15 Landor Road, London SW9 9RX
Basement Underpinning Structural Design Report

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Rev	Date	Comment
A0	10-03-2021	-

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1. Scope

The scope of this report is to provide the structural design report for the basement underpinning at subject address.

The report scope is the structural design for:

- Design of external side wall and party wall underpinning.
- Design of basement wall for front lightwell and rear garden wall.
- Design of Basement slab.
- Design of steel beams BB1, BB2, BB3 and BB4 supporting load bearing walls and/or floor joist.
- Design column BC1, BC2 supporting beam BB1.
- Design padstone BP1 supporting beams BB2 and BB4 (only one side).
- Design steel beams B4 and B5 supporting Chimney Breast.
- Design column C3 to support steel beam B4 and B5.
- Design Padstone P3 to support beam B4 (only one side) and P4 to support beam B5 (only one side).
- Design all connections: beam to beam, beam to column and column base plates.

Figure 1 15 Landor Road View

It is envisaged that a contractor who is experienced in this type of work is appointed to carry out the works. The contractor will be responsbile for the design and implementation of any temporary works necessary to build the basement. They will also be repsonsible for maintaining the stability of the existing structure throughout the works. A method statement showing the proposed temporary works shall be submitted by the chosen contractor for the review and approval of Beta Design Consultants.

2. Existing Structure

The existing property is end of terrace house arranged over three levels (ground, first and second floor levels).

Generally, the construction is typical of period properties in London. The main walls are masonry on corbelled out footings. The upper floors and roof are timber.

3. Proposal

The proposal invloves the underpinning of the new basement by 2400mm. Following the underpinning, and the laying of a new basement slab, double waterproofing systems shall be installed to comply with BS8102.

4. Potential Impact – Party Wall Matters

The structural design of the basement, wall and slab as well as the design of all necessary temporary works (contractor scope), and the sequencing of the construction; will take into account the all the geographical aspects and the locality of the adjoining properties. The underpinning of the existing house would be via sequantial mass concrete underpins in a sequential pattern, adopting contruction legs no wider than 1.0m folllowed by a reinforced concrete slab. This will avoid undue stresses being applied to the walls being underpinned. The works shall follow the Outline Method Statement included in this report. This method is generally accepted for basement construction of this type. Adoption of this method statement will limit any movement to the existing fabric of Adjoining Owner and any adjoining properties to ‘aesthetic’ as described by the BRE document for movement in buildings, or category of damage 1 under the Burland Scale.

5. Subterranean Conditions

The design is based on the following:

- Basement Impact assessment report document 18372/BIA_R38 prepared for the property by KK Facades Ltd, dated June 2020
- Geotechnical data are taken from Basement Impact assessment report .
- Borehole logs are taken from Basement Impact assessment report .

5.1 Site conditions are established from Basement Impact assessment report data. The ground condition in trial holes have describes as Made Ground typically comprised yellowish orange building SAND/concrete over soft black, slightly sandy,gravelly,light and dark brown sandy,slightly gravelly CLAY for 0.8m bgl. The London clay formation was incountered in one of the trial hole from directly below the Head to the final depth of 8.0m bgl. The soil conditions are known to be DRY at the proposed basement depth.

5.2 Records for borehole for the site show firm brown London clay formation from ~4m depth overlain by gravel. The basement would be founded on gravel (2.6m to 5.7m BGL) while the retained soil is stiff dark grey slightly sandy clay.

5.3 The groundwater was not encountered within any of the trial holes, but change of ground water level occur for seasonal effects and variations in drainage. The variable ground water was encountered at 2.39m bgl.The formation of the new basement will be above any water table and no de-watering to form the basement will be required.

5.4 The soils at new basement formation level will be LONDON CLAY OVERLAIN BY GRAVEL, and from Basement Impact assessment report data a safe bearing pressure on the clay of 85KN/m2, this value should ensure that differential and total settlements are very minimal.

5.5 The basement works will not affect any public services or utilities.

5.6 There are no nearby trees that will be affected by the works.

5.7 The design of the basement walls and bases and temporary works will take into account the locality of adjoining structures and any loading that may be imposed by these structures. The formation of the underpins and bases will be made in a sequential underpinning pattern adopting legs no wider than 1.0m will ensure help to avoid undue distress to the walls being underpinned.

