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STRUCTURAL CALCULATION PACKAGE

43A REDINGTON ROAD
LONDON NW3 7RA

REF: 21141 REV B

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Symmetrys
Unit 6 The Courtyard
Lynton Road
London N8 8SL

Revision History

Revision	Description	Date	By	Checked
A	First Issue	31.08.21	SB	DS
B	Second Issue	14.01.22	SB	DS

1. Introduction

1.1 The structural works for the BIA consisted of the following:

- The sub-structure designs associated with the new basement. This will include the design of the new ground bearing slab, design of underpins, new retaining walls and new suspended ground floors.
- The reconfiguration of the lower ground and ground floors with the removal of structure as can be seen from overlaying the existing plans with the proposed plans.

2.0 Design Codes

2.1 The following design codes/guidance were used to carry out the design:

- BS EN 1991-1-1:2002 – Actions on Structures
- BS EN 1992-1-1:2004 – Design of Concrete Structures
- BS EN 1993-1-1:2005 – Design of Steel Structures
- BS EN 1996-1-1:2005 – Design of Masonry Structures
- CP 111 – Masonry

3.0 Ground Conditions

3.1 Allowable bearing pressures of 60KN/m2 and 80KN/m2 have been adopted for the design of foundations at formation level of respectively +9.00 and +8.00. Refer to the site investigation report by Geofirma for details of the ground conditions.

4.0 Substructure Design

4.1 Existing footings determined from trial pits. Refer to the site investigation report by Geofirma.

5.0 Loading

5.1 The loadings used throughout the design are shown in the table below:

Item	DL (kPa)	LL (kPa)
Timber Roof		
Asphalt waterproofing	0.45	
Timber joists and insulation	0.20	
Ceiling and services	0.15	
Flat	**0.90**	**0.60**
Pitched	**1.30**	**0.60**
Timber Floor		
Ceiling and services	0.15	
Timber joists and insulation	0.20	
Plyboard and finishes	0.25	
Total	**0.60**	**1.5**
Existing Masonry (19kN/m^3)		
225mm thick	4.30	
Finishes	0.20	
Total	**4.50**	
Existing Masonry (19kN/m^3)		
330mm thick	6.30	
Finishes	0.20	
Total	**6.50**	
Cavity Wall (19kN/m^3)		
100mm thick blockwork	1.50	
102.5mm thick brickwork	2.00	
Finishes	0.50	
Total	**4.00**	

Refer to Architects drawings for all setting out details

The contractor shall be responsible for the design, installation and sequencing of all temporary works and must ensure that stability of the structure is not compromised during the works

Legend

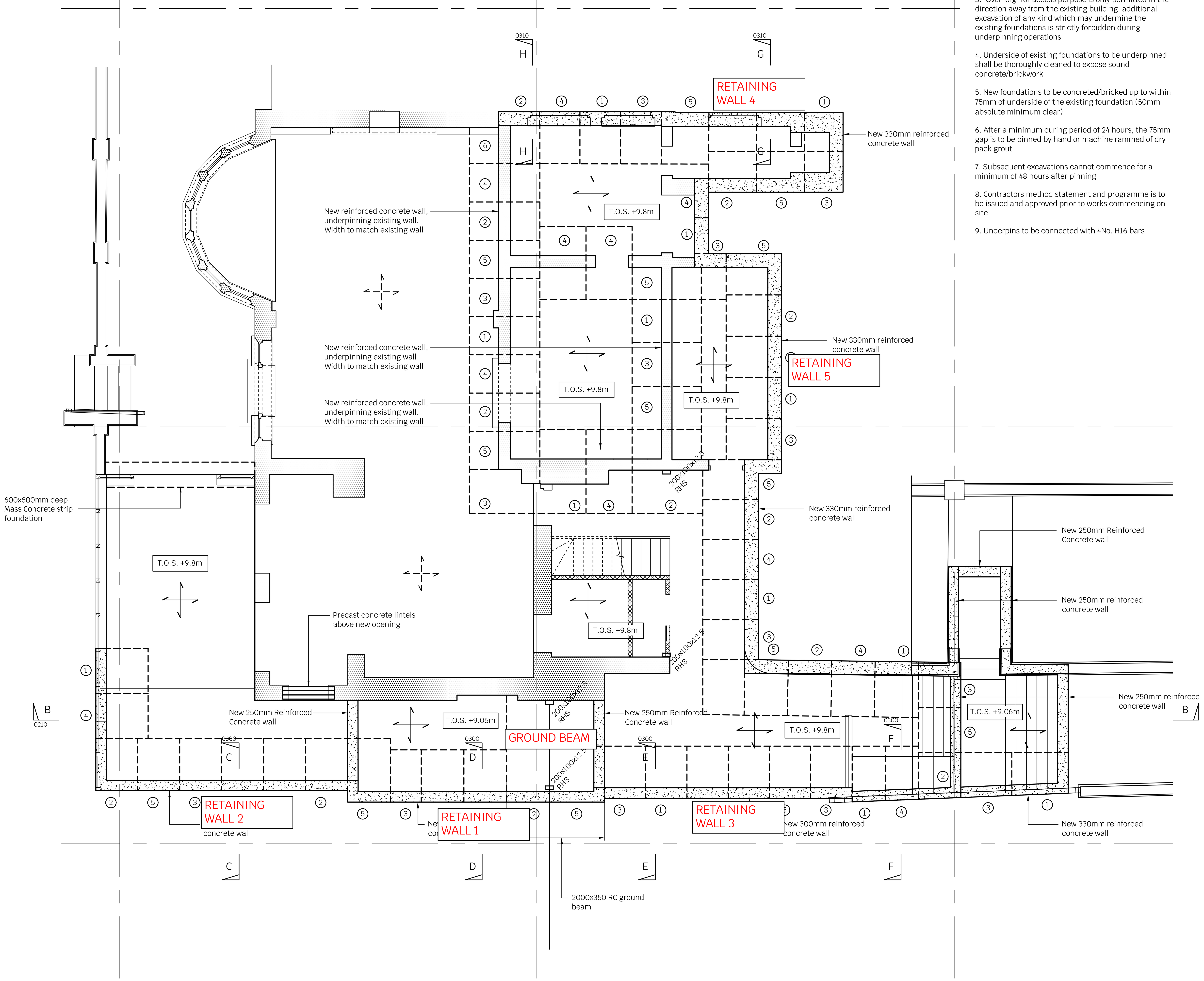
- Denotes existing masonry or timber structure
- Denotes new non load bearing stud wall by Architect
- Denotes existing slab to be retained
- Denotes new 250mm thick RC Slab
- T.O.S. +9.8m — Denotes top of slab (T.O.S.) level

Contractor/Specialist design element

1. All temporary works
2. All tanking details
3. All reinforcement drawings and bar bending schedules
4. Design of all steelwork connections. The fabricator will have to submit their calculations to building control for approval
5. Steel fabrication drawings
6. Design of all staircases, balustrades, glazing and secondary steel

Notes

- All steelwork in the external walls is to be galvanised (85 microns)
- All steelwork to be encased in concrete is to be un-painted
- Location of existing and proposed drain runs are to be confirmed by the Service Engineer
- Please refer to Architects drawings for all setting out details, insulation and ventilation details, damp proof courses and all tanking details
- For all fire work protection to steelwork refer to the Architects drawings
- Contractor should also review Architect's drawings for exact location of service penetration and confirm these with the Structural Engineer prior to cutting



Underpinning

1. Dry pack grout underpinning to be to the following specification:

a. Mix to be 3:1 of sharp sand o.p.c.
b. Sand to be sharpest that can be handled
c. Water content to be just enough to allow balling when compressed by hand

2. Excavation for new foundations to be carried out in maximum 1.0m lengths with a minimum clear space of 3.6m between open excavations at any one time

3. "Over-dig" for access purpose is only permitted in the direction away from the existing building. additional excavation of any kind which may undermine the existing foundations is strictly forbidden during underpinning operations

4. Underside of existing foundations to be underpinned shall be thoroughly cleaned to expose sound concrete/brickwork

5. New foundations to be concreted/bricked up to within 75mm of underside of the existing foundation (50mm absolute minimum clear)

6. After a minimum curing period of 24 hours, the 75mm gap is to be pinned by hand or machine rammed of dry pack grout

7. Subsequent excavations cannot commence for a minimum of 48 hours after pinning

8. Contractors method statement and programme is to be issued and approved prior to works commencing on site

9. Underpins to be connected with 4No. H16 bars

Notes

1. This drawing is to be read in conjunction with all relevant Architects & Engineers drawings and specifications
2. Do not scale from this drawing

General Propping Notes

1. The contractor will be responsible for all temporary works and will be responsible for the stability of the entire structure during the works
2. Our plans and section show a possible solution for the temporary works
3. All party wall notices are to be agreed
4. Install needles and props above the locations of the proposed beams
5. Ensure all props are laterally restrained and install all necessary bracing
6. Install any channel required across the head of the needles
7. If required ensure all temporary works are dry packed into place
8. Ensure no heavy materials are stored on the floors that are being propped
9. Break out the load bearing walls in question
10. Install new substructures
11. Install new steel frames and ensure that the are fully dry-packed into place in order to take the permanent loads
12. Once dry pack is set and approved by the building control officer strike the temporary props

Make good all areas of brickwork where needles where inserted

Rev	Date	Drwn	Chkd	Amendments
P1	00.08.21	JNS	XX	Preliminary Issue

Drawing Status PRELIMINARY

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Job Title
43A REDINGTON ROAD
LONDON, NW3

Drawing Title
TEMPORARY WORKS AND UNDERPINNING LAYOUT

Project	Company	Zones	Level	Type	Role	Number
21141	SYM	XX	XX	DR	S	0080

Scale: 1:50 AT A1 **Drawn By:** JNS **Revision :**
Date: AUG 2021 **Checked By:** SB P1

Refer to Architects drawings for all setting out details

The contractor shall be responsible for the design, installation and sequencing of all temporary works and must ensure that stability of the structure is not compromised during the works

Legend

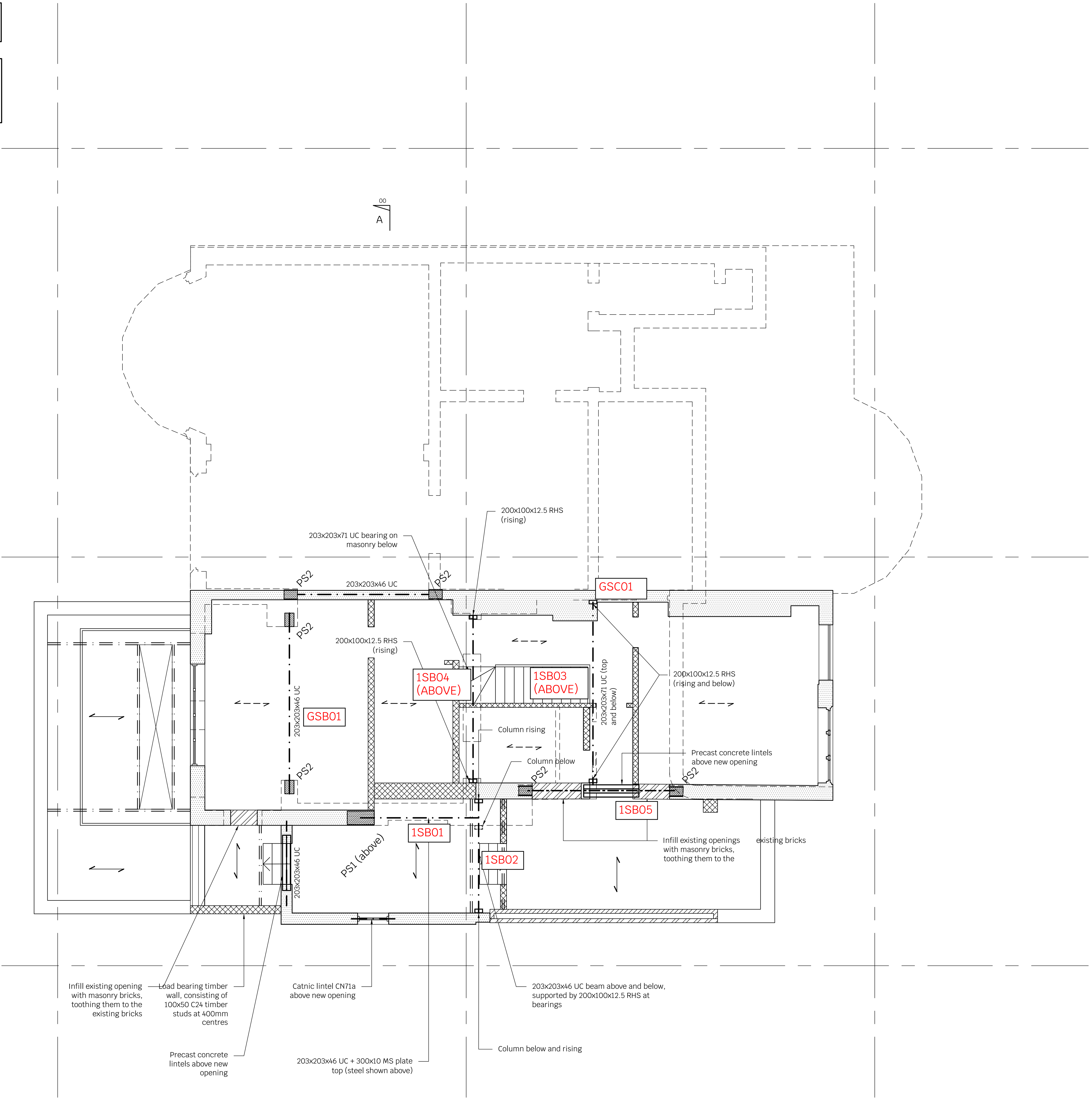
- Denotes existing masonry or timber structure
- Denotes new masonry walls built in 15N/mm² compressive strength brickwork and grade iii mortar
- Denotes new masonry walls built in 7.3N/mm² compressive strength blockwork and grade iii mortar
- Denotes new non load bearing stud wall by Architect, unless noted otherwise
- Denotes new floor joists 200x50mm C24 timber at 300mm centres
- PS1 Denotes 660 long x 330 wide x 215mm high mass concrete C30 padstone unless noted otherwise
- PS2 Denotes 330 long x 215 wide x 215mm high mass concrete C30 padstone unless noted otherwise
- Denotes joists doubled up and bolted together using M12 bolts at 400mm centres
- Denotes existing timber floor joists

Contractor/Specialist design element

1. All temporary works
2. All tanking details
3. All reinforcement drawings and bar bending schedules
4. Design of all steelwork connections. The fabricator will have to submit their calculations to building control for approval
5. Steel fabrication drawings
6. Design of all staircases, balustrades, glazing and secondary steel

Notes

- All steelwork in the external walls is to be galvanised (125 microns)
- All steelwork to be encased in concrete is to be un-painted
- Location of existing and proposed drain runs are to be confirmed by the Service Engineer
- Please refer to Architects drawings for all setting out details, insulation and ventilation details, damp proof courses and all tanking details
- For all fire work protection to steelwork refer to the Architects drawings
- Contractor should also review Architect's drawings for exact location of service penetration and confirm these with the Structural Engineer prior to cutting



Notes

1. This drawing is to be read in conjunction with all relevant Architects & Engineers drawings and specifications
2. Do not scale from this drawing

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Job Title
43A REDINGTON ROAD
LONDON, NW3

Drawing Title
GROUND FLOOR PLAN

Project	Company	Zones	Level	Type	Role	Number
21141	SYM	XX	XX	DR	S	0100

Scale: 1:50 AT A1
Date: AUG 2021
Drawn By: JNS
Checked By: SB
Revision : P1

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Job No.	Sheet No.	Revision
21141	2/2	

	Date	Made By	Checked By
Job Title 43a REDINGTON ROAD, LONDON			
Section STEEL FRAME - SF01 - FOUNDATION	20.8.21	SB	

AVERAGE UNIFORMLY DISTRIBUTED LOAD

$\Sigma P_{DL} = 39.7 + 89.4 = 129.1$ kN

$w_{DL} = \frac{129.1}{2.2} = 58.7$ kN/m

$\Sigma P_{LL} = 3.6 + 21.8 = 25.4$ kN

$w_{LL} = \frac{25.4}{2.2} = 11.5$ kN/m

REFER TO CALCULATION SHEET B-GB01 FOR GROUND BEAM DESIGN

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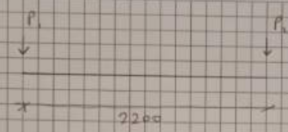
STRUCTURAL / CIVIL ENGINEERS

Job No.	Sheet No.	Revision
21141	1/2	

	Date	Made By	Checked By
Job Title 43a REDINGTON ROAD. LONDON			
Section STEEL FRAME - SF01 - FOUNDATION	20.8.21	SB	DS

GEOMETRY:



LOADING

$P_1 = 39.7$ kN (DL) $\quad \Sigma P_{1,DL+LL} = 43.3$ kN

3.6 kN (LL)

$P_2 = 89.4$ kN (DL) $\quad \Sigma P_{2,DL+LL} = 111.2$ kN

21.8 kN (LL)

GROUND BEARING PRESSURE:

$\Sigma P = 154.5$ kN

$\Sigma M = \frac{(111.2 - 43.3) \cdot 2.2}{2} = 74.7$ kNm

$e = \frac{\Sigma M}{\Sigma P} = \frac{74.7}{154.5} = 0.48 \text{ m} > \frac{L}{6}$ => PART OF THE FOUNDATION HAS PRESSURE = 0

TRY FOUNDATION WIDTH = 2m.

$GBP_{max} = \frac{2 \cdot \Sigma P}{3 \cdot B \left(\frac{L}{2} - e\right)} = \frac{2 \cdot 154.5}{3 \cdot 2 \left(\frac{2.2}{2} - 0.48\right)} = 83 \text{ kN/m}^2 < 95 \text{ kN/m}^2$ => OK

ADOPT 2m × 0.35m RC GROUND BEAM

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Job No.	Sheet No.	Revision
21141	1/2	

	Date	Made By	Checked By
Job Title 43a REDINGTON ROAD LONDON			
Section RETAINING WALLS LOAD TAKE DOWN AND STRIP FOUNDATIONS	20.8.21	SB	DS

VERTICAL LOADS

RET. WALL 1:

	DL	LL
275 mm masonry, h = 7.4 m	33.3	-
Roof (span 1.4 m)	1.8	0.8
G+First floor (span 1.4 m)	1.4 × 2	3.8 × 2
Total	37.9	8.4

RET. WALL 2:

	DL	LL
Cavity wall, h = 1.35 m	5.5	-
Parapet (cavity), h = 1.1 m	4.4	-
Ground floor (span 1 m)	1	2.5
	10.9	2.5

RET. WALL 3:

	DL	LL
Cavity wall, h = 4.62 m	18.5	-
Flat roof, span 1.45 m	1.45	0.9
Ground floor, span 1.45 m	1.45	3.9
	21.4	4.8

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Job No.	Sheet No.	Revision
21141	2/2	

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Job Title 43a REDINGTON ROAD, LONDON			
Section RETAINING WALLS LOAD TAKE DOWN AND STRIP FOUNDATIONS	20.8.21	SB	DS

VERTICAL LOADS

RET. WALL 4:

	DL	LL
340 masonry wall, h = 9.88 m	64.2	-
2+1+G floor, span 1.6 m	1.6 × 3	4.3 × 3
Roof, span 1.6 m	2.1	1
TOTAL	71.1	13.9

RET. WALL 5

	DL	LL
Ground floor, span 1.2 m	1.2	3.1
TOTAL	1.2	3.1

STRIP FOUNDATION 1

	DL	LL
225 masonry, h = 15 m	67.6	-
2+1+G floor, span 4.7 m	4.7 × 3	12.7 × 3
Roof, span 4.7 m	6.1	2.8
	87.8	40.9

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Job No.	Sheet No.	Revision
21141		
Date	**Made By**	**Checked By**
20.8.21	SB	DS

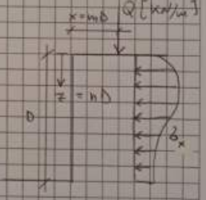
Job Title: 43a REDINGTON ROAD, LONDON

Section: RETAINING WALLS - SURCHARGE FROM No 41 REDINGTON ROAD

VERTICAL LOAD FROM FLANK WALL No 41 REDINGTON ROAD (ASSUMED)

	DL	LL
330 masonry ; h = 6,75 + 1,9 (TRIANGULAR)	56,1	/
2+1+G Floor :		
1 KN/m² · 3m	3 × 3	/
(1,5 + 1,2) KN/m² · 3m	/	8,1 × 3
TOTAL	65,1	24,3

→ SAY Q = (70 KN/m DL , 25 KN/m LL)



SURCHARGE CALCULATION BASED ON THE THEORY OF ELASTICITY :

$D = 2,7\,m$

$m = 0,37\,m \quad (x = 1m)$

As $m \le 0,4$:

$$\sigma_x = \frac{0,203\,Q\,n}{D(0,16 + n^2)^2}$$

$\sigma_{x,max,DL} \simeq 26,3\,KN/m^2 \quad (\text{for } n = 0,2)$

$\sigma_{x,max,LL} = 9,4\,KN/m^2 \quad (\text{for } n = 0,2)$

ACTIVE PRESSURE ($\phi = 27°$) = 0,37 (Coulomb)

ADDITIONALLY, AN IMPOSED SURCHARGE OF 2,5 KN/m² HAS BEEN CONSIDERED

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Job No.	Sheet No.	Revision
21141		
Date	**Made By**	**Checked By**
20.8.21	SB	DS

Job Title: 43a REDINGTON ROAD, LONDON

Section: PADSTONE P1

LOADING :

$V_{Ed} = 125,5\,KN$ (FROM '1F - 1SB01', REFER TO STEEL BEAM CALCULATION SHEET)

TRY 660 mm × 330 mm × 215 mm DEEP CONCRETE PADSTONE

$$\sigma_{Ed} = \frac{125,5 \cdot 1000}{660 \cdot 330} = 0,57\,\frac{N}{mm^2} < 0,63\,\frac{N}{mm^2} \Rightarrow \underline{OK}$$

WHERE :

$$\sigma_{Rd} = \frac{\sigma_K}{\gamma_m} = \frac{2,2}{3,5} = 0,63\,\frac{N}{mm^2}$$

RETAINING WALL ANALYSIS

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

Tedds calculation version 2.9.12

Retaining wall details

Stem type	Cantilever
Stem height	h_{stem} = **2500** mm
Stem thickness	t_{stem} = **225** mm
Angle to rear face of stem	α = **90** deg
Stem density	γ_{stem} = **25** kN/m^3
Toe length	l_{toe} = **2200** mm
Base thickness	t_{base} = **250** mm
Base density	γ_{base} = **25** kN/m^3
Height of retained soil	h_{ret} = **2500** mm
Angle of soil surface	β = **0** deg
Depth of cover	d_{cover} = **0** mm
Height of water	h_{water} = **1500** mm
Water density	γ_w = **9.8** kN/m^3

Retained soil properties

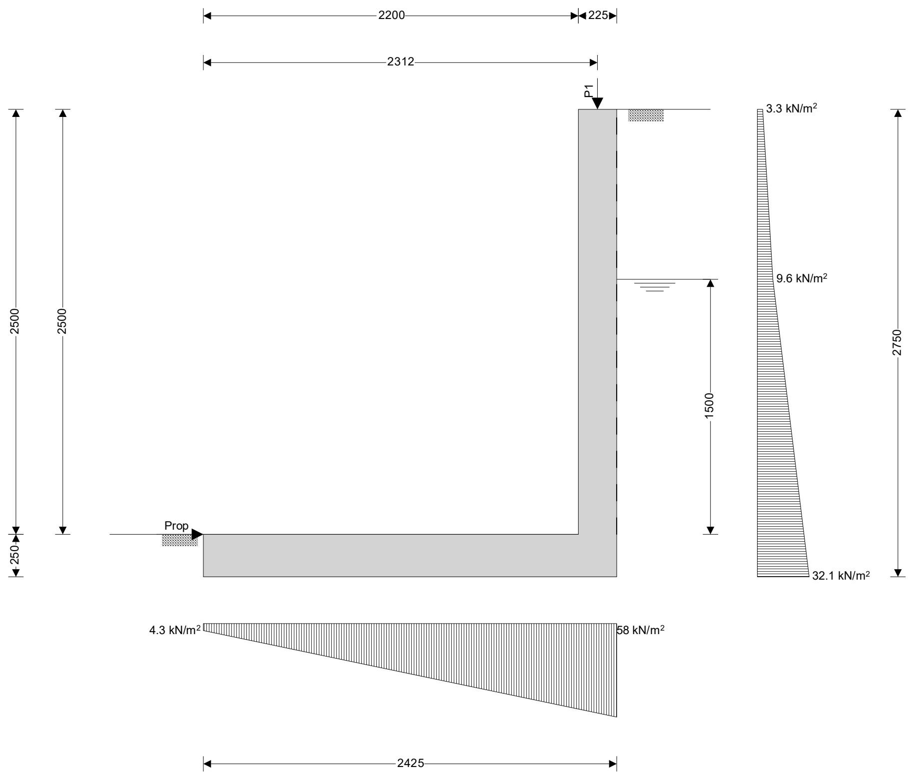
Soil type	Firm clay
Moist density	γ_{mr} = **19** kN/m^3
Saturated density	γ_{sr} = **19** kN/m^3
Characteristic effective shear resistance angle	$\phi'_{r.k}$ = **27** deg
Characteristic wall friction angle	$\delta_{r.k}$ = **13.5** deg

Base soil properties

Soil type	Firm clay
Soil density	γ_b = **19** kN/m^3
Characteristic effective shear resistance angle	$\phi'_{b.k}$ = **27** deg
Characteristic wall friction angle	$\delta_{b.k}$ = **13.5** deg
Characteristic base friction angle	$\delta_{bb.k}$ = **18** deg
Presumed bearing capacity	$P_{bearing}$ = **60** kN/m^2

Loading details

Variable surcharge load	$Surcharge_Q$ = **10** kN/m^2
Vertical line load at 2312 mm	P_{G1} = **37.9** kN/m
	P_{Q1} = **8.4** kN/m



General arrangement

Calculate retaining wall geometry

Base length	$l_{base} = l_{toe} + t_{stem} = \mathbf{2425}$ mm
Saturated soil height	$h_{sat} = h_{water} + d_{cover} = \mathbf{1500}$ mm
Moist soil height	$h_{moist} = h_{ret} - h_{water} = \mathbf{1000}$ mm
Length of surcharge load	$l_{sur} = l_{heel} = \mathbf{0}$ mm
- Distance to vertical component	$x_{sur\_v} = l_{base} - l_{heel} / 2 = \mathbf{2425}$ mm
Effective height of wall	$h_{eff} = h_{base} + d_{cover} + h_{ret} = \mathbf{2750}$ mm
- Distance to horizontal component	$x_{sur\_h} = h_{eff} / 2 = \mathbf{1375}$ mm
Area of wall stem	$A_{stem} = h_{stem} \times t_{stem} = \mathbf{0.563}$ m^2
- Distance to vertical component	$x_{stem} = l_{toe} + t_{stem} / 2 = \mathbf{2313}$ mm
Area of wall base	$A_{base} = l_{base} \times t_{base} = \mathbf{0.606}$ m^2
- Distance to vertical component	$x_{base} = l_{base} / 2 = \mathbf{1213}$ mm

Using Coulomb theory

Active pressure coefficient	$K_A = \sin(\alpha + \phi'_{r.k})^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta_{r.k}) \times [1 + \sqrt{[\sin(\phi'_{r.k} + \delta_{r.k}) \times \sin(\phi'_{r.k} - \beta) / (\sin(\alpha - \delta_{r.k}) \times \sin(\alpha + \beta))]}]^2) = \mathbf{0.340}$
Passive pressure coefficient	$K_P = \sin(90 - \phi'_{b.k})^2 / (\sin(90 + \delta_{b.k}) \times [1 - \sqrt{[\sin(\phi'_{b.k} + \delta_{b.k}) \times \sin(\phi'_{b.k}) / (\sin(90 + \delta_{b.k}))]}]^2) = \mathbf{4.044}$

