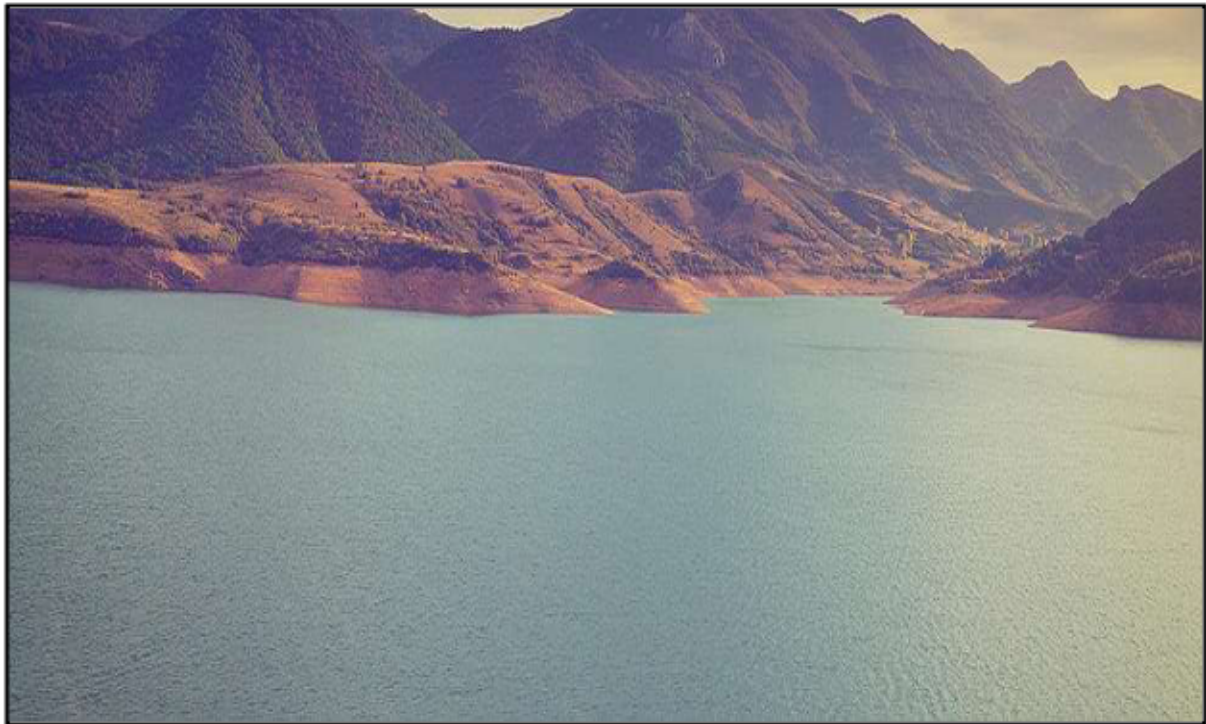




PACA GEOTECHNICAL ENGINEERING LTD

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Email: paolopitasi07@gmail.com

Description	Structural calculation of a basement and a rear house extension.
Site	8 Hughes Road, Hayes, UB3 3AN
Reference	A496.20 rev n.1
Status	Final
Client	CONSTRUCTION SERVICES MANAGEMENT LIMITED
Issue date	28/02/2020

Prepared and Reviewed by

Name	Paolo Pitasi <i>Paolo Pitasi</i>
Position	Chartered Engineer and BGA membership;
Date	10/03/2020



- **General construction notes.**

The temporary stability of the structure is the responsibility of the builder during all stages of the construction.

These calculations are for building regulations purpose and do not constitute full working drawings /details for the project.

All beam lengths used in these calculations are design lengths and are not to be used for ordering. All beams /joists lengths are to be taken to suit the site dimensions.

Foundation concrete to be minimum C20 mix in accordance with Building Regulations Approved Document A.

These calculations have been prepared based on architects plans and sections.

Any connections or details shown are for guidance only. Final connection details to be designed

steel by Fabricator based on the reaction given in these calculations, but to suit the site dimensions and relative levels that will be required during the construction stage.

Approval of these calculations and drawings by the Local Authority Building Control should be obtained prior to any ordering of material or fabrication. No liability is accepted for any changes that may be required as a result of work having commenced prior to such an approval having been obtained.

- **Introduction.**

The purpose of this report is to calculate the structure elements of a basement and a rear house extension.

This report was commissioned by client: CONSTRUCTION SERVICES MANAGEMENT LIMITED.

The plans and sections which will be examined in this structural calculation are represented in the structural drawings at the end of this report.

- **House extension calculation.**

- **Calculation of the timber rafters n.1;**

The timber rafters are simply propped at each end.

Analysis load.

The total unfactored uniform dead load acting on 1 m²;

$w_T = 1.1 \text{ KN/m}^2$ comprising the snow load .

Being the centre distance between the rafters 0.4m:

The Uniformly distributed load/rafters are $q_T = \text{rafters spacing} \times \text{load} = 0.4 * w_T = 0.44 \text{ kN/m}$;

The vertical component of the dead load acting on a timber rafter:

$Q_{\text{vert}} = 0.44 * \cos 7^\circ = 0.33 \text{ KN/m}$

The total unfactored uniform imposed load acting on 1m

$W_t = 0.75 \text{ KN/m}^2$

Being the centre distance between the rafters 0.4m:

The Uniformly distributed load/rafters are $q_T = \text{rafters spacing} \times \text{load} = 0.4 * w_T = 0.3 \text{ kN/m}$;

The vertical component of imposed load acting on a timber rafter:

$Q_{\text{vert}} = 0.3 * \cos 7^\circ = 0.226 \text{ KN/m}$

The total inclined length is $l = 2.8 \text{ m}$

The analysis performed by the software Tedds 2107 will be attached at the end of this report.

- **Calculation of the timber rafters n.2**

The timber rafters are simply propped at each end.

Analysis load.

The total unfactored uniform dead load acting on 1 m²;

$w_T = 1.1 \text{ KN/m}^2$ comprising the snow load .

Being the centre distance between the rafters 0.4m:

The Uniformly distributed load/rafters are $q_T = \text{rafters spacing} \times \text{load} = 0.4 * w_T = 0.44 \text{ kN/m}$;

The vertical component of the dead load acting on a timber rafter:

$$Q_{\text{vert}} = 0.44 * \cos 7^\circ = 0.33 \text{ KN/m}$$

The total unfactored uniform imposed load acting on 1m

$$W_t = 0.75 \text{ KN/m}^2$$

Being the centre distance between the rafters 0.4m:

The Uniformly distributed load/rafters are $q_T = \text{rafters spacing} \times \text{load} = 0.4 * w_T = 0.3 \text{ kN/m}$;

The vertical component of imposed load acting on a timber rafter:

$$Q_{\text{vert}} = 0.3 * \cos 7^\circ = 0.226 \text{ KN/m}$$

The total inclined length is $l = 4.5 \text{ m}$

The analysis performed by the software Tedds 2107 will be attached at the end of this report.

- **Calculation of the steel beam n.1a;**

The steel beam is simply propped at each end.

Analysis load.

The reaction forces of the timber rafters n.1

$$\text{Dead} = 0.5 * 2.5 = 1.25 \text{ KN/m}$$

$$\text{Imposed} = 0.36 * 2.5 = 0.9 \text{ KN/m}$$

The reaction forces of the timber rafters n.2

$$\text{Dead} = 0.94 * 2.5 = 2.35 \text{ KN/m}$$

$$\text{Imposed} = 0.6 * 2.5 = 1.5 \text{ KN/m}$$

The reaction forces of the skylight

$$\text{Dead} = 2.5 \text{ KN}$$

$$\text{Imposed} = 2.5 \text{ KN}$$

The total inclined length is $l = 5.2 \text{ m}$

The analysis performed by the software Tedds 2107 will be attached at the end of this report.

- **Calculation of the concrete padstone type 100*102*440mm supporting the steel beam N.1a at each end.**

Ultimate Beam End Reaction = 20.3KN;

Calculation of the characteristic strength of the masonry carried out in line with the BS 5628:

The design compressive strength of masonry is given by:

$$\text{Design compressive strength} = \frac{\beta f_k}{\gamma_m}$$

f_k = characteristic compressive strength of masonry formed by concrete block 7.3N

$$= 3.2 \text{ N/mm}^2$$

γ_m = partial safety factor for materials subject to compression = 2.5

β =capacity reduction factor obtained by Table 7, BS 5628 =8

SR= Slenderness ratio (SR)=12.5

$$SR = \frac{\text{effective height}}{\text{effective thickness}} = \frac{h_{ef}}{t_{ef}}$$

Where $h_{ef} = 0.75 \cdot h = 0.75 \cdot 5 = 3.75\text{m}$ and t_{ef} =thickness of the wall=0.25m

Eccentricity of the load at the top of wall, e_x , =0

The design compressive strength = $1 \cdot 3.2 / 2.5 = 1.08\text{N/mm}^2$

Width of beams end bearing =152mm;

Length of beams end bearing =100mm

Ultimate bearing stress acting on the top of the padstone $q = 20.3 \cdot 10^3 / (152 \cdot 100) = 1.33\text{N/mm}^2$;

Characteristics of the padstone:

Padstone nominal size formed by concrete padstone $100 \cdot 102 \cdot 440\text{mm}$

Concrete strength more than 50N/mm^2 at 28 days;

Design compressive strength of the padstone: $\sigma_y = 50 \cdot 0.45 = 22.5\text{N/mm}^2$;

The ultimate bearing stress acting on the top of the padstone $q = 1.33\text{N/mm}^2 < \text{design}$

compressive strength of the padstone= 22.5N/mm^2 **OK PASSED**

Stress under padstone= $20.3 \cdot 10^3 / (100 \cdot 440) = 0.46\text{N/mm}^2 < \text{Characteristic strength of the masonry} = 1.08\text{N/mm}^2$ Therefore masonry stress **OK PASSED**;

The padstone will be laid into the bearing wall with class (ii) mortar.

The builder will be responsible for all temporary supports and is to ensure that the steel beams are adequately supported during the works.

Any gap under the beams and padstone should be packed with mortar or steel and not slate or wood.

- **Calculation of the Catnic lintel n.1**

The lintel is subjected to:

The reaction forces of the timber rafters n.2

Dead= $0.94 \cdot 2.5 = 2.35\text{KN/m}$

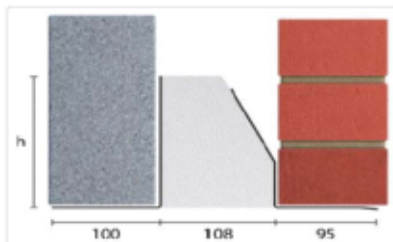
Imposed= $0.6 \cdot 2.5 = 1.5\text{KN/m}$

The weight of the wall over it: 1.0KN/m

The total length is $l = 2.4\text{m}$

The total ultimate load is 18KN

For which the suitable Catnic lintel is:



Standard Lengths (mm)	SWL (kN) 1:1/3:1	Weight (kg/m)	Nominal height 'h' (mm)
750-1500	15	8.0	153
1650-1800	18	11.9	202
1950-2100	20	11.9	202
2250-2400	22	15.8	233
2550-2700	26	15.8	233
2850-3600	26	16.9	229 (channel to inner leaf)

For which the suitable lintels is standard duty.

- **Calculation of the timber beam n.1b;**

The timber rafters are simply propped at each end.

Analysis load.

The reaction forces of the timber rafters n.1

Dead= $0.5 \times 2.5 = 1.25 \text{ kN/m}$

Imposed= $0.36 \times 2.5 = 0.9 \text{ kN/m}$

The total inclined length is $l = 1.50 \text{ m}$

The analysis performed by the software Tedds 2107 will be attached at the end of this report.

The timber beam n.3a is anchored at each end to the external bearing wall by n.2 bolts M8 grade 8.8 that are placed outside of the openings.

- **Basement calculation.**

- **Structural calculation of the basement roof and top floor.**

The top floor of the basement will be formed by an unpropped concrete metal deck floor placed on the top of the reinforced concrete basement walls and steel beams UC for a bearing length $> 0.15 \text{ m}$;

The mentioned deck floor will be designed by the supplier with the following characteristics:

Type n.1

Unique span: 4.00 m

Type n.2

Unique span: 1.20 m

Type n.3

Unique span: 3.0 m

And to withstand the following applied load

Dead loads:

Sedum green roof : 0.9 kN/m^2

Finishes : 1 kN/m^2

Snow load = 0.75 kN/m^2 ;

Imposed domestic floor load = 1.5 kN/m^2

Total applied ultimate load = 6.11 kN/m^2

The mentioned deck floor will be designed by the supplier with the following characteristics:

Type n.4

Unique span: 3.00 m

Type n.2

Unique span: 1.50 m

And to withstand the following applied load

Dead loads:

Finishes : 1 kN/m^2

Imposed domestic floor load = 1.5 kN/m^2

Total applied ultimate load = 3.8 kN/m^2

In addition it is required

- Anti- crack Mesh: A 142;
- Slab depth = 150 mm ;
- Fire resistance = 60 minutes;
- Concrete grade: Normal Concrete (NC) Grade C30/37 with a maximum aggregate size of 20 mm .

The weight of the wet concrete to be considered in our analysis is equal 2.8 kN/m^2

- **Steel beam n.F1;**

The steel beam is propped at each end

Analysis load.

The reaction forces of the unpropped concrete metal deck slab type n.2

Dead loads:

$5.45 \times 1.2 / 2 = 3.27 \text{ kN/m}$

Imposed domestic floor load:

$1.5 \times 1.2 / 2 = 0.9 \text{ kN/m}$

The reaction forces of the skylight or other

Dead = 1.0 kN

Imposed = 1.0 kN

The total length is $l = 1.5 \text{ m}$

The analysis performed by the software Tedds 2107 will be attached at the end of this report.

- **Steel beam n.F2;**

The steel beam is propped at each end

Analysis load.

The reaction forces of the steel beam n. F1

Dead loads: 3.1 kN

Imposed domestic floor load: 1.2 kN

The reaction forces of the skylight or other

Dead=1.0KN

Imposed=1.0KN

The total length is $l=4.00\text{m}$

The analysis performed by the software Tedds 2107 will be attached at the end of this report.

- **Steel beam n.F3;**

The steel beam is propped at each end

Analysis load.

The weight of the wall over it

Dead=7.5KN/m

The reaction forces of the timber rafters n.1

Dead= $0.5 \times 2.5 = 1.25\text{KN/m}$

Imposed= $0.36 \times 2.5 = 0.9\text{KN/m}$

The total length is $l=2.95\text{m}$

The analysis performed by the software Tedds 2107 will be attached at the end of this report.

- **Steel beam n.F4;**

The steel beam is propped at each end

Analysis load.

The reaction force of the steel beam n.3

Dead=13.3KN

Imposed=1.3KN

The weight of the wall over it

Dead=7.5KN/m

The reaction forces of the unpropped concrete metal deck slab types n.4 and n.5

Dead loads:

$3.8 \times 4.45 / 2 = 8.45\text{KN/m}$

Imposed domestic floor load:

$1.5 \times 4.45 / 2 = 3.33\text{KN/m}$

The reaction forces of the unpropped concrete metal deck slab type n.3

Dead loads:

$5.45 \times 2.95 / 2 = 8.03\text{KN/m}$

Imposed domestic floor load:

$1.5 \times 2.95 / 2 = 2.21\text{KN/m}$

The total length is $l=4.5\text{m}$

The analysis performed by the software Tedds 2107 will be attached at the end of this report.

- **Steel beam n.F5;**

The steel beam is propped at each end

Analysis load.

The reaction force of the steel beam n.F4

Dead=37.9KN

Imposed=6.8KN

The weight of the wall over it

Dead=7.5KN/m

The total length is $l=4.0\text{m}$

The analysis performed by the software Tedds 2107 will be attached at the end of this report.

- **Bolts connections checks between the steel beams at the ground floor by N.6 bolts diameter 12 mm grade 8.8 and steel plates thickness 8 mm at each connection.**

The bolts are subjected to shear forces.

The following check will be carried out:

Shear and bearing check.

1. Shear capacity of the bolt,

$$P_s = p_s A_s$$

2. Bearing capacity of bolt,

$$P_{bb} = d_b t_p p_{bb}$$

3. Bearing capacity of connected part,

$$P_{bs} = k_{bs} d_b t_p p_{bs} \leq 0.5 k_{bs} e t_p p_{bs}$$

Grade bolts: 4.6 for which the p_s =shear strength of the bolts= 375 N/mm²;

A_s = effective area of bolts in shear, normally taken as the tensile stress area, A_t =84.3mm²;

For which the shear capacity of the bolts is $P_s=84.3*375*6=189\text{KN}>63.90\text{KN}$ **OK PASSED;**

The d_b =diameter bolts= 16mm; t_p =thickness of connected part=8 mm; p_{bb} =bearing strength of the connected part=1000 N/mm²;(Grade bolt: 8.8)

The bearing capacity of the bolts: $P_{bb}=8*6*12*1000=576\text{KN}>63.9\text{KN}$; **OK PASSED;**

The K_{bs} =1.0; p_{bs} = bearing strength of the connected part=1000N/mm²(Grade steel S275);
 e =end distance=35mm;

Bearing capacity of connected part: $P_{bs}=576\text{KN}<840\text{KN}>63.9\text{KN}$ **OK PASSED;**

The bolts will be not subjected to tension forces.

- **Calculation of the retaining wall of the new basement (Section n.A) in temporary conditions.**
- **Analysis of the loads;**

The analysis will be carried out on 1m of a wall length and considering the wall.

The section more loaded will be considered.

The retaining wall is subjected to:

The reaction forces of the steel beam n.F5

Dead=34.9KN

Imposed=4.7KN

The reaction forces of the unpropped concrete metal deck slab type n.1

Dead loads:

$5.45*4/2=10.9\text{KN/m}$

Imposed domestic floor load:

$$1.5 \times 4 / 2 = 3.0 \text{ kN/m}$$

Finally, it is considered :

- the active earth and the passive earth pressure;
- Surcharge $q = 2.5 \text{ kN/m}^2$;
- Height of ground water equal to 1.00m

A cautious value of the effective angle of friction equal to 32 degree will be considered in the calculation of the basement.

A cautious value of the safe bearing capacity equal to 250 kPa is considered in the calculation of the basement .

The related verification of this section subjected to the mentioned loads performed by the software Tekla is attached at the end of this report.

- **Biography.**

- BS 6399;
- BS 8110;
- BS 5950;
- BS 5628;
- Foundation design and construction Mj Tomlinson;
- Design of Structural Elements Third edition Chanakya Arya;

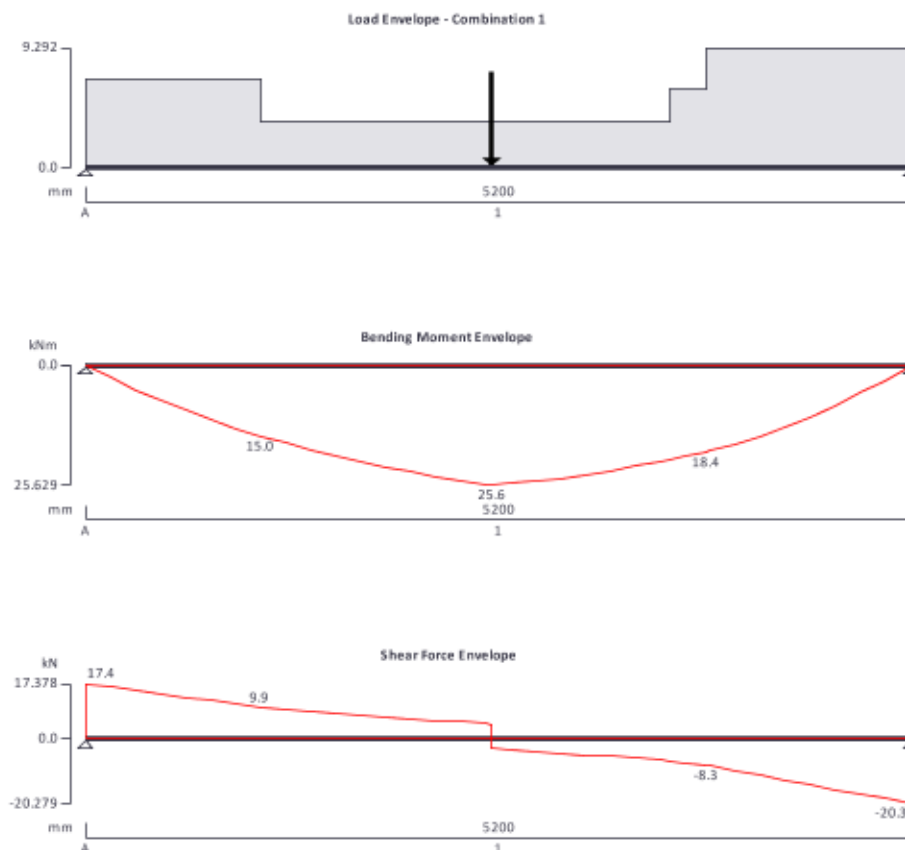
STRUCTURAL DRAWINGS
ALL THE DIMENSION MUST BE CHECKED AND
CONFIRMED ON SITE BY THE BUILDER
BEFORE STARTING THE WORKS WHO IS
RESPONSIBLE FOR
SETTING OUT THE PROPOSED WORKS.