Strata	Epoch	Depth Encountered (m bgl)		Typical Thickness (m)	Typical Description
		Top	Bottom		
MG	Anthropocene	GL	0.80 – 1.20[1]	0.90	Yellowish orange building SAND/concrete over soft black, slightly sandy, gravelly PEAT overlying soft, light and dark brown sandy, slightly gravelly CLAY with flint, concrete, brick, glass, clinker and wood gravel.
HEAD	Holocene	0.80	1.90	1.10	Soft to firm light orangish brown mottled grey very sandy CLAY.
LCF	Eocene	1.90	8.00[1] – 9.00[13]	Not proven[2]	Firm, orange brown mottled grey, slightly sandy CLAY becoming stiff, dark grey, slightly sandy CLAY.

Note: [1] Final depth of trial hole. [2] Base of strata not encountered. [3] Inferred from the results of dynamic probing.

Figure 2 Map showing site Ground condition taken from the Basement Impact assessment soils LIMITED

Contract Name: 15 Landor Road			Client: MrLeopold Ngouoto		Hole ID: WS1
Contract Number: 18372	Start and End Date: 01/05/20	Logged By: CVB/JO	Checked By: DVT	Status: FINAL	Hole Type: WS
Easting:	Northing:	Ground Level:	Plant Used: Premier 1	Print Date: 18/06/2020	Scale: 1:50
Weather:	Termination:				Sheet 1 of 1

Samples & In Situ Testing

Depth	Type	Results
0.00 - 1.00	B	
	J	
0.70	D	
1.20	D	
1.50	D	
1.80	D	
2.30	D	
2.80	D	
3.50	D	
4.50	D	
5.50	D	
6.50	D	
7.50	D	

Strata Details

Level (mAOD)	Depth (m) (Thickness)	Strata Description
	0.20	Yellowish orange building sand. Reworked ground. MADE GROUND.
	0.50 (0.30)	Dark brown very sandy slightly gravelly CLAY. Sand is fine to coarse. Gravel is fine to medium angular flint. Occasional to frequent fragments of brick. MADE GROUND.
	0.60	Light brown very sandy slightly gravelly CLAY. Sand is fine to coarse. Gravel is fine to medium angular flint. Occasional fragments of brick. MADE GROUND.
	0.80	Soft light brown very sandy slightly gravelly CLAY. Sand is fine to coarse. Gravel is fine to medium angular flint. Occasional fragments of brick. MADE GROUND.
	1.90 (1.10)	Soft to firm light orangish brown mottled grey very sandy CLAY. Sand is fine to coarse. Rare sub-angular fine chalk gravel. HEAD.
	4.10 (2.20)	Firm orange brown mottled grey slightly sandy CLAY. Sand is fine to coarse. LONDON CLAY FORMATION.
	4.20	firm brown sandy fragmented CLAYSTONE. LONDON CLAY FORMATION.
	7.10 (2.90)	Firm orange brown mottled grey slightly sandy CLAY. With occasional fine to medium, sub-rounded to sub angular claystone pockets Sand is fine to coarse. LONDON CLAY FORMATION.
	8.00 (0.90)	Stiff dark grey slightly sandy CLAY. LONDON CLAY FORMATION.
		End of Borehole at 8.00m

Groundwater: Water Strike | Backfill/Installation

Start & End of Shift Observations

Date	Time	Depth (m)	Casing (m)	Water (m)

Borehole Diameter

Depth (m)	Dia (mm)

Casing Diameter

Depth (m)	Dia (mm)

Remarks:
Roots and rootlets encountered to 1.90m bgl.
No groundwater encountered.

Chiselling

From (m)	To (m)	Duration	Remarks

Installation

Top (m)	Base (m)	Type	Dia (mm)
0.00	1.00	PLAIN	50
1.00	8.00	SLOTTED	50

Water Strikes

Strike (m)	Casing (m)	Sealed (m)	Time (mins)	Rose to (m)	Remarks

Hand vane (HV), Hand penetrometer (HP) reported in kPa. PID reported in ppm.