Bearing pressure check

Vertical forces on wall

Wall stem	$F_{stem} = A_{stem} \times \gamma_{stem} = \mathbf{14.1}$ kN/m
Wall base	$F_{base} = A_{base} \times \gamma_{base} = \mathbf{15.2}$ kN/m

Line loads $F_{P\_v} = P_{G1} + P_{Q1} = \mathbf{46.3}$ kN/m

Total $F_{total\_v} = F_{stem} + F_{base} + F_{P\_v} + F_{water\_v} = \mathbf{75.5}$ kN/m

Horizontal forces on wall

Surcharge load $F_{sur\_h} = K_A \times \cos(\delta_{r.k}) \times Surcharge_Q \times h_{eff} = \mathbf{9.1}$ kN/m

Saturated retained soil $F_{sat\_h} = K_A \times \cos(\delta_{r.k}) \times (\gamma_{sr} - \gamma_w) \times (h_{sat} + h_{base})^2 / 2 = \mathbf{4.6}$ kN/m

Water $F_{water\_h} = \gamma_w \times (h_{water} + d_{cover} + h_{base})^2 / 2 = \mathbf{15}$ kN/m

Moist retained soil $F_{moist\_h} = K_A \times \cos(\delta_{r.k}) \times \gamma_{mr} \times ((h_{eff} - h_{sat} - h_{base})^2 / 2 + (h_{eff} - h_{sat} - h_{base}) \times (h_{sat} + h_{base})) = \mathbf{14.1}$ kN/m

Base soil $F_{pass\_h} = -K_P \times \cos(\delta_{b.k}) \times \gamma_b \times (d_{cover} + h_{base})^2 / 2 = \mathbf{-2.3}$ kN/m

Total $F_{total\_h} = F_{sur\_h} + F_{sat\_h} + F_{water\_h} + F_{moist\_h} + F_{pass\_h} = \mathbf{40.5}$ kN/m

Moments on wall

Wall stem $M_{stem} = F_{stem} \times x_{stem} = \mathbf{32.5}$ kNm/m

Wall base $M_{base} = F_{base} \times x_{base} = \mathbf{18.4}$ kNm/m

Surcharge load $M_{sur} = -F_{sur\_h} \times x_{sur\_h} = \mathbf{-12.5}$ kNm/m

Line loads $M_P = (P_{G1} + P_{Q1}) \times p_1 = \mathbf{107}$ kNm/m

Saturated retained soil $M_{sat} = -F_{sat\_h} \times x_{sat\_h} = \mathbf{-2.7}$ kNm/m

Water $M_{water} = -F_{water\_h} \times x_{water\_h} = \mathbf{-8.8}$ kNm/m

Moist retained soil $M_{moist} = -F_{moist\_h} \times x_{moist\_h} = \mathbf{-16.1}$ kNm/m

Total $M_{total} = M_{stem} + M_{base} + M_{sur} + M_P + M_{sat} + M_{water} + M_{moist} = \mathbf{117.8}$ kNm/m

Check bearing pressure

Propping force $F_{prop\_base} = F_{total\_h} = \mathbf{40.5}$ kN/m

Distance to reaction $\bar{x} = M_{total} / F_{total\_v} = \mathbf{1560}$ mm

Eccentricity of reaction $e = \bar{x} - l_{base} / 2 = \mathbf{348}$ mm

Loaded length of base $l_{load} = l_{base} = \mathbf{2425}$ mm

Bearing pressure at toe $q_{toe} = F_{total\_v} / l_{base} \times (1 - 6 \times e / l_{base}) = \mathbf{4.3}$ kN/m^2

Bearing pressure at heel $q_{heel} = F_{total\_v} / l_{base} \times (1 + 6 \times e / l_{base}) = \mathbf{58}$ kN/m^2

Factor of safety $FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel}) = \mathbf{1.035}$

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 2.9.12

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

Concrete strength class C30/37

Characteristic compressive cylinder strength $f_{ck} = \mathbf{30}$ N/mm^2

Characteristic compressive cube strength $f_{ck,cube} = \mathbf{37}$ N/mm^2

Mean value of compressive cylinder strength $f_{cm} = f_{ck} + 8$ N/mm^2 $= \mathbf{38}$ N/mm^2

Mean value of axial tensile strength $f_{ctm} = 0.3$ N/mm$^2 \times (f_{ck} / 1$ N/mm$^2)^{2/3} = \mathbf{2.9}$ N/mm^2

5% fractile of axial tensile strength $f_{ctk,0.05} = 0.7 \times f_{ctm} = \mathbf{2.0}$ N/mm^2

Secant modulus of elasticity of concrete $E_{cm} = 22$ kN/mm$^2 \times (f_{cm} / 10$ N/mm$^2)^{0.3} = \mathbf{32837}$ 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 $f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C = \mathbf{17.0}$ N/mm^2

Maximum aggregate size $h_{agg} = \mathbf{20}$ mm

Ultimate strain - Table 3.1	ε_{cu2} = **0.0035**
Shortening strain - Table 3.1	ε_{cu3} = **0.0035**
Effective compression zone height factor	λ = **0.80**
Effective strength factor	η = **1.00**
Bending coefficient k_1	K_1 = **0.40**
Bending coefficient k_2	$K_2 = 1.00 \times (0.6 + 0.0014/\varepsilon_{cu2})$ = **1.00**
Bending coefficient k_3	K_3 =**0.40**
Bending coefficient k_4	$K_4 = 1.00 \times (0.6 + 0.0014/\varepsilon_{cu2})$ =**1.00**

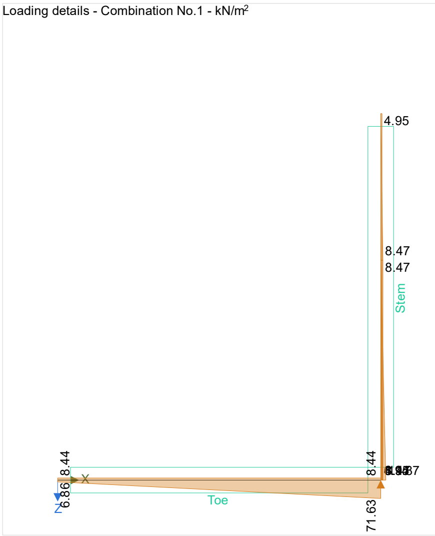
Reinforcement details

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

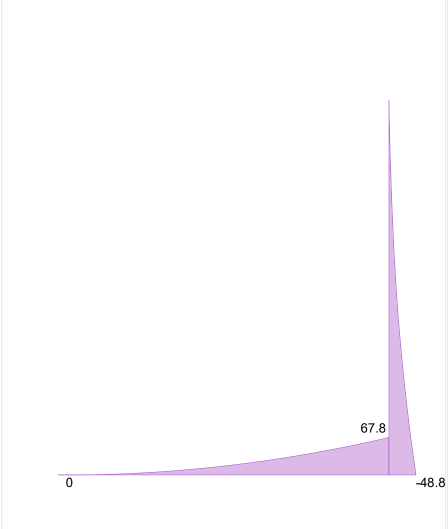
Cover to reinforcement

Front face of stem	c_{sf} = **40** mm
Rear face of stem	c_{sr} = **50** mm
Top face of base	c_{bt} = **50** mm
Bottom face of base	c_{bb} = **75** mm

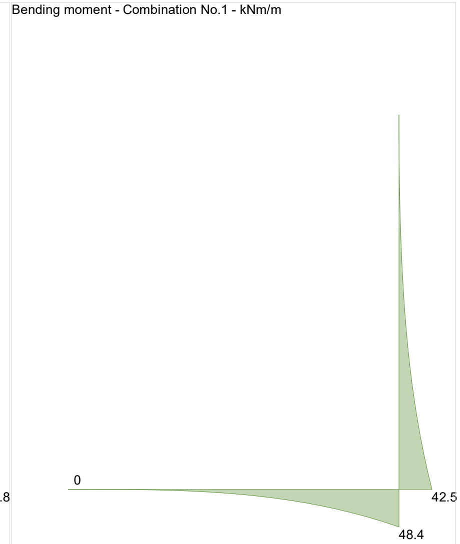
Loading details - Combination No.1 - kN/m^2

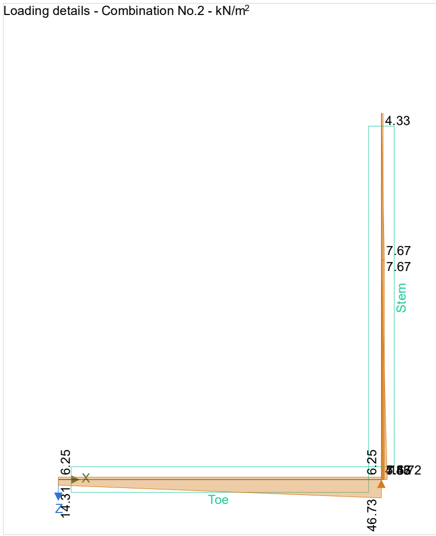


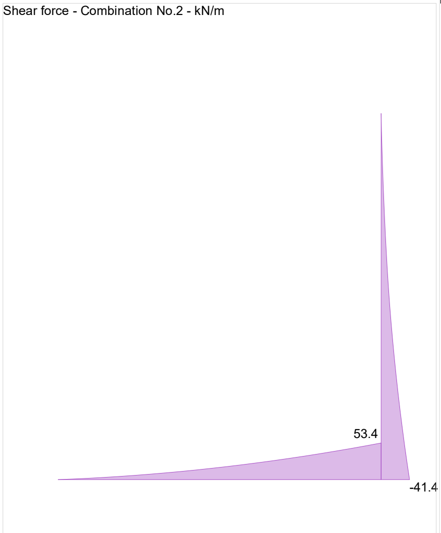
Shear force - Combination No.1 - kN/m

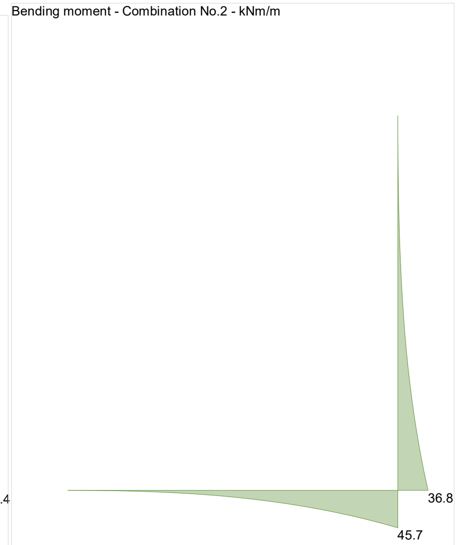


Bending moment - Combination No.1 - kNm/m









Check stem design at base of stem

Depth of section	h = **225** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1	M = **42.5** kNm/m
Depth to tension reinforcement	$d = h - c_{sr} - \phi_{sr} / 2$ = **167** mm
	$K = M / (d^2 \times f_{ck})$ = **0.051**
	$K' = (2 \times \eta \times \alpha_{cc}/\gamma_C)\times(1 - \lambda \times (\delta - K_1)/(2 \times K_2))\times(\lambda \times (\delta - K_1)/(2 \times K_2))$
	K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm	$z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d$ = **159** mm
Depth of neutral axis	$x = 2.5 \times (d – z)$ = **21** mm
Area of tension reinforcement required	$A_{sr.req} = M / (f_{yd} \times z)$ = **616** mm²/m
Tension reinforcement provided	16 dia.bars @ 150 c/c
Area of tension reinforcement provided	$A_{sr.prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr})$ = **1340** mm²/m
Minimum area of reinforcement - exp.9.1N	$A_{sr.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d$ = **252** mm²/m
Maximum area of reinforcement - cl.9.2.1.1(3)	$A_{sr.max} = 0.04 \times h$ = **9000** mm²/m
	$\max(A_{sr.req}, A_{sr.min}) / A_{sr.prov}$ = **0.46**

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

Library item: Rectangular single output

Deflection control - Section 7.4

Reference reinforcement ratio	$\rho_0 = \sqrt{}(f_{ck} / 1 \text{ N/mm}^2) / 1000$ = **0.005**
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	$K_s = \min(500 \text{ N/mm}^2 / (f_{yk} \times A_{sr.req} / A_{sr.prov}), 1.5)$ = **1.5**
Limiting span to depth ratio - exp.7.16.a	$\min(K_s \times K_b \times [11 + 1.5 \times \sqrt{}(f_{ck} / 1 \text{ N/mm}^2) \times \rho_0 / \rho + 3.2 \times \sqrt{}(f_{ck} / 1 \text{ N/mm}^2) \times (\rho_0 / \rho - 1)^{3/2}], 40 \times K_b)$ = **16**
Actual span to depth ratio	h_{stem} / d = **15**

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 — ψ_2 = **0.6**

Serviceability bending moment — M_{sls} = **26.2** kNm/m

Tensile stress in reinforcement — $\sigma_s = M_{sls} / (A_{sr.prov} \times z)$ = **123.3** N/mm^2

Load duration — Long term

Load duration factor — k_t = **0.4**

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

$A_{c.eff}$ = **68042** mm^2/m

Mean value of concrete tensile strength — $f_{ct.eff} = f_{ctm}$ = **2.9** N/mm^2

Reinforcement ratio — $\rho_{p.eff} = A_{sr.prov} / A_{c.eff}$ = **0.020**

Modular ratio — $\alpha_e = E_s / E_{cm}$ = **6.091**

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 — $s_{r.max} = k_3 \times c_{sr} + k_1 \times k_2 \times k_4 \times \phi_{sr} / \rho_{p.eff}$ = **308** 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 = **0.114** mm

w_k / w_{max} = **0.38**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force — V = **48.8** kN/m

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

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

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

$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5}$ = **0.542** N/mm^2

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

$V_{Rd.c}$ = **115.7** kN/m

$V / V_{Rd.c}$ = **0.422**

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) — $A_{sx.req} = \max(0.25 \times A_{sr.prov}, 0.001 \times t_{stem})$ = **335** mm^2/m

Maximum spacing of reinforcement – cl.9.6.3(2) — s_{sx_max} = **400** mm

Transverse reinforcement provided — 10 dia.bars @ 200 c/c

Area of transverse reinforcement provided — $A_{sx.prov} = \pi \times \phi_{sx}^2 / (4 \times s_{sx})$ = **393** mm^2/m

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

Check base design at toe

Depth of section — h = **250** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 — M = **48.4** kNm/m

Depth to tension reinforcement — $d = h - c_{bb} - \phi_{bb} / 2$ = **167** mm

$K = M / (d^2 \times f_{ck})$ = **0.058**

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

K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm	$z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d =$ **158** mm
Depth of neutral axis	$x = 2.5 \times (d - z) =$ **23** mm
Area of tension reinforcement required	$A_{bb.req} = M / (f_{yd} \times z) =$ **705** mm²/m
Tension reinforcement provided	16 dia.bars @ 150 c/c
Area of tension reinforcement provided	$A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb}) =$ **1340** mm²/m
Minimum area of reinforcement - exp.9.1N	$A_{bb.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d =$ **252** mm²/m
Maximum area of reinforcement - cl.9.2.1.1(3)	$A_{bb.max} = 0.04 \times h =$ **10000** mm²/m
	$\max(A_{bb.req}, A_{bb.min}) / A_{bb.prov} =$ **0.526**

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	$\psi_2 =$ **0.6**
Serviceability bending moment	$M_{sls} =$ **34.6** kNm/m
Tensile stress in reinforcement	$\sigma_s = M_{sls} / (A_{bb.prov} \times z) =$ **163.4** N/mm²
Load duration	Long term
Load duration factor	$k_t =$ **0.4**
Effective area of concrete in tension	$A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$
	$A_{c.eff} =$ **75818** mm²/m
Mean value of concrete tensile strength	$f_{ct.eff} = f_{ctm} =$ **2.9** N/mm²
Reinforcement ratio	$\rho_{p.eff} = A_{bb.prov} / A_{c.eff} =$ **0.018**
Modular ratio	$\alpha_e = E_s / E_{cm} =$ **6.091**
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	$s_{r.max} = k_3 \times c_{bb} + k_1 \times k_2 \times k_4 \times \phi_{bb} / \rho_{p.eff} =$ **409** mm
Maximum crack width - exp.7.8	$w_k = s_{r.max} \times \max(\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$
	$w_k =$ **0.2** mm
	$w_k / w_{max} =$ **0.668**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force	$V =$ **67.8** kN/m
	$C_{Rd,c} = 0.18 / \gamma_C =$ **0.120**
	$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2) =$ **2.000**
Longitudinal reinforcement ratio	$\rho_l = \min(A_{bb.prov} / d, 0.02) =$ **0.008**
	$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5} =$ **0.542** N/mm²
Design shear resistance - exp.6.2a & 6.2b	$V_{Rd.c} = \max(C_{Rd.c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$
	$V_{Rd.c} =$ **115.7** kN/m
	$V / V_{Rd.c} =$ **0.586**

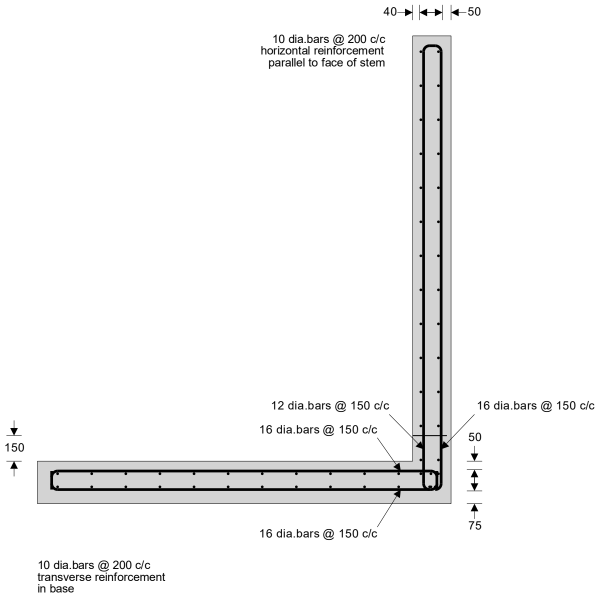
PASS - Design shear resistance exceeds design shear force

Secondary transverse reinforcement to base - Section 9.3

Minimum area of reinforcement – cl.9.3.1.1(2)	$A_{bx.req} = 0.2 \times A_{bb.prov} =$ **268** mm²/m
Maximum spacing of reinforcement – cl.9.3.1.1(3)	$s_{bx\_max} =$ **450** mm
Transverse reinforcement provided	10 dia.bars @ 200 c/c

Area of transverse reinforcement provided $A_{bx.prov} = \pi \times \phi_{bx}^2 / (4 \times s_{bx}) = \mathbf{393}\ mm^2/m$

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



Reinforcement details

RETAINING WALL ANALYSIS

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

Tedds calculation version 2.9.12

Retaining wall details

Stem type	Cantilever
Stem height	h_{stem} = **1700** mm
Stem thickness	t_{stem} = **225** mm
Angle to rear face of stem	α = **90** deg
Stem density	γ_{stem} = **25** kN/m^3
Toe length	l_{toe} = **2000** mm
Base thickness	t_{base} = **300** mm
Base density	γ_{base} = **25** kN/m^3
Height of retained soil	h_{ret} = **1700** mm
Angle of soil surface	β = **0** deg
Depth of cover	d_{cover} = **0** mm
Height of water	h_{water} = **700** mm
Water density	γ_w = **9.8** kN/m^3

Retained soil properties

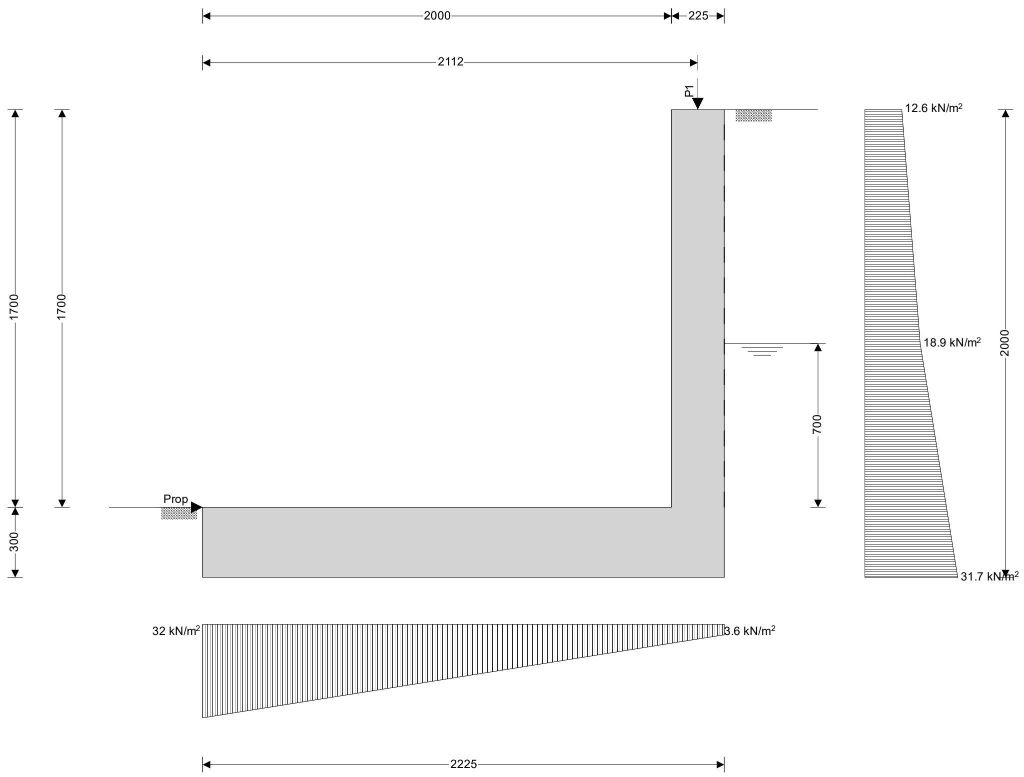
Soil type	Firm clay
Moist density	γ_{mr} = **19** kN/m^3
Saturated density	γ_{sr} = **19** kN/m^3
Characteristic effective shear resistance angle	$\phi'_{r.k}$ = **27** deg
Characteristic wall friction angle	$\delta_{r.k}$ = **13.5** deg

Base soil properties

Soil type	Firm clay
Soil density	γ_b = **19** kN/m^3
Characteristic effective shear resistance angle	$\phi'_{b.k}$ = **27** deg
Characteristic wall friction angle	$\delta_{b.k}$ = **13.5** deg
Characteristic base friction angle	$\delta_{bb.k}$ = **18** deg
Presumed bearing capacity	$P_{bearing}$ = **60** kN/m^2

Loading details

Permanent surcharge load	$Surcharge_G$ = **26.3** kN/m^2
Variable surcharge load	$Surcharge_Q$ = **11.9** kN/m^2
Vertical line load at 2112 mm	P_{G1} = **10.9** kN/m
	P_{Q1} = **2.5** kN/m



General arrangement

Calculate retaining wall geometry

Base length $l_{base} = l_{toe} + t_{stem} = \mathbf{2225}$ mm

Saturated soil height $h_{sat} = h_{water} + d_{cover} = \mathbf{700}$ mm

Moist soil height $h_{moist} = h_{ret} - h_{water} = \mathbf{1000}$ mm

Length of surcharge load $l_{sur} = l_{heel} = \mathbf{0}$ mm

- Distance to vertical component $x_{sur\_v} = l_{base} - l_{heel} / 2 = \mathbf{2225}$ mm

Effective height of wall $h_{eff} = h_{base} + d_{cover} + h_{ret} = \mathbf{2000}$ mm

- Distance to horizontal component $x_{sur\_h} = h_{eff} / 2 = \mathbf{1000}$ mm

Area of wall stem $A_{stem} = h_{stem} \times t_{stem} = \mathbf{0.383}$ m^2

- Distance to vertical component $x_{stem} = l_{toe} + t_{stem} / 2 = \mathbf{2113}$ mm

Area of wall base $A_{base} = l_{base} \times t_{base} = \mathbf{0.668}$ m^2

- Distance to vertical component $x_{base} = l_{base} / 2 = \mathbf{1113}$ mm

Using Coulomb theory

Active pressure coefficient $K_A = \sin(\alpha + \phi'_{r.k})^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta_{r.k}) \times [1 + \sqrt{[\sin(\phi'_{r.k} + \delta_{r.k}) \times \sin(\phi'_{r.k} - \beta) / (\sin(\alpha - \delta_{r.k}) \times \sin(\alpha + \beta))]}]^2) = \mathbf{0.340}$

Passive pressure coefficient $K_P = \sin(90 - \phi'_{b.k})^2 / (\sin(90 + \delta_{b.k}) \times [1 - \sqrt{[\sin(\phi'_{b.k} + \delta_{b.k}) \times \sin(\phi'_{b.k}) / (\sin(90 + \delta_{b.k}))]}]^2) = \mathbf{4.044}$

Bearing pressure check

Vertical forces on wall

Wall stem $F_{stem} = A_{stem} \times \gamma_{stem} = \mathbf{9.6}$ kN/m

Wall base $F_{base} = A_{base} \times \gamma_{base} = \mathbf{16.7}$ kN/m

Line loads	$F_{P\_v} = P_{G1} + P_{Q1} = \mathbf{13.4}$ kN/m
Total	$F_{total\_v} = F_{stem} + F_{base} + F_{P\_v} + F_{water\_v} = \mathbf{39.7}$ kN/m

Horizontal forces on wall

Surcharge load	$F_{sur\_h} = K_A \times \cos(\delta_{r.k}) \times (Surcharge_G + Surcharge_Q) \times h_{eff} = \mathbf{25.2}$ kN/m
Saturated retained soil	$F_{sat\_h} = K_A \times \cos(\delta_{r.k}) \times (\gamma_{sr} - \gamma_w) \times (h_{sat} + h_{base})^2 / 2 = \mathbf{1.5}$ kN/m
Water	$F_{water\_h} = \gamma_w \times (h_{water} + d_{cover} + h_{base})^2 / 2 = \mathbf{4.9}$ kN/m
Moist retained soil	$F_{moist\_h} = K_A \times \cos(\delta_{r.k}) \times \gamma_{mr} \times ((h_{eff} - h_{sat} - h_{base})^2 / 2 + (h_{eff} - h_{sat} - h_{base}) \times (h_{sat} + h_{base})) = \mathbf{9.4}$ kN/m
Base soil	$F_{pass\_h} = -K_P \times \cos(\delta_{b.k}) \times \gamma_b \times (d_{cover} + h_{base})^2 / 2 = \mathbf{-3.4}$ kN/m
Total	$F_{total\_h} = F_{sur\_h} + F_{sat\_h} + F_{water\_h} + F_{moist\_h} + F_{pass\_h} = \mathbf{37.7}$ kN/m

Moments on wall

Wall stem	$M_{stem} = F_{stem} \times x_{stem} = \mathbf{20.2}$ kNm/m
Wall base	$M_{base} = F_{base} \times x_{base} = \mathbf{18.6}$ kNm/m
Surcharge load	$M_{sur} = -F_{sur\_h} \times x_{sur\_h} = \mathbf{-25.2}$ kNm/m
Line loads	$M_P = (P_{G1} + P_{Q1}) \times p_1 = \mathbf{28.3}$ kNm/m
Saturated retained soil	$M_{sat} = -F_{sat\_h} \times x_{sat\_h} = \mathbf{-0.5}$ kNm/m
Water	$M_{water} = -F_{water\_h} \times x_{water\_h} = \mathbf{-1.6}$ kNm/m
Moist retained soil	$M_{moist} = -F_{moist\_h} \times x_{moist\_h} = \mathbf{-7.3}$ kNm/m
Total	$M_{total} = M_{stem} + M_{base} + M_{sur} + M_P + M_{sat} + M_{water} + M_{moist} = \mathbf{32.4}$ kNm/m

Check bearing pressure

Propping force	$F_{prop\_base} = F_{total\_h} = \mathbf{37.7}$ kN/m
Distance to reaction	$\bar{x} = M_{total} / F_{total\_v} = \mathbf{817}$ mm
Eccentricity of reaction	$e = \bar{x} - l_{base} / 2 = \mathbf{-296}$ mm
Loaded length of base	$l_{load} = l_{base} = \mathbf{2225}$ mm
Bearing pressure at toe	$q_{toe} = F_{total\_v} / l_{base} \times (1 - 6 \times e / l_{base}) = \mathbf{32}$ kN/m^2
Bearing pressure at heel	$q_{heel} = F_{total\_v} / l_{base} \times (1 + 6 \times e / l_{base}) = \mathbf{3.6}$ kN/m^2
Factor of safety	$FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel}) = \mathbf{1.873}$