 Paca Geotechnical engineering Ltd CM35TB	Project steel beam n.1a				Job no.	
	Calcs for				Start page no./Revision 1	
	Calcs by U	Calcs date 04/03/2020	Checked by	Checked date	Approved by	Approved date

STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.07



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Dead self weight of beam $\times 1$

Dead partial UDL 1.25 kN/m from 0 mm to 3680 mm

Imposed partial UDL 0.9 kN/m from 0 mm to 3680 mm

Dead partial UDL 2.35 kN/m from 3680 mm to 5200 mm

Imposed partial UDL 1.5 kN/m from 3680 mm to 5200 mm

Dead point load 2.5 kN at 2560 mm

Imposed point load 2.5 kN at 2560 mm

Dead partial UDL 1.25 kN/m from 0 mm to 1100 mm

Imposed partial UDL 0.9 kN/m from 0 mm to 1100 mm

Dead partial UDL 1.25 kN/m from 3910 mm to 5200 mm

Imposed partial UDL 0.9 kN/m from 3910 mm to 5200 mm

 Paca Geotechnical engineering Ltd CM35TB	Project steel beam n.1a				Job no.	
	Calcs for				Start page no./Revision 2	
	Calcs by U	Calcs date 04/03/2020	Checked by	Checked date	Approved by	Approved date

Load combinations

Load combination 1

Support A

Dead $\times 1.40$

Imposed $\times 1.60$

Dead $\times 1.40$

Imposed $\times 1.60$

Support B

Dead $\times 1.40$

Imposed $\times 1.60$

Analysis results

Maximum moment

$M_{max} = 25.6 \text{ kNm}$

$M_{min} = 0 \text{ kNm}$

Maximum shear

$V_{max} = 17.4 \text{ kN}$

$V_{min} = -20.3 \text{ kN}$

Deflection

$\delta_{max} = 5.3 \text{ mm}$

$\delta_{min} = 0 \text{ mm}$

Maximum reaction at support A

$R_{A_{max}} = 17.4 \text{ kN}$

$R_{A_{min}} = 17.4 \text{ kN}$

Unfactored dead load reaction at support A

$R_{A_{Dead}} = 7 \text{ kN}$

Unfactored imposed load reaction at support A

$R_{A_{Imposed}} = 4.8 \text{ kN}$

Maximum reaction at support B

$R_{B_{max}} = 20.3 \text{ kN}$

$R_{B_{min}} = 20.3 \text{ kN}$

Unfactored dead load reaction at support B

$R_{B_{Dead}} = 8.2 \text{ kN}$

Unfactored imposed load reaction at support B

$R_{B_{Imposed}} = 5.5 \text{ kN}$

Section details

Section type

UC 152x152x30 (BS4-1)

Steel grade

S275

From table 9: Design strength p_y

Thickness of element

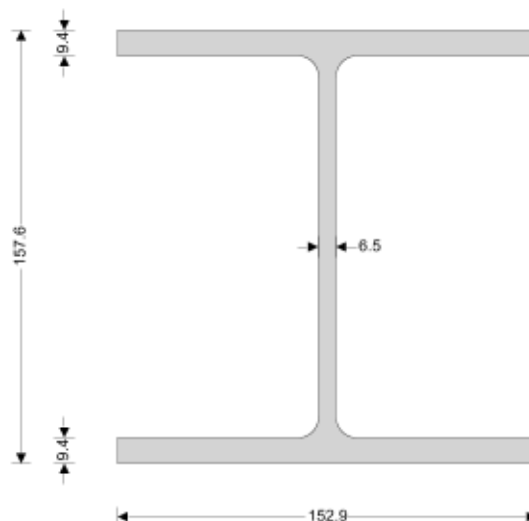
$\max(T, t) = 9.4 \text{ mm}$

Design strength

$p_y = 275 \text{ N/mm}^2$

Modulus of elasticity

$E = 205000 \text{ N/mm}^2$



Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis

$K_x = 1.00$

Effective length factor in minor axis

$K_y = 1.00$

Effective length factor for lateral-torsional buckling

$K_{LT,A} = 1.00$

$K_{LT,B} = 1.00$

 Paca Geotechnical engineering Ltd CM35TB	Project steel beam n.1a				Job no.	
	Calcs for				Start page no./Revision 3	
	Calcs by U	Calcs date 04/03/2020	Checked by	Checked date	Approved by	Approved date

Classification of cross sections - Section 3.5

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

Internal compression parts - Table 11

Depth of section

$$d = 123.6 \text{ mm}$$

$$d / t = 19.0 \times \varepsilon \leq 80 \times \varepsilon$$

Class 1 plastic

Outstand flanges - Table 11

Width of section

$$b = B / 2 = 76.5 \text{ mm}$$

$$b / T = 8.1 \times \varepsilon \leq 9 \times \varepsilon$$

Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force

$$F_v = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 20.3 \text{ kN}$$

$$d / t < 70 \times \varepsilon$$

Web does not need to be checked for shear buckling

Shear area

$$A_v = t \times D = 1024 \text{ mm}^2$$

Design shear resistance

$$P_v = 0.6 \times p_y \times A_v = 169 \text{ kN}$$

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment

$$M = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) = 25.6 \text{ kNm}$$

Moment capacity low shear - cl.4.2.5.2

$$M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = 68.1 \text{ kNm}$$

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling

$$L_E = 1.0 \times L_{s1} = 5200 \text{ mm}$$

Slenderness ratio

$$\lambda = L_E / r_{yy} = 135.869$$

Equivalent slenderness - Section 4.3.6.7

Buckling parameter

$$u = 0.849$$

Torsional index

$$x = 15.999$$

Slenderness factor

$$v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25} = 0.683$$

Ratio - cl.4.3.6.9

$$\beta_w = 1.000$$

Equivalent slenderness - cl.4.3.6.7

$$\lambda_{LT} = u \times v \times \lambda \times \sqrt{\beta_w} = 78.695$$

Limiting slenderness - Annex B.2.2

$$\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5} = 34.310$$

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

Bending strength - Section 4.3.6.5

Robertson constant

$$\alpha_{LT} = 7.0$$

Perry factor

$$\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) = 0.311$$

Euler stress

$$p_E = \pi^2 \times E / \lambda_{LT}^2 = 326.7 \text{ N/mm}^2$$

$$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 = 351.6 \text{ N/mm}^2$$

Bending strength - Annex B.2.1

$$p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) = 167.8 \text{ N/mm}^2$$

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment

$$M_2 = 16.9 \text{ kNm}$$

Moment at centre-line of segment

$$M_3 = 25.5 \text{ kNm}$$

Moment at three quarter point of segment

$$M_4 = 18.5 \text{ kNm}$$

Maximum moment in segment

$$M_{abs} = 25.6 \text{ kNm}$$

Maximum moment governing buckling resistance

$$M_{LT} = M_{abs} = 25.6 \text{ kNm}$$

Equivalent uniform moment factor for lateral-torsional buckling

$$m_{LT} = \max(0.2 + (0.15 \times M_2 + 0.5 \times M_3 + 0.15 \times M_4) / M_{abs}, 0.44) = 0.905$$

 Paca Geotechnical engineering Ltd CM35TB	Project				Job no.	
	steel beam n.1a				Start page no./Revision	
					4	
	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
	U	04/03/2020				

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment

$$M_b = p_b \times S_{xx} = 41.6 \text{ kNm}$$

$$M_b / m_{LT} = 45.9 \text{ kNm}$$

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection

$$\delta_{lim} = L_{st} / 360 = 14.444 \text{ mm}$$

Maximum deflection span 1

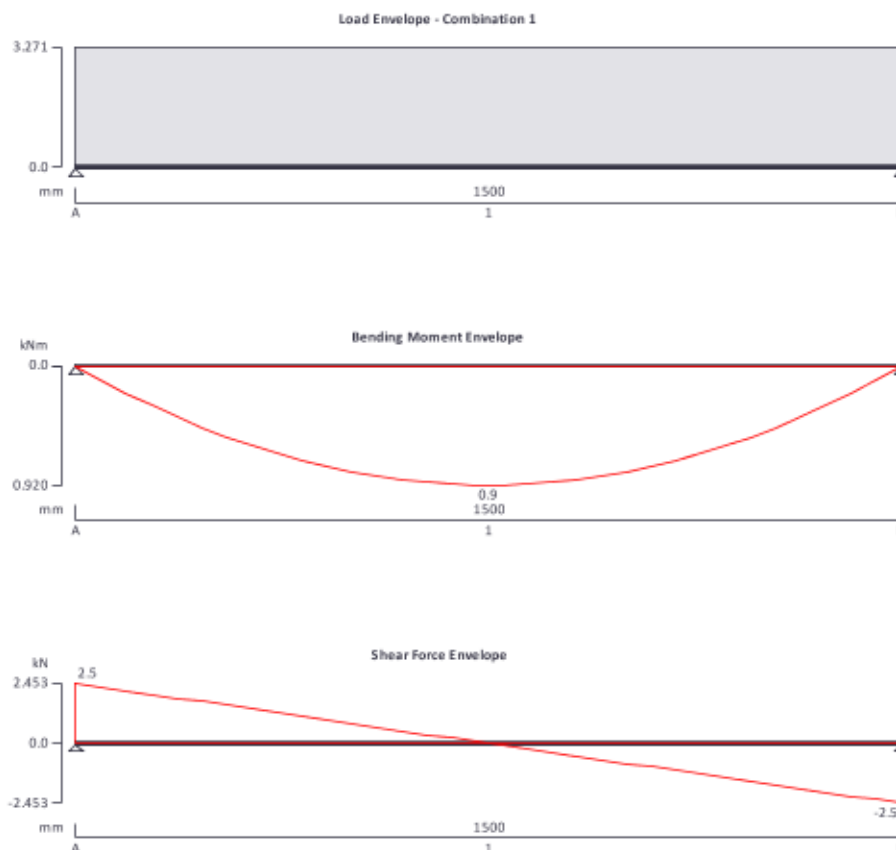
$$\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = 5.33 \text{ mm}$$

PASS - Maximum deflection does not exceed deflection limit

 Paca Geotechnical engineering Ltd CM35TB	Project timber beam n.1b				Job no.	
	Calcs for				Start page no./Revision 1	
	Calcs by U	Calcs date 04/03/2020	Checked by	Checked date	Approved by	Approved date

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.01



Applied loading

Beam loads

Dead self weight of beam $\times 1$
Dead full UDL 1.250 kN/m
Imposed full UDL 0.900 kN/m

Load combinations

Load combination 1

Support A	Dead $\times 1.40$ Imposed $\times 1.60$
Span 1	Dead $\times 1.40$ Imposed $\times 1.60$
Support B	Dead $\times 1.40$ Imposed $\times 1.60$

Analysis results

Maximum moment
Design moment
Maximum shear
Design shear
Total load on beam

$M_{\max} = 0.920 \text{ kNm}$ $M_{\min} = 0.000 \text{ kNm}$
 $M = \max(\text{abs}(M_{\max}), \text{abs}(M_{\min})) = 0.920 \text{ kNm}$
 $F_{\max} = 2.453 \text{ kN}$ $F_{\min} = -2.453 \text{ kN}$
 $F = \max(\text{abs}(F_{\max}), \text{abs}(F_{\min})) = 2.453 \text{ kN}$
 $W_{\text{tot}} = 4.907 \text{ kN}$

 Paca Geotechnical engineering Ltd CM35TB	Project timber beam n.1b				Job no.	
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Reactions at support A

$$R_{A\_max} = 2.453 \text{ kN}$$

$$R_{A\_min} = 2.453 \text{ kN}$$

Unfactored dead load reaction at support A

$$R_{A\_Dead} = 0.981 \text{ kN}$$

Unfactored imposed load reaction at support A

$$R_{A\_Imposed} = 0.675 \text{ kN}$$

Reactions at support B

$$R_{B\_max} = 2.453 \text{ kN}$$

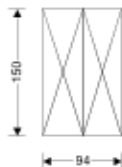
$$R_{B\_min} = 2.453 \text{ kN}$$

Unfactored dead load reaction at support B

$$R_{B\_Dead} = 0.981 \text{ kN}$$

Unfactored imposed load reaction at support B

$$R_{B\_Imposed} = 0.675 \text{ kN}$$



Timber section details

Breadth of sections

$$b = 47 \text{ mm}$$

Depth of sections

$$h = 150 \text{ mm}$$

Number of sections in member

$$N = 2$$

Overall breadth of member

$$b_b = N \times b = 94 \text{ mm}$$

Timber strength class

$$C24$$

Member details

Service class of timber

$$2$$

Load duration

$$\text{Long term}$$

Length of span

$$L_{s1} = 1500 \text{ mm}$$

Length of bearing

$$L_b = 100 \text{ mm}$$

Section properties

Cross sectional area of member

$$A = N \times b \times h = 14100 \text{ mm}^2$$

Section modulus

$$Z_x = N \times b \times h^2 / 6 = 352500 \text{ mm}^3$$

$$Z_y = h \times (N \times b)^2 / 6 = 220900 \text{ mm}^3$$

Second moment of area

$$I_x = N \times b \times h^3 / 12 = 26437500 \text{ mm}^4$$

$$I_y = h \times (N \times b)^3 / 12 = 10382300 \text{ mm}^4$$

Radius of gyration

$$i_x = \sqrt{I_x / A} = 43.3 \text{ mm}$$

$$i_y = \sqrt{I_y / A} = 27.1 \text{ mm}$$

Modification factors

Duration of loading - Table 17

$$K_3 = 1.00$$

Bearing stress - Table 18

$$K_4 = 1.00$$

Total depth of member - cl.2.10.6

$$K_7 = (300 \text{ mm} / h)^{0.11} = 1.08$$

Load sharing - cl.2.9

$$K_8 = 1.00$$

Lateral support - cl.2.10.8

Ends held in position and members held in line, as by direct connection of sheathing, deck or joists

Permissible depth-to-breadth ratio - Table 19

$$5.00$$

Actual depth-to-breadth ratio

$$h / (N \times b) = 1.60$$

PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress (no wane)

$$\sigma_{c\_adm} = \sigma_{cp1} \times K_3 \times K_4 \times K_8 = 2.400 \text{ N/mm}^2$$

Applied bearing stress

$$\sigma_{c\_a} = R_{A\_max} / (N \times b \times L_b) = 0.261 \text{ N/mm}^2$$

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$$\sigma_{c_a} / \sigma_{c_{adm}} = 0.109$$

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

Bending parallel to grain

Permissible bending stress

$$\sigma_{m_{adm}} = \sigma_m \times K_3 \times K_7 \times K_8 = 8.094 \text{ N/mm}^2$$

Applied bending stress

$$\sigma_{m_a} = M / Z_x = 2.610 \text{ N/mm}^2$$

$$\sigma_{m_a} / \sigma_{m_{adm}} = 0.322$$

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

Permissible shear stress

$$\tau_{adm} = \tau \times K_3 \times K_8 = 0.710 \text{ N/mm}^2$$

Applied shear stress

$$\tau_a = 3 \times F / (2 \times A) = 0.261 \text{ N/mm}^2$$

$$\tau_a / \tau_{adm} = 0.368$$

PASS - Applied shear stress is less than permissible shear stress

Deflection

Modulus of elasticity for deflection

$$E = E_{min} = 7200 \text{ N/mm}^2$$

Permissible deflection

$$\delta_{adm} = \min(0.551 \text{ in}, 0.003 \times L_{s1}) = 4.500 \text{ mm}$$

Bending deflection

$$\delta_{b_{s1}} = 0.765 \text{ mm}$$

Shear deflection

$$\delta_{v_{s1}} = 0.117 \text{ mm}$$

Total deflection

$$\delta_a = \delta_{b_{s1}} + \delta_{v_{s1}} = 0.882 \text{ mm}$$

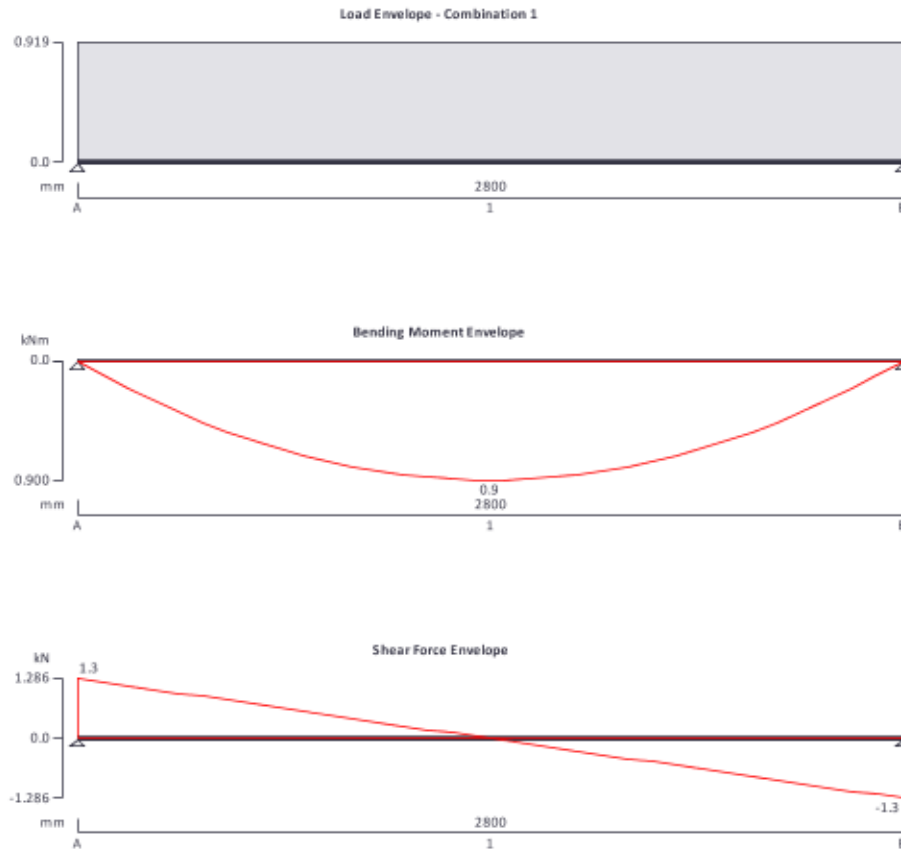
$$\delta_a / \delta_{adm} = 0.196$$

PASS - Total deflection is less than permissible deflection

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TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.01



Applied loading

Beam loads

Dead self weight of beam $\times 1$
Dead full UDL 0.330 kN/m
Imposed full UDL 0.260 kN/m

Load combinations

Load combination 1

Support A	Dead $\times 1.40$ Imposed $\times 1.60$
Span 1	Dead $\times 1.40$ Imposed $\times 1.60$
Support B	Dead $\times 1.40$ Imposed $\times 1.60$

Analysis results

Maximum moment
Design moment
Maximum shear
Design shear
Total load on beam

$M_{\max} = 0.900 \text{ kNm}$ $M_{\min} = 0.000 \text{ kNm}$
 $M = \max(\text{abs}(M_{\max}), \text{abs}(M_{\min})) = 0.900 \text{ kNm}$
 $F_{\max} = 1.286 \text{ kN}$ $F_{\min} = -1.286 \text{ kN}$
 $F = \max(\text{abs}(F_{\max}), \text{abs}(F_{\min})) = 1.286 \text{ kN}$
 $W_{\text{tot}} = 2.572 \text{ kN}$

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Reactions at support A

$$R_{A\_max} = 1.286 \text{ kN}$$

$$R_{A\_min} = 1.286 \text{ kN}$$

Unfactored dead load reaction at support A

$$R_{A\_Dead} = 0.503 \text{ kN}$$

Unfactored imposed load reaction at support A

$$R_{A\_Imposed} = 0.364 \text{ kN}$$

Reactions at support B

$$R_{B\_max} = 1.286 \text{ kN}$$

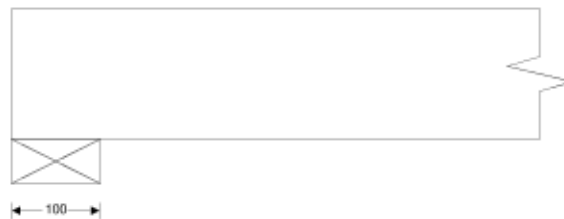
$$R_{B\_min} = 1.286 \text{ kN}$$

Unfactored dead load reaction at support B

$$R_{B\_Dead} = 0.503 \text{ kN}$$

Unfactored imposed load reaction at support B

$$R_{B\_Imposed} = 0.364 \text{ kN}$$



Timber section details

Breadth of sections

$$b = 47 \text{ mm}$$

Depth of sections

$$h = 150 \text{ mm}$$

Number of sections in member

$$N = 1$$

Overall breadth of member

$$b_b = N \times b = 47 \text{ mm}$$

Timber strength class

C24

Member details

Service class of timber

2

Load duration

Long term

Length of span

$$L_{s1} = 2800 \text{ mm}$$

Length of bearing

$$L_b = 100 \text{ mm}$$

Section properties

Cross sectional area of member

$$A = N \times b \times h = 7050 \text{ mm}^2$$

Section modulus

$$Z_x = N \times b \times h^2 / 6 = 176250 \text{ mm}^3$$

$$Z_y = h \times (N \times b)^2 / 6 = 55225 \text{ mm}^3$$

Second moment of area

$$I_x = N \times b \times h^3 / 12 = 13218750 \text{ mm}^4$$

$$I_y = h \times (N \times b)^3 / 12 = 1297787 \text{ mm}^4$$

Radius of gyration

$$i_x = \sqrt{I_x / A} = 43.3 \text{ mm}$$

$$i_y = \sqrt{I_y / A} = 13.6 \text{ mm}$$

Modification factors

Duration of loading - Table 17

$$K_3 = 1.00$$

Bearing stress - Table 18

$$K_4 = 1.00$$

Total depth of member - cl.2.10.6

$$K_7 = (300 \text{ mm} / h)^{0.11} = 1.08$$

Load sharing - cl.2.9

$$K_8 = 1.00$$

Lateral support - cl.2.10.8

Ends held in position and members held in line, as by direct connection of sheathing, deck or joists

Permissible depth-to-breadth ratio - Table 19

$$5.00$$

Actual depth-to-breadth ratio

$$h / (N \times b) = 3.19$$

PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress (no wane)

$$\sigma_{c\_adm} = \sigma_{cp1} \times K_3 \times K_4 \times K_8 = 2.400 \text{ N/mm}^2$$