Figure 3 Records for borehole in site taken from the Basement Impact assessment

6. Structural Design Principles

Underpins are designed as propped cantilievers in reinforced concrete. The basement slab acts as the prop at base level. The walls are designed using parameters relevant to the London Clay. Even though no water table was found in the trial pits, in boreholes in the area or in basements in the area, the design adopts a water head behind the wall to ¾ the height of the wall below ground in accordance with BS 8102. The surcharge load allowed on the external walls of the property will be 10KN/m^2 .The design will consider floor loading, any partition wall construction and will also take into account any loads from structure above.

The basement slab will be formed in reinforced concrete. It will be designed for uplift due to water pressure below. The basement slab will act as a prop to the base of the basement walls.

7. Design Criteria

Basement walls and bases are designed using the program 'TEDDS' parameters for the retained soils and bearing soils are as chosen for each particular project. The design is in accordance with BS 8002:1994. The design adopts the coulomb theory in calculating the active and passive earth pressures. Pressure coeficients in the design adopt ' at rest pressures'.

The wall and base are designed for the following

1. Vertical loads from walls above.
2. Party wall will be designed for a surcharge loading of 10kN/m2.
3. Other external walls, will be designed with a surchage load of 10KN/m2.
4. The design adopts a water head behind the wall to ¾ the height of the wall below ground in accordance with BS 8102. (EVEN THOUGH NO WATER TABLE WAS FOUND IN THE TRIAL PITS)
5. An allowable increase in bearing pressure at base formation on the LONDON CLAY will be taken at 85KN/m2 this will limit settlements as noted above.
6. Concrete will generally be grade C30/35 and Class 1 to BRE Digest 363. Reinforcement will be grade 500N/mm2.
7. Existing brickwork assumes 10N bricks in a lime mortar, CP.111 gives basic compressive stress for this makeup of 0.70N/mm2, and therefore allowable bearing stress will be 0.70N/mm2. Any bearings into existing external or party wall masonry will take account of this allowable stress. Mortar will be class (ii) or (iii) as required.

8. Design data

8.1 Materials

Steel Young's Modulus (Mpa)	E =	210000
Yield Strength Steel Grade S355 (Mpa)	F_y =	355
Steel Specific Gravity (kN/m3)	SG_S =	78.6
Masonry Specific Gravity – Brickwork (kN/m3)	SG_M =	22
Masonry Specific Gravity - Blockwork 7N/mm2	SG_B =	15
Masonry Specific Gravity – Aircrete (kN/m3)	SG_B =	15
Blockwork Specific Gravity - hollow/hourdi (kN/m3)	SG_P =	10
Timber Specific Gravity (kN/m3)	SG_T =	6
Plaster Specific Gravity (kN/m3)	SG_P =	10
Gypsum boards Specific Gravity (kN/m3)	SG_{GY} =	6.55
Reinforced concrete Specific Gravity (kN/m3)	SG_{RC} =	25.00
Concrete Specific Gravity (kN/m3)	SG_C =	24.00
Concrete Grade C30/37 (Mpa)	F_{yck} =	30

8.2 Design Codes

Component	Year	Code
Loading: imposed loading, material weights, wind	1996	BS6399
Design of Steelwork Structures	2005	EC3
Structural Use of Timber	2002	BS5268-Part 2
Code of Practice for Use of Masonry	2005	BS5628-Part 1
Code of Practice for Foundations	2015	BS8004
Code of Practice for earth retaining structures	1994	BS8002

8.3 Design Data

Floor Live Loads (First Floor, Future Loft)	LL_{Floor} =	1.5	kN/m^2	Ref BS EN 1991-1-1-2002 Cat A Floors
Roof Live Load	LL_{Roof} =	0.5	kN/m^2	Ref BS EN 1991-1-1-2002 30^0 Roofs
Factor of Safety, Permanent (Dead) Loads	γ_G =	1.4		Ref UK NA BS EN 1990
Factor of Safety, Variable (Live) Loads	γ_Q =	1.6		Ref UK NA BS EN 1990
Factor of Safety, Material Strength Bending	γ_M =	1.05		For structural steel in bending
Factor of Safety, Material Strength Shear	γ_{M01} =	1.15		For structural steel in shear
Factor of Safety, Material Strength Compression	γ_{MC} =	3.5		Normal Control, Cat II, Table 4 BS5628
Factor of Safety, Material Strength Bending	γ_{MC} =	3.0		For masonry in bending
Factor of Safety, Material Strength Bending	γ_M =	1.3		For structural timber in bending