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 2.9.12

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

Concrete strength class	C30/37
Characteristic compressive cylinder strength	$f_{ck} = \mathbf{30}$ N/mm^2
Characteristic compressive cube strength	$f_{ck,cube} = \mathbf{37}$ N/mm^2
Mean value of compressive cylinder strength	$f_{cm} = f_{ck} + 8$ N/mm$^2 = \mathbf{38}$ N/mm^2
Mean value of axial tensile strength	$f_{ctm} = 0.3$ N/mm$^2 \times (f_{ck} / 1$ N/mm$^2)^{2/3} = \mathbf{2.9}$ N/mm^2
5% fractile of axial tensile strength	$f_{ctk,0.05} = 0.7 \times f_{ctm} = \mathbf{2.0}$ N/mm^2
Secant modulus of elasticity of concrete	$E_{cm} = 22$ kN/mm$^2 \times (f_{cm} / 10$ N/mm$^2)^{0.3} = \mathbf{32837}$ 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	$f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C = \mathbf{17.0}$ N/mm^2
Maximum aggregate size	$h_{agg} = \mathbf{20}$ mm

Ultimate strain - Table 3.1	ε_{cu2} = **0.0035**
Shortening strain - Table 3.1	ε_{cu3} = **0.0035**
Effective compression zone height factor	λ = **0.80**
Effective strength factor	η = **1.00**
Bending coefficient k_1	K_1 = **0.40**
Bending coefficient k_2	$K_2 = 1.00 \times (0.6 + 0.0014/\varepsilon_{cu2})$ = **1.00**
Bending coefficient k_3	K_3 =**0.40**
Bending coefficient k_4	$K_4 = 1.00 \times (0.6 + 0.0014/\varepsilon_{cu2})$ =**1.00**

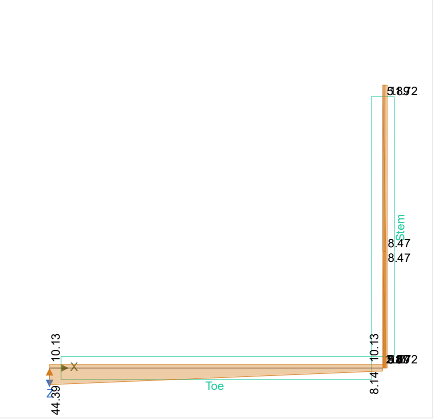
Reinforcement details

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

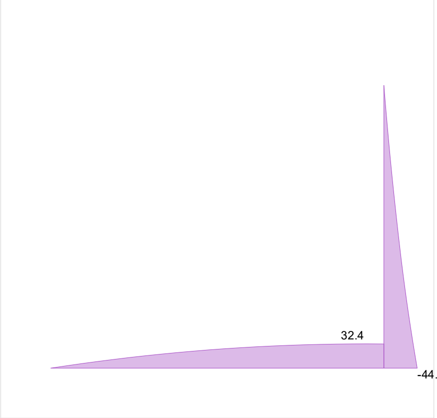
Cover to reinforcement

Front face of stem	c_{sf} = **40** mm
Rear face of stem	c_{sr} = **50** mm
Top face of base	c_{bt} = **50** mm
Bottom face of base	c_{bb} = **75** mm

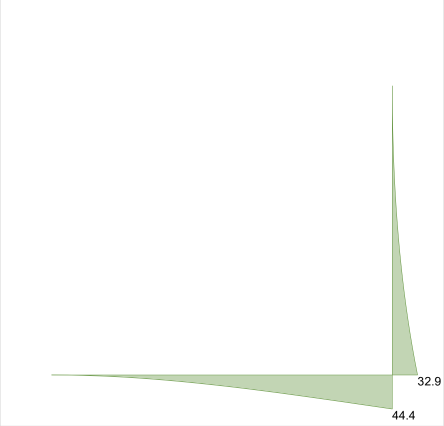
Loading details - Combination No.1 - kN/m²



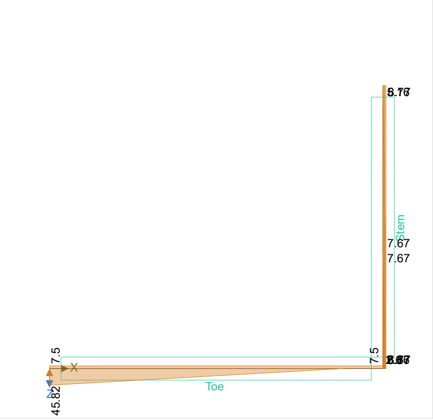
Shear force - Combination No.1 - kN/m



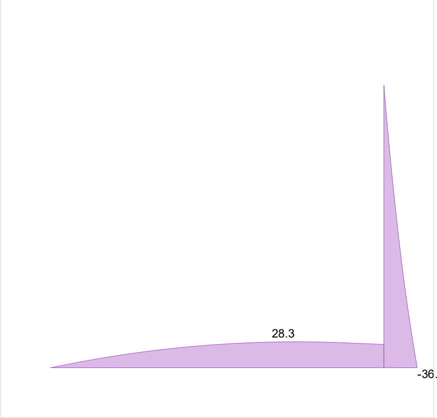
Bending moment - Combination No.1 - kNm/m



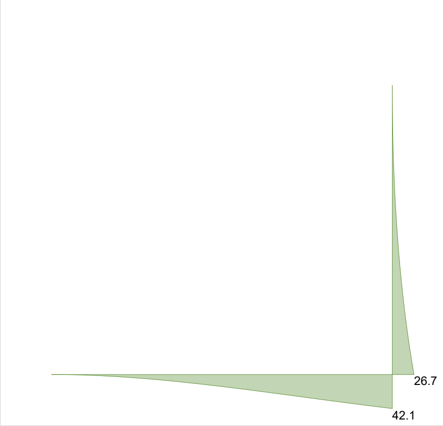
Loading details - Combination No.2 - kN/m²



Shear force - Combination No.2 - kN/m



Bending moment - Combination No.2 - kNm/m



Check stem design at base of stem

Depth of section — h = **225** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 — M = **32.9** kNm/m

Depth to tension reinforcement — $d = h - c_{sr} - \phi_{sr} / 2$ = **167** mm

$K = M / (d^2 \times f_{ck})$ = **0.039**

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

K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm — $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d$ = **159** mm

Depth of neutral axis — $x = 2.5 \times (d - z)$ = **21** mm

Area of tension reinforcement required — $A_{sr.req} = M / (f_{yd} \times z)$ = **477** mm²/m

Tension reinforcement provided — 16 dia.bars @ 150 c/c

Area of tension reinforcement provided — $A_{sr.prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr})$ = **1340** mm²/m

Minimum area of reinforcement - exp.9.1N — $A_{sr.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d$ = **252** mm²/m

Maximum area of reinforcement - cl.9.2.1.1(3) — $A_{sr.max} = 0.04 \times h$ = **9000** mm²/m

$\max(A_{sr.req}, A_{sr.min}) / A_{sr.prov}$ = **0.356**

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

Library item: Rectangular single output

Deflection control - Section 7.4

Reference reinforcement ratio — $\rho_0 = \sqrt{(f_{ck} / 1 \text{ N/mm}^2)} / 1000$ = **0.005**

Required tension reinforcement ratio — $\rho = A_{sr.req} / d$ = **0.003**

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 — $K_s = \min(500 \text{ N/mm}^2 / (f_{yk} \times A_{sr.req} / A_{sr.prov}), 1.5)$ = **1.5**

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

Actual span to depth ratio — h_{stem} / d = **10.2**

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 — ψ_2 = **0.6**

Serviceability bending moment — M_{sls} = **21.5** kNm/m

Tensile stress in reinforcement — $\sigma_s = M_{sls} / (A_{sr.prov} \times z)$ = **100.9** N/mm²

Load duration — Long term

Load duration factor — k_t = **0.4**

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

$A_{c.eff}$ = **68042** mm²/m

Mean value of concrete tensile strength — $f_{ct.eff} = f_{ctm}$ = **2.9** N/mm²

Reinforcement ratio — $\rho_{p.eff} = A_{sr.prov} / A_{c.eff}$ = **0.020**

Modular ratio — $\alpha_e = E_s / E_{cm}$ = **6.091**

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 — $s_{r.max} = k_3 \times c_{sr} + k_1 \times k_2 \times k_4 \times \phi_{sr} / \rho_{p.eff}$ = **308** 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 = **0.093** mm

w_k / w_{max} = **0.311**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force — V = **44.4** kN/m

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

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

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

$v_{min} = 0.035\ N^{1/2}/mm \times k^{3/2} \times f_{ck}^{0.5}$ = **0.542** N/mm²

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

$V_{Rd.c}$ = **115.7** kN/m

$V / V_{Rd.c}$ = **0.383**

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) — $A_{sx.req} = \max(0.25 \times A_{sr.prov}, 0.001 \times t_{stem})$ = **335** mm²/m

Maximum spacing of reinforcement – cl.9.6.3(2) — s_{sx_max} = **400** mm

Transverse reinforcement provided — 10 dia.bars @ 200 c/c

Area of transverse reinforcement provided — $A_{sx.prov} = \pi \times \phi_{sx}^2 / (4 \times s_{sx})$ = **393** mm²/m

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

Check base design at toe

Depth of section — h = **300** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 — M = **44.4** kNm/m

Depth to tension reinforcement — $d = h - c_{bb} - \phi_{bb} / 2$ = **217** mm

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

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

K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm — $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d$ = **206** mm

Depth of neutral axis — $x = 2.5 \times (d - z)$ = **27** mm

Area of tension reinforcement required — $A_{bb.req} = M / (f_{yd} \times z)$ = **495** mm²/m

Tension reinforcement provided — 16 dia.bars @ 150 c/c

Area of tension reinforcement provided — $A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb})$ = **1340** mm²/m

Minimum area of reinforcement - exp.9.1N — $A_{bb.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d$ = **327** mm²/m

Maximum area of reinforcement - cl.9.2.1.1(3) — $A_{bb.max} = 0.04 \times h$ = **12000** mm²/m

$\max(A_{bb.req}, A_{bb.min}) / A_{bb.prov}$ = **0.369**

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} = **32** kNm/m

Tensile stress in reinforcement — $\sigma_s = M_{sls} / (A_{bb.prov} \times z)$ = **115.9** N/mm²

Load duration	Long term
Load duration factor	k_t = **0.4**
Effective area of concrete in tension	$A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$
	$A_{c.eff}$ = **90958** mm²/m
Mean value of concrete tensile strength	$f_{ct.eff} = f_{ctm}$ = **2.9** N/mm²
Reinforcement ratio	$\rho_{p.eff} = A_{bb.prov} / A_{c.eff}$ = **0.015**
Modular ratio	$\alpha_e = E_s / E_{cm}$ = **6.091**
Bond property coefficient	k_1 = **0.8**
Strain distribution coefficient	k_2 = **0.5**
	k_3 = **3.4**
	k_4 = **0.425**
Maximum crack spacing - exp.7.11	$s_{r.max} = k_3 \times c_{bb} + k_1 \times k_2 \times k_4 \times \phi_{bb} / \rho_{p.eff}$ = **440** mm
Maximum crack width - exp.7.8	$w_k = s_{r.max} \times \max(\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$
	w_k = **0.153** mm
	w_k / w_{max} = **0.51**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force	V = **32.4** kN/m
	$C_{Rd,c} = 0.18 / \gamma_C$ = **0.120**
	$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2)$ = **1.960**
Longitudinal reinforcement ratio	$\rho_l = \min(A_{bb.prov} / d, 0.02)$ = **0.006**
	$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5}$ = **0.526** N/mm²
Design shear resistance - exp.6.2a & 6.2b	$V_{Rd.c} = \max(C_{Rd.c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$
	$V_{Rd.c}$ = **135.1** kN/m
	$V / V_{Rd.c}$ = **0.240**

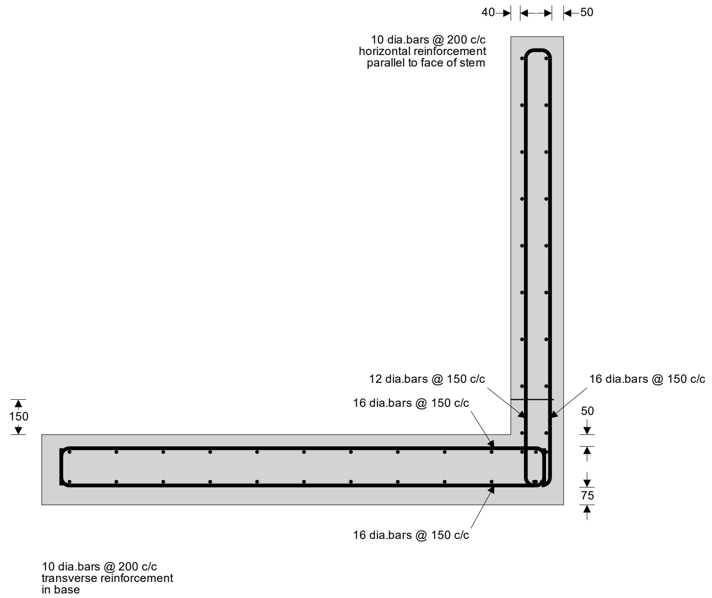
PASS - Design shear resistance exceeds design shear force

Secondary transverse reinforcement to base - Section 9.3

Minimum area of reinforcement – cl.9.3.1.1(2)	$A_{bx.req} = 0.2 \times A_{bb.prov}$ = **268** mm²/m
Maximum spacing of reinforcement – cl.9.3.1.1(3)	s_{bx_max} = **450** mm
Transverse reinforcement provided	10 dia.bars @ 200 c/c
Area of transverse reinforcement provided	$A_{bx.prov} = \pi \times \phi_{bx}^2 / (4 \times s_{bx})$ = **393** mm²/m

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

Symmetrys Unit 6 The Courtyard, Lynton Road London N8 8SL	Project 43a REDINGTON ROAD, LONDON NW3 7RA				Job no. 21141	
	Calcs for B-RW02				Start page no./Revision 8 B	
	Calcs by SB	Calcs date 14/01/2022	Checked by DS	Checked date 14/01/2022	Approved by DS	Approved date 14/01/2022



Reinforcement details

RETAINING WALL ANALYSIS

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

Tedds calculation version 2.9.12

Retaining wall details

Stem type	Cantilever
Stem height	h_{stem} = **1700** mm
Stem thickness	t_{stem} = **225** mm
Angle to rear face of stem	α = **90** deg
Stem density	γ_{stem} = **25** kN/m^3
Toe length	l_{toe} = **2000** mm
Base thickness	t_{base} = **300** mm
Base density	γ_{base} = **25** kN/m^3
Height of retained soil	h_{ret} = **1700** mm
Angle of soil surface	β = **0** deg
Depth of cover	d_{cover} = **0** mm
Height of water	h_{water} = **700** mm
Water density	γ_w = **9.8** kN/m^3

Retained soil properties

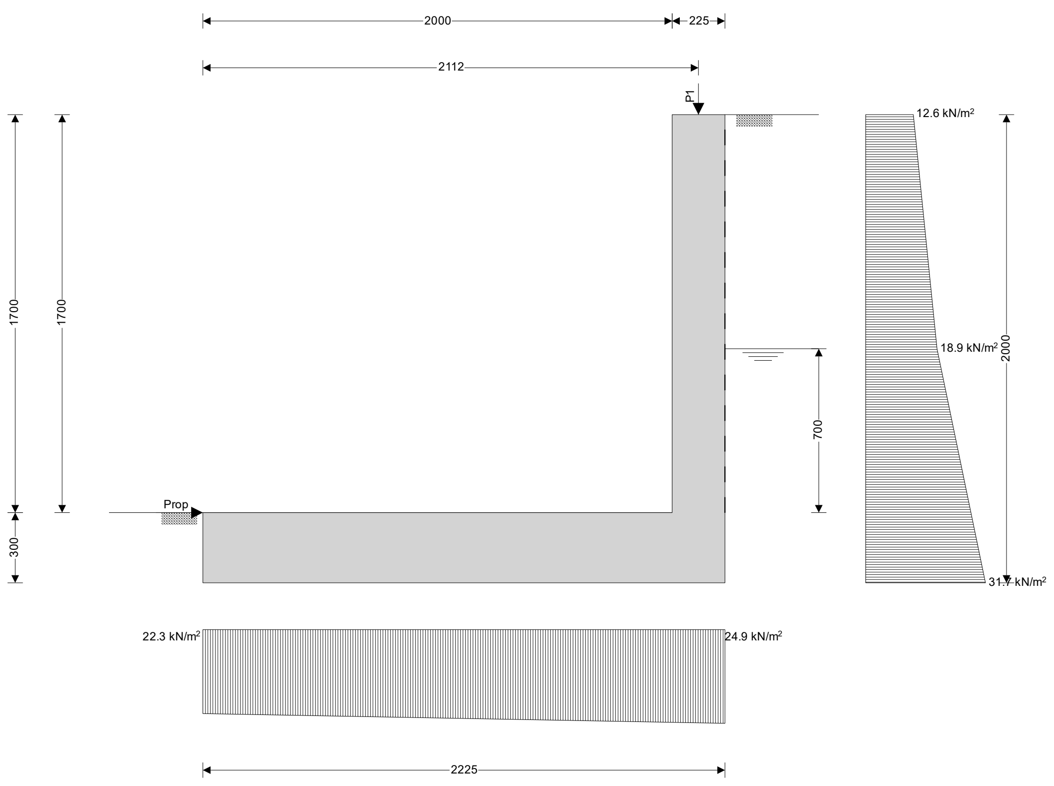
Soil type	Firm clay
Moist density	γ_{mr} = **19** kN/m^3
Saturated density	γ_{sr} = **19** kN/m^3
Characteristic effective shear resistance angle	$\phi'_{r.k}$ = **27** deg
Characteristic wall friction angle	$\delta_{r.k}$ = **13.5** deg

Base soil properties

Soil type	Firm clay
Soil density	γ_b = **19** kN/m^3
Characteristic effective shear resistance angle	$\phi'_{b.k}$ = **27** deg
Characteristic wall friction angle	$\delta_{b.k}$ = **13.5** deg
Characteristic base friction angle	$\delta_{bb.k}$ = **18** deg
Presumed bearing capacity	$P_{bearing}$ = **60** kN/m^2

Loading details

Permanent surcharge load	$Surcharge_G$ = **26.3** kN/m^2
Variable surcharge load	$Surcharge_Q$ = **11.9** kN/m^2
Vertical line load at 2112 mm	P_{G1} = **21.4** kN/m
	P_{Q1} = **4.8** kN/m



General arrangement

Calculate retaining wall geometry

Base length	$l_{base} = l_{toe} + t_{stem} =$ **2225** mm
Saturated soil height	$h_{sat} = h_{water} + d_{cover} =$ **700** mm
Moist soil height	$h_{moist} = h_{ret} - h_{water} =$ **1000** mm
Length of surcharge load	$l_{sur} = l_{heel} =$ **0** mm
- Distance to vertical component	$x_{sur\_v} = l_{base} - l_{heel} / 2 =$ **2225** mm
Effective height of wall	$h_{eff} = h_{base} + d_{cover} + h_{ret} =$ **2000** mm
- Distance to horizontal component	$x_{sur\_h} = h_{eff} / 2 =$ **1000** mm
Area of wall stem	$A_{stem} = h_{stem} \times t_{stem} =$ **0.383** m²
- Distance to vertical component	$x_{stem} = l_{toe} + t_{stem} / 2 =$ **2113** mm
Area of wall base	$A_{base} = l_{base} \times t_{base} =$ **0.668** m²
- Distance to vertical component	$x_{base} = l_{base} / 2 =$ **1113** mm

Using Coulomb theory

Active pressure coefficient	$K_A = \sin(\alpha + \phi'_{r.k})^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta_{r.k}) \times [1 + \sqrt{[\sin(\phi'_{r.k} + \delta_{r.k}) \times \sin(\phi'_{r.k} - \beta) / (\sin(\alpha - \delta_{r.k}) \times \sin(\alpha + \beta))]}]^2) =$ **0.340**
Passive pressure coefficient	$K_P = \sin(90 - \phi'_{b.k})^2 / (\sin(90 + \delta_{b.k}) \times [1 - \sqrt{[\sin(\phi'_{b.k} + \delta_{b.k}) \times \sin(\phi'_{b.k}) / (\sin(90 + \delta_{b.k}))]}]^2) =$ **4.044**

Bearing pressure check

Vertical forces on wall

Wall stem	$F_{stem} = A_{stem} \times \gamma_{stem} =$ **9.6** kN/m
Wall base	$F_{base} = A_{base} \times \gamma_{base} =$ **16.7** kN/m

Line loads $F_{P\_v} = P_{G1} + P_{Q1} = \mathbf{26.2}$ kN/m

Total $F_{total\_v} = F_{stem} + F_{base} + F_{P\_v} + F_{water\_v} = \mathbf{52.5}$ kN/m

Horizontal forces on wall

Surcharge load $F_{sur\_h} = K_A \times \cos(\delta_{r.k}) \times (Surcharge_G + Surcharge_Q) \times h_{eff} = \mathbf{25.2}$ kN/m

Saturated retained soil $F_{sat\_h} = K_A \times \cos(\delta_{r.k}) \times (\gamma_{sr} - \gamma_w) \times (h_{sat} + h_{base})^2 / 2 = \mathbf{1.5}$ kN/m

Water $F_{water\_h} = \gamma_w \times (h_{water} + d_{cover} + h_{base})^2 / 2 = \mathbf{4.9}$ kN/m

Moist retained soil $F_{moist\_h} = K_A \times \cos(\delta_{r.k}) \times \gamma_{mr} \times ((h_{eff} - h_{sat} - h_{base})^2 / 2 + (h_{eff} - h_{sat} - h_{base}) \times (h_{sat} + h_{base})) = \mathbf{9.4}$ kN/m

Base soil $F_{pass\_h} = -K_P \times \cos(\delta_{b.k}) \times \gamma_b \times (d_{cover} + h_{base})^2 / 2 = \mathbf{-3.4}$ kN/m

Total $F_{total\_h} = F_{sur\_h} + F_{sat\_h} + F_{water\_h} + F_{moist\_h} + F_{pass\_h} = \mathbf{37.7}$ kN/m

Moments on wall

Wall stem $M_{stem} = F_{stem} \times x_{stem} = \mathbf{20.2}$ kNm/m

Wall base $M_{base} = F_{base} \times x_{base} = \mathbf{18.6}$ kNm/m

Surcharge load $M_{sur} = -F_{sur\_h} \times x_{sur\_h} = \mathbf{-25.2}$ kNm/m

Line loads $M_P = (P_{G1} + P_{Q1}) \times p_1 = \mathbf{55.3}$ kNm/m

Saturated retained soil $M_{sat} = -F_{sat\_h} \times x_{sat\_h} = \mathbf{-0.5}$ kNm/m

Water $M_{water} = -F_{water\_h} \times x_{water\_h} = \mathbf{-1.6}$ kNm/m

Moist retained soil $M_{moist} = -F_{moist\_h} \times x_{moist\_h} = \mathbf{-7.3}$ kNm/m

Total $M_{total} = M_{stem} + M_{base} + M_{sur} + M_P + M_{sat} + M_{water} + M_{moist} = \mathbf{59.4}$ kNm/m

Check bearing pressure

Propping force $F_{prop\_base} = F_{total\_h} = \mathbf{37.7}$ kN/m

Distance to reaction $\bar{x} = M_{total} / F_{total\_v} = \mathbf{1133}$ mm

Eccentricity of reaction $e = \bar{x} - l_{base} / 2 = \mathbf{20}$ mm

Loaded length of base $l_{load} = l_{base} = \mathbf{2225}$ mm

Bearing pressure at toe $q_{toe} = F_{total\_v} / l_{base} \times (1 - 6 \times e / l_{base}) = \mathbf{22.3}$ kN/m^2

Bearing pressure at heel $q_{heel} = F_{total\_v} / l_{base} \times (1 + 6 \times e / l_{base}) = \mathbf{24.9}$ kN/m^2

Factor of safety $FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel}) = \mathbf{2.413}$

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 2.9.12

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

Concrete strength class C30/37

Characteristic compressive cylinder strength $f_{ck} = \mathbf{30}$ N/mm^2

Characteristic compressive cube strength $f_{ck,cube} = \mathbf{37}$ N/mm^2

Mean value of compressive cylinder strength $f_{cm} = f_{ck} + 8$ N/mm^2 $= \mathbf{38}$ N/mm^2

Mean value of axial tensile strength $f_{ctm} = 0.3$ N/mm$^2 \times (f_{ck} / 1$ N/mm$^2)^{2/3} = \mathbf{2.9}$ N/mm^2

5% fractile of axial tensile strength $f_{ctk,0.05} = 0.7 \times f_{ctm} = \mathbf{2.0}$ N/mm^2

Secant modulus of elasticity of concrete $E_{cm} = 22$ kN/mm$^2 \times (f_{cm} / 10$ N/mm$^2)^{0.3} = \mathbf{32837}$ 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 $f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C = \mathbf{17.0}$ N/mm^2

Maximum aggregate size $h_{agg} = \mathbf{20}$ mm

Ultimate strain - Table 3.1	ε_{cu2} = **0.0035**
Shortening strain - Table 3.1	ε_{cu3} = **0.0035**
Effective compression zone height factor	λ = **0.80**
Effective strength factor	η = **1.00**
Bending coefficient k_1	K_1 = **0.40**
Bending coefficient k_2	$K_2 = 1.00 \times (0.6 + 0.0014/\varepsilon_{cu2})$ = **1.00**
Bending coefficient k_3	K_3 =**0.40**
Bending coefficient k_4	$K_4 = 1.00 \times (0.6 + 0.0014/\varepsilon_{cu2})$ =**1.00**

Reinforcement details

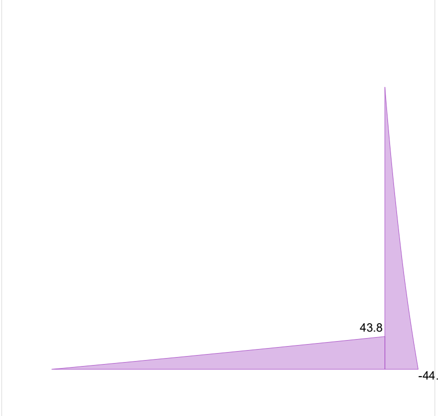
Characteristic yield strength of reinforcement	f_{yk} = **500** N/mm^2
Modulus of elasticity of reinforcement	E_s = **200000** N/mm^2
Partial factor for reinforcing steel - Table 2.1N	γ_S = **1.15**
Design yield strength of reinforcement	$f_{yd} = f_{yk} / \gamma_S$ = **435** N/mm^2

Cover to reinforcement

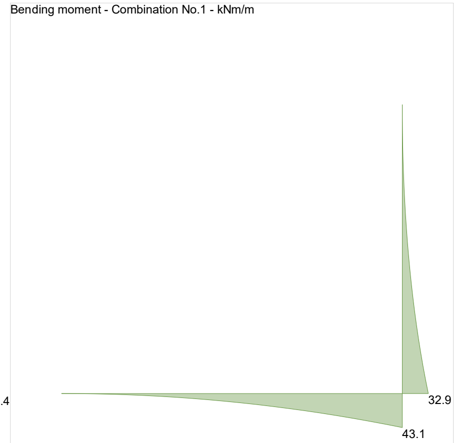
Front face of stem	c_{sf} = **40** mm
Rear face of stem	c_{sr} = **50** mm
Top face of base	c_{bt} = **50** mm
Bottom face of base	c_{bb} = **75** mm