Applied bearing stress

$$\sigma_{c\_a} = R_{A\_max} / (N \times b \times L_b) = 0.274 \text{ N/mm}^2$$

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$$\sigma_{c_a} / \sigma_{c_{adm}} = 0.114$$

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

Bending parallel to grain

Permissible bending stress

$$\sigma_{m_{adm}} = \sigma_m \times K_3 \times K_7 \times K_8 = 8.094 \text{ N/mm}^2$$

Applied bending stress

$$\sigma_{m_a} = M / Z_x = 5.108 \text{ N/mm}^2$$

$$\sigma_{m_a} / \sigma_{m_{adm}} = 0.631$$

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

Permissible shear stress

$$\tau_{adm} = \tau \times K_3 \times K_8 = 0.710 \text{ N/mm}^2$$

Applied shear stress

$$\tau_a = 3 \times F / (2 \times A) = 0.274 \text{ N/mm}^2$$

$$\tau_a / \tau_{adm} = 0.385$$

PASS - Applied shear stress is less than permissible shear stress

Deflection

Modulus of elasticity for deflection

$$E = E_{min} = 7200 \text{ N/mm}^2$$

Permissible deflection

$$\delta_{adm} = \min(0.551 \text{ in}, 0.003 \times L_{s1}) = 8.400 \text{ mm}$$

Bending deflection

$$\delta_{b_{s1}} = 5.206 \text{ mm}$$

Shear deflection

$$\delta_{v_{s1}} = 0.229 \text{ mm}$$

Total deflection

$$\delta_a = \delta_{b_{s1}} + \delta_{v_{s1}} = 5.435 \text{ mm}$$

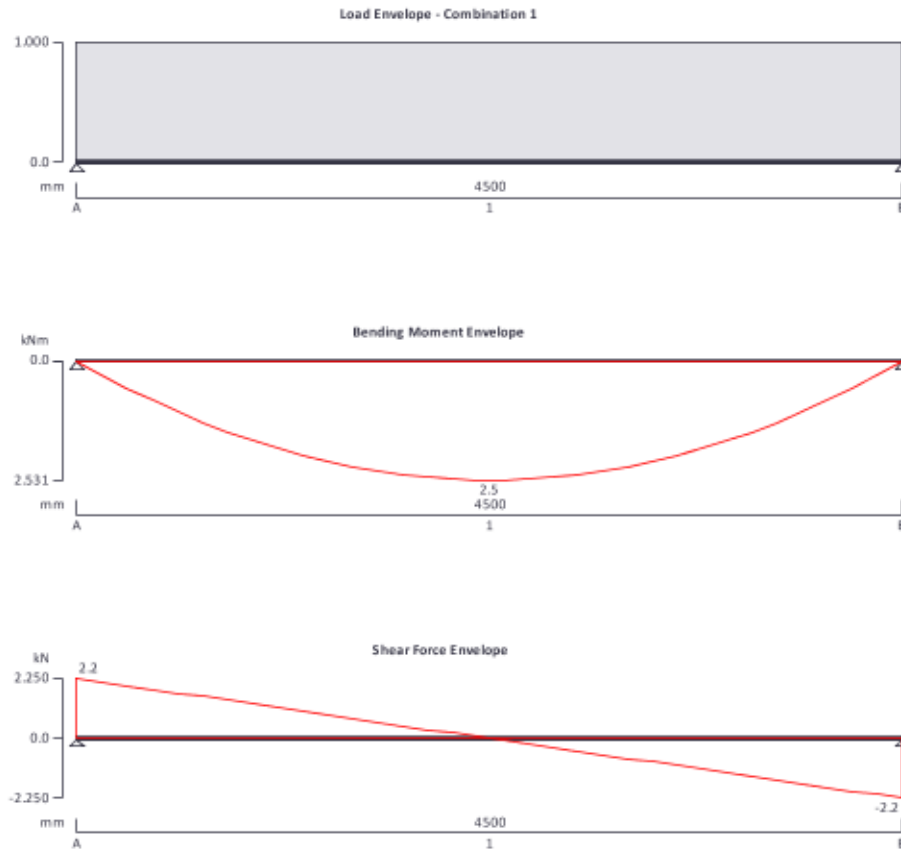
$$\delta_a / \delta_{adm} = 0.647$$

PASS - Total deflection is less than permissible deflection

 Paca Geotechnical engineering Ltd CM35TB	Project timber rafters n.2				Job no.	
	Calcs for				Start page no./Revision 1	
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TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.01



Applied loading

Beam loads

Dead self weight of beam $\times 1$
Dead full UDL 0.330 kN/m
Imposed full UDL 0.260 kN/m

Load combinations

Load combination 1

Support A	Dead $\times 1.40$ Imposed $\times 1.60$
Span 1	Dead $\times 1.40$ Imposed $\times 1.60$
Support B	Dead $\times 1.40$ Imposed $\times 1.60$

Analysis results

Maximum moment	$M_{\max} = 2.531 \text{ kNm}$	$M_{\min} = 0.000 \text{ kNm}$
Design moment	$M = \max(\text{abs}(M_{\max}), \text{abs}(M_{\min})) = 2.531 \text{ kNm}$	
Maximum shear	$F_{\max} = 2.250 \text{ kN}$	$F_{\min} = -2.250 \text{ kN}$
Design shear	$F = \max(\text{abs}(F_{\max}), \text{abs}(F_{\min})) = 2.250 \text{ kN}$	
Total load on beam	$W_{\text{tot}} = 4.500 \text{ kN}$	

 Paca Geotechnical engineering Ltd CM35TB	Project timber rafters n.2				Job no.	
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Reactions at support A

$$R_{A\_max} = 2.250 \text{ kN}$$

$$R_{A\_min} = 2.250 \text{ kN}$$

Unfactored dead load reaction at support A

$$R_{A\_Dead} = 0.939 \text{ kN}$$

Unfactored imposed load reaction at support A

$$R_{A\_Imposed} = 0.585 \text{ kN}$$

Reactions at support B

$$R_{B\_max} = 2.250 \text{ kN}$$

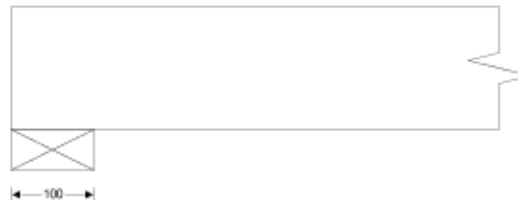
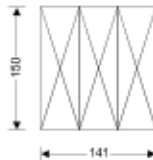
$$R_{B\_min} = 2.250 \text{ kN}$$

Unfactored dead load reaction at support B

$$R_{B\_Dead} = 0.939 \text{ kN}$$

Unfactored imposed load reaction at support B

$$R_{B\_Imposed} = 0.585 \text{ kN}$$



Timber section details

Breadth of sections

$$b = 47 \text{ mm}$$

Depth of sections

$$h = 150 \text{ mm}$$

Number of sections in member

$$N = 3$$

Overall breadth of member

$$b_b = N \times b = 141 \text{ mm}$$

Timber strength class

C24

Member details

Service class of timber

2

Load duration

Long term

Length of span

$$L_{s1} = 4500 \text{ mm}$$

Length of bearing

$$L_b = 100 \text{ mm}$$

Section properties

Cross sectional area of member

$$A = N \times b \times h = 21150 \text{ mm}^2$$

Section modulus

$$Z_x = N \times b \times h^2 / 6 = 528750 \text{ mm}^3$$

$$Z_y = h \times (N \times b)^2 / 6 = 497025 \text{ mm}^3$$

Second moment of area

$$I_x = N \times b \times h^3 / 12 = 39656250 \text{ mm}^4$$

$$I_y = h \times (N \times b)^3 / 12 = 35040263 \text{ mm}^4$$

Radius of gyration

$$i_x = \sqrt{I_x / A} = 43.3 \text{ mm}$$

$$i_y = \sqrt{I_y / A} = 40.7 \text{ mm}$$

Modification factors

Duration of loading - Table 17

$$K_3 = 1.00$$

Bearing stress - Table 18

$$K_4 = 1.00$$

Total depth of member - cl.2.10.6

$$K_7 = (300 \text{ mm} / h)^{0.11} = 1.08$$

Load sharing - cl.2.9

$$K_8 = 1.00$$

Lateral support - cl.2.10.8

Ends held in position and members held in line, as by direct connection of sheathing, deck or joists

Permissible depth-to-breadth ratio - Table 19

$$5.00$$

Actual depth-to-breadth ratio

$$h / (N \times b) = 1.06$$

PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress (no wane)

$$\sigma_{c\_adm} = \sigma_{cp1} \times K_3 \times K_4 \times K_8 = 2.400 \text{ N/mm}^2$$

Applied bearing stress

$$\sigma_{c\_a} = R_{A\_max} / (N \times b \times L_b) = 0.160 \text{ N/mm}^2$$

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$$\sigma_{c_a} / \sigma_{c_{adm}} = 0.066$$

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

Bending parallel to grain

Permissible bending stress

$$\sigma_{m_{adm}} = \sigma_m \times K_3 \times K_7 \times K_8 = 8.094 \text{ N/mm}^2$$

Applied bending stress

$$\sigma_{m_a} = M / Z_x = 4.787 \text{ N/mm}^2$$

$$\sigma_{m_a} / \sigma_{m_{adm}} = 0.591$$

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

Permissible shear stress

$$\tau_{adm} = \tau \times K_3 \times K_8 = 0.710 \text{ N/mm}^2$$

Applied shear stress

$$\tau_a = 3 \times F / (2 \times A) = 0.160 \text{ N/mm}^2$$

$$\tau_a / \tau_{adm} = 0.225$$

PASS - Applied shear stress is less than permissible shear stress

Deflection

Modulus of elasticity for deflection

$$E = E_{min} = 7200 \text{ N/mm}^2$$

Permissible deflection

$$\delta_{adm} = \min(0.551 \text{ in}, 0.003 \times L_{s1}) = 13.500 \text{ mm}$$

Bending deflection

$$\delta_{b_{s1}} = 12.662 \text{ mm}$$

Shear deflection

$$\delta_{v_{s1}} = 0.216 \text{ mm}$$

Total deflection

$$\delta_a = \delta_{b_{s1}} + \delta_{v_{s1}} = 12.878 \text{ mm}$$

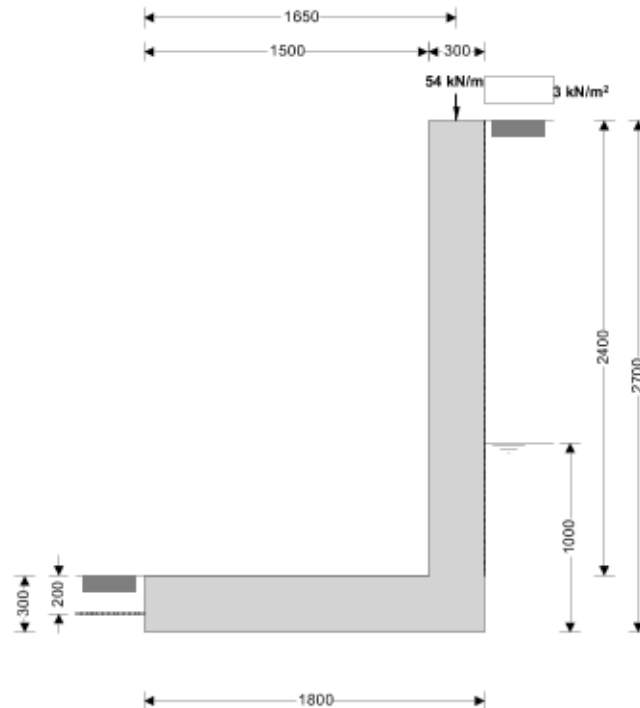
$$\delta_a / \delta_{adm} = 0.954$$

PASS - Total deflection is less than permissible deflection

 Paca Geotechnical engineering Ltd CM35TB	Project basement retaining wall pin more loaded				Job no.	
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RETAINING WALL ANALYSIS (BS 8002:1994)

TEDDS calculation version 1.2.01.06



Wall details

Retaining wall type
Height of retaining wall stem
Thickness of wall stem
Length of toe
Length of heel
Overall length of base
Thickness of base
Depth of downstand
Position of downstand
Thickness of downstand
Height of retaining wall
Depth of cover in front of wall
Depth of unplanned excavation
Height of ground water behind wall
Height of saturated fill above base
Density of wall construction
Density of base construction
Angle of rear face of wall
Angle of soil surface behind wall
Effective height at virtual back of wall

Unpropped cantilever

$h_{\text{stem}} = 2400 \text{ mm}$
 $t_{\text{wall}} = 300 \text{ mm}$
 $l_{\text{toe}} = 1500 \text{ mm}$
 $l_{\text{heel}} = 0 \text{ mm}$
 $l_{\text{base}} = l_{\text{toe}} + l_{\text{heel}} + t_{\text{wall}} = 1800 \text{ mm}$
 $t_{\text{base}} = 300 \text{ mm}$
 $d_{\text{ds}} = 0 \text{ mm}$
 $l_{\text{ds}} = 1400 \text{ mm}$
 $t_{\text{ds}} = 300 \text{ mm}$
 $h_{\text{wall}} = h_{\text{stem}} + t_{\text{base}} + d_{\text{ds}} = 2700 \text{ mm}$
 $d_{\text{cover}} = 0 \text{ mm}$
 $d_{\text{exc}} = 200 \text{ mm}$
 $h_{\text{water}} = 1000 \text{ mm}$
 $h_{\text{sat}} = \max(h_{\text{water}} - t_{\text{base}} - d_{\text{ds}}, 0 \text{ mm}) = 700 \text{ mm}$
 $\gamma_{\text{wall}} = 23.6 \text{ kN/m}^3$
 $\gamma_{\text{base}} = 23.6 \text{ kN/m}^3$
 $\alpha = 90.0 \text{ deg}$
 $\beta = 0.0 \text{ deg}$
 $h_{\text{eff}} = h_{\text{wall}} + l_{\text{heel}} \times \tan(\beta) = 2700 \text{ mm}$

Retained material details

Mobilisation factor
Moist density of retained material

$M = 1.5$
 $\gamma_m = 18.0 \text{ kN/m}^3$

 Paca Geotechnical engineering Ltd CM35TB	Project basement retaining wall pin more loaded				Job no.	
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Saturated density of retained material $\gamma_s = 21.0 \text{ kN/m}^3$

Design shear strength $\phi' = 32.0 \text{ deg}$

Angle of wall friction $\delta = 24.0 \text{ deg}$

Base material details

Moist density $\gamma_{mb} = 18.0 \text{ kN/m}^3$

Design shear strength $\phi'_b = 32.0 \text{ deg}$

Design base friction $\delta_b = 24.0 \text{ deg}$

Allowable bearing pressure $P_{\text{bearing}} = 150 \text{ kN/m}^2$

Using Coulomb theory

Active pressure coefficient for retained material

$$K_a = \sin(\alpha + \phi')^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta) \times [1 + \sqrt{(\sin(\phi' + \delta) \times \sin(\phi' - \beta) / (\sin(\alpha - \delta) \times \sin(\alpha + \beta)))^2}] = 0.275$$

Passive pressure coefficient for base material

$$K_p = \sin(90 - \phi'_b)^2 / (\sin(90 - \delta_b) \times [1 - \sqrt{(\sin(\phi'_b + \delta_b) \times \sin(\phi'_b) / (\sin(90 + \delta_b)))^2}] = 8.378$$

At-rest pressure

At-rest pressure for retained material $K_0 = 1 - \sin(\phi') = 0.470$

Loading details

Surcharge load on plan Surcharge = **2.5 kN/m²**

Applied vertical dead load on wall $W_{\text{dead}} = 45.8 \text{ kN/m}$

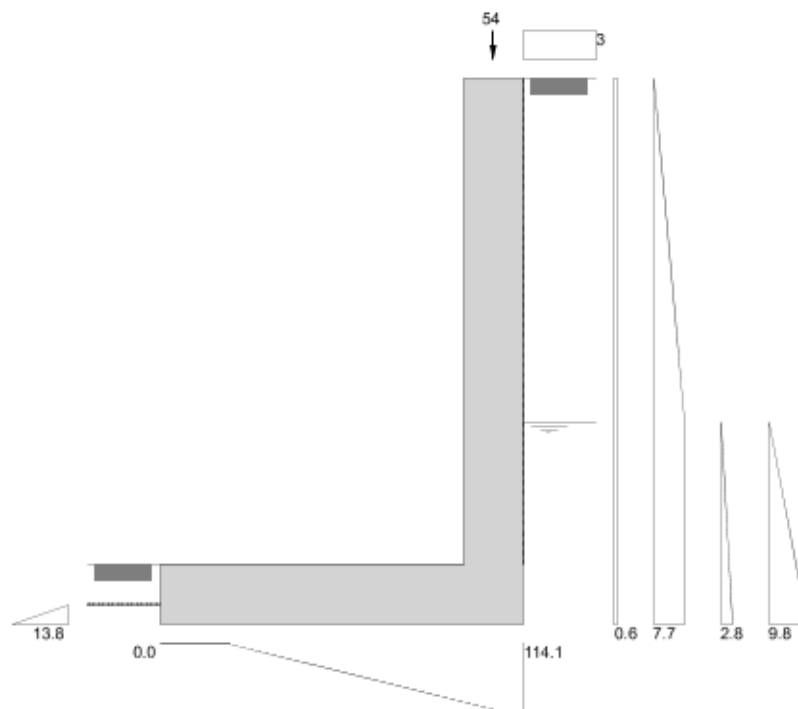
Applied vertical live load on wall $W_{\text{live}} = 7.7 \text{ kN/m}$

Position of applied vertical load on wall $l_{\text{load}} = 1650 \text{ mm}$

Applied horizontal dead load on wall $F_{\text{dead}} = 0.0 \text{ kN/m}$

Applied horizontal live load on wall $F_{\text{live}} = 0.0 \text{ kN/m}$

Height of applied horizontal load on wall $h_{\text{load}} = 0 \text{ mm}$



Loads shown in kN/m, pressures shown in kN/m²

 Paca Geotechnical engineering Ltd CM35TB	Project basement retaining wall pin more loaded				Job no.	
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Vertical forces on wall

Wall stem

$$W_{\text{wall}} = h_{\text{stem}} \times t_{\text{wall}} \times \gamma_{\text{wall}} = 17 \text{ kN/m}$$

Wall base

$$W_{\text{base}} = l_{\text{base}} \times t_{\text{base}} \times \gamma_{\text{base}} = 12.7 \text{ kN/m}$$

Applied vertical load

$$W_v = W_{\text{dead}} + W_{\text{live}} = 53.5 \text{ kN/m}$$

Total vertical load

$$W_{\text{total}} = W_{\text{wall}} + W_{\text{base}} + W_v = 83.2 \text{ kN/m}$$

Horizontal forces on wall

Surcharge

$$F_{\text{sur}} = K_a \times \cos(90 - \alpha + \delta) \times \text{Surcharge} \times h_{\text{eff}} = 1.7 \text{ kN/m}$$

Moist backfill above water table

$$F_{m_a} = 0.5 \times K_a \times \cos(90 - \alpha + \delta) \times \gamma_m \times (h_{\text{eff}} - h_{\text{water}})^2 = 6.5 \text{ kN/m}$$

Moist backfill below water table

$$F_{m_b} = K_a \times \cos(90 - \alpha + \delta) \times \gamma_m \times (h_{\text{eff}} - h_{\text{water}}) \times h_{\text{water}} = 7.7 \text{ kN/m}$$

Saturated backfill

$$F_s = 0.5 \times K_a \times \cos(90 - \alpha + \delta) \times (\gamma_s - \gamma_{\text{water}}) \times h_{\text{water}}^2 = 1.4 \text{ kN/m}$$

Water

$$F_{\text{water}} = 0.5 \times h_{\text{water}}^2 \times \gamma_{\text{water}} = 4.9 \text{ kN/m}$$

Total horizontal load

$$F_{\text{total}} = F_{\text{sur}} + F_{m_a} + F_{m_b} + F_s + F_{\text{water}} = 22.2 \text{ kN/m}$$

Calculate stability against sliding

Passive resistance of soil in front of wall

$$F_p = 0.5 \times K_p \times \cos(\delta_b) \times (d_{\text{cover}} + t_{\text{base}} + d_{\text{ds}} - d_{\text{exc}})^2 \times \gamma_{\text{mb}} = 0.7 \text{ kN/m}$$

Resistance to sliding

$$F_{\text{res}} = F_p + (W_{\text{total}} - W_{\text{live}}) \times \tan(\delta_b) = 34.3 \text{ kN/m}$$

PASS - Resistance force is greater than sliding force

Overturning moments

Surcharge

$$M_{\text{sur}} = F_{\text{sur}} \times (h_{\text{eff}} - 2 \times d_{\text{ds}}) / 2 = 2.3 \text{ kNm/m}$$

Moist backfill above water table

$$M_{m_a} = F_{m_a} \times (h_{\text{eff}} + 2 \times h_{\text{water}} - 3 \times d_{\text{ds}}) / 3 = 10.2 \text{ kNm/m}$$

Moist backfill below water table

$$M_{m_b} = F_{m_b} \times (h_{\text{water}} - 2 \times d_{\text{ds}}) / 2 = 3.8 \text{ kNm/m}$$

Saturated backfill

$$M_s = F_s \times (h_{\text{water}} - 3 \times d_{\text{ds}}) / 3 = 0.5 \text{ kNm/m}$$

Water

$$M_{\text{water}} = F_{\text{water}} \times (h_{\text{water}} - 3 \times d_{\text{ds}}) / 3 = 1.6 \text{ kNm/m}$$

Total overturning moment

$$M_{\text{ot}} = M_{\text{sur}} + M_{m_a} + M_{m_b} + M_s + M_{\text{water}} = 18.4 \text{ kNm/m}$$