8.4 Loading Data

Flat roof

Dead Loads	
Felt and chippings	0.45 kN/m^2
Boards and joists	0.20 kN/m^2
Ceiling	0.20 kN/m^2
Services	0.15 kN/m^2
Total Dead Load	**1.00 kN/m^2**
Imposed Load	**0.75 kN/m^2**

Pitched Roof

Dead Loads	
Slate and felt	0.35 kN/m^2
Boards and joists	0.25 kN/m^2
Ceiling	0.25 kN/m^2
Services	0.15 kN/m^2
Total Dead Load	**1.00 kN/m^2**
Imposed Load Roof	0.75 kN/m^2
Total Imposed Loading	**0.75 kN/m^2**

Timber Floors

Dead Loads	
Boards and joists	0.35 kN/m^2
Ceiling	0.20 kN/m^2
Services	0.20 kN/m^2
Total Dead Load	**0.75 kN/m^2**
Imposed Load	**1.50 kN/m^2**

Suspended slab Floors (200 thk slab)

Dead Loads	
Screed	1.20 kN/m^2
Floor swt	4.60 kN/m^2
Services	0.20 kN/m^2
Total Dead Load	**6.00 kN/m^2**
Imposed Load (plant)	**7.50 kN/m^2**

Walls

105 Brickwork + plaster	**2.60 kN/m^2**
215 Brickwork + plaster	**5.10 kN/m^2**
330 Brickwork + Plaster	**7.20 kN/m^2**
Cavity brick / block (100/100)	**4.20 kN/m^2**
Stud partitions (on elevation)	**0.70 kN/m^2**

9. BB1: UC254x254x73 S355 Span 4.5M

Steel Beam GF BB1		Section	Load	UDL (kN/m2)	Length (m)	Contribution Factor	Calculation	Loads	Units	Position
4500mm		wall Load 330thk	Dead	7.2	8.1	1	7.2 x 8.1 x 1	58.32	kN/m	For 1 m. width from LHS
		wall Load 330thk	Dead	7.2	8.1	1	7.2 x 8.1 x 1	58.32	kN/m	For 0.5 m. width from RHS
		Bifolding door load	Dead	0.5	3.2	1	0.5 x 3.2 x 1	1.60	kN/m	For 3.2m width from RHS
		Column reaction GF	Dead(kN/m)	45.48	3.2	0.5	45.48 x 3.2 x 0.5	72.77	kN	at 1.0m from LHS
		Column reaction GF	Dead(kN/m)	45.48	3.2	0.5	45.48 x 3.2 x 0.5	72.77	kN	at 0.5m from RHS
		Column reaction GF	Live(kN/m)	8.94	3.2	0.5	8.94 x 3.2 x 0.5	14.30	kN	at 1.0m from LHS
		Column reaction GF	Live(kN/m)	8.94	3.2	0.5	8.94 x 3.2 x 0.5	14.30	kN	at 0.5m from RHS
		Beam Reaction B3	Dead(kN/m)	1.91	4.5	0.5	1.9125 x 4.5 x 0.5	4.30	kN	1.m from LHS
		Beam Reaction B3	Live(kN/m)	3.83	4.5	0.5	3.825 x 4.5 x 0.5	8.61	kN	1.m from LHS
		Beam Reaction B3	Dead(kN/m)	4.30	1	0.5	4.3 x 1 x 0.5	2.15	kN	1.m from LHS
		Beam Reaction B3	Live(kN/m)	8.61	1	0.5	8.61 x 1 x 0.5	4.30	kN	1.m from LHS
	UDL	SUM UDL	Dead				Dead UDL =	58.32	kN/m	For 1 m. width from LHS
								58.32	kN/m	For 0.5 m. width from RHS
								1.60	kN/m	For 3.2m width from RHS
	UDL	SUM UDL	Live				Live UDL =	0.00	kN/m	Full UDL
	Point	Point	Dead				Dead Point =	79.22	kN	at 1.0m from LHS
	Point	Point	Dead				Dead Point =	72.77	kN	at 0.5m from RHS
	Point	Point	Live				Live Load =	27.21	kN	at 1.0m from LHS
UC 254x254x73	Point	Point	Live				Live Load =	14.30	kN	at 0.5m from RHS

Beam deflection is checked for L/250.