Loading details - Combination No.1 - kN/m²

Shear force - Combination No.1 - kN/m

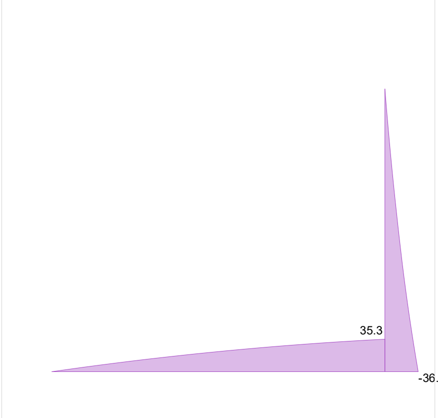


Bending moment - Combination No.1 - kNm/m

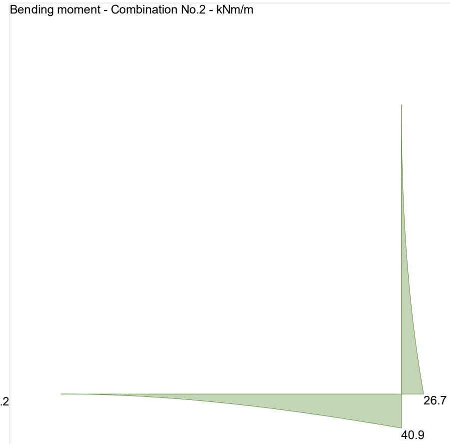


Loading details - Combination No.2 - kN/m²

Shear force - Combination No.2 - kN/m



Bending moment - Combination No.2 - kNm/m



Check stem design at base of stem

Depth of section h = **225** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 M = **32.9** kNm/m

Depth to tension reinforcement $d = h - c_{sr} - \phi_{sr} / 2$ = **167** mm

$K = M / (d^2 \times f_{ck})$ = **0.039**

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

K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d$ = **159** mm

Depth of neutral axis $x = 2.5 \times (d – z)$ = **21** mm

Area of tension reinforcement required $A_{sr.req} = M / (f_{yd} \times z)$ = **477** mm^2/m

Tension reinforcement provided 16 dia.bars @ 150 c/c

Area of tension reinforcement provided $A_{sr.prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr})$ = **1340** mm^2/m

Minimum area of reinforcement - exp.9.1N $A_{sr.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d$ = **252** mm^2/m

Maximum area of reinforcement - cl.9.2.1.1(3) $A_{sr.max} = 0.04 \times h$ = **9000** mm^2/m

$\max(A_{sr.req}, A_{sr.min}) / A_{sr.prov}$ = **0.356**

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 $\rho_0 = \sqrt{(f_{ck} / 1\ N/mm^2)} / 1000$ = **0.005**

Required tension reinforcement ratio $\rho = A_{sr.req} / d$ = **0.003**

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 = **10.2**

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 ψ_2 = **0.6**

Serviceability bending moment M_{sls} = **21.5** kNm/m

Tensile stress in reinforcement $\sigma_s = M_{sls} / (A_{sr.prov} \times z)$ = **100.9** N/mm^2

Load duration Long term

Load duration factor k_t = **0.4**

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

$A_{c.eff}$ = **68042** mm^2/m

Mean value of concrete tensile strength $f_{ct.eff} = f_{ctm}$ = **2.9** N/mm^2

Reinforcement ratio $\rho_{p.eff} = A_{sr.prov} / A_{c.eff}$ = **0.020**

Modular ratio $\alpha_e = E_s / E_{cm}$ = **6.091**

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 $s_{r.max} = k_3 \times c_{sr} + k_1 \times k_2 \times k_4 \times \phi_{sr} / \rho_{p.eff} = \mathbf{308}$ mm

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

$w_k = \mathbf{0.093}$ mm

$w_k / w_{max} = \mathbf{0.311}$

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force $V = \mathbf{44.4}$ kN/m

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

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

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

$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5} = \mathbf{0.542}$ N/mm^2

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

$V_{Rd.c} = \mathbf{115.7}$ kN/m

$V / V_{Rd.c} = \mathbf{0.383}$

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) $A_{sx.req} = \max(0.25 \times A_{sr.prov}, 0.001 \times t_{stem}) = \mathbf{335}$ mm^2/m

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

Transverse reinforcement provided 10 dia.bars @ 200 c/c

Area of transverse reinforcement provided $A_{sx.prov} = \pi \times \phi_{sx}^2 / (4 \times s_{sx}) = \mathbf{393}$ mm^2/m

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

Check base design at toe

Depth of section $h = \mathbf{300}$ mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 $M = \mathbf{43.1}$ kNm/m

Depth to tension reinforcement $d = h - c_{bb} - \phi_{bb} / 2 = \mathbf{217}$ mm

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

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

$K' = \mathbf{0.207}$

K' > K - No compression reinforcement is required

Lever arm $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d = \mathbf{206}$ mm

Depth of neutral axis $x = 2.5 \times (d - z) = \mathbf{27}$ mm

Area of tension reinforcement required $A_{bb.req} = M / (f_{yd} \times z) = \mathbf{481}$ mm^2/m

Tension reinforcement provided 16 dia.bars @ 150 c/c

Area of tension reinforcement provided $A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb}) = \mathbf{1340}$ mm^2/m

Minimum area of reinforcement - exp.9.1N $A_{bb.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = \mathbf{327}$ mm^2/m

Maximum area of reinforcement - cl.9.2.1.1(3) $A_{bb.max} = 0.04 \times h = \mathbf{12000}$ mm^2/m

$\max(A_{bb.req}, A_{bb.min}) / A_{bb.prov} = \mathbf{0.359}$

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 $M_{sls} = \mathbf{31.1}$ kNm/m

Tensile stress in reinforcement $\sigma_s = M_{sls} / (A_{bb.prov} \times z) = \mathbf{112.6}$ N/mm^2

Load duration	Long term
Load duration factor	k_t = **0.4**
Effective area of concrete in tension	$A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$
	$A_{c.eff}$ = **90958** mm^2/m
Mean value of concrete tensile strength	$f_{ct.eff} = f_{ctm}$ = **2.9** N/mm^2
Reinforcement ratio	$\rho_{p.eff} = A_{bb.prov} / A_{c.eff}$ = **0.015**
Modular ratio	$\alpha_e = E_s / E_{cm}$ = **6.091**
Bond property coefficient	k_1 = **0.8**
Strain distribution coefficient	k_2 = **0.5**
	k_3 = **3.4**
	k_4 = **0.425**
Maximum crack spacing - exp.7.11	$s_{r.max} = k_3 \times c_{bb} + k_1 \times k_2 \times k_4 \times \phi_{bb} / \rho_{p.eff}$ = **440** mm
Maximum crack width - exp.7.8	$w_k = s_{r.max} \times \max(\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$
	w_k = **0.148** mm
	w_k / w_{max} = **0.495**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

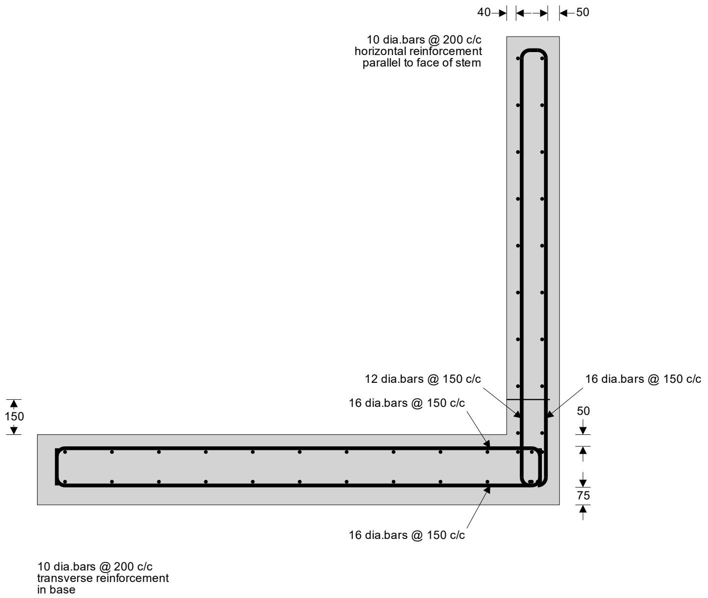
Design shear force	V = **43.8** kN/m
	$C_{Rd,c} = 0.18 / \gamma_C$ = **0.120**
	$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2)$ = **1.960**
Longitudinal reinforcement ratio	$\rho_l = \min(A_{bb.prov} / d, 0.02)$ = **0.006**
	$v_{min} = 0.035\ N^{1/2}/mm \times k^{3/2} \times f_{ck}^{0.5}$ = **0.526** N/mm^2
Design shear resistance - exp.6.2a & 6.2b	$V_{Rd.c} = \max(C_{Rd.c} \times k \times (100\ N^2/mm^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$
	$V_{Rd.c}$ = **135.1** kN/m
	$V / V_{Rd.c}$ = **0.324**

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)	$A_{bx.req} = 0.2 \times A_{bb.prov}$ = **268** mm^2/m
Maximum spacing of reinforcement – cl.9.3.1.1(3)	s_{bx_max} = **450** mm
Transverse reinforcement provided	10 dia.bars @ 200 c/c
Area of transverse reinforcement provided	$A_{bx.prov} = \pi \times \phi_{bx}^2 / (4 \times s_{bx})$ = **393** mm^2/m

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



Reinforcement details

RETAINING WALL ANALYSIS

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

Tedds calculation version 2.9.12

Retaining wall details

Stem type	Cantilever
Stem height	h_{stem} = **1030** mm
Stem thickness	t_{stem} = **330** mm
Angle to rear face of stem	α = **90** deg
Stem density	γ_{stem} = **25** kN/m^3
Toe length	l_{toe} = **1000** mm
Base thickness	t_{base} = **250** mm
Base density	γ_{base} = **25** kN/m^3
Height of retained soil	h_{ret} = **1030** mm
Angle of soil surface	β = **0** deg
Depth of cover	d_{cover} = **0** mm
Height of water	h_{water} = **100** mm
Water density	γ_w = **9.8** kN/m^3

Retained soil properties

Soil type	Firm clay
Moist density	γ_{mr} = **19** kN/m^3
Saturated density	γ_{sr} = **19** kN/m^3
Characteristic effective shear resistance angle	$\phi'_{r.k}$ = **27** deg
Characteristic wall friction angle	$\delta_{r.k}$ = **13.5** deg

Base soil properties

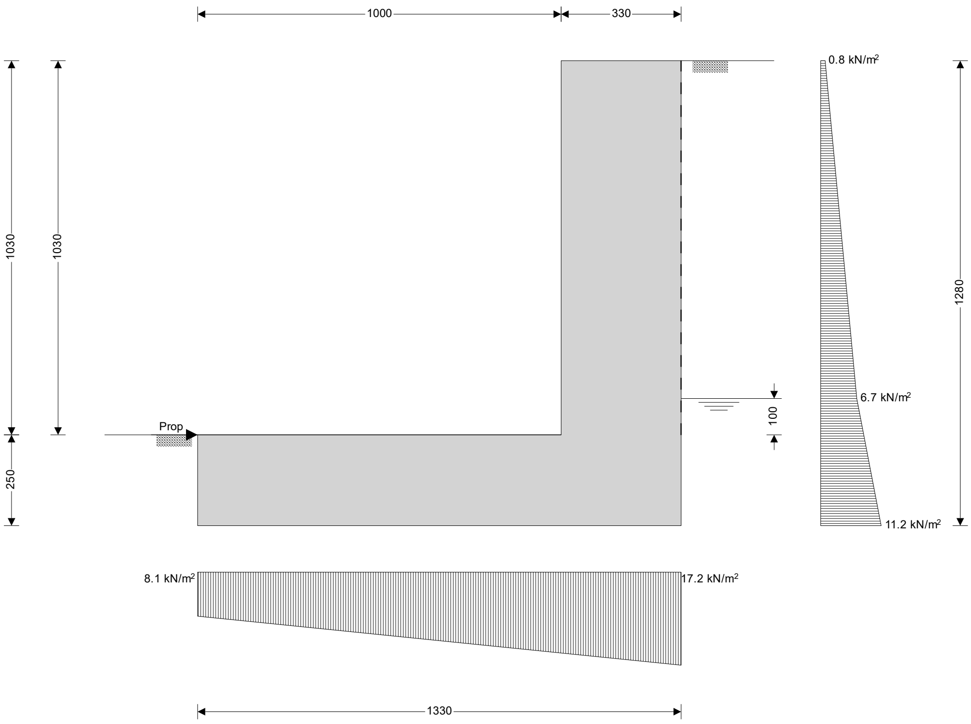
Soil type	Firm clay
Soil density	γ_b = **19** kN/m^3
Characteristic effective shear resistance angle	$\phi'_{b.k}$ = **27** deg
Characteristic wall friction angle	$\delta_{b.k}$ = **13.5** deg
Characteristic base friction angle	$\delta_{bb.k}$ = **18** deg
Presumed bearing capacity	$P_{bearing}$ = **60** kN/m^2

Loading details

Variable surcharge load	$Surcharge_Q$ = **2.5** kN/m^2

Note:

The retaining wall has been designed to support lateral pressures only.
Assuming the slab (retaining wall base) has sufficient stiffness, this will transfer the vertical load to a mass concrete strip foundation uniformly.
Refer to "Addendum to structural calculation package" for strip foundation and load bearing pressure check.



General arrangement

Calculate retaining wall geometry

Base length $l_{base} = l_{toe} + t_{stem} =$ **1330** mm

Saturated soil height $h_{sat} = h_{water} + d_{cover} =$ **100** mm

Moist soil height $h_{moist} = h_{ret} - h_{water} =$ **930** mm

Length of surcharge load $l_{sur} = l_{heel} =$ **0** mm

- Distance to vertical component $x_{sur\_v} = l_{base} - l_{heel} / 2 =$ **1330** mm

Effective height of wall $h_{eff} = h_{base} + d_{cover} + h_{ret} =$ **1280** mm

- Distance to horizontal component $x_{sur\_h} = h_{eff} / 2 =$ **640** mm

Area of wall stem $A_{stem} = h_{stem} \times t_{stem} =$ **0.34** m^2

- Distance to vertical component $x_{stem} = l_{toe} + t_{stem} / 2 =$ **1165** mm

Area of wall base $A_{base} = l_{base} \times t_{base} =$ **0.333** m^2

- Distance to vertical component $x_{base} = l_{base} / 2 =$ **665** mm

Using Coulomb theory

Active pressure coefficient $K_A = \sin(\alpha + \phi'_{r.k})^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta_{r.k}) \times [1 + \sqrt{[\sin(\phi'_{r.k} + \delta_{r.k}) \times \sin(\phi'_{r.k} - \beta) / (\sin(\alpha - \delta_{r.k}) \times \sin(\alpha + \beta))]}]^2) =$ **0.340**

Passive pressure coefficient $K_P = \sin(90 - \phi'_{b.k})^2 / (\sin(90 + \delta_{b.k}) \times [1 - \sqrt{[\sin(\phi'_{b.k} + \delta_{b.k}) \times \sin(\phi'_{b.k}) / (\sin(90 + \delta_{b.k}))]}]^2) =$ **4.044**

Bearing pressure check

Vertical forces on wall

Wall stem $F_{stem} = A_{stem} \times \gamma_{stem} =$ **8.5** kN/m

Wall base $F_{base} = A_{base} \times \gamma_{base} =$ **8.3** kN/m

Total | $F_{total\_v} = F_{stem} + F_{base} + F_{water\_v} =$ **16.8** kN/m

Horizontal forces on wall

Surcharge load | $F_{sur\_h} = K_A \times \cos(\delta_{r.k}) \times Surcharge_Q \times h_{eff} =$ **1.1** kN/m

Saturated retained soil | $F_{sat\_h} = K_A \times \cos(\delta_{r.k}) \times (\gamma_{sr} - \gamma_w) \times (h_{sat} + h_{base})^2 / 2 =$ **0.2** kN/m

Water | $F_{water\_h} = \gamma_w \times (h_{water} + d_{cover} + h_{base})^2 / 2 =$ **0.6** kN/m

Moist retained soil | $F_{moist\_h} = K_A \times \cos(\delta_{r.k}) \times \gamma_{mr} \times ((h_{eff} - h_{sat} - h_{base})^2 / 2 + (h_{eff} - h_{sat} - h_{base}) \times (h_{sat} + h_{base})) =$ **4.8** kN/m

Base soil | $F_{pass\_h} = -K_P \times \cos(\delta_{b.k}) \times \gamma_b \times (d_{cover} + h_{base})^2 / 2 =$ **-2.3** kN/m

Total | $F_{total\_h} = F_{sur\_h} + F_{sat\_h} + F_{water\_h} + F_{moist\_h} + F_{pass\_h} =$ **4.3** kN/m

Moments on wall

Wall stem | $M_{stem} = F_{stem} \times x_{stem} =$ **9.9** kNm/m

Wall base | $M_{base} = F_{base} \times x_{base} =$ **5.5** kNm/m

Surcharge load | $M_{sur} = -F_{sur\_h} \times x_{sur\_h} =$ **-0.7** kNm/m

Saturated retained soil | $M_{sat} = -F_{sat\_h} \times x_{sat\_h} =$ **0** kNm/m

Water | $M_{water} = -F_{water\_h} \times x_{water\_h} =$ **-0.1** kNm/m

Moist retained soil | $M_{moist} = -F_{moist\_h} \times x_{moist\_h} =$ **-2.1** kNm/m

Total | $M_{total} = M_{stem} + M_{base} + M_{sur} + M_{sat} + M_{water} + M_{moist} =$ **12.5** kNm/m

Check bearing pressure

Propping force | $F_{prop\_base} = F_{total\_h} =$ **4.3** kN/m

Distance to reaction | $\bar{x} = M_{total} / F_{total\_v} =$ **744** mm

Eccentricity of reaction | $e = \bar{x} - l_{base} / 2 =$ **79** mm

Loaded length of base | $l_{load} = l_{base} =$ **1330** mm

Bearing pressure at toe | $q_{toe} = F_{total\_v} / l_{base} \times (1 - 6 \times e / l_{base}) =$ **8.1** kN/m²

Bearing pressure at heel | $q_{heel} = F_{total\_v} / l_{base} \times (1 + 6 \times e / l_{base}) =$ **17.2** kN/m²

Factor of safety | $FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel}) =$ **3.496**

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 2.9.12

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

Concrete strength class | C30/37

Characteristic compressive cylinder strength | $f_{ck} =$ **30** N/mm²

Characteristic compressive cube strength | $f_{ck,cube} =$ **37** N/mm²

Mean value of compressive cylinder strength | $f_{cm} = f_{ck} + 8$ N/mm² = **38** N/mm²

Mean value of axial tensile strength | $f_{ctm} = 0.3$ N/mm² $\times (f_{ck} / 1$ N/mm²$)^{2/3} =$ **2.9** N/mm²

5% fractile of axial tensile strength | $f_{ctk,0.05} = 0.7 \times f_{ctm} =$ **2.0** N/mm²

Secant modulus of elasticity of concrete | $E_{cm} = 22$ kN/mm² $\times (f_{cm} / 10$ N/mm²$)^{0.3} =$ **32837** N/mm²

Partial factor for concrete - Table 2.1N | $\gamma_C =$ **1.50**

Compressive strength coefficient - cl.3.1.6(1) | $\alpha_{cc} =$ **0.85**

Design compressive concrete strength - exp.3.15 | $f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C =$ **17.0** N/mm²

Maximum aggregate size | $h_{agg} =$ **20** mm

Ultimate strain - Table 3.1 | $\varepsilon_{cu2} =$ **0.0035**

Shortening strain - Table 3.1 | $\varepsilon_{cu3} =$ **0.0035**

Effective compression zone height factor λ = **0.80**

Effective strength factor η = **1.00**

Bending coefficient k_1 K_1 = **0.40**

Bending coefficient k_2 $K_2 = 1.00 \times (0.6 + 0.0014/\varepsilon_{cu2})$ = **1.00**

Bending coefficient k_3 K_3 =**0.40**

Bending coefficient k_4 $K_4 = 1.00 \times (0.6 + 0.0014/\varepsilon_{cu2})$ =**1.00**

Reinforcement details

Characteristic yield strength of reinforcement f_{yk} = **500** N/mm^2

Modulus of elasticity of reinforcement E_s = **200000** N/mm^2

Partial factor for reinforcing steel - Table 2.1N γ_S = **1.15**

Design yield strength of reinforcement $f_{yd} = f_{yk} / \gamma_S$ = **435** N/mm^2

Cover to reinforcement

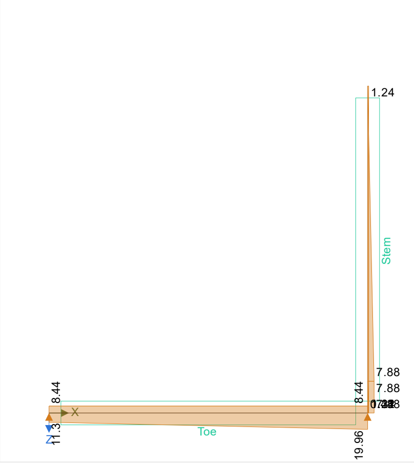
Front face of stem c_{sf} = **40** mm

Rear face of stem c_{sr} = **50** mm

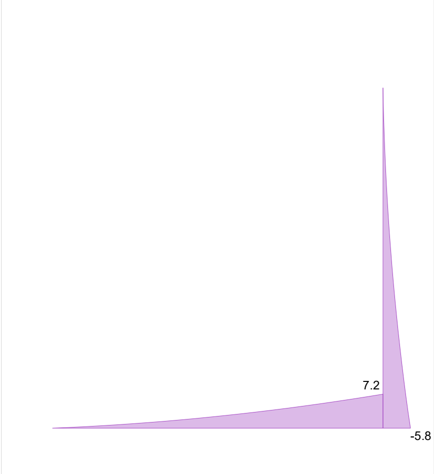
Top face of base c_{bt} = **50** mm

Bottom face of base c_{bb} = **75** mm

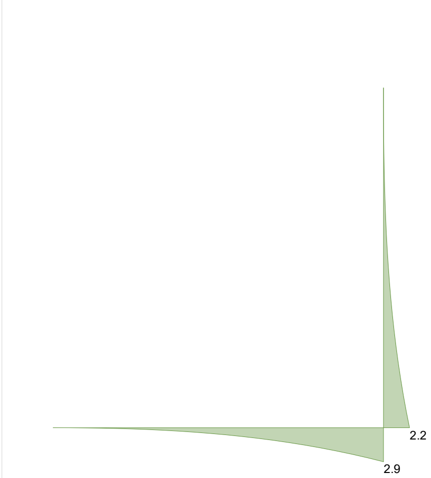
Loading details - Combination No.1 - kN/m²



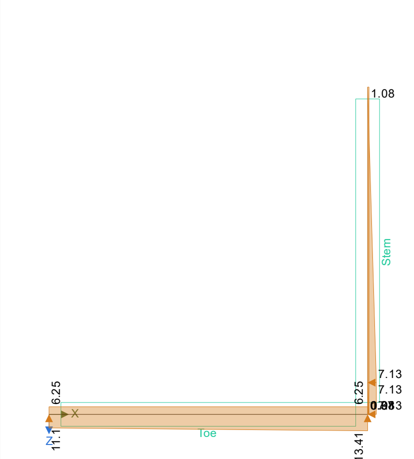
Shear force - Combination No.1 - kN/m



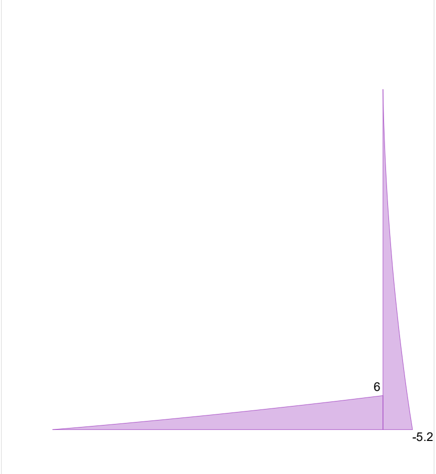
Bending moment - Combination No.1 - kNm/m



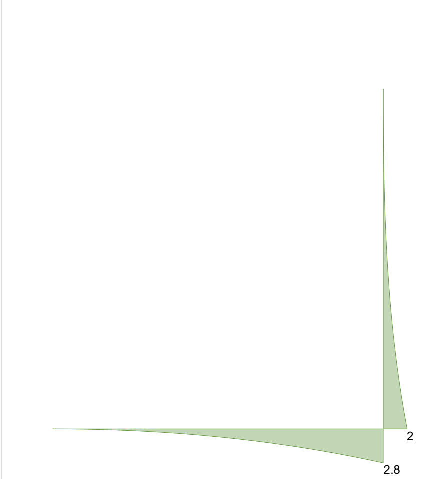
Loading details - Combination No.2 - kN/m²



Shear force - Combination No.2 - kN/m



Bending moment - Combination No.2 - kNm/m



Check stem design at base of stem

Depth of section — h = **330** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 — M = **2.2** kNm/m

Depth to tension reinforcement — $d = h - c_{sr} - \phi_{sr} / 2 =$ **272** mm

$K = M / (d^2 \times f_{ck}) =$ **0.001**

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

$K' =$ **0.207**

K' > K - No compression reinforcement is required

Lever arm — $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d =$ **258** mm

Depth of neutral axis — $x = 2.5 \times (d – z) =$ **34** mm

Area of tension reinforcement required — $A_{sr.req} = M / (f_{yd} \times z) =$ **20** mm²/m

Tension reinforcement provided — 16 dia.bars @ 150 c/c

Area of tension reinforcement provided — $A_{sr.prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr}) =$ **1340** mm²/m

Minimum area of reinforcement - exp.9.1N — $A_{sr.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d =$ **410** mm²/m

Maximum area of reinforcement - cl.9.2.1.1(3) — $A_{sr.max} = 0.04 \times h =$ **13200** mm²/m

$\max(A_{sr.req}, A_{sr.min}) / A_{sr.prov} =$ **0.306**

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 — $\rho_0 = \sqrt{}(f_{ck} / 1\ N/mm^2) / 1000 =$ **0.005**

Required tension reinforcement ratio — $\rho = A_{sr.req} / d =$ **0.000**

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 — $h_{stem} / d =$ **3.8**

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 — $M_{sls} =$ **1.4** kNm/m

Tensile stress in reinforcement — $\sigma_s = M_{sls} / (A_{sr.prov} \times z) =$ **4.1** N/mm²

Load duration — Long term

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

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

$A_{c.eff} =$ **98667** mm²/m

Mean value of concrete tensile strength — $f_{ct.eff} = f_{ctm} =$ **2.9** N/mm²

Reinforcement ratio — $\rho_{p.eff} = A_{sr.prov} / A_{c.eff} =$ **0.014**

Modular ratio — $\alpha_e = E_s / E_{cm} =$ **6.091**

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 $s_{r.max} = k_3 \times c_{sr} + k_1 \times k_2 \times k_4 \times \phi_{sr} / \rho_{p.eff}$ = **370** 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 = **0.005** mm

w_k / w_{max} = **0.015**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force V = **5.8** kN/m

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

$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2)$ = **1.857**

Longitudinal reinforcement ratio $\rho_l = \min(A_{sr.prov} / d, 0.02)$ = **0.005**

$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5}$ = **0.485** N/mm^2

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

$V_{Rd.c}$ = **148.8** kN/m

$V / V_{Rd.c}$ = **0.039**

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) $A_{sx.req} = \max(0.25 \times A_{sr.prov}, 0.001 \times t_{stem})$ = **335** mm^2/m

Maximum spacing of reinforcement – cl.9.6.3(2) s_{sx_max} = **400** mm

Transverse reinforcement provided 10 dia.bars @ 200 c/c

Area of transverse reinforcement provided $A_{sx.prov} = \pi \times \phi_{sx}^2 / (4 \times s_{sx})$ = **393** mm^2/m

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

Check base design at toe

Depth of section h = **250** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 M = **2.9** kNm/m

Depth to tension reinforcement $d = h - c_{bb} - \phi_{bb} / 2$ = **167** mm

$K = M / (d^2 \times f_{ck})$ = **0.003**

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

K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d$ = **159** mm

Depth of neutral axis $x = 2.5 \times (d – z)$ = **21** mm

Area of tension reinforcement required $A_{bb.req} = M / (f_{yd} \times z)$ = **42** mm^2/m

Tension reinforcement provided 16 dia.bars @ 150 c/c

Area of tension reinforcement provided $A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb})$ = **1340** mm^2/m