Restoring moments

Wall stem

$$M_{\text{wall}} = W_{\text{wall}} \times (l_{\text{os}} + t_{\text{wall}} / 2) = 28 \text{ kNm/m}$$

Wall base

$$M_{\text{base}} = W_{\text{base}} \times l_{\text{base}} / 2 = 11.5 \text{ kNm/m}$$

Design vertical dead load

$$M_{\text{dead}} = W_{\text{dead}} \times l_{\text{load}} = 75.6 \text{ kNm/m}$$

Total restoring moment

$$M_{\text{rest}} = M_{\text{wall}} + M_{\text{base}} + M_{\text{dead}} = 115.1 \text{ kNm/m}$$

Check stability against overturning

Total overturning moment

$$M_{\text{ot}} = 18.4 \text{ kNm/m}$$

Total restoring moment

$$M_{\text{rest}} = 115.1 \text{ kNm/m}$$

PASS - Restoring moment is greater than overturning moment

Check bearing pressure

Design vertical live load

$$M_{\text{live}} = W_{\text{live}} \times l_{\text{load}} = 12.7 \text{ kNm/m}$$

Total moment for bearing

$$M_{\text{total}} = M_{\text{rest}} - M_{\text{ot}} + M_{\text{live}} = 109.3 \text{ kNm/m}$$

Total vertical reaction

$$R = W_{\text{total}} = 83.2 \text{ kN/m}$$

Distance to reaction

$$x_{\text{bar}} = M_{\text{total}} / R = 1314 \text{ mm}$$

Eccentricity of reaction

$$e = \text{abs}((l_{\text{base}} / 2) - x_{\text{bar}}) = 414 \text{ mm}$$

Reaction acts outside middle third of base

Bearing pressure at toe

$$p_{\text{toe}} = 0 \text{ kN/m}^2 = 0 \text{ kN/m}^2$$

Bearing pressure at heel

$$p_{\text{heel}} = R / (1.5 \times (l_{\text{base}} - x_{\text{bar}})) = 114.1 \text{ kN/m}^2$$

PASS - Maximum bearing pressure is less than allowable bearing pressure

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RETAINING WALL DESIGN (BS 8002:1994)

TEDDS calculation version 1.2.01.06

Ultimate limit state load factors

Dead load factor	$\gamma_{f,d} = 1.4$
Live load factor	$\gamma_{f,l} = 1.6$
Earth and water pressure factor	$\gamma_{f,e} = 1.4$

Factored vertical forces on wall

Wall stem	$W_{wall,f} = \gamma_{f,d} \times h_{stem} \times t_{wall} \times \gamma_{wall} = 23.8 \text{ kN/m}$
Wall base	$W_{base,f} = \gamma_{f,d} \times l_{base} \times t_{base} \times \gamma_{base} = 17.8 \text{ kN/m}$
Applied vertical load	$W_{v,f} = \gamma_{f,d} \times W_{dead} + \gamma_{f,l} \times W_{live} = 76.4 \text{ kN/m}$
Total vertical load	$W_{total,f} = W_{wall,f} + W_{base,f} + W_{v,f} = 118.1 \text{ kN/m}$

Factored horizontal at-rest forces on wall

Surcharge	$F_{sur,f} = \gamma_{f,l} \times K_0 \times \text{Surcharge} \times h_{eff} = 5.1 \text{ kN/m}$
Moist backfill above water table	$F_{m,a,f} = \gamma_{f,e} \times 0.5 \times K_0 \times \gamma_m \times (h_{eff} - h_{water})^2 = 17.1 \text{ kN/m}$
Moist backfill below water table	$F_{m,b,f} = \gamma_{f,e} \times K_0 \times \gamma_m \times (h_{eff} - h_{water}) \times h_{water} = 20.1 \text{ kN/m}$
Saturated backfill	$F_{s,f} = \gamma_{f,e} \times 0.5 \times K_0 \times (\gamma_s - \gamma_{water}) \times h_{water}^2 = 3.7 \text{ kN/m}$
Water	$F_{water,f} = \gamma_{f,e} \times 0.5 \times h_{water}^2 \times \gamma_{water} = 6.9 \text{ kN/m}$
Total horizontal load	$F_{total,f} = F_{sur,f} + F_{m,a,f} + F_{m,b,f} + F_{s,f} + F_{water,f} = 52.9 \text{ kN/m}$
Passive resistance of soil in front of wall	$F_{p,f} = \gamma_{f,e} \times 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb} = 1 \text{ kN/m}$

Factored overturning moments

Surcharge	$M_{sur,f} = F_{sur,f} \times (h_{eff} - 2 \times d_{ds}) / 2 = 6.9 \text{ kNm/m}$
Moist backfill above water table	$M_{m,a,f} = F_{m,a,f} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3 = 26.8 \text{ kNm/m}$
Moist backfill below water table	$M_{m,b,f} = F_{m,b,f} \times (h_{water} - 2 \times d_{ds}) / 2 = 10.1 \text{ kNm/m}$
Saturated backfill	$M_{s,f} = F_{s,f} \times (h_{water} - 3 \times d_{ds}) / 3 = 1.2 \text{ kNm/m}$
Water	$M_{water,f} = F_{water,f} \times (h_{water} - 3 \times d_{ds}) / 3 = 2.3 \text{ kNm/m}$
Total overturning moment	$M_{ot,f} = M_{sur,f} + M_{m,a,f} + M_{m,b,f} + M_{s,f} + M_{water,f} = 47.3 \text{ kNm/m}$

Restoring moments

Wall stem	$M_{wall,f} = W_{wall,f} \times (l_{toe} + t_{wall} / 2) = 39.3 \text{ kNm/m}$
Wall base	$M_{base,f} = W_{base,f} \times l_{base} / 2 = 16.1 \text{ kNm/m}$
Design vertical load	$M_{v,f} = W_{v,f} \times l_{load} = 126.1 \text{ kNm/m}$
Total restoring moment	$M_{rest,f} = M_{wall,f} + M_{base,f} + M_{v,f} = 181.4 \text{ kNm/m}$

Factored bearing pressure

Total moment for bearing	$M_{total,f} = M_{rest,f} - M_{ot,f} = 134.2 \text{ kNm/m}$
Total vertical reaction	$R_f = W_{total,f} = 118.1 \text{ kN/m}$
Distance to reaction	$x_{bar,f} = M_{total,f} / R_f = 1136 \text{ mm}$
Eccentricity of reaction	$e_f = \text{abs}((l_{base} / 2) - x_{bar,f}) = 236 \text{ mm}$

Reaction acts within middle third of base

Bearing pressure at toe	$p_{toe,f} = (R_f / l_{base}) - (6 \times R_f \times e_f / l_{base}^2) = 13.9 \text{ kN/m}^2$
Bearing pressure at heel	$p_{heel,f} = (R_f / l_{base}) + (6 \times R_f \times e_f / l_{base}^2) = 117.3 \text{ kN/m}^2$
Rate of change of base reaction	$\text{rate} = (p_{toe,f} - p_{heel,f}) / l_{base} = -57.44 \text{ kN/m}^2/\text{m}$
Bearing pressure at stem / toe	$p_{stem\_toe,f} = \text{max}(p_{heel,f} + (\text{rate} \times (l_{heel} + t_{wall})), 0 \text{ kN/m}^2) = 100.1 \text{ kN/m}^2$
Bearing pressure at mid stem	$p_{stem\_mid,f} = \text{max}(p_{heel,f} + (\text{rate} \times (l_{heel} + t_{wall} / 2)), 0 \text{ kN/m}^2) = 108.7 \text{ kN/m}^2$
Bearing pressure at stem / heel	$p_{stem\_heel,f} = \text{max}(p_{heel,f} + (\text{rate} \times l_{heel}), 0 \text{ kN/m}^2) = 117.3 \text{ kN/m}^2$

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

Material properties

Characteristic strength of concrete $f_{cu} = 40 \text{ N/mm}^2$
Characteristic strength of reinforcement $f_y = 500 \text{ N/mm}^2$

Base details

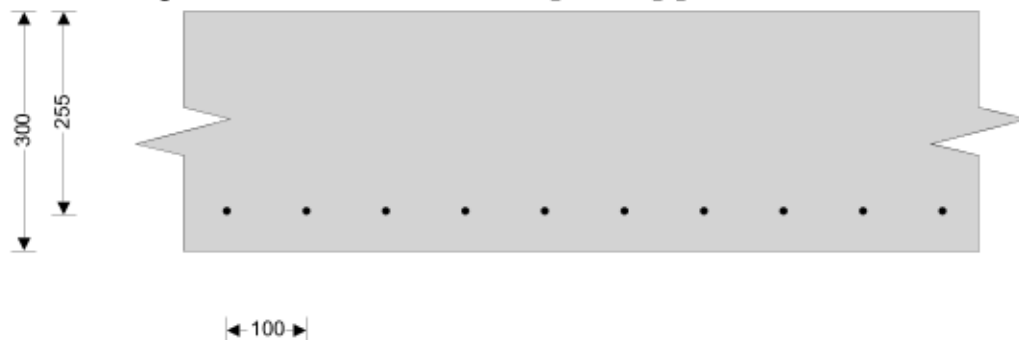
Minimum area of reinforcement $k = 0.13 \%$
Cover to reinforcement in toe $c_{toe} = 40 \text{ mm}$

Calculate shear for toe design

Shear from bearing pressure $V_{toe\_bear} = (p_{toe\_f} + p_{stem\_toe\_f}) \times l_{toe} / 2 = 85.5 \text{ kN/m}$
Shear from weight of base $V_{toe\_wt\_base} = \gamma_{fd} \times \gamma_{base} \times l_{toe} \times t_{base} = 14.9 \text{ kN/m}$
Total shear for toe design $V_{toe} = V_{toe\_bear} - V_{toe\_wt\_base} = 70.6 \text{ kN/m}$

Calculate moment for toe design

Moment from bearing pressure $M_{toe\_bear} = (2 \times p_{toe\_f} + p_{stem\_mid\_f}) \times (l_{toe} + t_{wall} / 2)^2 / 6 = 61.9 \text{ kNm/m}$
Moment from weight of base $M_{toe\_wt\_base} = (\gamma_{fd} \times \gamma_{base} \times t_{base} \times (l_{toe} + t_{wall} / 2)^2 / 2) = 13.5 \text{ kNm/m}$
Total moment for toe design $M_{toe} = M_{toe\_bear} - M_{toe\_wt\_base} = 48.4 \text{ kNm/m}$



Check toe in bending

Width of toe $b = 1000 \text{ mm/m}$
Depth of reinforcement $d_{toe} = t_{base} - c_{toe} - (\phi_{toe} / 2) = 255.0 \text{ mm}$
Constant $K_{toe} = M_{toe} / (b \times d_{toe}^2 \times f_{cu}) = 0.019$

Compression reinforcement is not required

Lever arm $z_{toe} = \min(0.5 + \sqrt{(0.25 - (\min(K_{toe}, 0.225) / 0.9))}, 0.95) \times d_{toe}$
 $z_{toe} = 242 \text{ mm}$
Area of tension reinforcement required $A_{s\_toe\_des} = M_{toe} / (0.87 \times f_y \times z_{toe}) = 460 \text{ mm}^2/\text{m}$
Minimum area of tension reinforcement $A_{s\_toe\_min} = k \times b \times t_{base} = 390 \text{ mm}^2/\text{m}$
Area of tension reinforcement required $A_{s\_toe\_req} = \text{Max}(A_{s\_toe\_des}, A_{s\_toe\_min}) = 460 \text{ mm}^2/\text{m}$
Reinforcement provided **B785 mesh**
Area of reinforcement provided $A_{s\_toe\_prov} = 785 \text{ mm}^2/\text{m}$

PASS - Reinforcement provided at the retaining wall toe is adequate

Check shear resistance at toe

Design shear stress $v_{toe} = V_{toe} / (b \times d_{toe}) = 0.277 \text{ N/mm}^2$
Allowable shear stress $v_{adm} = \min(0.8 \times \sqrt{f_{cu}} / 1 \text{ N/mm}^2, 5) \times 1 \text{ N/mm}^2 = 5.000 \text{ N/mm}^2$

PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress $v_{c\_toe} = 0.559 \text{ N/mm}^2$

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

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

Material properties

Characteristic strength of concrete $f_{cu} = 40 \text{ N/mm}^2$
Characteristic strength of reinforcement $f_y = 500 \text{ N/mm}^2$

Wall details

Minimum area of reinforcement $k = 0.13 \%$
Cover to reinforcement in stem $C_{stem} = 40 \text{ mm}$
Cover to reinforcement in wall $C_{wall} = 30 \text{ mm}$

Factored horizontal at-rest forces on stem

Surcharge $F_{s\_sur\_f} = \gamma_{f\_1} \times K_0 \times \text{Surcharge} \times (h_{eff} - t_{base} - d_{ds}) = 4.5 \text{ kN/m}$
Moist backfill above water table $F_{s\_m\_a\_f} = 0.5 \times \gamma_{f\_e} \times K_0 \times \gamma_m \times (h_{eff} - t_{base} - d_{ds} - h_{sat})^2 = 17.1 \text{ kN/m}$
Moist backfill below water table $F_{s\_m\_b\_f} = \gamma_{f\_e} \times K_0 \times \gamma_m \times (h_{eff} - t_{base} - d_{ds} - h_{sat}) \times h_{sat} = 14.1 \text{ kN/m}$
Saturated backfill $F_{s\_s\_f} = 0.5 \times \gamma_{f\_e} \times K_0 \times (\gamma_s - \gamma_{water}) \times h_{sat}^2 = 1.8 \text{ kN/m}$
Water $F_{s\_water\_f} = 0.5 \times \gamma_{f\_e} \times \gamma_{water} \times h_{sat}^2 = 3.4 \text{ kN/m}$

Calculate shear for stem design

Shear at base of stem $V_{stem} = F_{s\_sur\_f} + F_{s\_m\_a\_f} + F_{s\_m\_b\_f} + F_{s\_s\_f} + F_{s\_water\_f} = 40.9 \text{ kN/m}$

Calculate moment for stem design

Surcharge $M_{s\_sur} = F_{s\_sur\_f} \times (h_{stem} + t_{base}) / 2 = 6.1 \text{ kNm/m}$
Moist backfill above water table $M_{s\_m\_a} = F_{s\_m\_a\_f} \times (2 \times h_{sat} + h_{eff} - d_{ds} + t_{base} / 2) / 3 = 24.2 \text{ kNm/m}$
Moist backfill below water table $M_{s\_m\_b} = F_{s\_m\_b\_f} \times h_{sat} / 2 = 4.9 \text{ kNm/m}$
Saturated backfill $M_{s\_s} = F_{s\_s\_f} \times h_{sat} / 3 = 0.4 \text{ kNm/m}$
Water $M_{s\_water} = F_{s\_water\_f} \times h_{sat} / 3 = 0.8 \text{ kNm/m}$
Total moment for stem design $M_{stem} = M_{s\_sur} + M_{s\_m\_a} + M_{s\_m\_b} + M_{s\_s} + M_{s\_water} = 36.5 \text{ kNm/m}$



Check wall stem in bending

Width of wall stem $b = 1000 \text{ mm/m}$
Depth of reinforcement $d_{stem} = t_{wall} - C_{stem} - (\phi_{stem} / 2) = 255.0 \text{ mm}$
Constant $K_{stem} = M_{stem} / (b \times d_{stem}^2 \times f_{cu}) = 0.014$

Compression reinforcement is not required

Lever arm $Z_{stem} = \min(0.5 + \sqrt{(0.25 - (\min(K_{stem}, 0.225) / 0.9))}, 0.95) \times d_{stem}$
 $Z_{stem} = 242 \text{ mm}$

Area of tension reinforcement required $A_{s\_stem\_des} = M_{stem} / (0.87 \times f_y \times Z_{stem}) = 346 \text{ mm}^2/\text{m}$
Minimum area of tension reinforcement $A_{s\_stem\_min} = k \times b \times t_{wall} = 390 \text{ mm}^2/\text{m}$
Area of tension reinforcement required $A_{s\_stem\_req} = \text{Max}(A_{s\_stem\_des}, A_{s\_stem\_min}) = 390 \text{ mm}^2/\text{m}$
Reinforcement provided **B785 mesh**

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Area of reinforcement provided

$$A_{s\_stem\_prov} = 785 \text{ mm}^2/\text{m}$$

PASS - Reinforcement provided at the retaining wall stem is adequate

Check shear resistance at wall stem

Design shear stress

$$V_{stem} = V_{stem} / (b \times d_{stem}) = 0.160 \text{ N/mm}^2$$

Allowable shear stress

$$V_{adm} = \min(0.8 \times \sqrt{f_{cu}} / 1 \text{ N/mm}^2, 5) \times 1 \text{ N/mm}^2 = 5.000 \text{ N/mm}^2$$

PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress

$$V_{c\_stem} = 0.559 \text{ N/mm}^2$$

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

Check retaining wall deflection

Basic span/effective depth ratio

$$\text{ratio}_{bas} = 7$$

Design service stress

$$f_s = 2 \times f_y \times A_{s\_stem\_req} / (3 \times A_{s\_stem\_prov}) = 165.5 \text{ N/mm}^2$$

Modification factor

$$\text{factor}_{tens} = \min(0.55 + (477 \text{ N/mm}^2 - f_s) / (120 \times (0.9 \text{ N/mm}^2 + (M_{stem} / (b \times d_{stem}^2)))), 2) = 2.00$$

Maximum span/effective depth ratio

$$\text{ratio}_{max} = \text{ratio}_{bas} \times \text{factor}_{tens} = 14.00$$

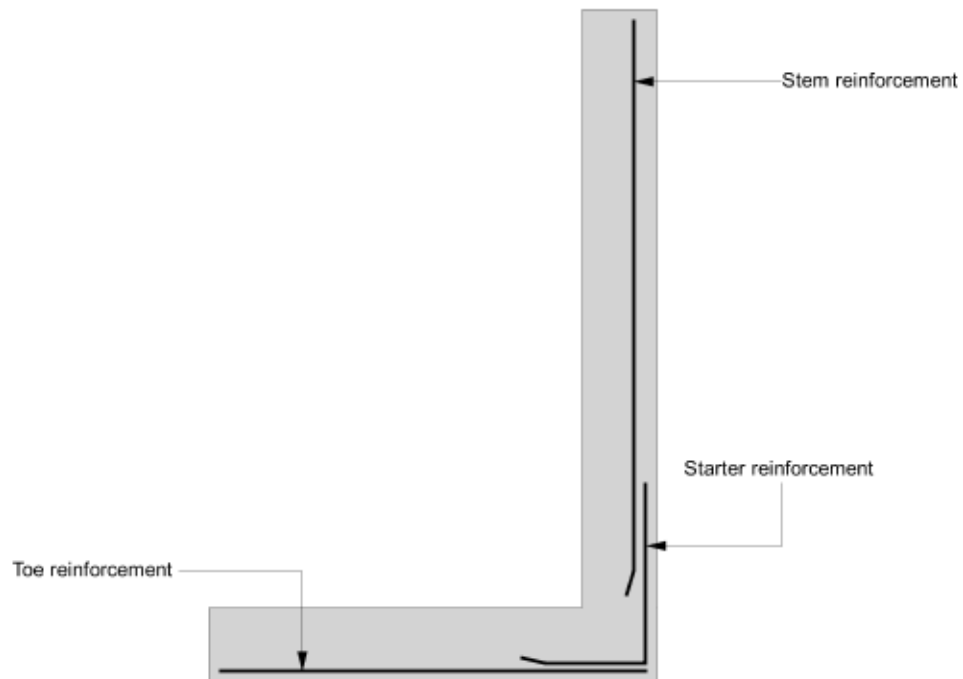
Actual span/effective depth ratio

$$\text{ratio}_{act} = h_{stem} / d_{stem} = 9.41$$

PASS - Span to depth ratio is acceptable

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Indicative retaining wall reinforcement diagram



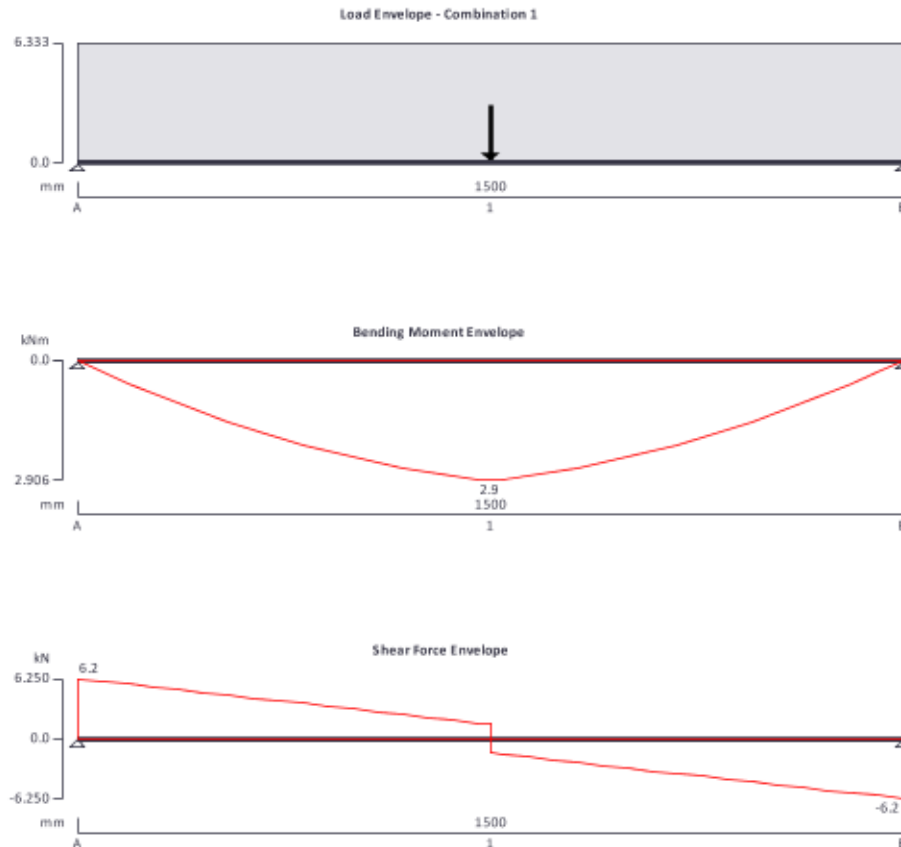
Toe mesh - B785 - (785 mm²/m)
Stem mesh - B785 - (785 mm²/m)