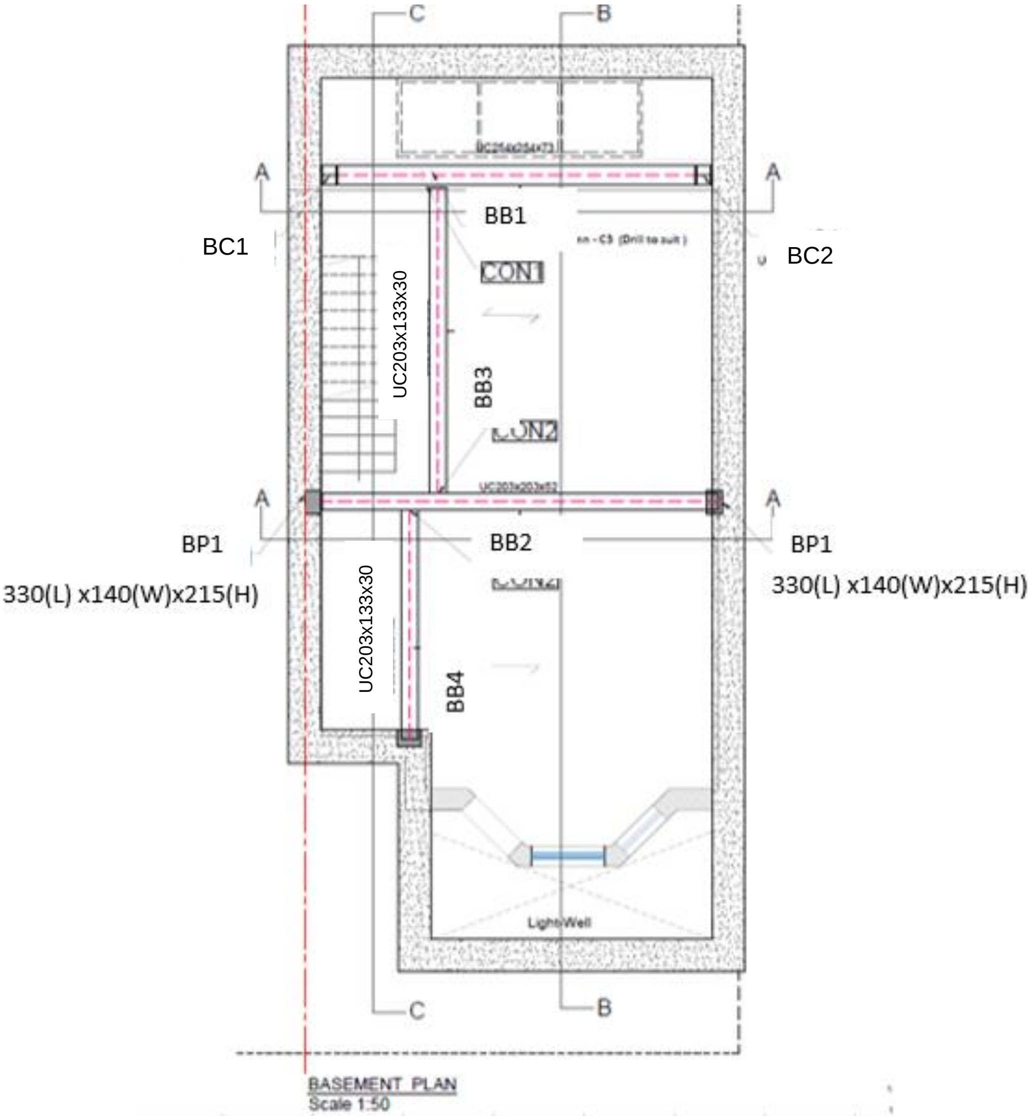


Figure 4: Location of Steel Beams on Plan showing BB1

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

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

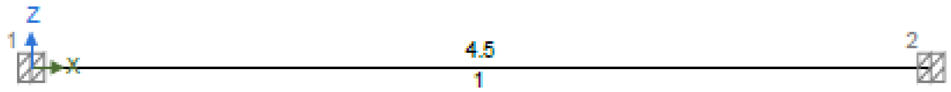
Tedds calculation version 4.3.04

ANALYSIS

Tedds calculation version 1.0.27

Geometry

Geometry (m) - Steel (EC3) - UC 254x254x73

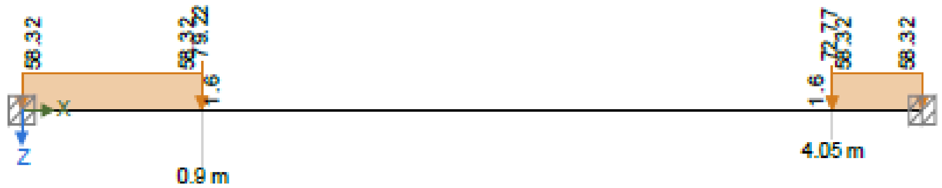


Span	Length (m)	Section	Start Support	End Support
1	4.5	UC 254x254x73	Fixed	Fixed
UC 254x254x73: Area 93 cm², Inertia Major 11407 cm⁴, Inertia Minor 3908 cm⁴, Shear area parallel to Minor 22 cm², Shear area parallel to Major = 65 cm²				
Steel (EC3): Density 7850 kg/m³, Youngs 210 kN/mm², Shear 80.8 kN/mm², Thermal 0.000012 °C⁻¹				

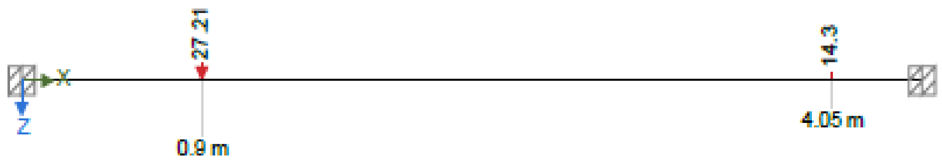
Loading

Self weight included

Permanent - Loading (kN/m,kN)



Imposed - Loading (kN)



Load combination factors

Load combination	Self Weight	Permanent	Imposed
1.35G + 1.5Q + 1.5RQ (Strength)	1.35	1.35	1.50