Minimum area of reinforcement - exp.9.1N $A_{bb.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d$ = **252** mm^2/m

Maximum area of reinforcement - cl.9.2.1.1(3) $A_{bb.max} = 0.04 \times h$ = **10000** mm^2/m

$\max(A_{bb.req}, A_{bb.min}) / A_{bb.prov}$ = **0.188**

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} = **2.1** kNm/m

Tensile stress in reinforcement $\sigma_s = M_{sls} / (A_{bb.prov} \times z)$ = **9.7** 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 × (h - d), (h - x) / 3, h / 2)

$A_{c.eff}$ = **76375** mm²/m

Mean value of concrete tensile strength — $f_{ct.eff} = f_{ctm}$ = **2.9** N/mm²

Reinforcement ratio — $\rho_{p.eff} = A_{bb.prov} / A_{c.eff}$ = **0.018**

Modular ratio — $\alpha_e = E_s / E_{cm}$ = **6.091**

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 — $s_{r.max} = k_3 \times c_{bb} + k_1 \times k_2 \times k_4 \times \phi_{bb} / \rho_{p.eff}$ = **410** 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 = **0.012** mm

w_k / w_{max} = **0.04**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force — V = **7.2** kN/m

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

k = min(1 + √(200 mm / d), 2) = **2.000**

Longitudinal reinforcement ratio — $\rho_l = \min(A_{bb.prov} / d, 0.02)$ = **0.008**

$v_{min} = 0.035\ N^{1/2}/mm \times k^{3/2} \times f_{ck}^{0.5}$ = **0.542** N/mm²

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

$V_{Rd.c}$ = **115.7** kN/m

$V / V_{Rd.c}$ = **0.062**

PASS - Design shear resistance exceeds design shear force

Secondary transverse reinforcement to base - Section 9.3

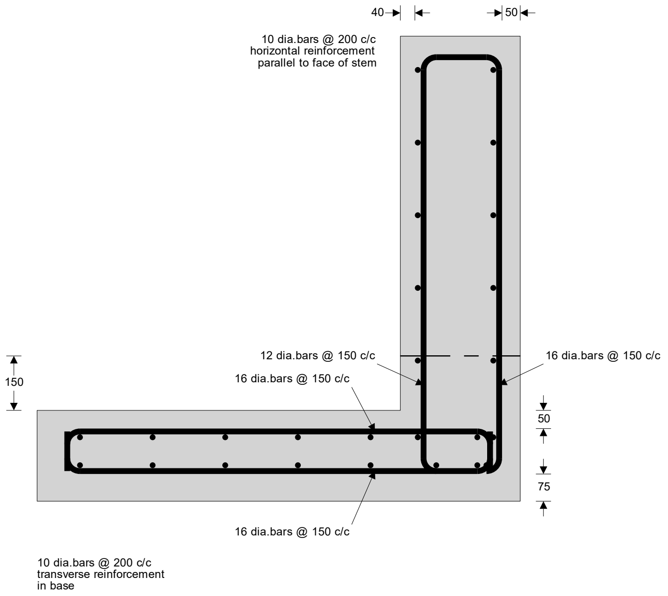
Minimum area of reinforcement – cl.9.3.1.1(2) — $A_{bx.req} = 0.2 \times A_{bb.prov}$ = **268** mm²/m

Maximum spacing of reinforcement – cl.9.3.1.1(3) — s_{bx_max} = **450** mm

Transverse reinforcement provided — 10 dia.bars @ 200 c/c

Area of transverse reinforcement provided — $A_{bx.prov} = \pi \times \phi_{bx}^2 / (4 \times s_{bx})$ = **393** mm²/m

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



Reinforcement details

RETAINING WALL ANALYSIS

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

Tedds calculation version 2.9.12

Retaining wall details

Stem type	Cantilever
Stem height	h_{stem} = **2700** mm
Stem thickness	t_{stem} = **330** mm
Angle to rear face of stem	α = **90** deg
Stem density	γ_{stem} = **25** kN/m^3
Toe length	l_{toe} = **2000** mm
Base thickness	t_{base} = **250** mm
Base density	γ_{base} = **25** kN/m^3
Height of retained soil	h_{ret} = **2700** mm
Angle of soil surface	β = **0** deg
Depth of cover	d_{cover} = **0** mm
Height of water	h_{water} = **1700** mm
Water density	γ_w = **9.8** kN/m^3

Retained soil properties

Soil type	Firm clay
Moist density	γ_{mr} = **19** kN/m^3
Saturated density	γ_{sr} = **19** kN/m^3
Characteristic effective shear resistance angle	$\phi'_{r.k}$ = **27** deg
Characteristic wall friction angle	$\delta_{r.k}$ = **13.5** deg

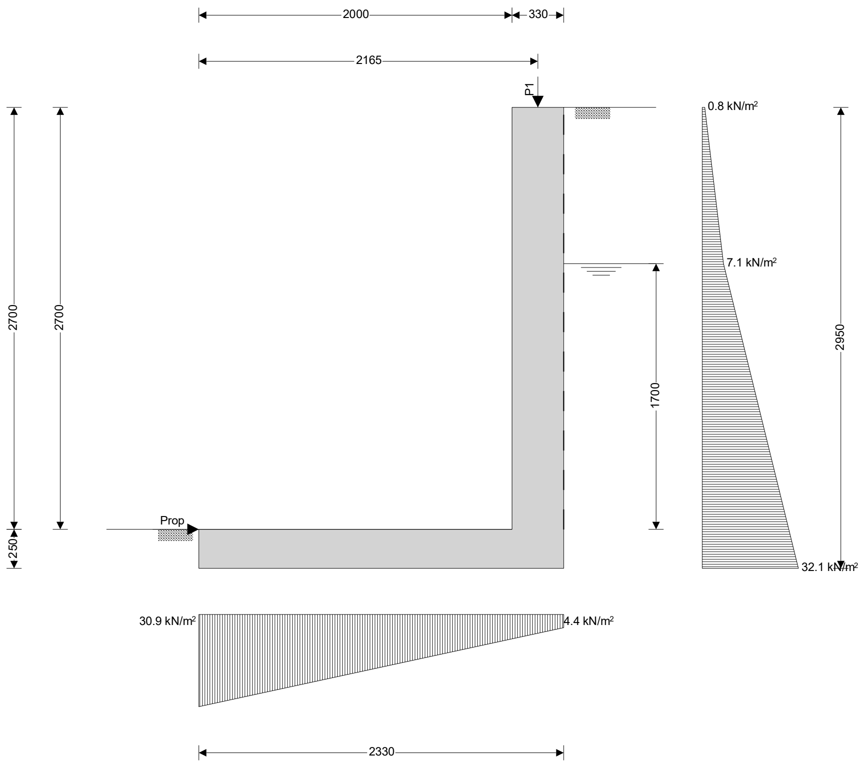
Base soil properties

Soil type	Firm clay
Soil density	γ_b = **19** kN/m^3
Characteristic effective shear resistance angle	$\phi'_{b.k}$ = **27** deg
Characteristic wall friction angle	$\delta_{b.k}$ = **13.5** deg
Characteristic base friction angle	$\delta_{bb.k}$ = **18** deg
Presumed bearing capacity	$P_{bearing}$ = **60** kN/m^2

Loading details

Variable surcharge load	$Surcharge_Q$ = **2.5** kN/m^2
Vertical line load at 2165 mm	P_{G1} = **1.2** kN/m
	P_{Q1} = **3.1** kN/m

Symmetrys Unit 6 The Courtyard, Lynton Road London N8 8SL	Project 43a REDINGTON ROAD, LONDON NW3 7RA				Job no. 21141	
	Calcs for B-RW05				Start page no./Revision 2 B	
	Calcs by SB	Calcs date 14/01/2022	Checked by DS	Checked date 14/01/2022	Approved by DS	Approved date 14/01/2022



General arrangement

Calculate retaining wall geometry

Base length $l_{base} = l_{toe} + t_{stem} =$ **2330** mm

Saturated soil height $h_{sat} = h_{water} + d_{cover} =$ **1700** mm

Moist soil height $h_{moist} = h_{ret} - h_{water} =$ **1000** mm

Length of surcharge load $l_{sur} = l_{heel} =$ **0** mm

 - Distance to vertical component $x_{sur\_v} = l_{base} - l_{heel} / 2 =$ **2330** mm

Effective height of wall $h_{eff} = h_{base} + d_{cover} + h_{ret} =$ **2950** mm

 - Distance to horizontal component $x_{sur\_h} = h_{eff} / 2 =$ **1475** mm

Area of wall stem $A_{stem} = h_{stem} \times t_{stem} =$ **0.891** m^2

 - Distance to vertical component $x_{stem} = l_{toe} + t_{stem} / 2 =$ **2165** mm

Area of wall base $A_{base} = l_{base} \times t_{base} =$ **0.583** m^2

 - Distance to vertical component $x_{base} = l_{base} / 2 =$ **1165** mm

Using Coulomb theory

Active pressure coefficient $K_A = \sin(\alpha + \phi'_{r.k})^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta_{r.k}) \times [1 + \sqrt{[\sin(\phi'_{r.k} + \delta_{r.k}) \times \sin(\phi'_{r.k} - \beta) / (\sin(\alpha - \delta_{r.k}) \times \sin(\alpha + \beta))]}]^2) =$ **0.340**

Passive pressure coefficient $K_P = \sin(90 - \phi'_{b.k})^2 / (\sin(90 + \delta_{b.k}) \times [1 - \sqrt{[\sin(\phi'_{b.k} + \delta_{b.k}) \times \sin(\phi'_{b.k}) / (\sin(90 + \delta_{b.k}))]}]^2) =$ **4.044**

Bearing pressure check

Vertical forces on wall

Wall stem $F_{stem} = A_{stem} \times \gamma_{stem} =$ **22.3** kN/m

Wall base $F_{base} = A_{base} \times \gamma_{base} =$ **14.6** kN/m

Line loads	$F_{P\_v} = P_{G1} + P_{Q1} = \mathbf{4.3}$ kN/m
Total	$F_{total\_v} = F_{stem} + F_{base} + F_{P\_v} + F_{water\_v} = \mathbf{41.1}$ kN/m

Horizontal forces on wall

Surcharge load	$F_{sur\_h} = K_A \times \cos(\delta_{r.k}) \times Surcharge_Q \times h_{eff} = \mathbf{2.4}$ kN/m
Saturated retained soil	$F_{sat\_h} = K_A \times \cos(\delta_{r.k}) \times (\gamma_{sr} - \gamma_w) \times (h_{sat} + h_{base})^2 / 2 = \mathbf{5.8}$ kN/m
Water	$F_{water\_h} = \gamma_w \times (h_{water} + d_{cover} + h_{base})^2 / 2 = \mathbf{18.7}$ kN/m
Moist retained soil	$F_{moist\_h} = K_A \times \cos(\delta_{r.k}) \times \gamma_{mr} \times ((h_{eff} - h_{sat} - h_{base})^2 / 2 + (h_{eff} - h_{sat} - h_{base}) \times (h_{sat} + h_{base})) = \mathbf{15.4}$ kN/m
Base soil	$F_{pass\_h} = -K_P \times \cos(\delta_{b.k}) \times \gamma_b \times (d_{cover} + h_{base})^2 / 2 = \mathbf{-2.3}$ kN/m
Total	$F_{total\_h} = F_{sur\_h} + F_{sat\_h} + F_{water\_h} + F_{moist\_h} + F_{pass\_h} = \mathbf{39.9}$ kN/m

Moments on wall

Wall stem	$M_{stem} = F_{stem} \times x_{stem} = \mathbf{48.2}$ kNm/m
Wall base	$M_{base} = F_{base} \times x_{base} = \mathbf{17}$ kNm/m
Surcharge load	$M_{sur} = -F_{sur\_h} \times x_{sur\_h} = \mathbf{-3.6}$ kNm/m
Line loads	$M_P = (P_{G1} + P_{Q1}) \times p_1 = \mathbf{9.3}$ kNm/m
Saturated retained soil	$M_{sat} = -F_{sat\_h} \times x_{sat\_h} = \mathbf{-3.7}$ kNm/m
Water	$M_{water} = -F_{water\_h} \times x_{water\_h} = \mathbf{-12.1}$ kNm/m
Moist retained soil	$M_{moist} = -F_{moist\_h} \times x_{moist\_h} = \mathbf{-19.1}$ kNm/m
Total	$M_{total} = M_{stem} + M_{base} + M_{sur} + M_P + M_{sat} + M_{water} + M_{moist} = \mathbf{35.9}$ kNm/m

Check bearing pressure

Propping force	$F_{prop\_base} = F_{total\_h} = \mathbf{39.9}$ kN/m
Distance to reaction	$\bar{x} = M_{total} / F_{total\_v} = \mathbf{874}$ mm
Eccentricity of reaction	$e = \bar{x} - l_{base} / 2 = \mathbf{-291}$ mm
Loaded length of base	$l_{load} = l_{base} = \mathbf{2330}$ mm
Bearing pressure at toe	$q_{toe} = F_{total\_v} / l_{base} \times (1 - 6 \times e / l_{base}) = \mathbf{30.9}$ kN/m^2
Bearing pressure at heel	$q_{heel} = F_{total\_v} / l_{base} \times (1 + 6 \times e / l_{base}) = \mathbf{4.4}$ kN/m^2
Factor of safety	$FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel}) = \mathbf{1.942}$

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 2.9.12

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

Concrete strength class	C30/37
Characteristic compressive cylinder strength	$f_{ck} = \mathbf{30}$ N/mm^2
Characteristic compressive cube strength	$f_{ck,cube} = \mathbf{37}$ N/mm^2
Mean value of compressive cylinder strength	$f_{cm} = f_{ck} + 8$ N/mm^2 $= \mathbf{38}$ N/mm^2
Mean value of axial tensile strength	$f_{ctm} = 0.3$ N/mm^2 $\times (f_{ck} / 1$ N/mm$^2)^{2/3} = \mathbf{2.9}$ N/mm^2
5% fractile of axial tensile strength	$f_{ctk,0.05} = 0.7 \times f_{ctm} = \mathbf{2.0}$ N/mm^2
Secant modulus of elasticity of concrete	$E_{cm} = 22$ kN/mm^2 $\times (f_{cm} / 10$ N/mm$^2)^{0.3} = \mathbf{32837}$ 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	$f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C = \mathbf{17.0}$ N/mm^2
Maximum aggregate size	$h_{agg} = \mathbf{20}$ mm

Ultimate strain - Table 3.1	ε_{cu2} = **0.0035**
Shortening strain - Table 3.1	ε_{cu3} = **0.0035**
Effective compression zone height factor	λ = **0.80**
Effective strength factor	η = **1.00**
Bending coefficient k_1	K_1 = **0.40**
Bending coefficient k_2	$K_2 = 1.00 \times (0.6 + 0.0014/\varepsilon_{cu2})$ = **1.00**
Bending coefficient k_3	K_3 =**0.40**
Bending coefficient k_4	$K_4 = 1.00 \times (0.6 + 0.0014/\varepsilon_{cu2})$ =**1.00**

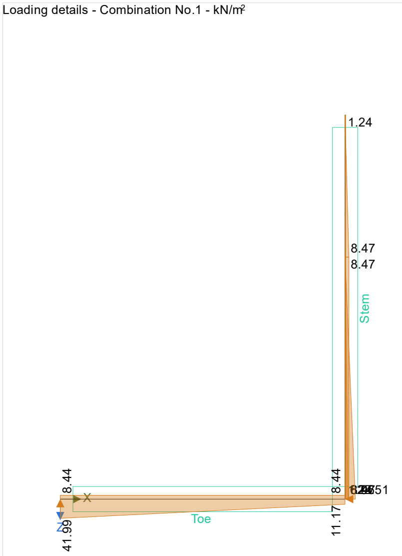
Reinforcement details

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

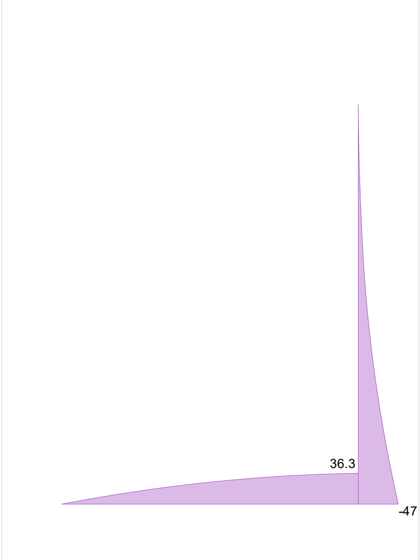
Cover to reinforcement

Front face of stem	c_{sf} = **40** mm
Rear face of stem	c_{sr} = **50** mm
Top face of base	c_{bt} = **50** mm
Bottom face of base	c_{bb} = **75** mm

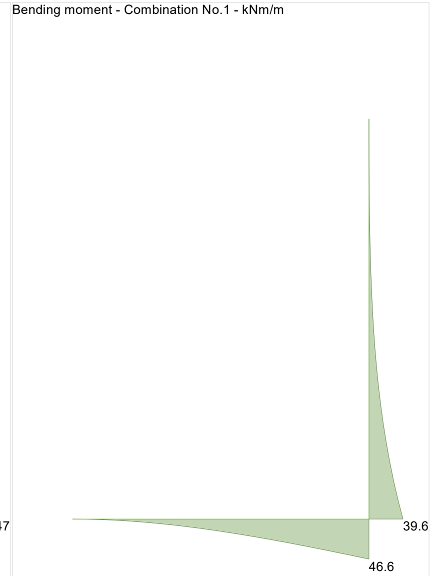
Loading details - Combination No.1 - kN/m²

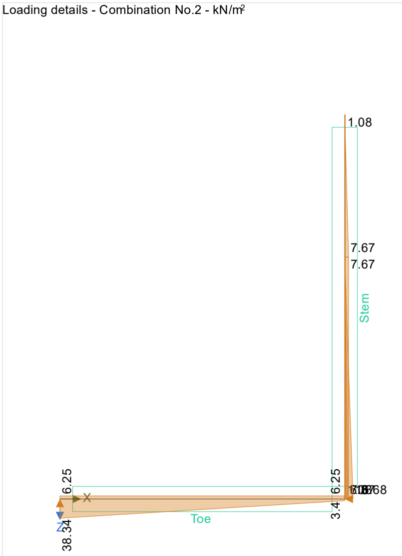


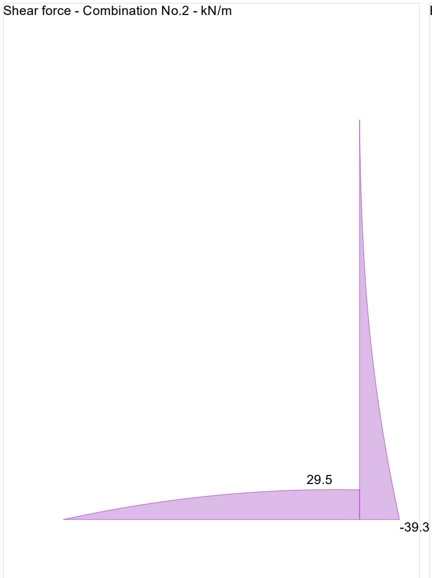
Shear force - Combination No.1 - kN/m

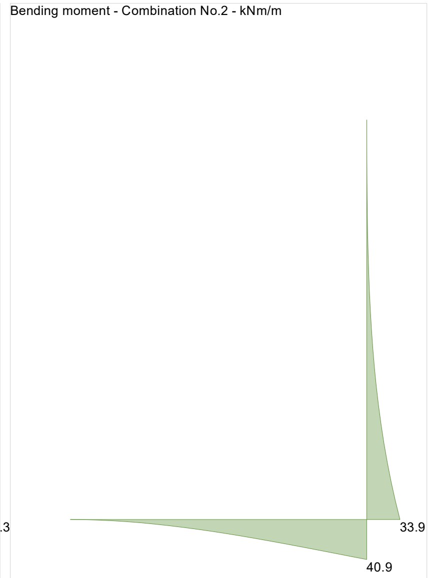


Bending moment - Combination No.1 - kNm/m









Check stem design at base of stem

Depth of section $h = \mathbf{330}$ mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 $M = \mathbf{39.6}$ kNm/m

Depth to tension reinforcement $d = h - c_{sr} - \phi_{sr} / 2 = \mathbf{272}$ mm

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

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

$K' = \mathbf{0.207}$

K' > K - No compression reinforcement is required

Lever arm $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d = \mathbf{258}$ mm

Depth of neutral axis $x = 2.5 \times (d – z) = \mathbf{34}$ mm

Area of tension reinforcement required $A_{sr.req} = M / (f_{yd} \times z) = \mathbf{352}$ mm²/m

Tension reinforcement provided 16 dia.bars @ 150 c/c

Area of tension reinforcement provided $A_{sr.prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr}) = \mathbf{1340}$ mm²/m

Minimum area of reinforcement - exp.9.1N $A_{sr.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = \mathbf{410}$ mm²/m

Maximum area of reinforcement - cl.9.2.1.1(3) $A_{sr.max} = 0.04 \times h = \mathbf{13200}$ mm²/m

$\max(A_{sr.req}, A_{sr.min}) / A_{sr.prov} = \mathbf{0.306}$

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

Library item: Rectangular single output

Deflection control - Section 7.4

Reference reinforcement ratio $\rho_0 = \sqrt{}(f_{ck} / 1\ \text{N/mm}^2) / 1000 = \mathbf{0.005}$

Required tension reinforcement ratio $\rho = A_{sr.req} / d = \mathbf{0.001}$

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 $K_s = \min(500\ \text{N/mm}^2 / (f_{yk} \times A_{sr.req} / A_{sr.prov}), 1.5) = \mathbf{1.5}$

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

Actual span to depth ratio $h_{stem} / d = \mathbf{9.9}$

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 — ψ_2 = **0.6**

Serviceability bending moment — M_{sls} = **27.8** kNm/m

Tensile stress in reinforcement — $\sigma_s = M_{sls} / (A_{sr.prov} \times z)$ = **80.2** N/mm²

Load duration — Long term

Load duration factor — k_t = **0.4**

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

$A_{c.eff}$ = **98667** mm²/m

Mean value of concrete tensile strength — $f_{ct.eff} = f_{ctm}$ = **2.9** N/mm²

Reinforcement ratio — $\rho_{p.eff} = A_{sr.prov} / A_{c.eff}$ = **0.014**

Modular ratio — $\alpha_e = E_s / E_{cm}$ = **6.091**

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 — $s_{r.max} = k_3 \times c_{sr} + k_1 \times k_2 \times k_4 \times \phi_{sr} / \rho_{p.eff}$ = **370** 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 = **0.089** mm

w_k / w_{max} = **0.297**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force — V = **47** kN/m

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

$k = \min(1 + \sqrt{(200\ mm / d)}, 2)$ = **1.857**

Longitudinal reinforcement ratio — $\rho_l = \min(A_{sr.prov} / d, 0.02)$ = **0.005**

$v_{min} = 0.035\ N^{1/2}/mm \times k^{3/2} \times f_{ck}^{0.5}$ = **0.485** N/mm²

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

$V_{Rd.c}$ = **148.8** kN/m

$V / V_{Rd.c}$ = **0.316**

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) — $A_{sx.req} = \max(0.25 \times A_{sr.prov}, 0.001 \times t_{stem})$ = **335** mm²/m

Maximum spacing of reinforcement – cl.9.6.3(2) — s_{sx_max} = **400** mm

Transverse reinforcement provided — 10 dia.bars @ 200 c/c

Area of transverse reinforcement provided — $A_{sx.prov} = \pi \times \phi_{sx}^2 / (4 \times s_{sx})$ = **393** mm²/m

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

Check base design at toe

Depth of section — h = **250** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 — M = **46.6** kNm/m

Depth to tension reinforcement — $d = h - c_{bb} - \phi_{bb} / 2$ = **167** mm

$K = M / (d^2 \times f_{ck})$ = **0.056**

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

K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm — $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d =$ **158** mm

Depth of neutral axis — $x = 2.5 \times (d - z) =$ **22** mm

Area of tension reinforcement required — $A_{bb.req} = M / (f_{yd} \times z) =$ **676** mm²/m

Tension reinforcement provided — 16 dia.bars @ 150 c/c

Area of tension reinforcement provided — $A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb}) =$ **1340** mm²/m

Minimum area of reinforcement - exp.9.1N — $A_{bb.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d =$ **252** mm²/m

Maximum area of reinforcement - cl.9.2.1.1(3) — $A_{bb.max} = 0.04 \times h =$ **10000** mm²/m

$\max(A_{bb.req}, A_{bb.min}) / A_{bb.prov} =$ **0.504**

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 — $\psi_2 =$ **0.6**

Serviceability bending moment — $M_{sls} =$ **34.1** kNm/m

Tensile stress in reinforcement — $\sigma_s = M_{sls} / (A_{bb.prov} \times z) =$ **160.8** N/mm²

Load duration — Long term

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

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

$A_{c.eff} =$ **76128** mm²/m

Mean value of concrete tensile strength — $f_{ct.eff} = f_{ctm} =$ **2.9** N/mm²

Reinforcement ratio — $\rho_{p.eff} = A_{bb.prov} / A_{c.eff} =$ **0.018**

Modular ratio — $\alpha_e = E_s / E_{cm} =$ **6.091**

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 — $s_{r.max} = k_3 \times c_{bb} + k_1 \times k_2 \times k_4 \times \phi_{bb} / \rho_{p.eff} =$ **409** mm

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

$w_k =$ **0.198** mm

$w_k / w_{max} =$ **0.659**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force — $V =$ **36.3** kN/m

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

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

Longitudinal reinforcement ratio — $\rho_l = \min(A_{bb.prov} / d, 0.02) =$ **0.008**

$v_{min} = 0.035\ N^{1/2}/mm \times k^{3/2} \times f_{ck}^{0.5} =$ **0.542** N/mm²

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

$V_{Rd.c} =$ **115.7** kN/m

$V / V_{Rd.c} =$ **0.313**

PASS - Design shear resistance exceeds design shear force

Secondary transverse reinforcement to base - Section 9.3

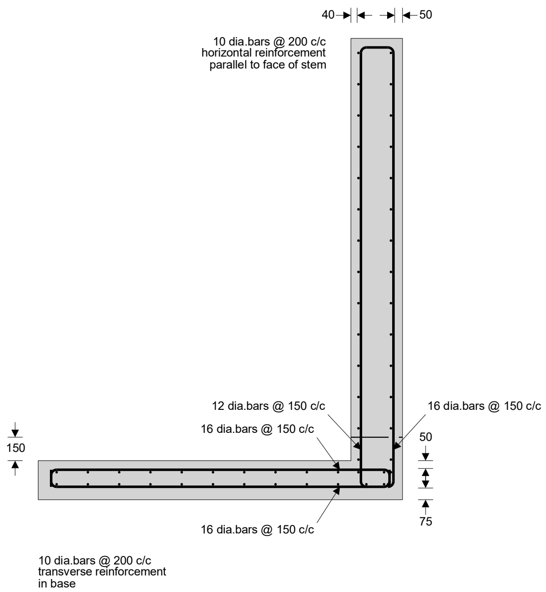
Minimum area of reinforcement – cl.9.3.1.1(2) — $A_{bx.req} = 0.2 \times A_{bb.prov} =$ **268** mm²/m

Maximum spacing of reinforcement – cl.9.3.1.1(3) — $s_{bx\_max} =$ **450** mm

Transverse reinforcement provided — 10 dia.bars @ 200 c/c

Area of transverse reinforcement provided $A_{bx.prov} = \pi \times \phi_{bx}^2 / (4 \times s_{bx}) =$ **393** mm^2/m