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STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.07



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Dead self weight of beam $\times 1$

Dead full UDL 3.27 kN/m

Imposed full UDL 0.9 kN/m

Dead point load 1 kN at 750 mm

Imposed point load 1 kN at 750 mm

Load combinations

Load combination 1

Support A

Dead $\times 1.40$

Imposed $\times 1.60$

Dead $\times 1.40$

Imposed $\times 1.60$

Support B

Dead $\times 1.40$

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Imposed $\times 1.60$

Analysis results

Maximum moment

$$M_{\max} = 2.9 \text{ kNm}$$

$$M_{\min} = 0 \text{ kNm}$$

Maximum shear

$$V_{\max} = 6.2 \text{ kN}$$

$$V_{\min} = -6.2 \text{ kN}$$

Deflection

$$\delta_{\max} = 0.1 \text{ mm}$$

$$\delta_{\min} = 0 \text{ mm}$$

Maximum reaction at support A

$$R_{A_{\max}} = 6.2 \text{ kN}$$

$$R_{A_{\min}} = 6.2 \text{ kN}$$

Unfactored dead load reaction at support A

$$R_{A_{\text{Dead}}} = 3.1 \text{ kN}$$

Unfactored imposed load reaction at support A

$$R_{A_{\text{Imposed}}} = 1.2 \text{ kN}$$

Maximum reaction at support B

$$R_{B_{\max}} = 6.2 \text{ kN}$$

$$R_{B_{\min}} = 6.2 \text{ kN}$$

Unfactored dead load reaction at support B

$$R_{B_{\text{Dead}}} = 3.1 \text{ kN}$$

Unfactored imposed load reaction at support B

$$R_{B_{\text{Imposed}}} = 1.2 \text{ kN}$$

Section details

Section type

UC 152x152x23 (BS4-1)

Steel grade

S275

From table 9: Design strength p_y

Thickness of element

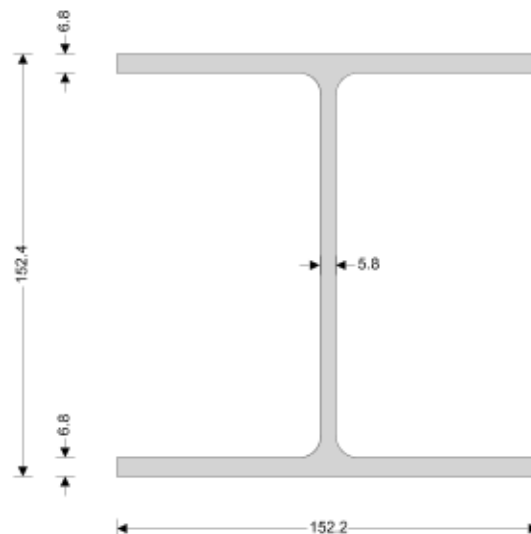
$$\max(T, t) = 6.8 \text{ mm}$$

Design strength

$$p_y = 275 \text{ N/mm}^2$$

Modulus of elasticity

$$E = 205000 \text{ N/mm}^2$$



Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis

$$K_x = 1.00$$

Effective length factor in minor axis

$$K_y = 1.00$$

Effective length factor for lateral-torsional buckling

$$K_{LTA} = 1.00$$

$$K_{LTB} = 1.00$$

Classification of cross sections - Section 3.5

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

Internal compression parts - Table 11

Depth of section

$$d = 123.6 \text{ mm}$$

$$d / t = 21.3 \times \varepsilon \leq 80 \times \varepsilon$$

Class 1 plastic

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Outstand flanges - Table 11

Width of section

$$b = B / 2 = \mathbf{76.1 \text{ mm}}$$

$$b / T = 11.2 \times \varepsilon \leq 15 \times \varepsilon$$

Class 3 semi-compact

Section is class 3 semi-compact

Shear capacity - Section 4.2.3

Design shear force

$$F_v = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = \mathbf{6.2 \text{ kN}}$$

$$d / t < 70 \times \varepsilon$$

Web does not need to be checked for shear buckling

Shear area

$$A_v = t \times D = \mathbf{884 \text{ mm}^2}$$

Design shear resistance

$$P_v = 0.6 \times p_y \times A_v = \mathbf{145.8 \text{ kN}}$$

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment

$$M = \max(\text{abs}(M_{s1_{\max}}), \text{abs}(M_{s1_{\min}})) = \mathbf{2.9 \text{ kNm}}$$

Effective plastic modulus - Section 3.5.6

Limiting value for class 2 compact flange

$$\beta_{2f} = 10 \times \varepsilon = \mathbf{10}$$

Limiting value for class 3 semi-compact flange

$$\beta_{3f} = 15 \times \varepsilon = \mathbf{15}$$

Limiting value for class 2 compact web

$$\beta_{2w} = 100 \times \varepsilon = \mathbf{100}$$

Limiting value for class 3 semi-compact web

$$\beta_{3w} = 120 \times \varepsilon = \mathbf{120}$$

Effective plastic modulus - cl.3.5.6.2

$$S_{\text{eff}} = \min(Z_{xx} + (S_{xx} - Z_{xx}) \times \min(\frac{(\beta_{3w} / (d / t))^2 - 1}{(\beta_{3w} / \beta_{2w})^2 - 1}, \frac{(\beta_{3f} / (b / T) - 1)}{(\beta_{3f} / \beta_{2f} - 1)}), S_{xx}) = \mathbf{176245 \text{ mm}^3}$$

Moment capacity low shear - cl.4.2.5.2

$$M_c = \min(p_y \times S_{\text{eff}}, 1.2 \times p_y \times Z_{xx}) = \mathbf{48.5 \text{ kNm}}$$

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling

$$L_E = 1.0 \times L_{s1} = \mathbf{1500 \text{ mm}}$$

Slenderness ratio

$$\lambda = L_E / r_{yy} = \mathbf{40.563}$$

Equivalent slenderness - Section 4.3.6.7

Buckling parameter

$$u = \mathbf{0.840}$$

Torsional index

$$x = \mathbf{20.701}$$

Slenderness factor

$$v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25} = \mathbf{0.957}$$

Ratio - cl.4.3.6.9

$$\beta_w = S_{\text{eff}} / S_{xx} = \mathbf{0.968}$$

Equivalent slenderness - cl.4.3.6.7

$$\lambda_{LT} = u \times v \times \lambda \times \sqrt{\beta_w} = \mathbf{32.075}$$

Limiting slenderness - Annex B.2.2

$$\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5} = \mathbf{34.310}$$

$\lambda_{LT} < \lambda_{L0}$ - No allowance need be made for lateral-torsional buckling

Buckling resistance moment - Section 4.3.6.4

Bending strength

$$p_b = p_y = \mathbf{275 \text{ N/mm}^2}$$

Buckling resistance moment

$$M_b = p_b \times S_{\text{eff}} = \mathbf{48.5 \text{ kNm}}$$

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection

$$\delta_{\text{lim}} = L_{s1} / 360 = \mathbf{4.167 \text{ mm}}$$

Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{\max}), \text{abs}(\delta_{\min})) = \mathbf{0.051 \text{ mm}}$$

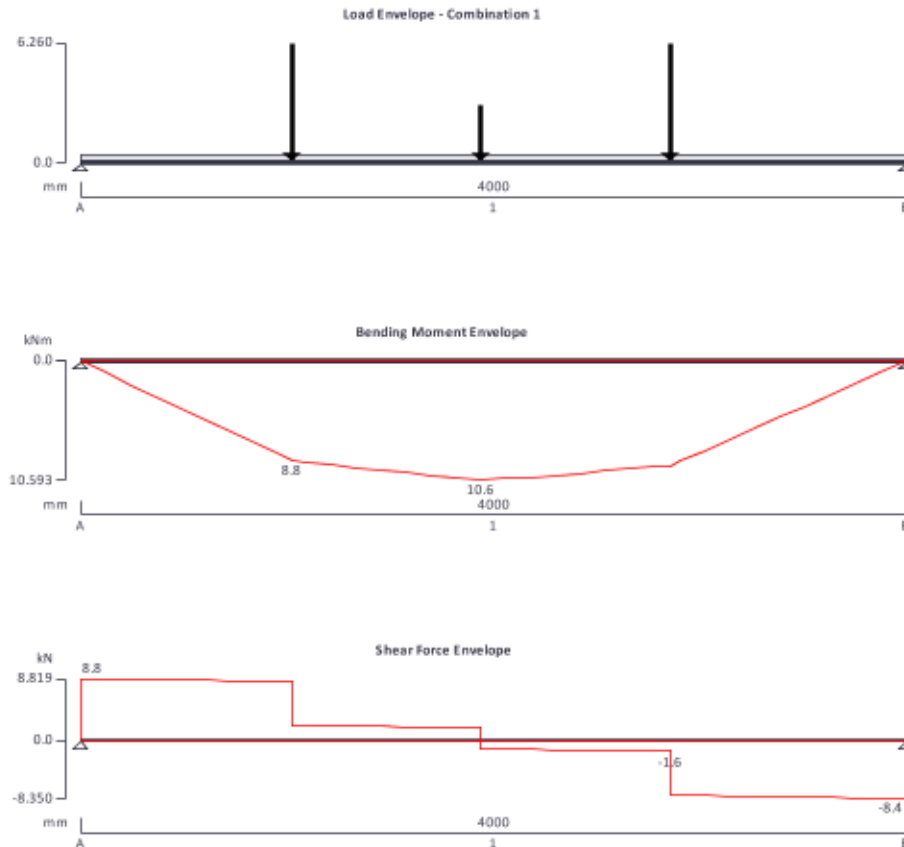
PASS - Maximum deflection does not exceed deflection limit

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Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Dead self weight of beam $\times 1$

Dead point load 3.1 kN at 1023 mm

Imposed point load 1.2 kN at 1023 mm

Dead point load 1 kN at 1940 mm

Imposed point load 1 kN at 1940 mm

Dead point load 3.1 kN at 2856 mm

Imposed point load 1.2 kN at 2856 mm

Load combinations

Load combination 1

Support A

Dead $\times 1.40$

Imposed $\times 1.60$

Dead $\times 1.40$

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Analysis results

Maximum moment

$$M_{\max} = 10.6 \text{ kNm}$$

$$\text{Imposed} \times 1.60$$

Maximum shear

$$V_{\max} = 8.8 \text{ kN}$$

$$\text{Dead} \times 1.40$$

Deflection

$$\delta_{\max} = 1 \text{ mm}$$

$$\text{Imposed} \times 1.60$$

Maximum reaction at support A

$$R_{A_{\max}} = 8.8 \text{ kN}$$

$$M_{\min} = 0 \text{ kNm}$$

Unfactored dead load reaction at support A

$$R_{A_{\text{Dead}}} = 4.3 \text{ kN}$$

$$V_{\min} = -8.4 \text{ kN}$$

Unfactored imposed load reaction at support A

$$R_{A_{\text{Imposed}}} = 1.8 \text{ kN}$$

$$\delta_{\min} = 0 \text{ mm}$$

Maximum reaction at support B

$$R_{B_{\max}} = 8.4 \text{ kN}$$

$$R_{A_{\min}} = 8.8 \text{ kN}$$

Unfactored dead load reaction at support B

$$R_{B_{\text{Dead}}} = 4.1 \text{ kN}$$

$$R_{B_{\min}} = 8.4 \text{ kN}$$

Unfactored imposed load reaction at support B

$$R_{B_{\text{Imposed}}} = 1.6 \text{ kN}$$

Section details

Section type

UC 152x152x30 (BS4-1)

Steel grade

S275

From table 9: Design strength p_y

Thickness of element

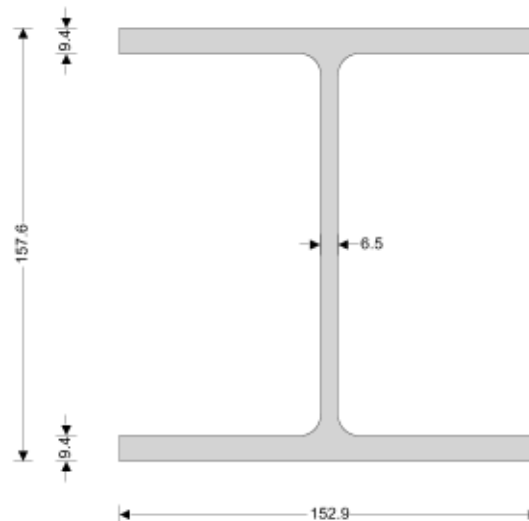
$$\max(T, t) = 9.4 \text{ mm}$$

Design strength

$$p_y = 275 \text{ N/mm}^2$$

Modulus of elasticity

$$E = 205000 \text{ N/mm}^2$$



Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis

$$K_x = 1.00$$

Effective length factor in minor axis

$$K_y = 1.00$$

Effective length factor for lateral-torsional buckling

$$K_{LTA} = 1.00$$

$$K_{LTB} = 1.00$$

Classification of cross sections - Section 3.5

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

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Internal compression parts - Table 11

Depth of section $d = 123.6 \text{ mm}$
 $d / t = 19.0 \times \varepsilon \leq 80 \times \varepsilon$ Class 1 plastic

Outstand flanges - Table 11

Width of section $b = B / 2 = 76.5 \text{ mm}$
 $b / T = 8.1 \times \varepsilon \leq 9 \times \varepsilon$ Class 1 plastic
Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force $F_v = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 8.8 \text{ kN}$
 $d / t < 70 \times \varepsilon$
Web does not need to be checked for shear buckling
Shear area $A_v = t \times D = 1024 \text{ mm}^2$
Design shear resistance $P_v = 0.6 \times p_y \times A_v = 169 \text{ kN}$
PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment $M = \max(\text{abs}(M_{s1_{\max}}), \text{abs}(M_{s1_{\min}})) = 10.6 \text{ kNm}$
Moment capacity low shear - cl.4.2.5.2 $M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = 68.1 \text{ kNm}$

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling $L_E = 1.0 \times L_{s1} = 4000 \text{ mm}$
Slenderness ratio $\lambda = L_E / r_{yy} = 104.514$

Equivalent slenderness - Section 4.3.6.7

Buckling parameter $u = 0.849$
Torsional index $x = 15.999$
Slenderness factor $v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25} = 0.752$
Ratio - cl.4.3.6.9 $\beta_w = 1.000$
Equivalent slenderness - cl.4.3.6.7 $\lambda_{LT} = u \times v \times \lambda \times \sqrt{\beta_w} = 66.653$
Limiting slenderness - Annex B.2.2 $\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5} = 34.310$
 $\lambda_{LT} > \lambda_{L0}$ - Allowance should be made for lateral-torsional buckling

Bending strength - Section 4.3.6.5

Robertson constant $\alpha_{LT} = 7.0$
Perry factor $\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) = 0.226$
Euler stress $p_E = \pi^2 \times E / \lambda_{LT}^2 = 455.4 \text{ N/mm}^2$
 $\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 = 416.8 \text{ N/mm}^2$
 $p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) = 196.6 \text{ N/mm}^2$
Bending strength - Annex B.2.1

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment $M_2 = 8.6 \text{ kNm}$
Moment at centre-line of segment $M_3 = 10.5 \text{ kNm}$
Moment at three quarter point of segment $M_4 = 8.1 \text{ kNm}$
Maximum moment in segment $M_{\text{abs}} = 10.6 \text{ kNm}$
Maximum moment governing buckling resistance $M_{LT} = M_{\text{abs}} = 10.6 \text{ kNm}$
Equivalent uniform moment factor for lateral-torsional buckling
 $m_{LT} = \max(0.2 + (0.15 \times M_2 + 0.5 \times M_3 + 0.15 \times M_4) / M_{\text{abs}}, 0.44) = 0.934$

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment $M_b = p_b \times S_{xx} = 48.7 \text{ kNm}$

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$$M_b / m_{LT} = 52.2 \text{ kNm}$$

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection

$$\delta_{lim} = L_{s1} / 360 = 11.111 \text{ mm}$$

Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = 1.026 \text{ mm}$$

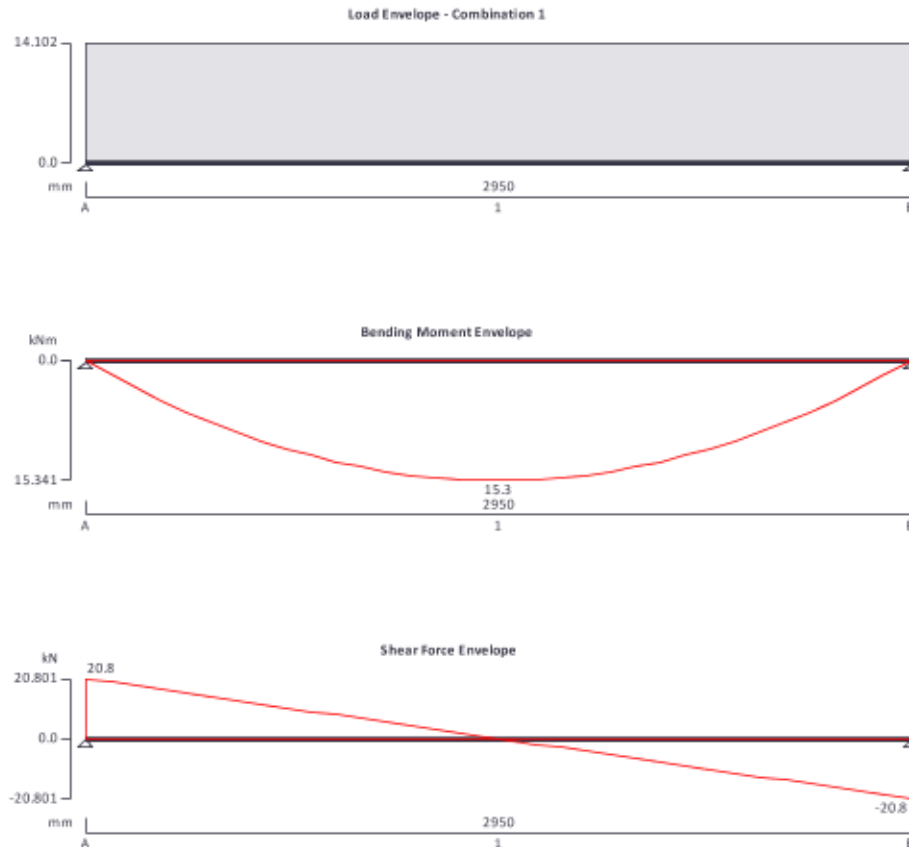
PASS - Maximum deflection does not exceed deflection limit

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Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Dead self weight of beam $\times 1$

Dead full UDL 7.5 kN/m

Dead full UDL 1.25 kN/m

Imposed full UDL 0.9 kN/m

Load combinations

Load combination 1

Support A

Dead $\times 1.40$

Imposed $\times 1.60$

Dead $\times 1.40$

Imposed $\times 1.60$

Support B

Dead $\times 1.40$

Imposed $\times 1.60$

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Analysis results

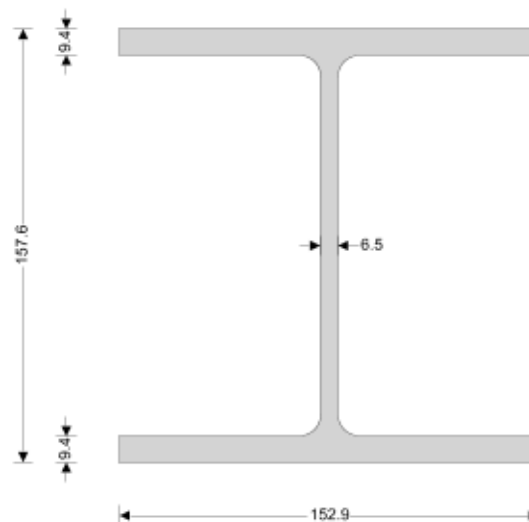
Maximum moment	$M_{max} = 15.3 \text{ kNm}$	$M_{min} = 0 \text{ kNm}$
Maximum shear	$V_{max} = 20.8 \text{ kN}$	$V_{min} = -20.8 \text{ kN}$
Deflection	$\delta_{max} = 0.2 \text{ mm}$	$\delta_{min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_{max}} = 20.8 \text{ kN}$	$R_{A_{min}} = 20.8 \text{ kN}$
Unfactored dead load reaction at support A	$R_{A_{Dead}} = 13.3 \text{ kN}$	
Unfactored imposed load reaction at support A	$R_{A_{Imposed}} = 1.3 \text{ kN}$	
Maximum reaction at support B	$R_{B_{max}} = 20.8 \text{ kN}$	$R_{B_{min}} = 20.8 \text{ kN}$
Unfactored dead load reaction at support B	$R_{B_{Dead}} = 13.3 \text{ kN}$	
Unfactored imposed load reaction at support B	$R_{B_{Imposed}} = 1.3 \text{ kN}$	

Section details

Section type	UC 152x152x30 (BS4-1)
Steel grade	S275

From table 9: Design strength p_y

Thickness of element	$\max(T, t) = 9.4 \text{ mm}$
Design strength	$p_y = 275 \text{ N/mm}^2$
Modulus of elasticity	$E = 205000 \text{ N/mm}^2$



Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis	$K_x = 1.00$
Effective length factor in minor axis	$K_y = 1.00$
Effective length factor for lateral-torsional buckling	$K_{LT,A} = 1.00$
	$K_{LT,B} = 1.00$

Classification of cross sections - Section 3.5

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

Internal compression parts - Table 11

Depth of section	$d = 123.6 \text{ mm}$	
	$d / t = 19.0 \times \varepsilon \leq 80 \times \varepsilon$	Class 1 plastic

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Outstand flanges - Table 11

Width of section

$$b = B / 2 = 76.5 \text{ mm}$$

$$b / T = 8.1 \times \epsilon \leq 9 \times \epsilon$$

Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force

$$F_v = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 20.8 \text{ kN}$$

$$d / t < 70 \times \epsilon$$

Web does not need to be checked for shear buckling

Shear area

$$A_v = t \times D = 1024 \text{ mm}^2$$

Design shear resistance

$$P_v = 0.6 \times p_y \times A_v = 169 \text{ kN}$$

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment

$$M = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) = 15.3 \text{ kNm}$$

Moment capacity low shear - cl.4.2.5.2

$$M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = 68.1 \text{ kNm}$$

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling

$$L_E = 1.0 \times L_{s1} = 2950 \text{ mm}$$

Slenderness ratio

$$\lambda = L_E / r_{yy} = 77.079$$

Equivalent slenderness - Section 4.3.6.7

Buckling parameter

$$u = 0.849$$

Torsional index

$$x = 15.999$$

Slenderness factor

$$v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25} = 0.825$$

Ratio - cl.4.3.6.9

$$\beta_W = 1.000$$

Equivalent slenderness - cl.4.3.6.7

$$\lambda_{LT} = u \times v \times \lambda \times \sqrt{\beta_W} = 53.946$$

Limiting slenderness - Annex B.2.2

$$\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5} = 34.310$$

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

Bending strength - Section 4.3.6.5

Robertson constant

$$\alpha_{LT} = 7.0$$

Perry factor

$$\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) = 0.137$$

Euler stress

$$p_E = \pi^2 \times E / \lambda_{LT}^2 = 695.2 \text{ N/mm}^2$$

$$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 = 532.9 \text{ N/mm}^2$$

Bending strength - Annex B.2.1

$$p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) = 228.3 \text{ N/mm}^2$$

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment

$$M_2 = 11.5 \text{ kNm}$$

Moment at centre-line of segment

$$M_3 = 15.3 \text{ kNm}$$

Moment at three quarter point of segment

$$M_4 = 11.5 \text{ kNm}$$

Maximum moment in segment

$$M_{abs} = 15.3 \text{ kNm}$$

Maximum moment governing buckling resistance

$$M_{LT} = M_{abs} = 15.3 \text{ kNm}$$

Equivalent uniform moment factor for lateral-torsional buckling

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

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment

$$M_b = p_b \times S_{xx} = 56.5 \text{ kNm}$$

$$M_b / m_{LT} = 61.1 \text{ kNm}$$

PASS - Buckling resistance moment exceeds design bending moment

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Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection

$$\delta_{lim} = L_{s1} / 360 = \mathbf{8.194 \text{ mm}}$$

Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = \mathbf{0.248 \text{ mm}}$$

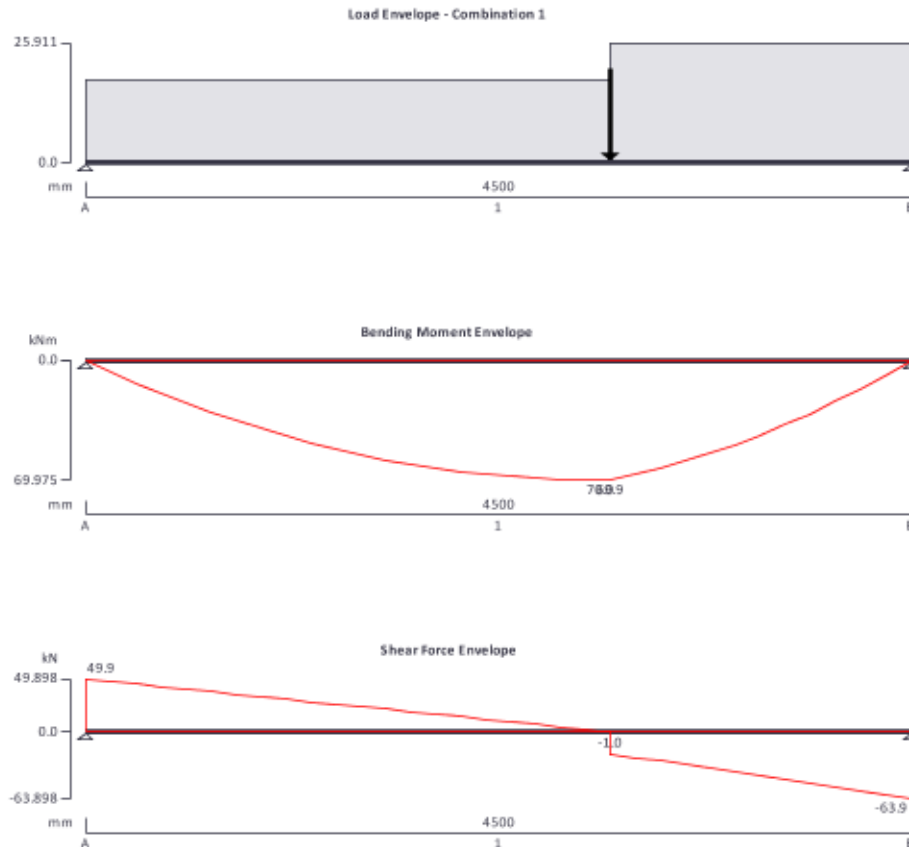
PASS - Maximum deflection does not exceed deflection limit

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Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Dead self weight of beam $\times 1$

Dead partial UDL 7.5 kN/m from 2860 mm to 4500 mm

Dead partial UDL 8.45 kN/m from 0 mm to 2860 mm

Imposed partial UDL 3.33 kN/m from 0 mm to 2860 mm

Dead partial UDL 8.03 kN/m from 2860 mm to 4500 mm

Imposed partial UDL 2.21 kN/m from 2860 mm to 4500 mm

Dead point load 13.1 kN at 2860 mm

Imposed point load 1.3 kN at 2860 mm

Load combinations

Load combination 1

Support A

Dead $\times 1.40$

Imposed $\times 1.60$

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Analysis results

Maximum moment
Maximum shear
Deflection
Maximum reaction at support A
Unfactored dead load reaction at support A
Unfactored imposed load reaction at support A
Maximum reaction at support B
Unfactored dead load reaction at support B
Unfactored imposed load reaction at support B

Support B

$M_{max} = 70 \text{ kNm}$
 $V_{max} = 49.9 \text{ kN}$
 $\delta_{max} = 2 \text{ mm}$
 $R_{A\_max} = 49.9 \text{ kN}$
 $R_{A\_Dead} = 26.9 \text{ kN}$
 $R_{A\_Imposed} = 7.6 \text{ kN}$
 $R_{B\_max} = 63.9 \text{ kN}$
 $R_{B\_Dead} = 37.9 \text{ kN}$
 $R_{B\_Imposed} = 6.8 \text{ kN}$

Dead $\times 1.40$
Imposed $\times 1.60$
Dead $\times 1.40$
Imposed $\times 1.60$

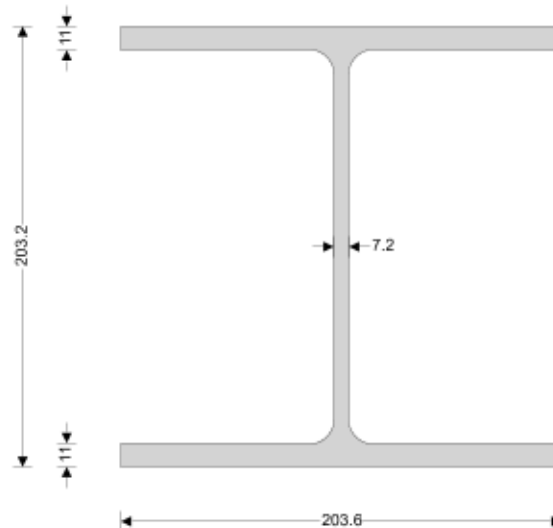
$M_{min} = 0 \text{ kNm}$
 $V_{min} = -63.9 \text{ kN}$
 $\delta_{min} = 0 \text{ mm}$
 $R_{A\_min} = 49.9 \text{ kN}$
 $R_{B\_min} = 63.9 \text{ kN}$

Section details

Section type
Steel grade
From table 9: Design strength p_y
Thickness of element
Design strength
Modulus of elasticity

UC 203x203x46 (BS4-1)
S275

$\max(T, t) = 11.0 \text{ mm}$
 $p_y = 275 \text{ N/mm}^2$
 $E = 205000 \text{ N/mm}^2$



Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis
Effective length factor in minor axis
Effective length factor for lateral-torsional buckling

$K_x = 1.00$
 $K_y = 1.00$
 $K_{LT,A} = 1.00$
 $K_{LT,B} = 1.00$

Classification of cross sections - Section 3.5

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

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Internal compression parts - Table 11

Depth of section $d = 160.8 \text{ mm}$
 $d / t = 22.3 \times \varepsilon \leq 80 \times \varepsilon$ Class 1 plastic

Outstand flanges - Table 11

Width of section $b = B / 2 = 101.8 \text{ mm}$
 $b / T = 9.3 \times \varepsilon \leq 10 \times \varepsilon$ Class 2 compact
Section is class 2 compact

Shear capacity - Section 4.2.3

Design shear force $F_v = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 63.9 \text{ kN}$
 $d / t < 70 \times \varepsilon$
Web does not need to be checked for shear buckling
Shear area $A_v = t \times D = 1463 \text{ mm}^2$
Design shear resistance $P_v = 0.6 \times p_y \times A_v = 241.4 \text{ kN}$
PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment $M = \max(\text{abs}(M_{s1_{\max}}), \text{abs}(M_{s1_{\min}})) = 70 \text{ kNm}$
Moment capacity low shear - cl.4.2.5.2 $M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = 136.8 \text{ kNm}$

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling $L_E = 1.0 \times L_{s1} = 4500 \text{ mm}$
Slenderness ratio $\lambda = L_E / r_{yy} = 87.646$

Equivalent slenderness - Section 4.3.6.7

Buckling parameter $u = 0.847$
Torsional index $x = 17.713$
Slenderness factor $v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25} = 0.819$
Ratio - cl.4.3.6.9 $\beta_w = 1.000$
Equivalent slenderness - cl.4.3.6.7 $\lambda_{LT} = u \times v \times \lambda \times \sqrt{\beta_w} = 60.753$
Limiting slenderness - Annex B.2.2 $\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5} = 34.310$
 $\lambda_{LT} > \lambda_{L0}$ - Allowance should be made for lateral-torsional buckling

Bending strength - Section 4.3.6.5

Robertson constant $\alpha_{LT} = 7.0$
Perry factor $\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) = 0.185$
Euler stress $p_E = \pi^2 \times E / \lambda_{LT}^2 = 548.2 \text{ N/mm}^2$
 $\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 = 462.3 \text{ N/mm}^2$
 $p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) = 211.3 \text{ N/mm}^2$

Bending strength - Annex B.2.1

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment $M_2 = 44.9 \text{ kNm}$
Moment at centre-line of segment $M_3 = 67.2 \text{ kNm}$
Moment at three quarter point of segment $M_4 = 55.5 \text{ kNm}$
Maximum moment in segment $M_{\text{abs}} = 70 \text{ kNm}$
Maximum moment governing buckling resistance $M_{LT} = M_{\text{abs}} = 70 \text{ kNm}$
Equivalent uniform moment factor for lateral-torsional buckling
 $m_{LT} = \max(0.2 + (0.15 \times M_2 + 0.5 \times M_3 + 0.15 \times M_4) / M_{\text{abs}}, 0.44) = 0.896$

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment $M_b = p_b \times S_{xx} = 105.1 \text{ kNm}$

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$$M_b / m_{LT} = 117.4 \text{ kNm}$$

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection

$$\delta_{lim} = L_{s1} / 360 = 12.5 \text{ mm}$$

Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = 1.95 \text{ mm}$$

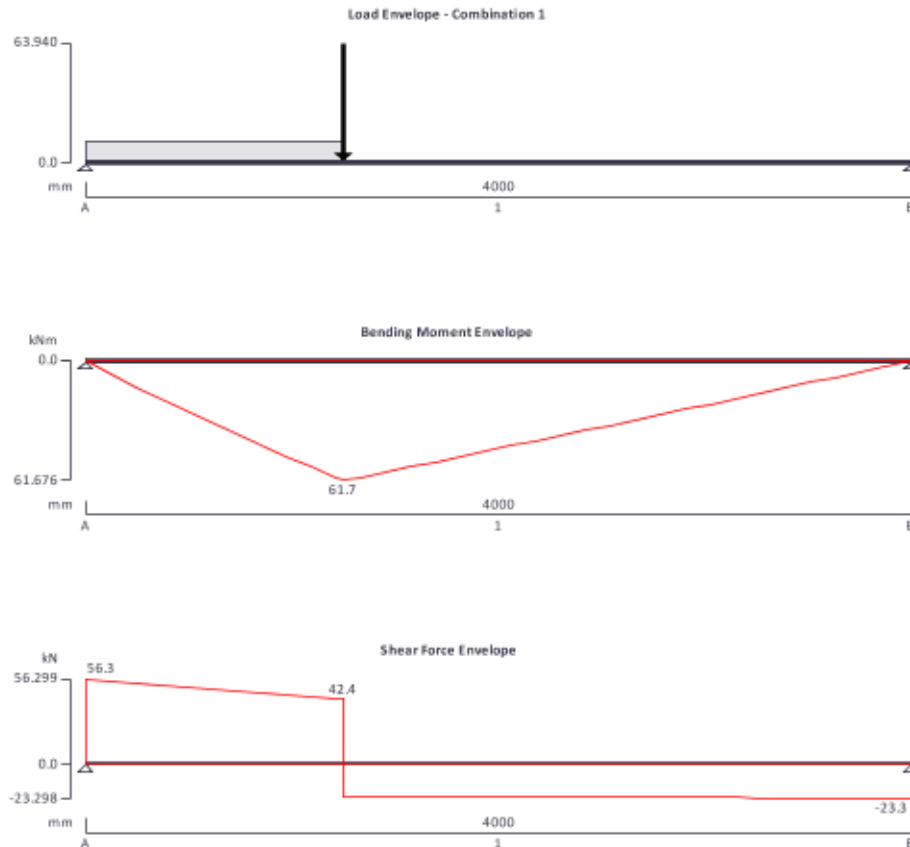
PASS - Maximum deflection does not exceed deflection limit

 Paca Geotechnical engineering Ltd CM35TB	Project steel beam n.F5				Job no.	
	Calcs for				Start page no./Revision 1	
	Calcs by U	Calcs date 09/03/2020	Checked by	Checked date	Approved by	Approved date

STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.07



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Dead self weight of beam $\times 1$

Dead partial UDL 7.5 kN/m from 0 mm to 1250 mm

Dead point load 37.9 kN at 1250 mm

Imposed point load 6.8 kN at 1250 mm

Load combinations

Load combination 1

Support A

Dead $\times 1.40$

Imposed $\times 1.60$

Dead $\times 1.40$

Imposed $\times 1.60$

Support B

Dead $\times 1.40$

Imposed $\times 1.60$

 Paca Geotechnical engineering Ltd CM35TB	Project steel beam n.F5				Job no.	
	Calcs for				Start page no./Revision 2	
	Calcs by U	Calcs date 09/03/2020	Checked by	Checked date	Approved by	Approved date

Analysis results

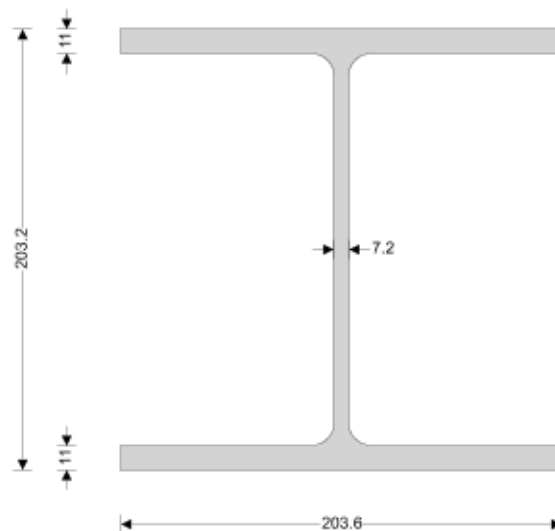
Maximum moment	$M_{max} = 61.7 \text{ kNm}$	$M_{min} = 0 \text{ kNm}$
Maximum shear	$V_{max} = 56.3 \text{ kN}$	$V_{min} = -23.3 \text{ kN}$
Deflection	$\delta_{max} = 0.8 \text{ mm}$	$\delta_{min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_{max}} = 56.3 \text{ kN}$	$R_{A_{min}} = 56.3 \text{ kN}$
Unfactored dead load reaction at support A	$R_{A_{Dead}} = 34.9 \text{ kN}$	
Unfactored imposed load reaction at support A	$R_{A_{Imposed}} = 4.7 \text{ kN}$	
Maximum reaction at support B	$R_{B_{max}} = 23.3 \text{ kN}$	$R_{B_{min}} = 23.3 \text{ kN}$
Unfactored dead load reaction at support B	$R_{B_{Dead}} = 14.2 \text{ kN}$	
Unfactored imposed load reaction at support B	$R_{B_{Imposed}} = 2.1 \text{ kN}$	

Section details

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

From table 9: Design strength p_y

Thickness of element	$\max(T, t) = 11.0 \text{ mm}$
Design strength	$p_y = 275 \text{ N/mm}^2$
Modulus of elasticity	$E = 205000 \text{ N/mm}^2$



Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis	$K_x = 1.00$
Effective length factor in minor axis	$K_y = 1.00$
Effective length factor for lateral-torsional buckling	$K_{LTA} = 1.00$
	$K_{LTB} = 1.00$

Classification of cross sections - Section 3.5

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

Internal compression parts - Table 11

Depth of section	$d = 160.8 \text{ mm}$	
	$d / t = 22.3 \times \varepsilon \leq 80 \times \varepsilon$	Class 1 plastic

 Paca Geotechnical engineering Ltd CM35TB	Project steel beam n.F5				Job no.	
	Calcs for				Start page no./Revision 3	
	Calcs by U	Calcs date 09/03/2020	Checked by	Checked date	Approved by	Approved date

Outstand flanges - Table 11

Width of section

$$b = B / 2 = 101.8 \text{ mm}$$

$$b / T = 9.3 \times \epsilon \leq 10 \times \epsilon$$

Class 2 compact

Section is class 2 compact

Shear capacity - Section 4.2.3

Design shear force

$$F_v = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 56.3 \text{ kN}$$

$$d / t < 70 \times \epsilon$$

Web does not need to be checked for shear buckling

Shear area

$$A_v = t \times D = 1463 \text{ mm}^2$$

Design shear resistance

$$P_v = 0.6 \times p_y \times A_v = 241.4 \text{ kN}$$

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment

$$M = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) = 61.7 \text{ kNm}$$

Moment capacity low shear - cl.4.2.5.2

$$M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = 136.8 \text{ kNm}$$

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling

$$L_E = 1.0 \times L_{s1} = 4000 \text{ mm}$$

Slenderness ratio

$$\lambda = L_E / r_{yy} = 77.908$$

Equivalent slenderness - Section 4.3.6.7

Buckling parameter

$$u = 0.847$$

Torsional index

$$x = 17.713$$

Slenderness factor

$$v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25} = 0.844$$

Ratio - cl.4.3.6.9

$$\beta_W = 1.000$$

Equivalent slenderness - cl.4.3.6.7

$$\lambda_{LT} = u \times v \times \lambda \times \sqrt{\beta_W} = 55.686$$

Limiting slenderness - Annex B.2.2

$$\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5} = 34.310$$

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

Bending strength - Section 4.3.6.5

Robertson constant

$$\alpha_{LT} = 7.0$$

Perry factor

$$\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) = 0.150$$

Euler stress

$$p_E = \pi^2 \times E / \lambda_{LT}^2 = 652.5 \text{ N/mm}^2$$

$$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 = 512.6 \text{ N/mm}^2$$

Bending strength - Annex B.2.1

$$p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) = 224 \text{ N/mm}^2$$

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment

$$M_2 = 50.7 \text{ kNm}$$

Moment at centre-line of segment

$$M_3 = 45.3 \text{ kNm}$$

Moment at three quarter point of segment

$$M_4 = 23 \text{ kNm}$$

Maximum moment in segment

$$M_{abs} = 61.7 \text{ kNm}$$

Maximum moment governing buckling resistance

$$M_{LT} = M_{abs} = 61.7 \text{ kNm}$$

Equivalent uniform moment factor for lateral-torsional buckling

$$m_{LT} = \max(0.2 + (0.15 \times M_2 + 0.5 \times M_3 + 0.15 \times M_4) / M_{abs}, 0.44) = 0.747$$

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment

$$M_b = p_b \times S_{xx} = 111.4 \text{ kNm}$$

$$M_b / m_{LT} = 149.2 \text{ kNm}$$

PASS - Moment capacity exceeds design bending moment

 Paca Geotechnical engineering Ltd CM35TB	Project				Job no.	
	steel beam n.F5				Start page no./Revision	
	Calcs for				4	
	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
	U	09/03/2020				