1.35G + 1.5Q + 1.5ψ₀S (Strength)	1.35	1.35	1.50
1.35G + 1.5ψ₀Q + 1.5S (Strength)	1.35	1.35	1.05

Load combination	Self Weight	Permanent	Imposed
$1.35G + 1.5Q + 1.5\psi_0 S + 1.5\psi_0 W$ (Strength)	1.35	1.35	1.50
$1.35G + 1.5\psi_0 Q + 1.5S + 1.5\psi_0 W$ (Strength)	1.35	1.35	1.05
$1.35G + 1.5\psi_0 Q + 1.5\psi_0 S + 1.5W$ (Strength)	1.35	1.35	1.05
$1.0G + 1.5W$ (Strength)	1.00	1.00	
$1.0G + 1.0W$ (Service)	1.00	1.00	
$1.35G + 1.5\psi_0 Q + 1.5\psi_0 RQ$ (Strength)	1.35	1.35	1.05
$1.35G + 1.5\psi_0 Q + 1.5\psi_0 S$ (Strength)	1.35	1.35	1.05
$1.35\xi G + 1.5Q + 1.5RQ$ (Strength)	1.25	1.25	1.50
$1.35\xi G + 1.5Q + 1.5\psi_0 S$ (Strength)	1.25	1.25	1.50
$1.35\xi G + 1.5\psi_0 Q + 1.5S$ (Strength)	1.25	1.25	1.05
$1.35G + 1.5\psi_0 Q + 1.5\psi_0 S + 1.5\psi_0 W$ (Strength)	1.35	1.35	1.05
$1.35\xi G + 1.5Q + 1.5\psi_0 S + 1.5\psi_0 W$ (Strength)	1.25	1.25	1.50
$1.35\xi G + 1.5\psi_0 Q + 1.5S + 1.5\psi_0 W$ (Strength)	1.25	1.25	1.05
$1.35\xi G + 1.5\psi_0 Q + 1.5\psi_0 S + 1.5W$ (Strength)	1.25	1.25	1.05
SERVICE LOAD CASE (Service)	1.00	1.00	1.00