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



Reinforcement details

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

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

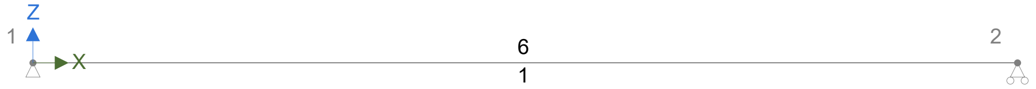
Tedds calculation version 3.3.06

ANALYSIS

Tedds calculation version 1.0.36

Geometry

Geometry (m) - Concrete (C30 2500 Quartzite) - R 2000x350



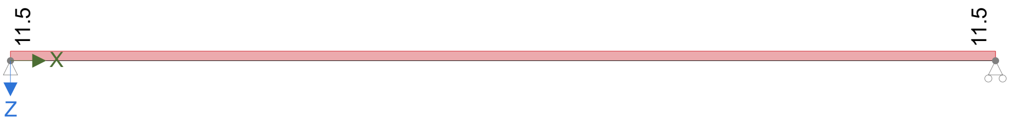
Span	Length (m)	Section	Start Support	End Support
1	6	R 2000x350	Pinned	Roller Pin X
R 2000x350: Area 7000 cm², Inertia Major 714583 cm⁴, Inertia Minor 2.×10⁷ cm⁴, Shear area parallel to Minor 5833 cm², Shear area parallel to Major = 5833 cm²				
Concrete (C30 2500 Quartzite): Density 2500 kg/m³, Youngs 32.836568 kN/mm², Shear 13.6819033 kN/mm², Thermal 0.00001 °C⁻¹				

Loading

Self weight included

Permanent - Loading (kN/m)

Imposed - Loading (kN/m)



Load combination factors

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

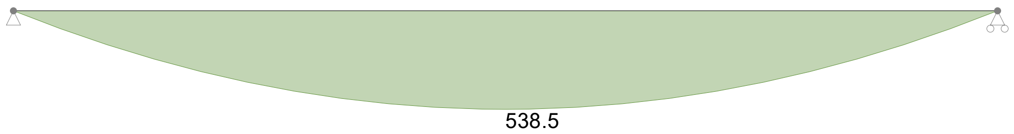
Member Loads

Member	Load case	Load Type	Orientation	Description
Beam	Permanent	UDL	GlobalZ	58.7 kN/m
Beam	Imposed	UDL	GlobalZ	11.5 kN/m

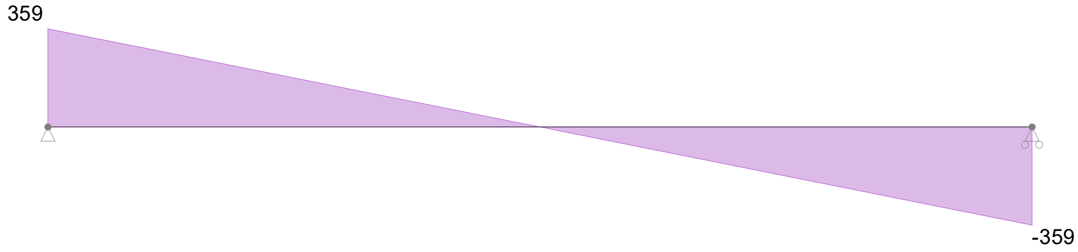
Results

Forces

Strength combinations - Moment envelope (kNm)



Strength combinations - Shear envelope (kN)



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

Concrete strength class	C30/37
Aggregate type	Quartzite
Aggregate adjustment factor - cl.3.1.3(2)	AAF = **1.0**
Characteristic compressive cylinder strength	f_{ck} = **30** N/mm^2
Mean value of compressive cylinder strength	$f_{cm} = f_{ck} + 8$ N/mm^2 = **38** N/mm^2
Mean value of axial tensile strength	$f_{ctm} = 0.3$ N/mm$^2 \times (f_{ck}/ 1$ N/mm$^2)^{2/3}$ = **2.9** N/mm^2
Secant modulus of elasticity of concrete	$E_{cm} = 22$ kN/mm$^2\times(f_{cm} / 10$ N/mm$^2)^{0.3} \times$ AAF = **32837** N/mm^2
Ultimate strain - Table 3.1	ε_{cu2} = **0.0035**
Shortening strain - Table 3.1	ε_{cu3} = **0.0035**
Effective compression zone height factor	λ = **0.80**
Effective strength factor	η = **1.00**
Coefficient k_1	k_1 = **0.40**
Coefficient k_2	$k_2 = 1.0 \times (0.6 + 0.0014 / \varepsilon_{cu2})$ = **1.00**
Coefficient k_3	k_3 = **0.40**
Coefficient k_4	$k_4 = 1.0 \times (0.6 + 0.0014 / \varepsilon_{cu2})$ = **1.00**
Partial factor for concrete -Table 2.1N	γ_C = **1.50**
Compressive strength coefficient - cl.3.1.6(1)	α_{cc} = **0.85**
Design compressive concrete strength - exp.3.15	$f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C$ = **17.0** N/mm^2
Compressive strength coefficient - cl.3.1.6(1)	α_{ccw} = **1.00**
Design compressive concrete strength - exp.3.15	$f_{cwd} = \alpha_{ccw} \times f_{ck} / \gamma_C$ = **20.0** N/mm^2
Maximum aggregate size	h_{agg} = **20** mm
Density of reinforced concrete	ρ = **2500** kg/m^3
Monolithic simple support moment factor	β_1 = **0.25**

Reinforcement details

Characteristic yield strength of reinforcement	f_{yk} = **500** N/mm^2

Partial factor for reinforcing steel - Table 2.1N γ_S = **1.15**

Design yield strength of reinforcement $f_{yd} = f_{yk} / \gamma_S$ = **435** N/mm^2

Nominal cover to reinforcement

Nominal cover to top reinforcement c_{nom_t} = **50** mm

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

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

Fire resistance

Standard fire resistance period R = **60** min

Number of sides exposed to fire 3

Minimum width of beam - EN1992-1-2 Table 5.5 b_{min} = **120** mm

Beam - Span 1

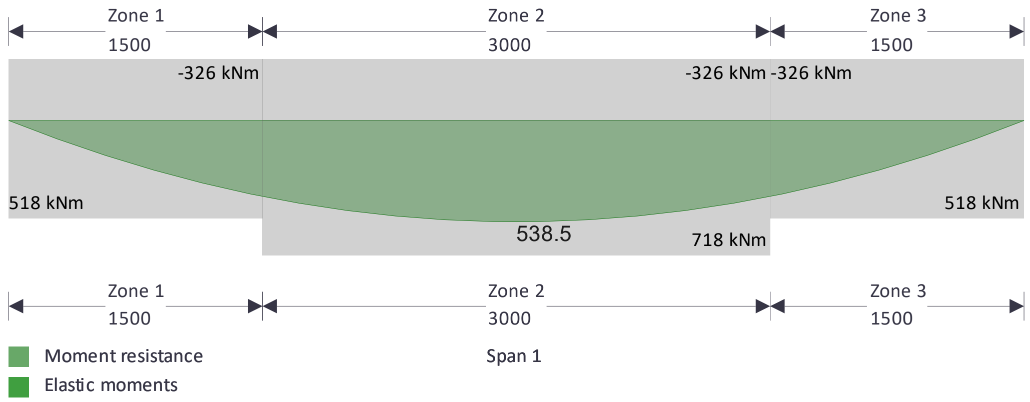
Rectangular section details

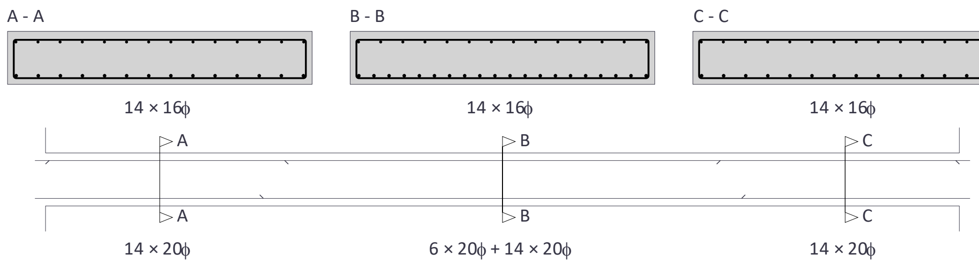
Section width b = **2000** mm

Section depth h = **350** mm

PASS - Minimum dimensions for fire resistance met

Moment design





Zone 1 (0 mm - 1500 mm) Positive moment - section 6.1

Design bending moment $M = abs(M_{m1\_s1\_z1\_max\_red})$ = **403.9** kNm

Effective depth of tension reinforcement d = **293** mm

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

$K = M / (b \times d^2 \times f_{ck})$ = **0.078**

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

= **0.207**

K' > K - No compression reinforcement is required

Lever arm $z = \min(0.5 \times d \times [1 + (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}], 0.95 \times d)$ = **271** mm

Depth of neutral axis $x = 2 \times (d - z) / \lambda$ = **55** mm

Area of tension reinforcement required	$A_{s,req} = M / (f_{yd} \times z) = \mathbf{3426}$ mm^2
Tension reinforcement provided	$14 \times 20\phi$
Area of tension reinforcement provided	$A_{s,prov} = \mathbf{4398}$ mm^2
Minimum area of reinforcement - exp.9.1N	$A_{s,min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b \times d = \mathbf{883}$ mm^2
Maximum area of reinforcement - cl.9.2.1.1(3)	$A_{s,max} = 0.04 \times b \times h = \mathbf{28000}$ mm^2

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

Crack control - Section 7.3

Maximum crack width	$w_k = \mathbf{0.3}$ mm
Design value modulus of elasticity reinf – 3.2.7(4)	$E_s = \mathbf{200000}$ N/mm^2
Mean value of concrete tensile strength	$f_{ct,eff} = f_{ctm} = \mathbf{2.9}$ N/mm^2
Stress distribution coefficient	$k_c = \mathbf{0.4}$
Non-uniform self-equilibrating stress coefficient	$k = \min(\max(1 + (300 \text{ mm} - \min(h, b)) \times 0.35 / 500 \text{ mm}, 0.65), 1) = \mathbf{0.97}$
Actual tension bar spacing	$s_{bar} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z1\_v}) + \phi_{m1\_s1\_z1\_b\_L1} \times N_{m1\_s1\_z1\_b\_L1})) / (N_{m1\_s1\_z1\_b\_L1} - 1) + \phi_{m1\_s1\_z1\_b\_L1} = \mathbf{145.1}$ mm
Maximum stress permitted - Table 7.3N	$\sigma_s = \mathbf{284}$ N/mm^2
Steel to concrete modulus of elast. ratio	$\alpha_{cr} = E_s / E_{cm} = \mathbf{6.09}$
Distance of the Elastic NA from bottom of beam	$y = (b \times h^2 / 2 + A_{s,prov} \times (\alpha_{cr} - 1) \times (h - d)) / (b \times h + A_{s,prov} \times (\alpha_{cr} - 1)) = \mathbf{171}$ mm
Area of concrete in the tensile zone	$A_{ct} = b \times y = \mathbf{342685}$ mm^2
Minimum area of reinforcement required - exp.7.1	$A_{sc,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / \sigma_s = \mathbf{1349}$ mm^2

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

Quasi-permanent moment	$M_{QP} = \max(\beta_1 \times \text{abs}(M_{m1_s1_z2_neg_quasi}), \text{abs}(M_{m1_s1_z1_pos_quasi})) = \mathbf{267.7}$kNm
Permanent load ratio	$R_{PL} = M_{QP} / M = \mathbf{0.66}$
Service stress in reinforcement	$\sigma_{sr} = f_{yd} \times A_{s,req} / A_{s,prov} \times R_{PL} = \mathbf{225}$ N/mm^2
Maximum bar spacing - Tables 7.3N	$s_{bar,max} = \mathbf{219.4}$ mm

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

Zone 1 (0 mm - 1500 mm) Negative moment - section 6.1

Design bending moment	$M = \max(\beta_1 \times \text{abs}(M_{m1\_s1\_max\_red}), \text{abs}(M_{m1\_s1\_z1\_min\_red})) = \mathbf{134.6}$ kNm
Effective depth of tension reinforcement	$d = \mathbf{280}$ mm
Redistribution ratio	$\delta = 1 = \mathbf{1.000}$
	$K = M / (b \times d^2 \times f_{ck}) = \mathbf{0.029}$
	$K' = (2 \times \eta \times \alpha_{cc} / \gamma_C) \times (1 - \lambda \times (\delta - k_1) / (2 \times k_2)) \times (\lambda \times (\delta - k_1) / (2 \times k_2)) = \mathbf{0.207}$

K' > K - No compression reinforcement is required

Lever arm	$z = \min(0.5 \times d \times [1 + (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}], 0.95 \times d) = \mathbf{266}$ mm
Depth of neutral axis	$x = 2 \times (d - z) / \lambda = \mathbf{35}$ mm
Area of tension reinforcement required	$A_{s,req} = M / (f_{yd} \times z) = \mathbf{1164}$ mm^2
Tension reinforcement provided	$14 \times 16\phi$
Area of tension reinforcement provided	$A_{s,prov} = \mathbf{2815}$ mm^2
Minimum area of reinforcement - exp.9.1N	$A_{s,min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b \times d = \mathbf{843}$ mm^2
Maximum area of reinforcement - cl.9.2.1.1(3)	$A_{s,max} = 0.04 \times b \times h = \mathbf{28000}$ mm^2

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

Crack control - Section 7.3

Maximum crack width	$w_k = \mathbf{0.3}$ mm

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

Mean value of concrete tensile strength $f_{ct,eff} = f_{ctm}$ = **2.9** N/mm^2

Stress distribution coefficient k_c = **0.4**

Non-uniform self-equilibrating stress coefficient $k = \min(\max(1 + (300\text{ mm} - \min(h, b)) \times 0.35 / 500\text{ mm}, 0.65), 1)$ = **0.97**

Actual tension bar spacing $s_{bar} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z1\_v}) + \phi_{m1\_s1\_z1\_t\_L1} \times N_{m1\_s1\_z1\_t\_L1})) / (N_{m1\_s1\_z1\_t\_L1} - 1) + \phi_{m1\_s1\_z1\_t\_L1}$ = **145.4** mm

Maximum stress permitted - Table 7.3N σ_s = **284** N/mm^2

Steel to concrete modulus of elast. ratio $\alpha_{cr} = E_s / E_{cm}$ = **6.09**

Distance of the Elastic NA from bottom of beam $y = (b \times h^2 / 2 + A_{s,prov} \times (\alpha_{cr} - 1) \times (h - d)) / (b \times h + A_{s,prov} \times (\alpha_{cr} - 1))$ = **173** mm

Area of concrete in the tensile zone $A_{ct} = b \times y$ = **345787** mm^2

Minimum area of reinforcement required - exp.7.1 $A_{sc,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / \sigma_s$ = **1363** mm^2

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

Quasi-permanent moment $M_{QP} = \max(\beta_1 \times \text{abs}(M_{m1\_s1\_z2\_pos\_quasi}), \text{abs}(M_{m1\_s1\_z1\_neg\_quasi}))$ = **89.2**kNm

Permanent load ratio $R_{PL} = M_{QP} / M$ = **0.66**

Service stress in reinforcement $\sigma_{sr} = f_{yd} \times A_{s,req} / A_{s,prov} \times R_{PL}$ = **119** N/mm^2

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

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

Minimum bar spacing (Section 8.2)

Top bar spacing $s_{top} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z1\_v}) + \phi_{m1\_s1\_z1\_t\_L1} \times N_{m1\_s1\_z1\_t\_L1})) / (N_{m1\_s1\_z1\_t\_L1} - 1)$ = **129.4** mm

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

PASS - Actual bar spacing exceeds minimum allowable

Bottom bar spacing $s_{bot} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z1\_v}) + \phi_{m1\_s1\_z1\_b\_L1} \times N_{m1\_s1\_z1\_b\_L1})) / (N_{m1\_s1\_z1\_b\_L1} - 1)$ = **125.1** mm

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

PASS - Actual bar spacing exceeds minimum allowable

Zone 2 (1500 mm - 4500 mm) Positive moment - section 6.1

Design bending moment $M = \text{abs}(M_{m1\_s1\_z2\_max\_red})$ = **538.5** kNm

Effective depth of tension reinforcement d = **293** mm

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

$K = M / (b \times d^2 \times f_{ck})$ = **0.105**

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

K' > K - No compression reinforcement is required

Lever arm $z = \min(0.5 \times d \times [1 + (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}], 0.95 \times d)$ = **263** mm

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

Area of tension reinforcement required $A_{s,req} = M / (f_{yd} \times z)$ = **4711** mm^2

Tension reinforcement provided $6 \times 20\phi + 14 \times 20\phi$

Area of tension reinforcement provided $A_{s,prov}$ = **6283** mm^2

Minimum area of reinforcement - exp.9.1N $A_{s,min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b \times d$ = **883** mm^2

Maximum area of reinforcement - cl.9.2.1.1(3) $A_{s,max} = 0.04 \times b \times h$ = **28000** mm^2

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

Crack control - Section 7.3

Maximum crack width	w_k = **0.3** mm
Design value modulus of elasticity reinf – 3.2.7(4)	E_s = **200000** N/mm^2
Mean value of concrete tensile strength	$f_{ct,eff} = f_{ctm}$ = **2.9** N/mm^2
Stress distribution coefficient	k_c = **0.4**
Non-uniform self-equilibrating stress coefficient	k = min(max(1 + (300 mm - min(h, b)) × 0.35 / 500 mm, 0.65), 1) = **0.97**
Actual tension bar spacing	$s_{bar} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z2\_v}) + \phi_{m1\_s1\_z2\_b\_L1} \times N_{m1\_s1\_z2\_b\_L1} + \phi_{m1\_s1\_z1\_b\_L1} \times N_{m1\_s1\_z1\_b\_L1})) / ((N_{m1\_s1\_z2\_b\_L1} + N_{m1\_s1\_z1\_b\_L1}) - 1) + \phi_{m1\_s1\_z2\_b\_L1}$ = **99.3** mm
Maximum stress permitted - Table 7.3N	σ_s = **321** N/mm^2
Steel to concrete modulus of elast. ratio	$\alpha_{cr} = E_s / E_{cm}$ = **6.09**
Distance of the Elastic NA from bottom of beam	$y = (b \times h^2 / 2 + A_{s,prov} \times (\alpha_{cr} - 1) \times (h - d)) / (b \times h + A_{s,prov} \times (\alpha_{cr} - 1))$ = **170** mm
Area of concrete in the tensile zone	$A_{ct} = b \times y$ = **339687** mm^2
Minimum area of reinforcement required - exp.7.1	$A_{sc,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / \sigma_s$ = **1185** mm^2

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

Quasi-permanent moment	$M_{QP} = abs(M_{m1\_s1\_z2\_pos\_quasi})$ = **356.9**kNm
Permanent load ratio	$R_{PL} = M_{QP} / M$ = **0.66**
Service stress in reinforcement	$\sigma_{sr} = f_{yd} \times A_{s,req} / A_{s,prov} \times R_{PL}$ = **216** N/mm^2
Maximum bar spacing - Tables 7.3N	$s_{bar,max}$ = **229.9** mm

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

Deflection control - Section 7.4

Reference reinforcement ratio	$\rho_{m0} = (f_{ck} / 1\ N/mm^2)^{0.5} / 1000$ = **0.00548**
Required tension reinforcement ratio	$\rho_m = A_{s,req} / (b \times d)$ = **0.00804**
Required compression reinforcement ratio	$\rho'_m = A_{s2,req} / (b \times d)$ = **0.00000**
Structural system factor - Table 7.4N	K_b = **1.0**
Basic allowable span to depth ratio	$span\_to\_depth_{basic} = K_b \times [11 + 1.5 \times (f_{ck} / 1\ N/mm^2)^{0.5} \times \rho_{m0} / (\rho_m - \rho'_m) + (f_{ck} / 1\ N/mm^2)^{0.5} \times (\rho'_m / \rho_{m0})^{0.5} / 12]$ = **16.597**
Reinforcement factor - exp.7.17	$K_s = \min(A_{s,prov} / A_{s,req} \times 500\ N/mm^2 / f_{yk}, 1.5)$ = **1.334**
Flange width factor	F1 = 1 = **1.000**
Long span supporting brittle partition factor	F2 = 1 = **1.000**
Allowable span to depth ratio	$span\_to\_depth_{allow} = \min(span\_to\_depth_{basic} \times K_s \times F1 \times F2, 40 \times K_b)$ = **22.134**
Actual span to depth ratio	$span\_to\_depth_{actual} = L_{m1\_s1} / d$ = **20.478**

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

Minimum bar spacing (Section 8.2)

Top bar spacing	$s_{top} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z2\_v}) + \phi_{m1\_s1\_z2\_t\_L1} \times N_{m1\_s1\_z2\_t\_L1})) / (N_{m1\_s1\_z2\_t\_L1} - 1)$ = **129.4** mm
Minimum allowable top bar spacing	$s_{top,min} = \max(\phi_{m1\_s1\_z2\_t\_L1} \times k_{s1}, h_{agg} + k_{s2}, 20mm)$ = **25.0** mm

PASS - Actual bar spacing exceeds minimum allowable

Bottom bar spacing	$s_{bot} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z2\_v}) + \phi_{m1\_s1\_z2\_b\_L1} \times N_{m1\_s1\_z2\_b\_L1} + \phi_{m1\_s1\_z1\_b\_L1} \times N_{m1\_s1\_z1\_b\_L1})) / ((N_{m1\_s1\_z2\_b\_L1} + N_{m1\_s1\_z1\_b\_L1}) - 1)$ = **79.3** mm
Minimum allowable bottom bar spacing	$s_{bot,min} = \max(\phi_{m1\_s1\_z2\_b\_L1} \times k_{s1}, h_{agg} + k_{s2}, 20mm)$ = **25.0** mm

PASS - Actual bar spacing exceeds minimum allowable

Zone 3 (4500 mm - 6000 mm) Positive moment - section 6.1

Design bending moment — $M = abs(M_{m1\_s1\_z3\_max\_red})$ = **403.9** kNm

Effective depth of tension reinforcement — d = **293** mm

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

$K = M / (b \times d^2 \times f_{ck})$ = **0.078**

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

K' > K - No compression reinforcement is required

Lever arm — $z = min(0.5 \times d \times [1 + (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}], 0.95 \times d)$ = **271** mm

Depth of neutral axis — $x = 2 \times (d - z) / \lambda$ = **55** mm

Area of tension reinforcement required — $A_{s,req} = M / (f_{yd} \times z)$ = **3426** mm^2

Tension reinforcement provided — $14 \times 20\phi$

Area of tension reinforcement provided — $A_{s,prov}$ = **4398** mm^2

Minimum area of reinforcement - exp.9.1N — $A_{s,min} = max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b \times d$ = **883** mm^2

Maximum area of reinforcement - cl.9.2.1.1(3) — $A_{s,max} = 0.04 \times b \times h$ = **28000** mm^2

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

Crack control - Section 7.3

Maximum crack width — w_k = **0.3** mm

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

Mean value of concrete tensile strength — $f_{ct,eff} = f_{ctm}$ = **2.9** N/mm^2

Stress distribution coefficient — k_c = **0.4**

Non-uniform self-equilibrating stress coefficient — k = min(max(1 + (300 mm - min(h, b)) × 0.35 / 500 mm, 0.65), 1) = **0.97**

Actual tension bar spacing — $s_{bar} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z3\_v}) + \phi_{m1\_s1\_z3\_b\_L1} \times N_{m1\_s1\_z3\_b\_L1})) / (N_{m1\_s1\_z3\_b\_L1} - 1) + \phi_{m1\_s1\_z3\_b\_L1}$ = **145.1** mm

Maximum stress permitted - Table 7.3N — σ_s = **284** N/mm^2

Steel to concrete modulus of elast. ratio — $\alpha_{cr} = E_s / E_{cm}$ = **6.09**

Distance of the Elastic NA from bottom of beam — $y = (b \times h^2 / 2 + A_{s,prov} \times (\alpha_{cr} - 1) \times (h - d)) / (b \times h + A_{s,prov} \times (\alpha_{cr} - 1))$ = **171** mm

Area of concrete in the tensile zone — $A_{ct} = b \times y$ = **342685** mm^2

Minimum area of reinforcement required - exp.7.1 — $A_{sc,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / \sigma_s$ = **1349** mm^2

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

Quasi-permanent moment — $M_{QP} = max(\beta_1 \times abs(M_{m1\_s1\_z2\_neg\_quasi}), abs(M_{m1\_s1\_z3\_pos\_quasi}))$ = **267.7**kNm

Permanent load ratio — $R_{PL} = M_{QP} / M$ = **0.66**

Service stress in reinforcement — $\sigma_{sr} = f_{yd} \times A_{s,req} / A_{s,prov} \times R_{PL}$ = **225** N/mm^2

Maximum bar spacing - Tables 7.3N — $s_{bar,max}$ = **219.4** mm

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

Zone 3 (4500 mm - 6000 mm) Negative moment - section 6.1

Design bending moment — $M = max(\beta_1 \times abs(M_{m1\_s1\_max\_red}), abs(M_{m1\_s1\_z3\_min\_red}))$ = **134.6** kNm

Effective depth of tension reinforcement — d = **280** mm

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

$K = M / (b \times d^2 \times f_{ck})$ = **0.029**

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

K' > K - No compression reinforcement is required

Lever arm $z = \min(0.5 \times d \times [1 + (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}], 0.95 \times d) =$ **266** mm

Depth of neutral axis $x = 2 \times (d - z) / \lambda =$ **35** mm

Area of tension reinforcement required $A_{s,req} = M / (f_{yd} \times z) =$ **1164** mm^2

Tension reinforcement provided $14 \times 16\phi$

Area of tension reinforcement provided $A_{s,prov} =$ **2815** mm^2

Minimum area of reinforcement - exp.9.1N $A_{s,min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b \times d =$ **843** mm^2

Maximum area of reinforcement - cl.9.2.1.1(3) $A_{s,max} = 0.04 \times b \times h =$ **28000** mm^2

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

Crack control - Section 7.3

Maximum crack width $w_k =$ **0.3** mm

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

Mean value of concrete tensile strength $f_{ct,eff} = f_{ctm} =$ **2.9** N/mm^2

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

Non-uniform self-equilibrating stress coefficient $k = \min(\max(1 + (300\text{ mm} - \min(h, b)) \times 0.35 / 500\text{ mm}, 0.65), 1) =$ **0.97**

Actual tension bar spacing $s_{bar} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z3\_v}) + \phi_{m1\_s1\_z3\_t\_L1} \times N_{m1\_s1\_z3\_t\_L1})) / (N_{m1\_s1\_z3\_t\_L1} - 1) + \phi_{m1\_s1\_z3\_t\_L1} =$ **145.4** mm

Maximum stress permitted - Table 7.3N $\sigma_s =$ **284** N/mm^2

Steel to concrete modulus of elast. ratio $\alpha_{cr} = E_s / E_{cm} =$ **6.09**

Distance of the Elastic NA from bottom of beam $y = (b \times h^2 / 2 + A_{s,prov} \times (\alpha_{cr} - 1) \times (h - d)) / (b \times h + A_{s,prov} \times (\alpha_{cr} - 1)) =$ **173** mm

Area of concrete in the tensile zone $A_{ct} = b \times y =$ **345787** mm^2

Minimum area of reinforcement required - exp.7.1 $A_{sc,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / \sigma_s =$ **1363** mm^2

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

Quasi-permanent moment $M_{QP} = \max(\beta_1 \times abs(M_{m1\_s1\_z2\_pos\_quasi}), abs(M_{m1\_s1\_z3\_neg\_quasi})) =$ **89.2**kNm

Permanent load ratio $R_{PL} = M_{QP} / M =$ **0.66**

Service stress in reinforcement $\sigma_{sr} = f_{yd} \times A_{s,req} / A_{s,prov} \times R_{PL} =$ **119** N/mm^2

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

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

Minimum bar spacing (Section 8.2)

Top bar spacing $s_{top} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z3\_v}) + \phi_{m1\_s1\_z3\_t\_L1} \times N_{m1\_s1\_z3\_t\_L1})) / (N_{m1\_s1\_z3\_t\_L1} - 1) =$ **129.4** mm