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

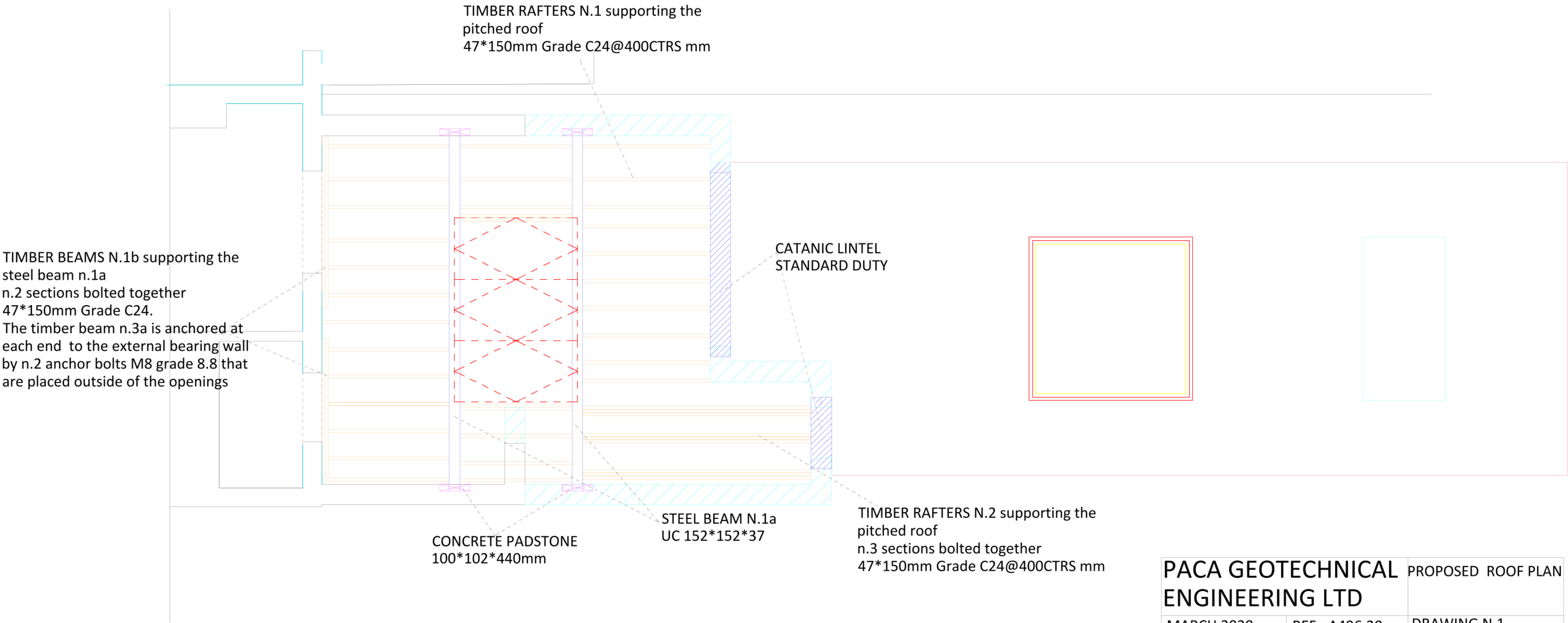
Limiting deflection

$$\delta_{lim} = L_{s1} / 360 = 11.111 \text{ mm}$$

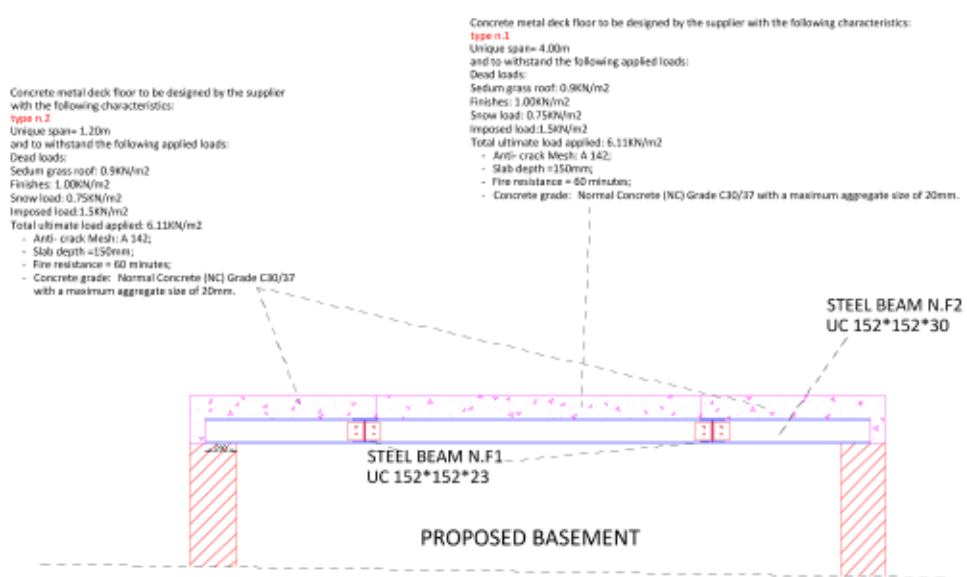
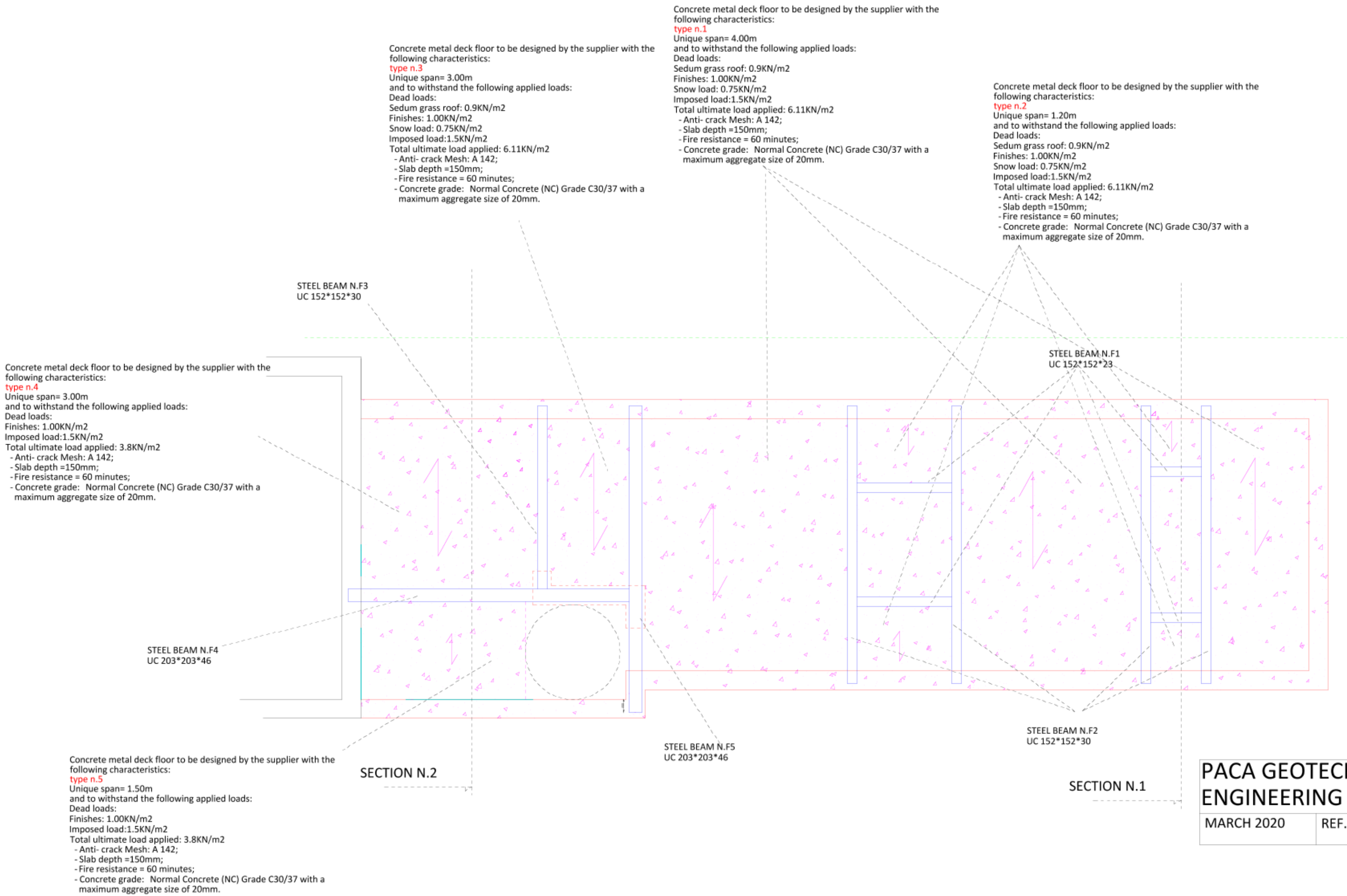
Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = 0.799 \text{ mm}$$

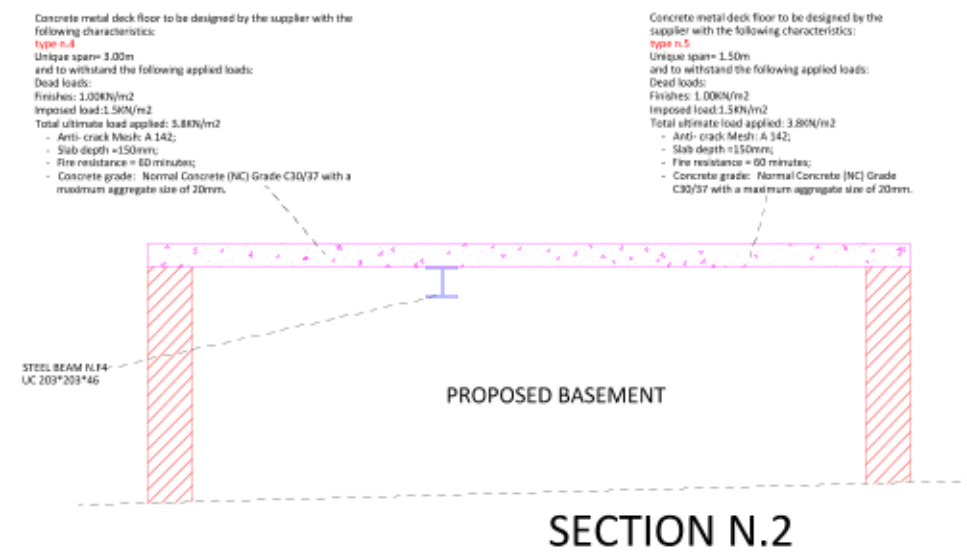
PASS - Maximum deflection does not exceed deflection limit



PACA GEOTECHNICAL ENGINEERING LTD		PROPOSED ROOF PLAN
MARCH 2020	REF. :A496.20	DRAWING N.1 SCALE 1:30@A1



SECTION N.1



SECTION N.2

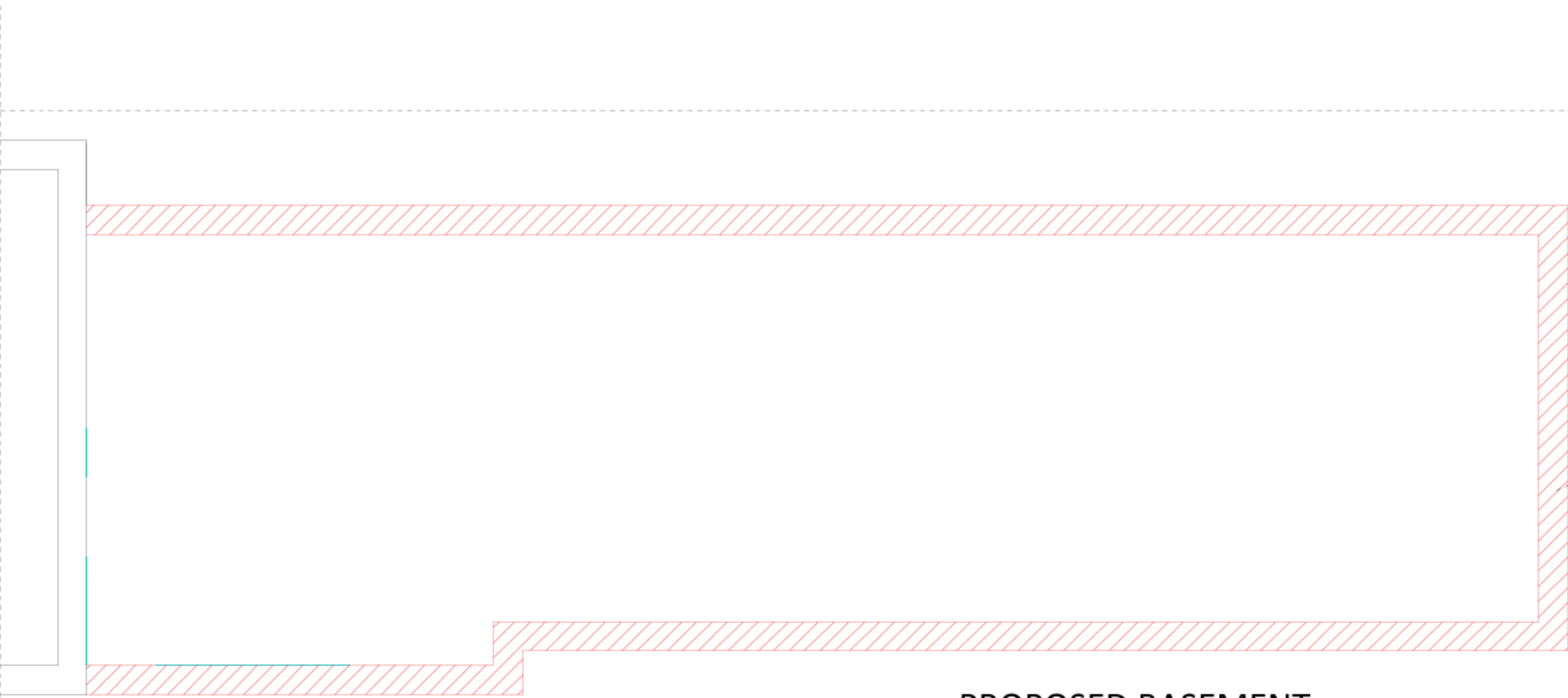
GENERAL NOTES:

STEEL BEARING LENGTHS OF THE STEEL BEAMS EQUAL TO 200mm WHICH WILL BE PROPPED AND INSERTED INTO THE BASEMENT REINFORCED CONCRETE WALLS .

STEEL BEAMS CONNECTIONS BY N.6 BOLTS DIAM. 12mm grade 8.8 and steel plate thickness=8.00mm grade S275;
Dh=nominal diam.=db + 1 mm=13mm for db <12mm;
p=Spacing between centres of bolts, pitch = 2.5db ≤ p ≤ 14t;
32.5mm<p<= 112mm;
e=Minimum edge distance>2.5*Dh=32.5mm

Design strength of the fillet weld: pw= 220N/mm2
(Eletrocode classification:35; Steel grade: S275);
Weld leg length s= 8mm;
CFW = CONTINUOUS FILLET WELD;
FPBW = FULL PENETRATION BUTT WELD.

PACA GEOTECHNICAL ENGINEERING LTD		PROPOSED GROUND FLOOR OF THE HOUSE EXTENSION AND BASEMENT ROOF PLAN
MARCH 2020	REF. :A496.20	DRAWING N.2 SCALE 1:50@A1



PROPOSED BASEMENT
STAGE N.1

FIRST HEIGHT OF 1.35m OF THE BASEMENT RETAINING WALLS;
As the basement retaining walls will be built in two stages , the minimum lap to the mesh reinforcement will have to be 450 mm.
In addition to connect the two parts of the retaining wall built in different stages and the adjacent pins, longitudinal bars mild steel dowel length 600mm, diam. 10mm and distance centre to centre 400mm must be inserted into the reinforced concrete wall for 300mm and embedded into the other part of the wall for 300mm for connecting the two elements of the one built in different stages;

GENERAL NOTE:
THE WALLS TO OF THE NEW BASEMENT WILL BE BUILT IN TWO STAGES ;
FIRST STAGE : THE FIRST HEIGHT OF 1.35m WILL BE BUILT IN ALL THE SIDES OF BASEMENT.
SECOND STAGE : THE REMAINING WALL HEIGHT TO REACH THE PROPOSED HEIGHT OF THE BASEMENT WILL BE BUILT WITH THE UNDERPINNING TECHNIQUE. FOR WHICH REINFORCED CONCRETE PINS WITH A MAXIMUM LENGTH EQUAL TO 0.8/1.0m WILL BE BUILT TO TAKE THE VERTICAL LOADS FROM THE WALLS AND HORIZONTAL LOADS FROM THE EARTH AND ADJACENT BUILDING;
DURING THE CONSTRUCTION, THE PINS WALLS AND BASES WILL REQUIRE LATERALLY PROPPING IN THE TEMPORARY CONDITION. PROPPING WILL BE MADE AGAINST THE CENTRAL EARTH PUDDING.
THE REINFORCED CONCRETE RETAINING WALL PINS THAT WILL BE BUILT UNDER THE CONTINUOUS FIRST WALL HEIGHT OF 1.35m WILL BE CONSTRUCTED STAGGERED.
THE SEQUENCE OF THE CONSTRUCTION OF THE REINFORCED CONCRETE PINS WILL BE
IN THE 1,3,5,2 AND 4 SEQUENCE,
IN THE a, c,e,b AND d SEQUENCE AND SUCH THAT ANY GIVEN UNDERPIN WILL BE COMPLETED , DRY PACKED AND A MINIMUM PERIOD OF 48 HOURS LAPSED BEFORE AN ADJACENT EXCAVATION COMMENCED TO FORM ANOTHER UNDERPIN.
TO CONNECT THE REINFORCED CONCRETE RETAINING WALL PINS BETWEEN THEM , LONGITUDINAL BARS MILD STEEL DOWEL LENGTH 600mm WILL BE INSERTED INTO EACH PIN AND EMBEDDED INTO THE ADJACENT ONE FOR 300mm;
THE SEQUENCE OF THE CONSTRUCTION STAGES OF THE SIDES BASEMENT WALL WILL BE SHOWN IN THE NEXT DRAWINGS;
CONCRETE WILL GENERALLY BE GRADE C35/40 and Class 1 to BRE Digest 363; Reinforcement will be grade 500N/mm2;



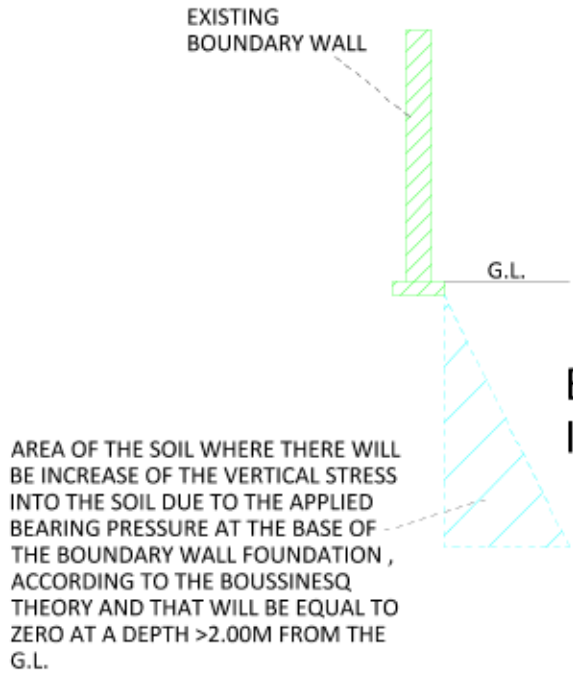
PROPOSED BASEMENT
STAGE N.2

REINFORCED CONCRETE RETAINING WALL PINS
THAT WILL BE BUILT UNDER THE FIRST HEIGHT OF
WALL EQUAL TO 1.35m.
LENGTH OF THE PINS=800/1000mm

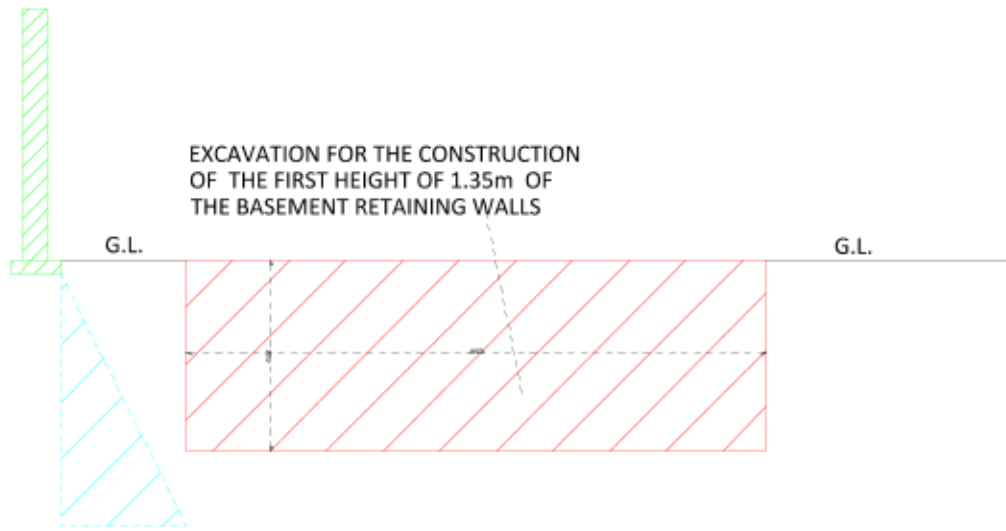
PART OF THE EXISTING FOUNDATION
THAT WILL HAVE TO BE UNDERPINNED TO CREATE
THE PROPOSED BASEMENT .
THE MENTIONED REINFORCED CONCRETE PINS MUST
BE BULT STAGGERED AND WITH A LENGTH
=800/1000mm

FOUNDATION OF THE
REINFORCED CONCRETE
RETAINING WALLS (PINS)