Member Loads

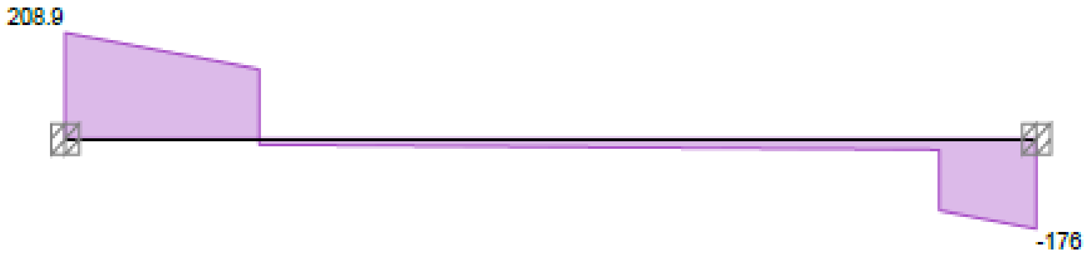
Member	Load case	Load Type	Orientation	Description
Member1	Permanent	UDL	GlobalZ	58.32 kN/m at 0 m to 0.9 m
Member1	Permanent	Point load	GlobalZ	79.22 kN at 0.9 m
Member1	Permanent	Point load	GlobalZ	72.77 kN at 4.05 m
Member1	Permanent	UDL	GlobalZ	58.32 kN/m at 4.05 m to 4.5 m
Member1	Permanent	UDL	GlobalZ	1.6 kN/m at 0.9 m to 4.05 m
Member1	Imposed	Point load	GlobalZ	27.21 kN at 0.9 m
Member1	Imposed	Point load	GlobalZ	14.3 kN at 4.05 m

Results

Forces

Strength combinations - Moment envelope (kNm)

Strength combinations - Shear envelope (kN)



Service combinations - Deflection envelope (mm)

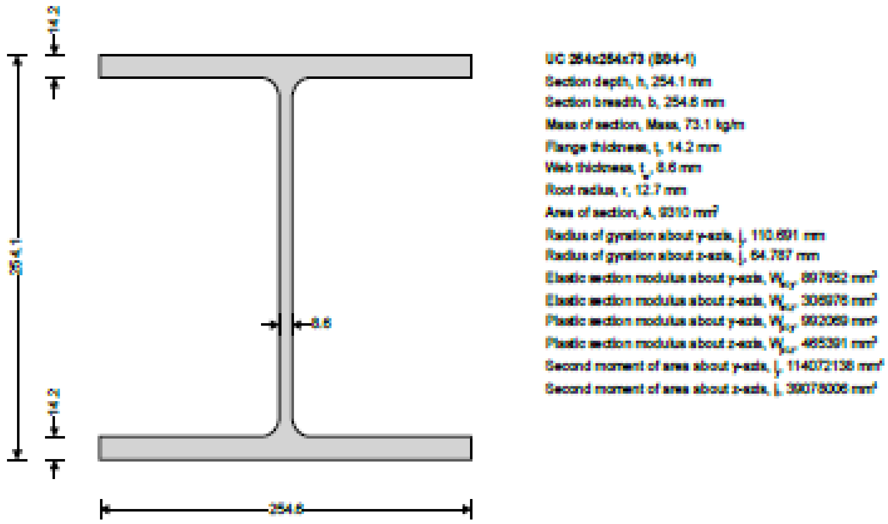
Partial factors - Section 6.1

Resistance of cross-sections	γ_{M0} = **1**
Resistance of members to instability	γ_{M1} = **1**
Resistance of tensile members to fracture	γ_{M2} = **1.1**

Member1 design

Section details

Section type	UC 254x254x73 (BS4-1)
Steel grade - EN 10025-2:2004	S355
Nominal thickness of element	$t_{nom} = \max(t_f, t_w)$ = **14.2** mm
Nominal yield strength	f_y = **355** N/mm^2
Nominal ultimate tensile strength	f_u = **470** N/mm^2
Modulus of elasticity	E = **210000** N/mm^2



Lateral restraint

Both flanges have lateral restraint at supports only

Consider Combination 6 - 1.35G + 1.5Q + 1.5ψ_0S + 1.5ψ_0W (Strength)

Classification of cross sections - Section 5.5

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

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

Width of section — $c = d$ = **200.3** mm

$c / t_w = 23.3 = 28.6 \times \varepsilon <= 72 \times \varepsilon$ — Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section — $c = (b - t_w - 2 \times r) / 2$ = **110.3** mm