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

PASS - Actual bar spacing exceeds minimum allowable

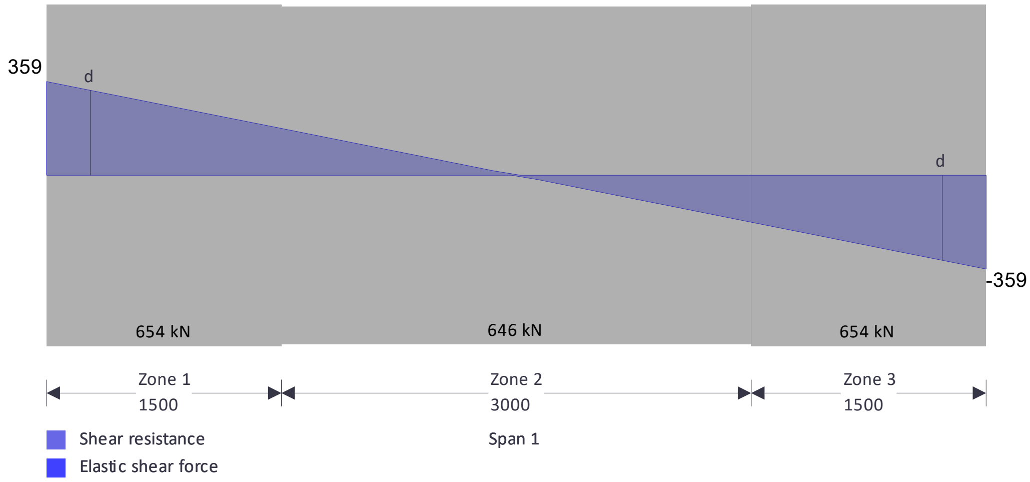
Bottom bar spacing $s_{bot} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z3\_v}) + \phi_{m1\_s1\_z3\_b\_L1} \times N_{m1\_s1\_z3\_b\_L1})) / (N_{m1\_s1\_z3\_b\_L1} - 1) =$ **125.1** mm

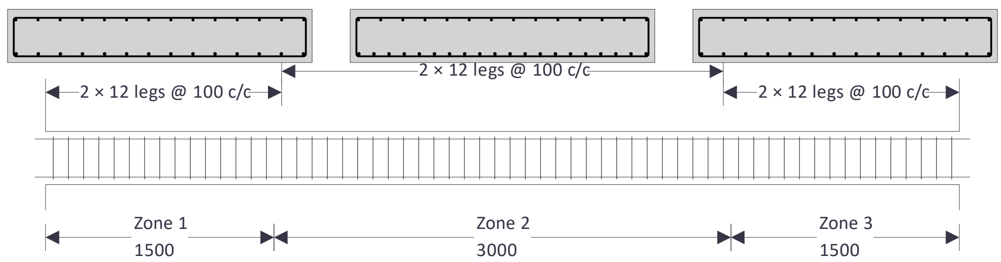
Minimum allowable bottom bar spacing $s_{bot,min} = \max(\phi_{m1\_s1\_z3\_b\_L1} \times k_{s1}, h_{agg} + k_{s2}, 20mm) =$ **25.0** mm

PASS - Actual bar spacing exceeds minimum allowable

Symmetrys Unit 6 The Courtyard, Lynton Road London N8 8SL	Project 43a REDINGTON ROAD, LONDON NW3 7RA				Job no. 21141	
	Calcs for B-GB01				Start page no./Revision 9	
	Calcs by SB	Calcs date 27/08/2021	Checked by DS	Checked date	Approved by DS	Approved date

Shear design





Angle of comp. shear strut for maximum shear	θ_{max} = 45 deg
Strength reduction factor - cl.6.2.3(3)	$v_1 = 0.6 \times (1 - f_{ck} / 250\ N/mm^2)$ = **0.528**
Compression chord coefficient - cl.6.2.3(3)	α_{cw} = **1.00**
Minimum area of shear reinforcement - exp.9.5N	$A_{sv,min} = 0.08\ N/mm^2 \times b \times (f_{ck} / 1\ N/mm^2)^{0.5} / f_{yk}$ = **1753** mm^2/m

Zone 1 (0 mm - 1500 mm) shear - section 6.2

Design shear force at support	$V_{Ed,max} = \max(\text{abs}(V_{z1\_max}), \text{abs}(V_{z1\_red\_max}))$ = **359** kN
Min lever arm in shear zone	z = **266** mm
Maximum design shear resistance - exp.6.9	$V_{Rd,max} = \alpha_{cw} \times b \times z \times v_1 \times f_{cwd} / (\cot(\theta_{max}) + \tan(\theta_{max}))$ = **2809** kN

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

Design shear force at 280mm from support	V_{Ed} = **325** kN
Design shear stress	$v_{Ed} = V_{Ed} / (b \times z)$ = **0.612** N/mm^2
Angle of concrete compression strut - cl.6.2.3	$\theta = \min(\max(0.5 \times \text{Asin}(\min(2 \times v_{Ed} / (\alpha_{cw} \times f_{cwd} \times v_1),1)), 21.8\ \text{deg}), 45\text{deg})$ = **21.8** deg
Area of shear reinforcement required - exp.6.8	$A_{sv,des} = v_{Ed} \times b / (f_{yd} \times \cot(\theta))$ = **1126** mm^2/m
Area of shear reinforcement required	$A_{sv,req} = \max(A_{sv,min}, A_{sv,des})$ = **1753** mm^2/m
Shear reinforcement provided	2 × 12 legs @ 100 c/c
Area of shear reinforcement provided	$A_{sv,prov}$ = **2262** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing - exp.9.6N	$s_{vl,max} = 0.75 \times d$ = **210** mm

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

Zone 2 (1500 mm - 4500 mm) shear - section 6.2

Design shear force at support	$V_{Ed,max} = \max(\text{abs}(V_{z2\_max}), \text{abs}(V_{z2\_red\_max}))$ = **179** kN

Min lever arm in shear zone z = **263** mm

Maximum design shear resistance - exp.6.9 $V_{Rd,max} = \alpha_{cw} \times b \times z \times v_1 \times f_{cwd} / (\cot(\theta_{max}) + \tan(\theta_{max}))$ = **2776** kN

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

Design shear force within zone V_{Ed} = **179** kN

Design shear stress $v_{Ed} = V_{Ed} / (b \times z)$ = **0.341** N/mm^2

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

Area of shear reinforcement required - exp.6.8 $A_{sv,des} = v_{Ed} \times b / (f_{yd} \times \cot(\theta))$ = **628** mm^2/m

Area of shear reinforcement required $A_{sv,req} = \max(A_{sv,min}, A_{sv,des})$ = **1753** mm^2/m

Shear reinforcement provided 2 × 12 legs @ 100 c/c

Area of shear reinforcement provided $A_{sv,prov}$ = **2262** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing - exp.9.6N $s_{vl,max} = 0.75 \times d$ = **220** mm

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

Zone 3 (4500 mm - 6000 mm) shear - section 6.2

Design shear force at support $V_{Ed,max} = \max(\mathrm{abs}(V_{z3\_max}), \mathrm{abs}(V_{z3\_red\_max}))$ = **359** kN

Min lever arm in shear zone z = **266** mm

Maximum design shear resistance - exp.6.9 $V_{Rd,max} = \alpha_{cw} \times b \times z \times v_1 \times f_{cwd} / (\cot(\theta_{max}) + \tan(\theta_{max}))$ = **2809** kN

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

Design shear force at 280mm from support V_{Ed} = **325** kN

Design shear stress $v_{Ed} = V_{Ed} / (b \times z)$ = **0.612** N/mm^2

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

Area of shear reinforcement required - exp.6.8 $A_{sv,des} = v_{Ed} \times b / (f_{yd} \times \cot(\theta))$ = **1126** mm^2/m

Area of shear reinforcement required $A_{sv,req} = \max(A_{sv,min}, A_{sv,des})$ = **1753** mm^2/m

Shear reinforcement provided 2 × 12 legs @ 100 c/c

Area of shear reinforcement provided $A_{sv,prov}$ = **2262** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

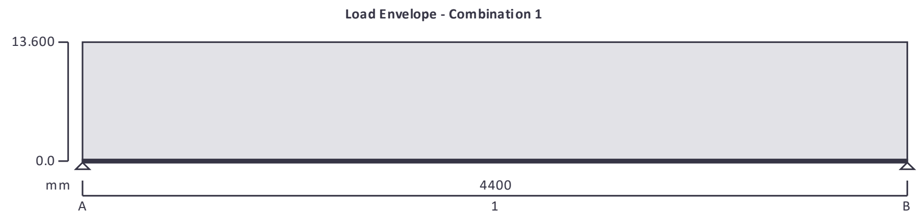
Maximum longitudinal spacing - exp.9.6N $s_{vl,max} = 0.75 \times d$ = **210** mm

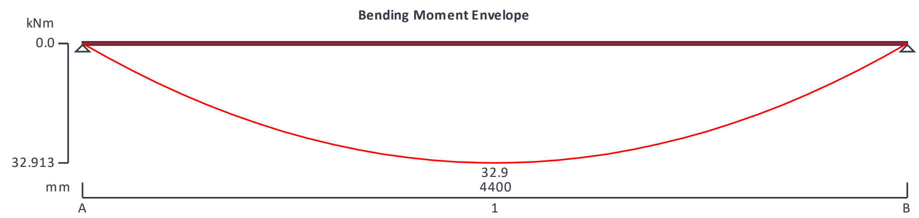
PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

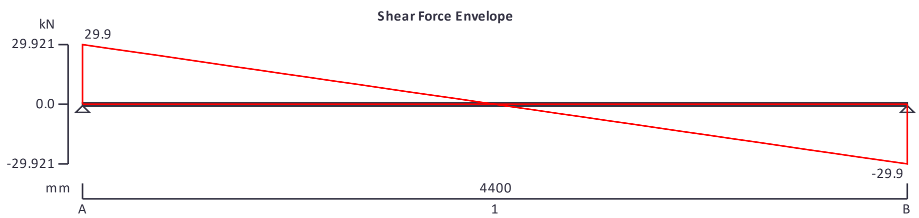
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

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 Ground floor. 1KN/m2 x 4.8m/2 - Permanent full UDL 2.4 kN/m Ground floor + partitions. 2.7KN/m2 x 4.8m/2 - Variable full UDL 6.5 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

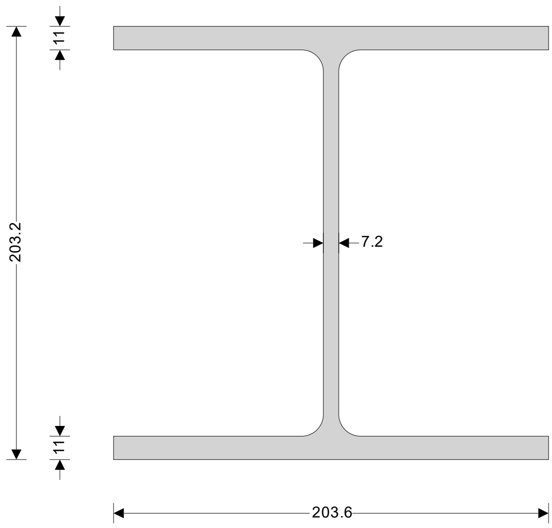
Maximum moment	M_{max} = **32.9** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **29.9** kN	V_{min} = **-29.9** kN
Deflection	δ_{max} = **4.8** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **29.9** kN	R_{A_min} = **29.9** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **6.3** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **14.3** kN	
Maximum reaction at support B	R_{B_max} = **29.9** kN	R_{B_min} = **29.9** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **6.3** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **14.3** kN	

Section details

Section type	**UC 203x203x46 (BS4-1)**
Steel grade	**S355**

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 = **355** N/mm^2
Nominal ultimate tensile strength	f_u = **470** 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 lateral restraint at supports only

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

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]} = \mathbf{0.81}$

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

Width of section — $c = d = \mathbf{160.8}$ mm

$c / t_w = 27.4 \times \varepsilon <= 72 \times \varepsilon$ — Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section — $c = (b - t_w - 2 \times r) / 2 = \mathbf{88}$ mm

$c / t_f = 9.8 \times \varepsilon <= 10 \times \varepsilon$ — Class 2

Section is class 2

Check shear - Section 6.2.6

Height of web — $h_w = h - 2 \times t_f = \mathbf{181.2}$ mm

Shear area factor — $\eta = \mathbf{1.000}$

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

Shear buckling resistance can be ignored

Design shear force — $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = \mathbf{29.9}$ kN

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

Design shear resistance - cl 6.2.6(2) — $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0} = \mathbf{347.9}$ kN

PASS - Design shear resistance exceeds design shear force

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

Design bending moment — $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) = \mathbf{32.9}$ kNm

Design bending resistance moment - eq 6.13 — $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0} = \mathbf{176.6}$ kNm

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 — $k_c = \mathbf{0.94}$

$C_1 = 1 / k_c^2 = \mathbf{1.132}$

Curvature factor — $g = \sqrt{[1 - (I_z / I_y)]} = \mathbf{0.813}$

Poissons ratio — $\nu = \mathbf{0.3}$

Shear modulus — $G = E / [2 \times (1 + \nu)] = \mathbf{80769}$ N/mm^2

Unrestrained length — $L = 1.0 \times L_{s1} = \mathbf{4400}$ mm

Elastic critical buckling moment — $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} = \mathbf{326.5}$ kNm

Slenderness ratio for lateral torsional buckling — $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})} = \mathbf{0.735}$

Limiting slenderness ratio — $\bar{\lambda}_{LT,0} = \mathbf{0.4}$

$\bar{\lambda}_{LT} > \bar{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5 — b

Imperfection factor - Table 6.3 — $\alpha_{LT} = \mathbf{0.34}$

Correction factor for rolled sections — $\beta = \mathbf{0.75}$

LTB reduction determination factor — $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = \mathbf{0.760}$

LTB reduction factor - eq 6.57 — $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = \mathbf{0.852}$

Modification factor — $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = \mathbf{0.970}$

Modified LTB reduction factor - eq 6.58 — $\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = \mathbf{0.878}$

Design buckling resistance moment - eq 6.55 — $M_{b,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1} = \mathbf{155}$ kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection $\delta_{lim} = L_{s1} / 250 =$ **17.6** mm

Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) =$ **4.758** mm

PASS - Maximum deflection does not exceed deflection limit

STEEL COLUMN DESIGN

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

Tedds calculation version 1.1.06

Design summary

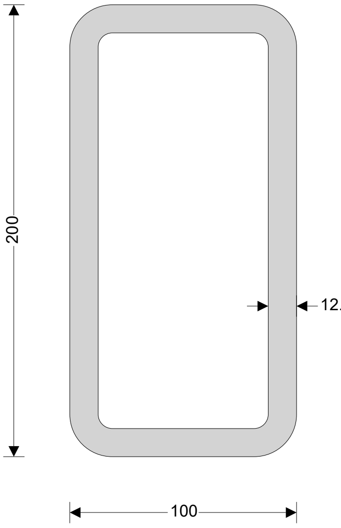
Description	Unit	Provided	Required	Utilisation	Result
Shear resistance (y-y)	kN	916	1	0.001	PASS
Shear resistance (z-z)	kN	458	8	0.017	PASS
Axial compression	kN	2381	154	0.065	PASS
Bending resistance (z-z)	kNm	87	23	0.265	PASS
Combined bending & axial	kNm	87	23	0.265	PASS
Buckling in compression	kN	1560	154	0.099	PASS
Combined buckling				0.280	PASS

Partial factors - Section 6.1

Resistance of cross-sections γ_{M0} = **1**

Resistance of members to instability γ_{M1} = **1**

Resistance of cross-sections in tension to fracture γ_{M2} = **1.1**



RHS 200x100x12.5 (Tata Steel Celsius (Gr355 Gr420))
Section depth, h, 200 mm
Section breadth, b, 100 mm
Mass of section, Mass, 52.7 kg/m
Section thickness, t, 12.5 mm
Area of section, A, 6707 mm^2
Radius of gyration about y-axis, i_y, 68.377 mm
Radius of gyration about z-axis, i_z, 38.688 mm
Elastic section modulus about y-axis, $W_{el.y}$, 313596 mm^3
Elastic section modulus about z-axis, $W_{el.z}$, 200791 mm^3
Plastic section modulus about y-axis, $W_{pl.y}$, 408228 mm^3
Plastic section modulus about z-axis, $W_{pl.z}$, 244736 mm^3
Second moment of area about y-axis, I_y, 31359643 mm^4
Second moment of area about z-axis, I_z, 10039542 mm^4

Column details

Column section **RHS 200x100x12.5**

Steel grade **User defined**

Yield strength f_y = **355** N/mm^2

Ultimate strength f_u = **470** N/mm^2

Modulus of elasticity E = **210** kN/mm^2

Poisson's ratio ν = **0.3**

Shear modulus $G = E / [2 \times (1 + \nu)]$ = **80.8** kN/mm^2

Column geometry

System length for buckling - Major axis L_y = **3000** mm

System length for buckling - Minor axis L_z = **3000** mm

The column is not part of a sway frame in the direction of the minor axis

The column is not part of a sway frame in the direction of the major axis

Column loading

Axial load — N_{Ed} = **154** kN (Compression)

Major axis moment at end 1 - Bottom — $M_{y,Ed1}$ = **0.0** kNm

Major axis moment at end 2 - Top — $M_{y,Ed2}$ = **0.0** kNm

Minor axis moment at end 1 - Bottom — $M_{z,Ed1}$ = **23.0** kNm

Minor axis moment at end 2 - Top — $M_{z,Ed2}$ = **2.0** kNm

Minor axis bending is single curvature

Major axis shear force — $V_{y,Ed}$ = **1** kN

Minor axis shear force — $V_{z,Ed}$ = **8** kN

Buckling length for flexural buckling - Major axis

End restraint factor — K_y = **1.000**

Buckling length — $L_{cr\_y} = L_y \times K_y$ = **3000** mm

Buckling length for flexural buckling - Minor axis

End restraint factor — K_z = **1.000**

Buckling length — $L_{cr\_z} = L_z \times K_z$ = **3000** mm

Web section classification (Table 5.2)

Coefficient depending on f_y — $\varepsilon = \sqrt{(235 \text{ N/mm}^2 / f_y)}$ = **0.814**

Depth between fillets — $c_w = h - 3 \times t$ = **162.5** mm

Ratio of c/t — $ratio_w = c_w / t$ = **13.00**

Length of web taken by axial load — $l_w = \min(N_{Ed} / (2 \times f_y \times t), c_w)$ = **17.4** mm

For class 1 & 2 proportion in compression — $\alpha = (c_w/2 + l_w/2) / c_w$ = **0.553**

Limit for class 1 web — $Limit_{1w} = (396 \times \varepsilon) / (13 \times \alpha - 1)$ = **52.02**

The web is class 1

Flange section classification (Table 5.2)

Depth between fillets — $c_f = b - 3 \times t$ = **62.5** mm

Ratio of c/t — $ratio_f = c_f / t$ = **5.00**

Conservatively assume uniform compression in flange

Limit for class 1 flange — $Limit_{1f} = 33 \times \varepsilon$ = **26.85**

Limit for class 2 flange — $Limit_{2f} = 38 \times \varepsilon$ = **30.92**

Limit for class 3 flange — $Limit_{3f} = 42 \times \varepsilon$ = **34.17**

The flange is class 1

Overall section classification

The section is class 1

Resistance of cross section (cl. 6.2)

Shear - Major axis (cl. 6.2.6)

Design shear force — $V_{y,Ed}$ = **1.0** kN

Shear area — $A_{vy} = A \times h / (b + h)$ = **4472** mm²

Plastic shear resistance — $V_{pl,y,Rd} = A_{vy} \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **916.5** kN

$V_{y,Ed} / V_{pl,y,Rd}$ = **0.001**

PASS - Shear resistance exceeds the design shear force

$V_{y,Ed} <= 0.5 \times V_{pl,y,Rd}$ - No reduction in f_y required for bending/axial force

Shear - Minor axis (cl. 6.2.6)

Design shear force — $V_{z,Ed}$ = **8.0** kN

Shear area $A_{vz} = A \times b / (b + h) =$ **2236** mm^2

Plastic shear resistance $V_{pl,z,Rd} = A_{vz} \times (f_y / \sqrt{(3)}) / \gamma_{M0} =$ **458.2** kN

$V_{z,Ed} / V_{pl,z,Rd} =$ **0.017**

PASS - Shear resistance exceeds the design shear force

$V_{z,Ed} <= 0.5 \times V_{pl,z,Rd}$ - No reduction in f_y required for bending/axial force

Compression (cl. 6.2.4)

Design force $N_{Ed} =$ **154** kN

Design resistance $N_{c,Rd} = N_{pl,Rd} = A \times f_y / \gamma_{M0} =$ **2381** kN

$N_{Ed} / N_{c,Rd} =$ **0.065**

PASS - The compression design resistance exceeds the design force

Bending - Major axis(cl. 6.2.5)

Design bending moment $M_{z,Ed} = \max(\text{abs}(M_{z,Ed1}), \text{abs}(M_{z,Ed2})) =$ **23.0** kNm

Section modulus $W_z = W_{pl.z} =$ **244.7** cm^3

Design resistance $M_{c,z,Rd} = W_z \times f_y / \gamma_{M0} =$ **86.9** kNm

$M_{z,Ed} / M_{c,z,Rd} =$ **0.265**

PASS - The bending design resistance exceeds the design moment

Combined bending and axial force (cl. 6.2.9)

Ratio design axial to design plastic resistance $n = \text{abs}(N_{Ed}) / N_{pl,Rd} =$ **0.065**

Ratio web area to gross area $a_w = \min(0.5, (A - 2 \times b \times t) / A) =$ **0.500**

Ratio flange area to gross area $a_f = \min(0.5, (A - 2 \times h \times t) / A) =$ **0.255**

Bending - Minor axis (cl. 6.2.9.1)

Design bending moment $M_{z,Ed} = \max(\text{abs}(M_{z,Ed1}), \text{abs}(M_{z,Ed2})) =$ **23.0** kNm

Plastic design resistance $M_{pl,z,Rd} = W_{pl.z} \times f_y / \gamma_{M0} =$ **86.9** kNm

Modified design resistance $M_{N,z,Rd} = M_{pl,z,Rd} \times \min(1, (1 - n) / (1 - 0.5 \times a_f)) =$ **86.9** kNm

$M_{z,Ed} / M_{N,z,Rd} =$ **0.265**

PASS - Bending resistance in presence of axial load exceeds design moment

Buckling resistance (cl. 6.3)

Yield strength for buckling resistance $f_y =$ **355** N/mm^2

Flexural buckling - Major axis

Elastic critical buckling force $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr\_y}^2 =$ **7222** kN

Non-dimensional slenderness $\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})} =$ **0.574**

Buckling curve (Table 6.2) **a**

Imperfection factor (Table 6.1) $\alpha_y =$ **0.21**

Parameter Φ $\Phi_y = 0.5 \times [1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2] =$ **0.704**

Reduction factor $\chi_y = \min(1.0, 1 / [\Phi_y + \sqrt{(\Phi_y^2 - \bar{\lambda}_y^2)}]) =$ **0.900**

Design buckling resistance $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1} =$ **2141.8** kN

$N_{Ed} / N_{b,y,Rd} =$ **0.072**

PASS - The flexural buckling resistance exceeds the design axial load

Flexural buckling - Minor axis

Elastic critical buckling force $N_{cr,z} = \pi^2 \times E \times I_z / L_{cr\_z}^2 =$ **2312** kN

Non-dimensional slenderness $\bar{\lambda}_z = \sqrt{(A \times f_y / N_{cr,z})} =$ **1.015**

Buckling curve (Table 6.2) **a**

Imperfection factor (Table 6.1) $\alpha_z =$ **0.21**

Parameter Φ $\Phi_z = 0.5 \times [1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2] =$ **1.100**

Reduction factor $\chi_z = \min(1.0, 1 / [\Phi_z + \sqrt{(\Phi_z^2 - \bar{\lambda}_z^2)}]) = \mathbf{0.655}$

Design buckling resistance $N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1} = \mathbf{1560.2}$ kN

$N_{Ed} / N_{b,z,Rd} = \mathbf{0.099}$

PASS - The flexural buckling resistance exceeds the design axial load

Minimum buckling resistance

Minimum buckling resistance $N_{b,Rd} = \min(N_{b,y,Rd}, N_{b,z,Rd}) = \mathbf{1560.2}$ kN

$N_{Ed} / N_{b,Rd} = \mathbf{0.099}$

PASS - The axial load buckling resistance exceeds the design axial load

Combined bending and axial compression (cl. 6.3.3)

Characteristic resistance to normal force $N_{Rk} = A \times f_y = \mathbf{2381}$ kN

Characteristic moment resistance - Major axis $M_{y,Rk} = W_{pl.y} \times f_y = \mathbf{144.9}$ kNm

Characteristic moment resistance - Minor axis $M_{z,Rk} = W_{pl.z} \times f_y = \mathbf{86.9}$ kNm

ψ_y = if(abs($M_{y,Ed1}$)<=abs($M_{y,Ed2}$), $M_{y,Ed1}$ / if($M_{y,Ed2}$>=0 kNm,max($M_{y,Ed2}$,0.0001 kNm),$M_{y,Ed2}$), $M_{y,Ed2}$ / if($M_{y,Ed1}$>=0 kNm,max($M_{y,Ed1}$,0.0001 kNm),$M_{y,Ed1}$)) = **0.000**

Moment distribution factor - Major axis $\psi_y = M_{y,Ed1} / M_{y,Ed2} = \mathbf{0.000}$

Moment factor - Major axis $C_{my} = \max(0.4, 0.6 + 0.4 \times \psi_y) = \mathbf{0.600}$

Moment distribution factor - Minor axis $\psi_z = M_{z,Ed2} / M_{z,Ed1} = \mathbf{0.087}$

Moment factor - Minor axis $C_{mz} = \max(0.4, 0.6 + 0.4 \times \psi_z) = \mathbf{0.635}$

Moment distribution factor for LTB $\psi_{LT} = M_{y,Ed1} / M_{y,Ed2} = \mathbf{0.000}$

Moment factor for LTB $C_{mLT} = \max(0.4, 0.6 + 0.4 \times \psi_{LT}) = \mathbf{0.600}$

Interaction factor k_{yy} $k_{yy} = C_{my} \times [1 + \min(0.8, \bar{\lambda}_y - 0.2) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1})] = \mathbf{0.616}$

Interaction factor k_{zy} $k_{zy} = 1 - \min(0.1, 0.1 \times \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times (\chi_z \times N_{Rk}/\gamma_{M1})) = \mathbf{0.972}$

Interaction factor k_{zz} $k_{zz} = C_{mz} \times [1 + \min(0.8, \bar{\lambda}_z - 0.2) \times N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1})] = \mathbf{0.685}$

Interaction factor k_{yz} $k_{yz} = 0.6 \times k_{zz} = \mathbf{0.411}$

Section utilisation $UR_{B\_1} = N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yz} \times M_{z,Ed} / (M_{z,Rk} / \gamma_{M1}) = \mathbf{0.181}$

$UR_{B\_2} = N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zz} \times M_{z,Ed} / (M_{z,Rk} / \gamma_{M1}) = \mathbf{0.280}$

PASS - The buckling resistance is adequate

	Project				Job no.	
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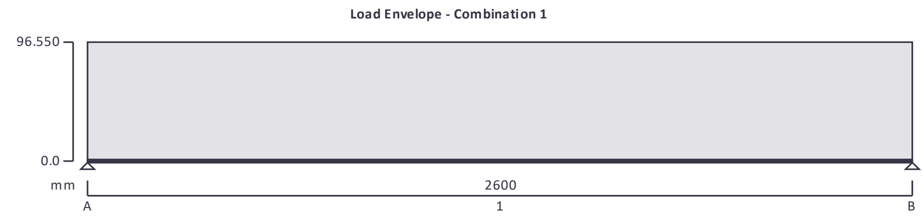
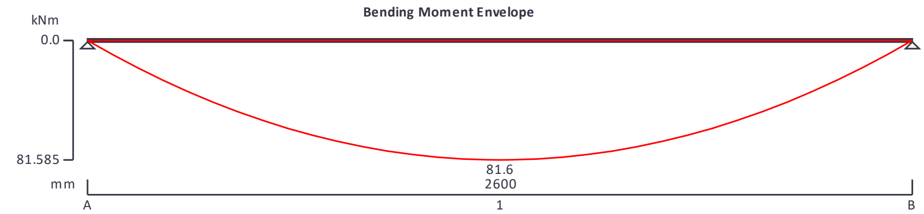
	Calcs for				Start page no./Revision	
	1F-1SB01				1	
	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
	SB	27/08/2021	DS		DS	