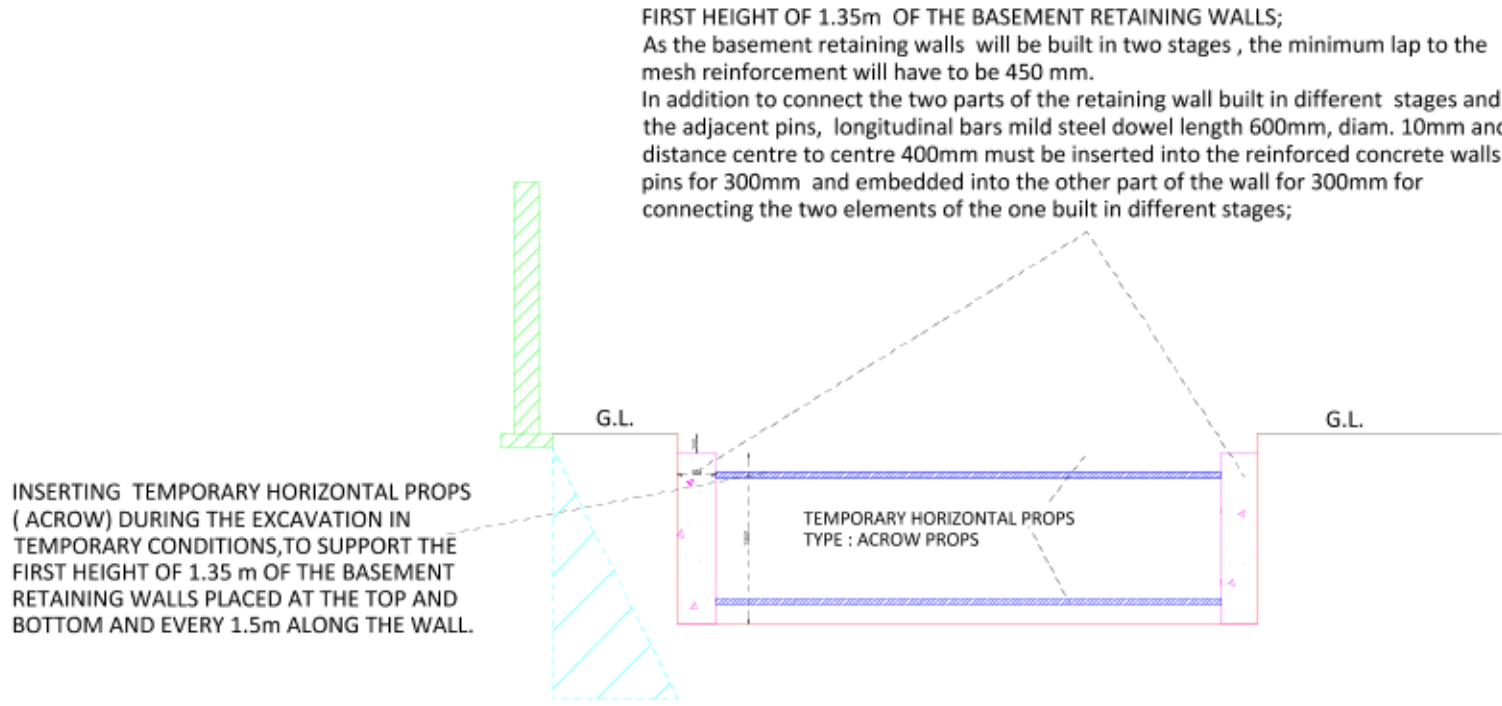
PACA GEOTECHNICAL ENGINEERING LTD			PROPOSED BASEMENT PLAN
MARCH 2020	REF. :A496.20	DRAWING N.3 SCALE 1:50@A1	



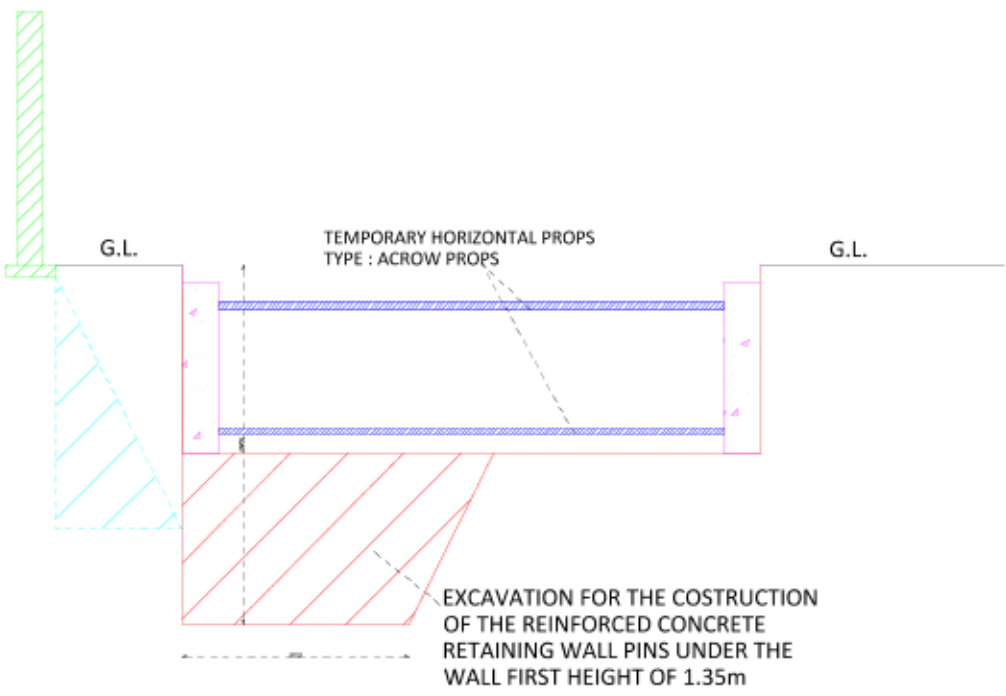
EXISTING SECTION N.A:
INITIAL STAGE



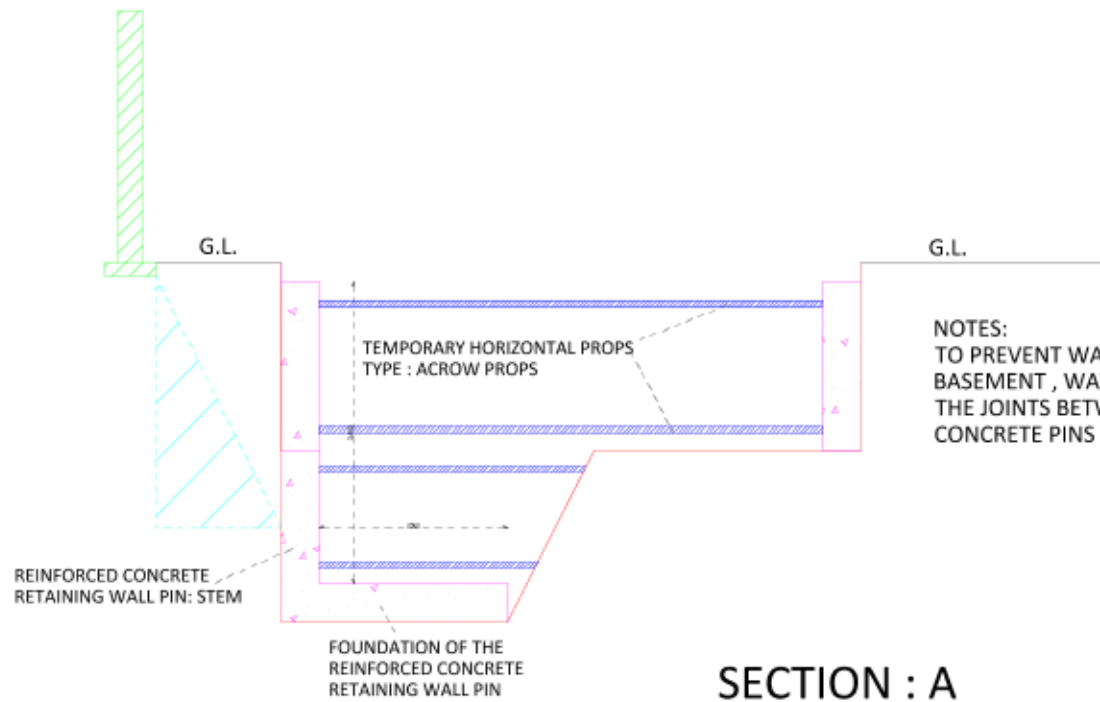
SECTION : A
STAGE N.1 : EXCAVATION FOR THE CONSTRUCTION OF THE FIRST HEIGHT OF 1.35m FROM THE GROUND LEVEL OF THE BASEMENT RETAINING WALL .
If the soil is unstable , inserting for propping the soil:
n.3 x 300mm steel trench sheets Type Standard S275JRC;
Placed by hand in front the face of the excavation and propped by Acrow props.



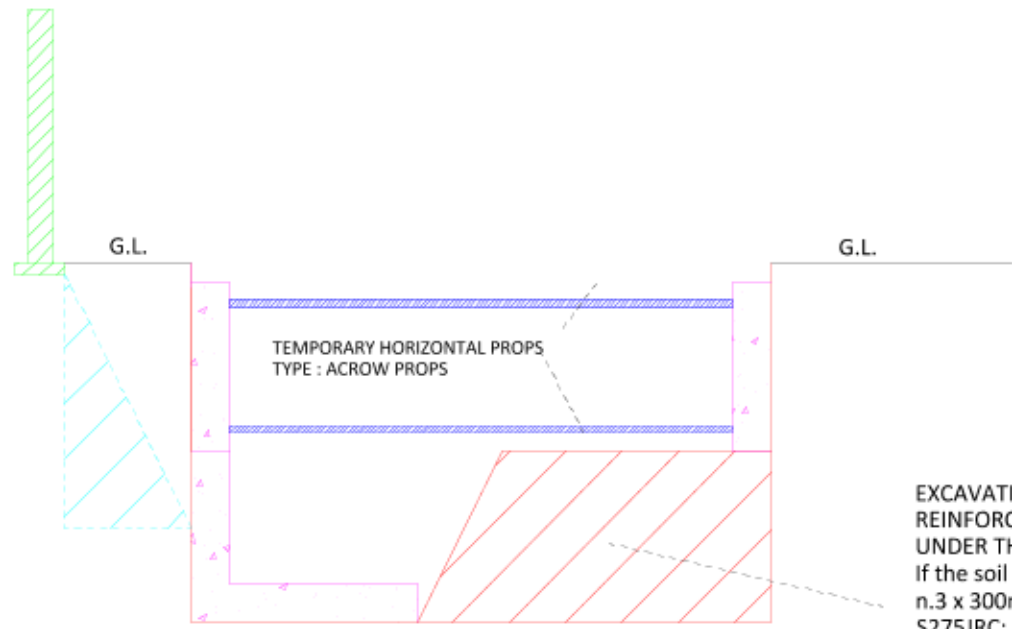
SECTION : A
STAGE N.2 : CONSTRUCTION OF THE FIRST HEIGHT OF 1.35m FROM THE G.L. OF THE BASEMENT RETAINING WALLS;



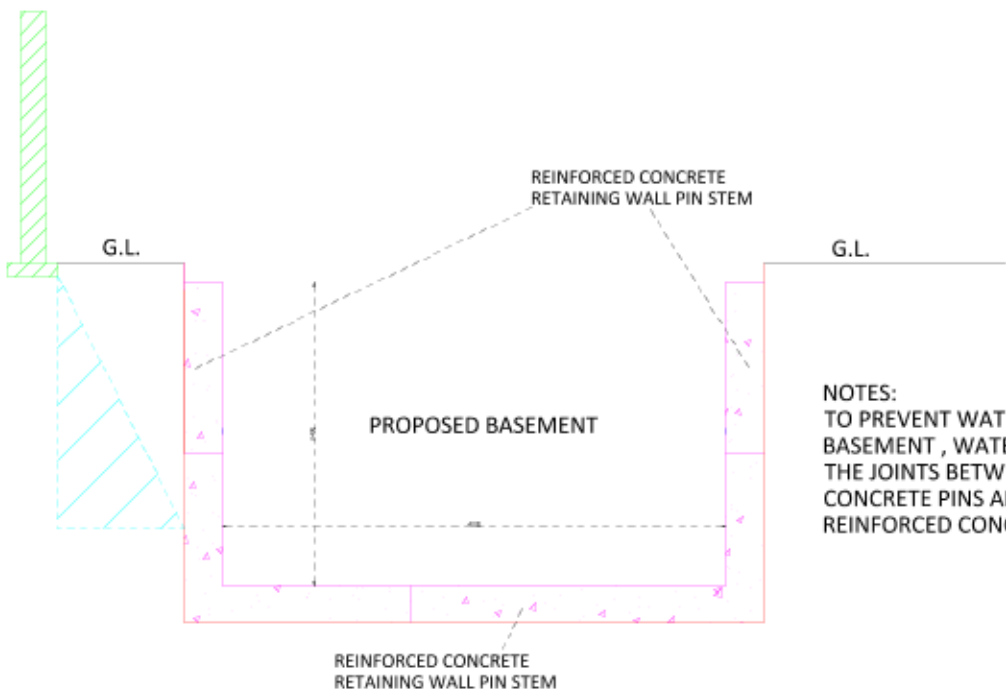
SECTION : A
STAGE N.3: EXCAVATION FOR THE CONSTRUCTION OF THE REMAINING HEIGHT OF THE RETAINING WALL TO REACH THE PROPOSED HEIGHT;
If the soil is unstable , inserting for propping the soil:
n.3 x 300mm steel trench sheets Type Standard S275JRC;
Placed by hand in front the face of the excavation and propped by Acrow props.



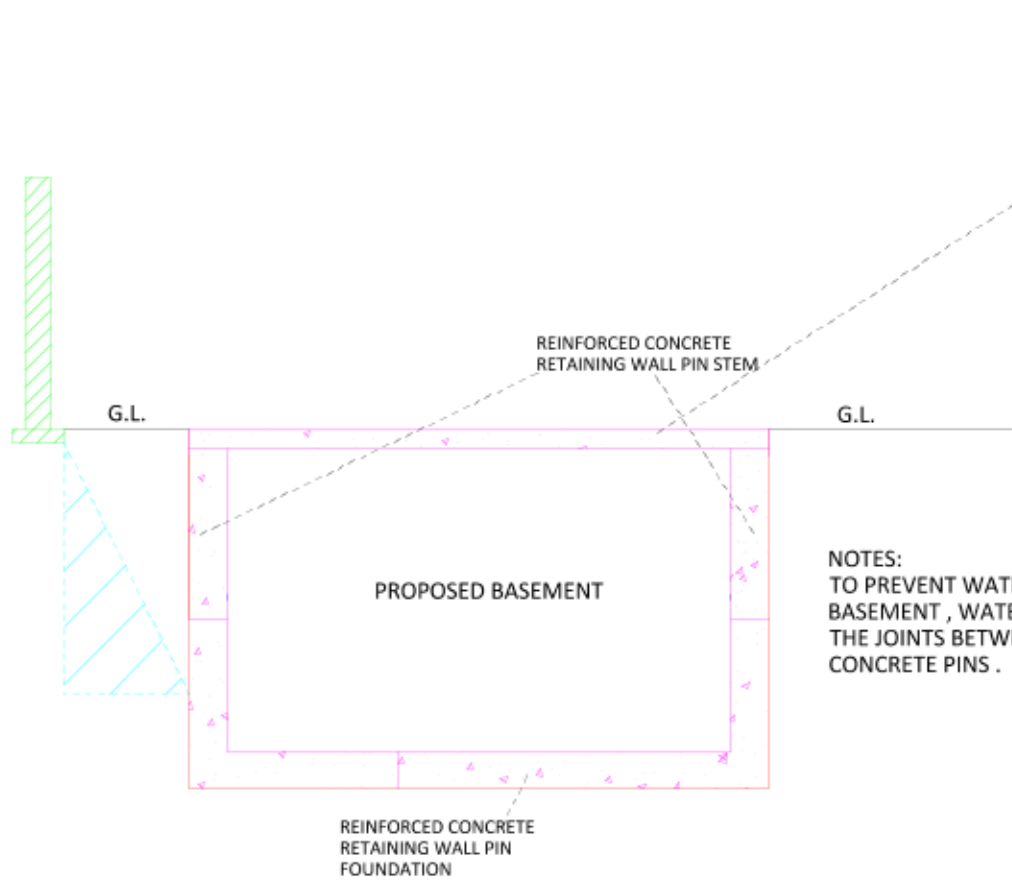
SECTION : A
STAGE N.4 :
CONSTRUCTION OF THE RETAINING WALL PINS UNDER THE FIRST HEIGHT OF 1.350m OF THE BASEMENT RETAINING WALLS ;



SECTION : A
STAGE N.5 :
CONSTRUCTION OF THE RETAINING WALL PINS UNDER THE FIRST HEIGHT OF 1.35m OF THE BASEMENT RETAINING WALLS ;

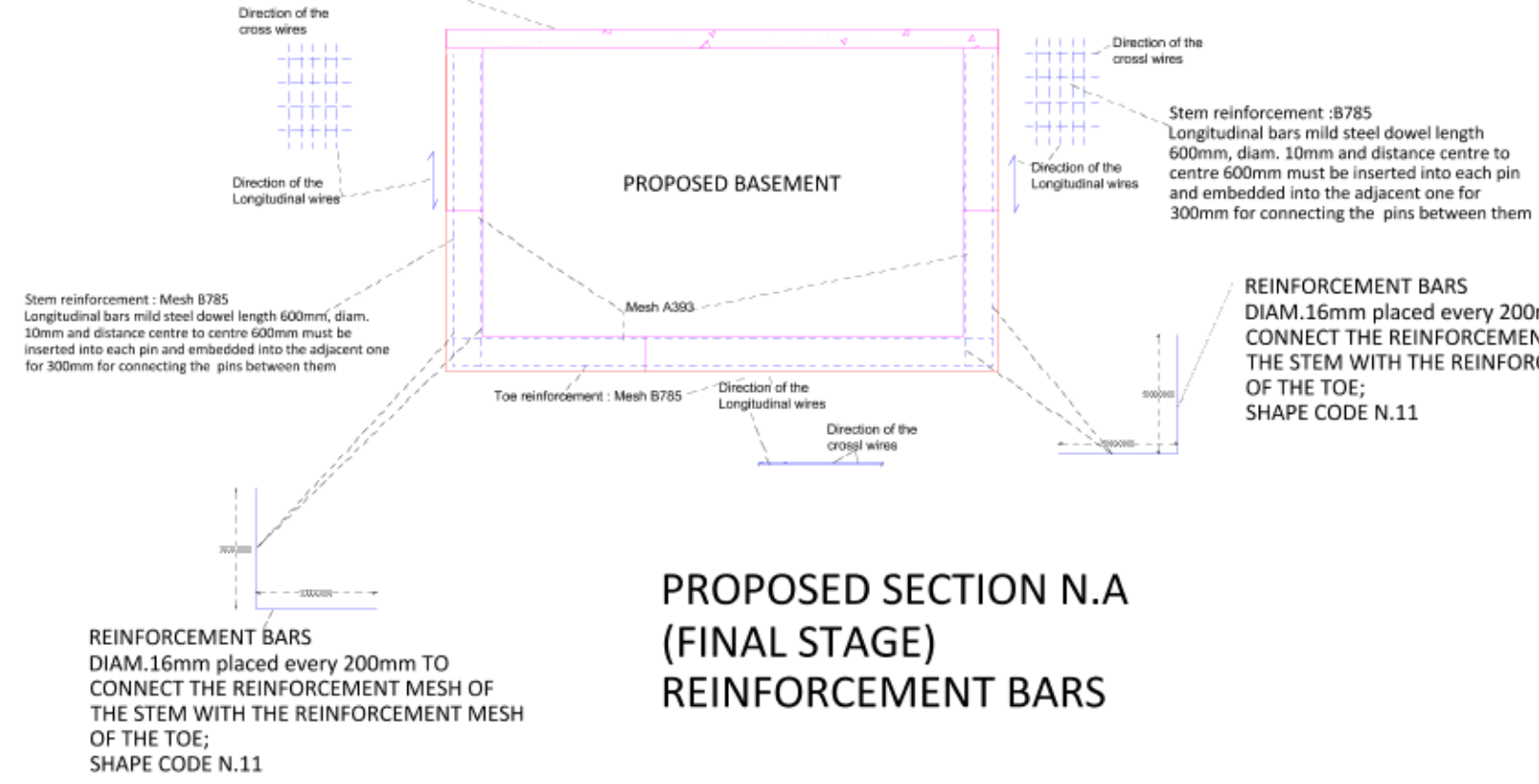


SECTION : A
STAGE N.6 :
CONSTRUCTION OF THE RETAINING WALL PINS UNDER THE FIRST HEIGHT OF 1.35m OF THE BASEMENT RETAINING WALLS ;



SECTION : A
FINAL STAGE

Concrete metal deck floor to be designed by the supplier with the following characteristics:
Type n.1
Unique span= 4.00m
and to withstand the following applied loads:
Dead loads:
Sedum grass roof: 0.9kN/m2
Finishes: 1.00kN/m2
Snow load: 0.75kN/m2
Imposed load: 1.5kN/m2
Total ultimate load applied: 6.11kN/m2
- Anti-crack Mesh: A 142;
- Slab depth =150mm;
- Fire resistance = 60 minutes;
- Concrete grade: Normal Concrete (NC) Grade C30/37 with a maximum aggregate size of 20mm.



PROPOSED SECTION N.A
(FINAL STAGE)
REINFORCEMENT BARS

PACA GEOTECHNICAL ENGINEERING LTD		
MARCH 2020	REF. :A496.20	PROPOSED SECTION N.A OF THE BASEMENT : CONSTRUCTION STAGES DRAWING N.5 SCALE 1:60@A1