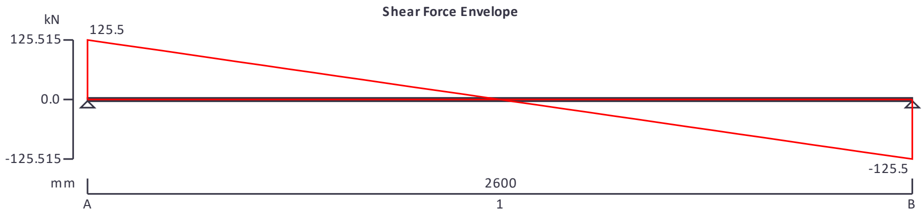
<u>STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)</u>

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
	330mm Masonry. 6.5KN/m2 x 5.8m - Permanent full UDL 38 kN/m
	Roof. 1.3KN/m2 x 7.8m/2 - Permanent full UDL 5 kN/m
	Roof. 0.6KN/m2 x 7.8m/2 - Variable full UDL 2.3 kN/m
	2nd floor. 1KN/m2 x 5m/2 - Permanent full UDL 2.5 kN/m
	2nd floor + partitions. 2.7KN/m2 x 5m/2 - Variable full UDL 6.7 kN/m
	1st floor. 1KN/m2 x 7.8m/2 - Permanent full UDL 3.9 kN/m
	1st floor + partitions. 2.7KN/m2 x 7.8m/2 - Variable full UDL 10.5 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} = **81.6** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **125.5** kN	V_{min} = **-125.5** kN
Deflection	δ_{max} = **1.2** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **125.5** kN	R_{A_min} = **125.5** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **64.8** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **25.4** kN	
Maximum reaction at support B	R_{B_max} = **125.5** kN	R_{B_min} = **125.5** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **64.8** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **25.4** kN	

Section details

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

Steel grade — **S355**

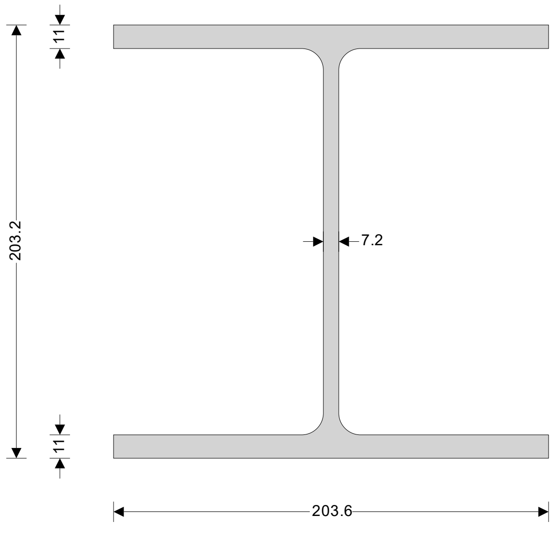
EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element — $t = \max(t_f, t_w)$ = **11.0** mm

Nominal yield strength — f_y = **355** N/mm^2

Nominal ultimate tensile strength — f_u = **470** 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 lateral restraint at supports only

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

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]}$ = **0.81**

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

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

$c / t_w = 27.4 \times \varepsilon <= 72 \times \varepsilon$ — Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

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

$c / t_f = 9.8 \times \varepsilon <= 10 \times \varepsilon$ — Class 2

Section is class 2

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 — $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **125.5** 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) — $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0}$ = **347.9** kN

PASS - Design shear resistance exceeds design shear force

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

Design bending moment — $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **81.6** kNm

Design bending resistance moment - eq 6.13 — $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **176.6** kNm

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 — k_c = **0.94**

$C_1 = 1 / k_c^2$ = **1.132**

Curvature factor — $g = \sqrt{[1 - (I_z / I_y)]}$ = **0.813**

Poissons ratio — ν = **0.3**

Shear modulus — $G = E / [2 \times (1 + \nu)]$ = **80769** N/mm^2

Unrestrained length — $L = 1.0 \times L_{s1}$ = **2600** mm

Elastic critical buckling moment — $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]}$ = **753.5** kNm

Slenderness ratio for lateral torsional buckling — $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **0.484**

Limiting slenderness ratio — $\bar{\lambda}_{LT,0}$ = **0.4**

$\bar{\lambda}_{LT} > \bar{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5 — b

Imperfection factor - Table 6.3 — α_{LT} = **0.34**

Correction factor for rolled sections — β = **0.75**

LTB reduction determination factor — $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **0.602**

LTB reduction factor - eq 6.57 — $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.967**

Modification factor $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = \mathbf{0.976}$

Modified LTB reduction factor - eq 6.58 $\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = \mathbf{0.990}$

Design buckling resistance moment - eq 6.55 $M_{b,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1} = \mathbf{174.9}$ kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to variable loads

Limiting deflection $\delta_{lim} = L_{s1} / 360 = \mathbf{7.2}$ mm

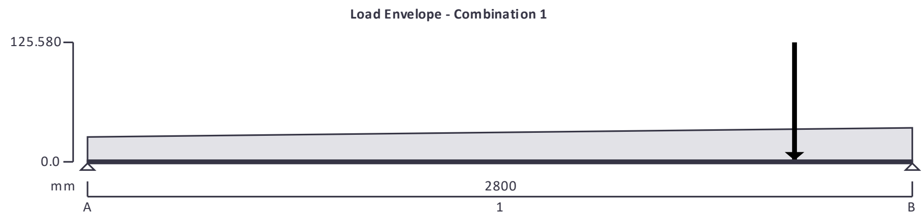
Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = \mathbf{1.21}$ mm

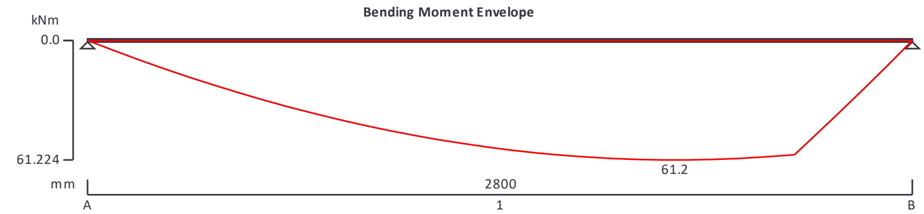
PASS - Maximum deflection does not exceed deflection limit

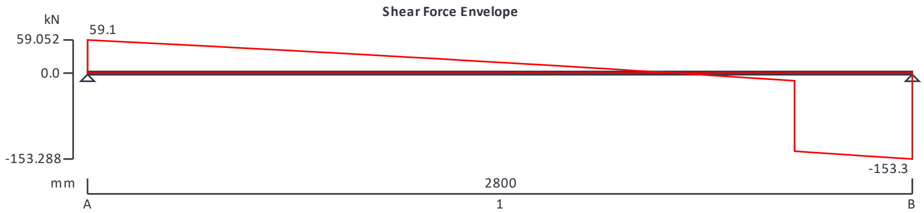
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

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
	225mm Masonry. 4.5KN/m2 x 4.2m to 5.8m - Permanent full VDL 18.9 kN/m to 26.1 kN/m
	1F-1SB01 - Permanent point load 64.8 kN at 2400 mm
	1F-1SB01 - Variable point load 25.4 kN at 2400 mm

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} = **61.2** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **59.1** kN	V_{min} = **-153.3** kN
Deflection	δ_{max} = **0.5** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **59.1** kN	R_{A_min} = **59.1** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **39.7** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **3.6** kN	
Maximum reaction at support B	R_{B_max} = **153.3** kN	R_{B_min} = **153.3** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **89.4** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **21.8** kN	

Section details

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

Steel grade — **S355**

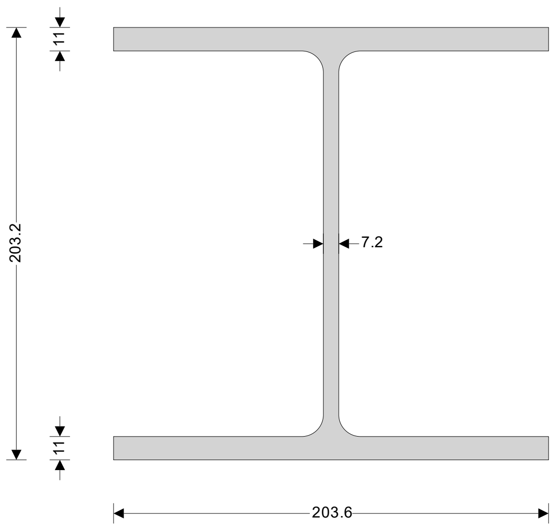
EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element — $t = \max(t_f, t_w)$ = **11.0** mm

Nominal yield strength — f_y = **355** N/mm^2

Nominal ultimate tensile strength — f_u = **470** 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 lateral restraint at supports only

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

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]}$ = **0.81**

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

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

$c / t_w = 27.4 \times \varepsilon <= 72 \times \varepsilon$ — Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

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

$c / t_f = 9.8 \times \varepsilon <= 10 \times \varepsilon$ — Class 2

Section is class 2

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 — $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **153.3** 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) — $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0}$ = **347.9** kN

PASS - Design shear resistance exceeds design shear force

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

Design bending moment — $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **61.2** kNm

Design bending resistance moment - eq 6.13 — $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **176.6** kNm

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 — k_c = **0.94**

$C_1 = 1 / k_c^2$ = **1.132**

Curvature factor — $g = \sqrt{[1 - (I_z / I_y)]}$ = **0.813**

Poissons ratio — ν = **0.3**

Shear modulus — $G = E / [2 \times (1 + \nu)]$ = **80769** N/mm^2

Unrestrained length — $L = 1.0 \times L_{s1}$ = **2800** mm

Elastic critical buckling moment — $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]}$ = **664.6** kNm

Slenderness ratio for lateral torsional buckling — $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **0.515**

Limiting slenderness ratio — $\bar{\lambda}_{LT,0}$ = **0.4**

$\bar{\lambda}_{LT}$ > $\bar{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5 — b

Imperfection factor - Table 6.3 — α_{LT} = **0.34**

Correction factor for rolled sections — β = **0.75**

LTB reduction determination factor — $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **0.619**

LTB reduction factor - eq 6.57 — $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.954**

Modification factor — $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.975**

Modified LTB reduction factor - eq 6.58 — $\chi_{LT,mod} = \min(\chi_{LT} / f, 1)$ = **0.978**

Design buckling resistance moment - eq 6.55 — $M_{b,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1}$ = **172.8** kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to variable loads

Limiting deflection $\delta_{lim} = L_{s1} / 360 =$ **7.8** mm

Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) =$ **0.516** mm

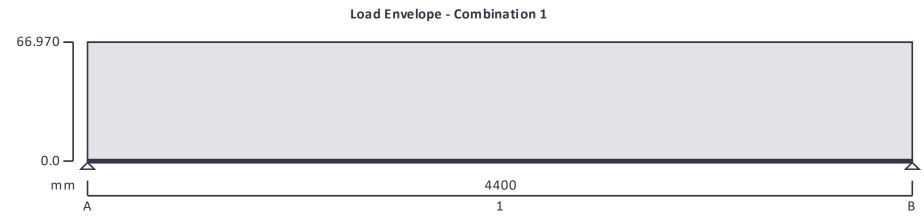
PASS - Maximum deflection does not exceed deflection limit

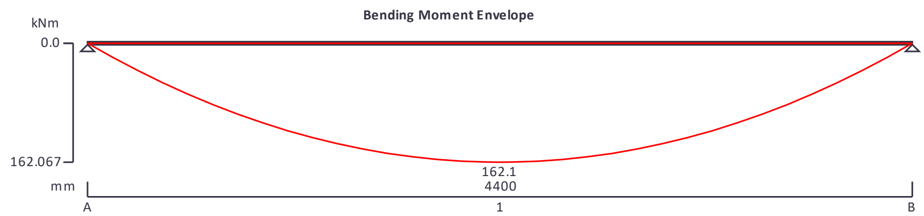
Symmetrys Unit 6 The Courtyard, Lynton Road London N8 8SL	Project 43a REDINGTON ROAD, LONDON NW3 7RA				Job no. 21141	
	Calcs for 1F-1SB03				Start page no./Revision 1	
	Calcs by SB	Calcs date 27/08/2021	Checked by DS	Checked date	Approved by DS	Approved date

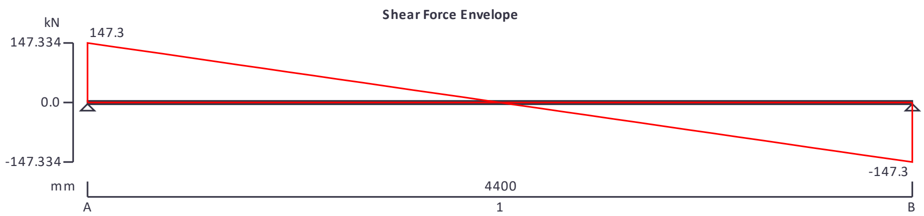
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

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 × 1 102.5mm Masonry. 2.2KN/m2 x 7m - Permanent full UDL 15.4 kN/m 2nd floor. 1KN/m2 x 8.4m/2 - Permanent full UDL 4.2 kN/m 2nd floor + partitions. 2.7KN/m2 x 8.4m/2 - Variable full UDL 11.3 kN/m 1st floor. 1KN/m2 x 8.4m/2 - Permanent full UDL 4.2 kN/m 1st floor + partitions. 2.7KN/m2 x 8.4m/2 - Variable full UDL 11.3 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

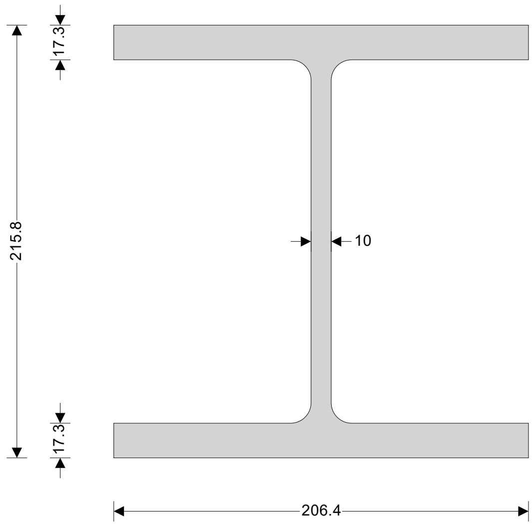
Maximum moment	M_{max} = **162.1** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **147.3** kN	V_{min} = **-147.3** kN
Deflection	δ_{max} = **14.4** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **147.3** kN	R_{A_min} = **147.3** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **53.9** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **49.7** kN	
Maximum reaction at support B	R_{B_max} = **147.3** kN	R_{B_min} = **147.3** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **53.9** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **49.7** kN	

Section details

Section type	**UC 203x203x71 (BS4-1)**
Steel grade	**S355**

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

Nominal thickness of element	$t = \max(t_f, t_w)$ = **17.3** mm
Nominal yield strength	f_y = **345** N/mm^2
Nominal ultimate tensile strength	f_u = **470** 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 lateral restraint at supports only

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

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]}$ = **0.83**

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

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

$c / t_w = 19.5 \times \varepsilon <= 72 \times \varepsilon$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

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

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

Section is class 1

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f$ = **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 $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **147.3** kN

Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **2427** mm²

Design shear resistance - cl 6.2.6(2) $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0}$ = **483.5** kN

PASS - Design shear resistance exceeds design shear force

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

Design bending moment $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **162.1** kNm

Design bending resistance moment - eq 6.13 $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **275.6** kNm

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 k_c = **0.94**

$C_1 = 1 / k_c^2$ = **1.132**

Curvature factor $g = \sqrt{[1 - (I_z / I_y)]}$ = **0.817**

Poissons ratio ν = **0.3**

Shear modulus $G = E / [2 \times (1 + \nu)]$ = **80769** N/mm²

Unrestrained length $L = 1.0 \times L_{s1}$ = **4400** mm

Elastic critical buckling moment $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]}$ = **691.2** kNm

Slenderness ratio for lateral torsional buckling $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **0.631**

Limiting slenderness ratio $\bar{\lambda}_{LT,0}$ = **0.4**

$\bar{\lambda}_{LT} > \bar{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5 b

Imperfection factor - Table 6.3 α_{LT} = **0.34**

Correction factor for rolled sections β = **0.75**

LTB reduction determination factor $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **0.689**

LTB reduction factor - eq 6.57 $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.903**

Modification factor $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.972**

Modified LTB reduction factor - eq 6.58 $\chi_{LT,mod} = \min(\chi_{LT} / f, 1)$ = **0.929**

Design buckling resistance moment - eq 6.55 $M_{b,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1}$ = **256** kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection $\delta_{lim} = L_{s1} / 250 =$ **17.6** mm

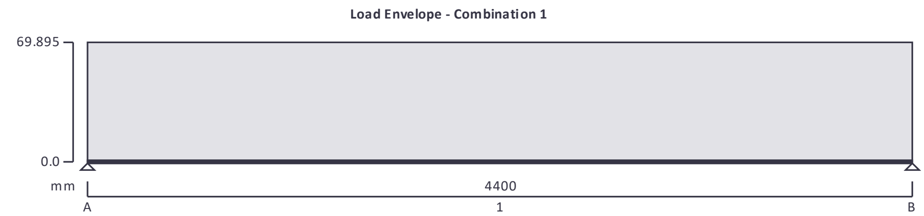
Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) =$ **14.367** mm

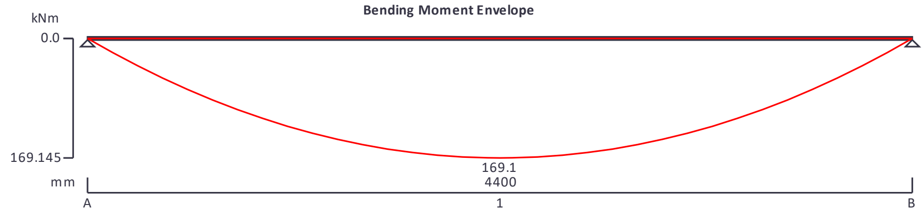
PASS - Maximum deflection does not exceed deflection limit

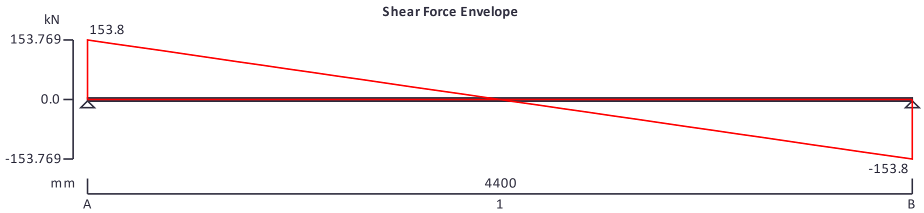
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

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
	102.5mm Masonry. 2.2KN/m2 x 7m - Permanent full UDL 15.4 kN/m
	2nd floor. 1KN/m2 x 9.5m/2 - Permanent full UDL 4.7 kN/m
	2nd floor + partitions. 2.7KN/m2 x 9.5m/2 - Variable full UDL 12.8 kN/m
	1st floor. 1KN/m2 x 8.4m/2 - Permanent full UDL 4.2 kN/m
	1st floor + partitions. 2.7KN/m2 x 8.4m/2 - Variable full UDL 11.3 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

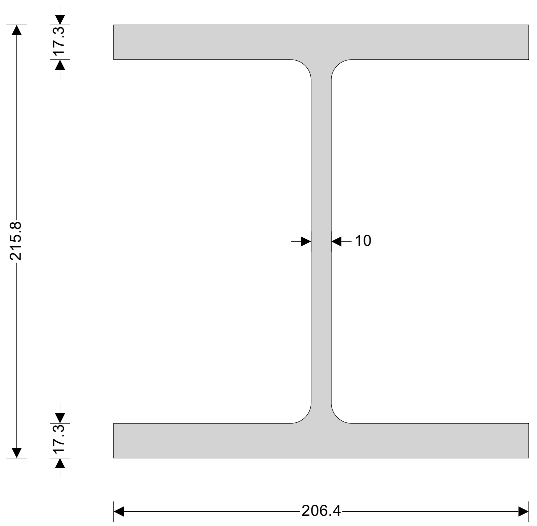
Maximum moment	M_{max} = **169.1** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **153.8** kN	V_{min} = **-153.8** kN
Deflection	δ_{max} = **15** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **153.8** kN	R_{A_min} = **153.8** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **55** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **53** kN	
Maximum reaction at support B	R_{B_max} = **153.8** kN	R_{B_min} = **153.8** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **55** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **53** kN	

Section details

Section type	**UC 203x203x71 (BS4-1)**
Steel grade	**S355**

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

Nominal thickness of element	$t = \max(t_f, t_w)$ = **17.3** mm
Nominal yield strength	f_y = **345** N/mm^2
Nominal ultimate tensile strength	f_u = **470** 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 lateral restraint at supports only

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

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]}$ = **0.83**

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

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

$c / t_w = 19.5 \times \varepsilon <= 72 \times \varepsilon$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

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

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

Section is class 1

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f$ = **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 $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **153.8** kN

Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **2427** mm^2

Design shear resistance - cl 6.2.6(2) $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0}$ = **483.5** kN

PASS - Design shear resistance exceeds design shear force

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

Design bending moment $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **169.1** kNm

Design bending resistance moment - eq 6.13 $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **275.6** kNm

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 k_c = **0.94**

$C_1 = 1 / k_c^2$ = **1.132**

Curvature factor $g = \sqrt{[1 - (I_z / I_y)]}$ = **0.817**

Poissons ratio ν = **0.3**

Shear modulus $G = E / [2 \times (1 + \nu)]$ = **80769** N/mm^2

Unrestrained length $L = 1.0 \times L_{s1}$ = **4400** mm

Elastic critical buckling moment $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]}$ = **691.2** kNm

Slenderness ratio for lateral torsional buckling $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **0.631**

Limiting slenderness ratio $\bar{\lambda}_{LT,0}$ = **0.4**

$\bar{\lambda}_{LT} > \bar{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5 b

Imperfection factor - Table 6.3 α_{LT} = **0.34**

Correction factor for rolled sections β = **0.75**

LTB reduction determination factor $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **0.689**

LTB reduction factor - eq 6.57 $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.903**

Modification factor $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.972**

Modified LTB reduction factor - eq 6.58 $\chi_{LT,mod} = \min(\chi_{LT} / f, 1)$ = **0.929**

Design buckling resistance moment - eq 6.55 $M_{b,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1}$ = **256** kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection $\delta_{lim} = L_{s1} / 250 =$ **17.6** mm

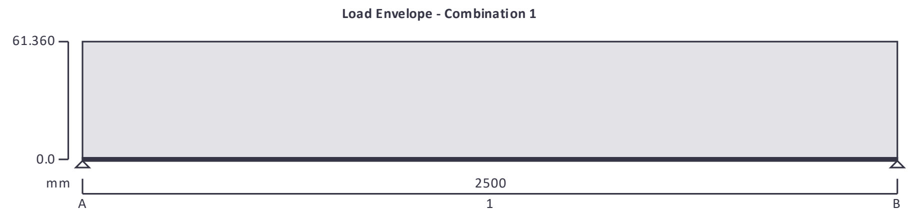
Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) =$ **14.977** mm

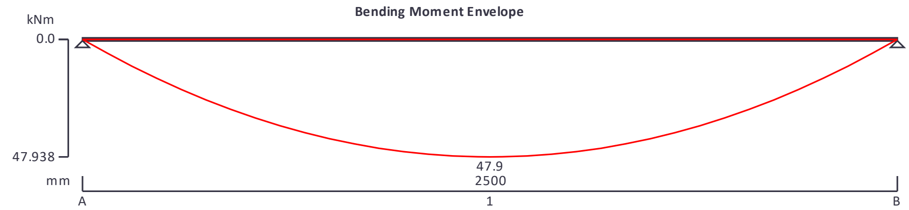
PASS - Maximum deflection does not exceed deflection limit

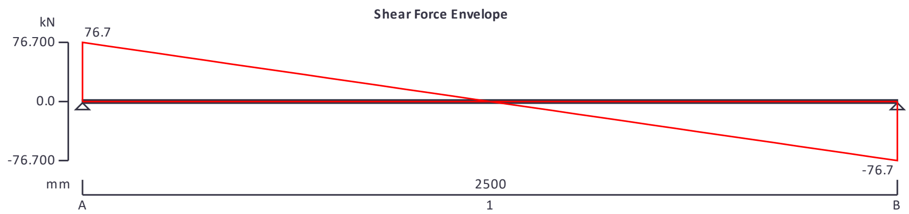
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

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
	330mm Masonry. 6.5N/m2 x 6m - Permanent full UDL 39 kN/m
	Ground floor. 1KN/m2 x 1.5m/2 - Permanent full UDL 1.5 kN/m
	1st floor + partitions. 2.7KN/m2 x 1.5m/2 - Variable full UDL 4.05 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

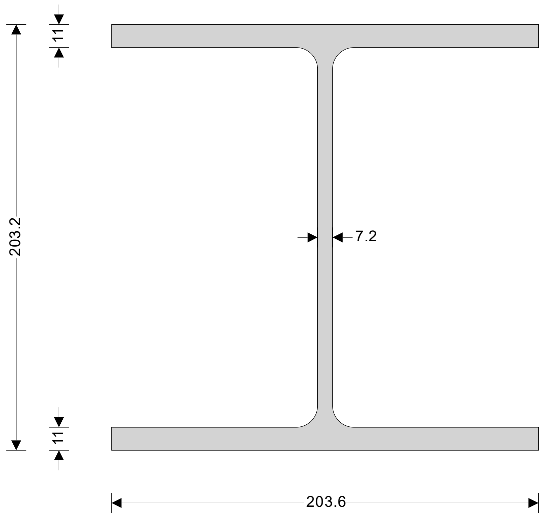
Maximum moment	M_{max} = **47.9** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **76.7** kN	V_{min} = **-76.7** kN
Deflection	δ_{max} = **2.4** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **76.7** kN	R_{A_min} = **76.7** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **51.2** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **5.1** kN	
Maximum reaction at support B	R_{B_max} = **76.7** kN	R_{B_min} = **76.7** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **51.2** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **5.1** kN	

Section details

Section type	**UC 203x203x46 (BS4-1)**
Steel grade	**S355**

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 = **355** N/mm^2
Nominal ultimate tensile strength	f_u = **470** 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 lateral restraint at supports only

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

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]}$ = **0.81**

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

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

$c / t_w = 27.4 \times \varepsilon <= 72 \times \varepsilon$ — Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

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

$c / t_f = 9.8 \times \varepsilon <= 10 \times \varepsilon$ — Class 2

Section is class 2

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 — $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **76.7** 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) — $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0}$ = **347.9** kN

PASS - Design shear resistance exceeds design shear force

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

Design bending moment — $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **47.9** kNm

Design bending resistance moment - eq 6.13 — $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **176.6** kNm

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 — k_c = **0.94**

$C_1 = 1 / k_c^2$ = **1.132**

Curvature factor — $g = \sqrt{[1 - (I_z / I_y)]}$ = **0.813**

Poissons ratio — ν = **0.3**

Shear modulus — $G = E / [2 \times (1 + \nu)]$ = **80769** N/mm^2

Unrestrained length — $L = 1.0 \times L_{s1}$ = **2500** mm

Elastic critical buckling moment — $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]}$ = **806** kNm

Slenderness ratio for lateral torsional buckling — $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **0.468**

Limiting slenderness ratio — $\bar{\lambda}_{LT,0}$ = **0.4**

$\bar{\lambda}_{LT}$ > $\bar{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5 — b

Imperfection factor - Table 6.3 — α_{LT} = **0.34**

Correction factor for rolled sections — β = **0.75**

LTB reduction determination factor — $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **0.594**

LTB reduction factor - eq 6.57 — $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.973**

Modification factor — $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.977**

Modified LTB reduction factor - eq 6.58 — $\chi_{LT,mod} = \min(\chi_{LT} / f, 1)$ = **0.996**

Design buckling resistance moment - eq 6.55 — $M_{b,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1}$ = **176** kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection $\delta_{lim} = L_{s1} / 250 =$ **10** mm

Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) =$ **2.386** mm

PASS - Maximum deflection does not exceed deflection limit

CONTINUE THE CONVERSATION

**TO DISCUSS YOUR PROJECT WITH US
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