$c / t_f = 7.8 = 9.5 \times \varepsilon <= 10 \times \varepsilon$ — Class 2

Section is class 2

Check design at start of span

Check shear - Section 6.2.6

Height of web — $h_w = h - 2 \times t_f$ = **225.7** mm — η = **1.000**

$h_w / t_w = 26.2 = 32.3 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force — $V_{y,Ed}$ = **208.9** kN

Shear area - cl 6.2.6(3) — $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **2562** mm^2

Design shear resistance - cl 6.2.6(2) — $V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **525.2** kN

$V_{y,Ed} / V_{c,y,Rd}$ = **0.398**

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment — $M_{y,Ed}$ = **117.6** kNm

Design bending resistance moment - eq 6.13 — $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0}$ = **352.2** kNm

$M_{y,Ed} / M_{c,y,Rd}$ = **0.334**

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 — k_c = **0.472**

$C_1 = 1 / k_c^2$ = **4.48**

Poissons ratio — ν = **0.3**

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

Unrestrained effective length — $L = 1.0 \times L_{m1\_s1\_seg1\_B}$ = **4500** mm

Elastic critical buckling moment — $M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **2890.6** kNm

Slenderness ratio for lateral torsional buckling — $\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})}$ = **0.349**

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

$\bar{\lambda}_{LT} < \bar{\lambda}_{LT,0}$ - Lateral torsional buckling can be ignored

Check design 900 mm along span

Check shear - Section 6.2.6

Height of web — $h_w = h - 2 \times t_f$ = **225.7** mm — η = **1.000**

$h_w / t_w = 26.2 = 32.3 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force $V_{y,Ed}$ = **137.2** kN

Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **2562** mm²

Design shear resistance - cl 6.2.6(2) $V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **525.2** kN

$V_{y,Ed} / V_{c,y,Rd}$ = **0.261**

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment $M_{y,Ed}$ = **38.1** kNm

Design bending resistance moment - eq 6.13 $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0}$ = **352.2** kNm

$M_{y,Ed} / M_{c,y,Rd}$ = **0.108**

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 k_c = **0.472**

$C_1 = 1 / k_c^2$ = **4.48**

Poissons ratio ν = **0.3**

Shear modulus $G = E / [2 \times (1 + \nu)]$ = **80769** N/mm²

Unrestrained effective length $L = 1.0 \times L_{m1\_s1\_seg1\_T}$ = **4500** mm

Elastic critical buckling moment $M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **2890.6** kNm

Slenderness ratio for lateral torsional buckling $\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})}$ = **0.349**

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

$\bar{\lambda}_{LT} < \bar{\lambda}_{LT,0}$ - Lateral torsional buckling can be ignored

Consider Combination 23 - SERVICE LOAD CASE (Service)

Check design 1785 mm along span

Check y-y axis deflection - Section 7.2.1

Maximum deflection δ_y = **1.8** mm

Allowable deflection $\delta_{y,Allowable} = \mathrm{Min}(L_{m1\_s1} / 250, 15\ mm)$ = **15** mm

$\delta_y / \delta_{y,Allowable}$ = **0.117**

PASS - Allowable deflection exceeds design deflection

10. BB2: UC203x203x52 S355 Span 5.1M

Steel Beam GF B2		Section	Load	UDL (kN/m2)	Length (m)	Contribution Factor	Calculation	Loads	Units	Position
5100mm										
		Beam Reaction B3	Dead(kN/m)	1.91	4.5	0.5	1.91 x 4.5 x 0.5	4.30	kN	1.m from LHS
		Beam Reaction B3	Live(kN/m)	3.83	4.5	0.5	3.83 x 4.5 x 0.5	8.61	kN	1.m from LHS
		Beam Reaction B3	Dead(kN/m)	4.30	1	0.5	4.3 x 1 x 0.5	2.15	kN	1.m from LHS
		Beam Reaction B3	Live(kN/m)	8.61	1	0.5	8.61 x 1 x 0.5	4.30	kN	1.m from LHS
		Beam Reaction B4	Dead(kN/m)	1.91	3.5	0.5	1.91 x 3.5 x 0.5	3.35	kN	0.8m from LHS
		Beam Reaction B4	Live(kN/m)	3.83	3.5	0.5	3.83 x 3.5 x 0.5	6.69	kN	0.8m from LHS
	UDL	SUM UDL	Dead				Dead Point =	6.45	kN	1.m from LHS
	UDL	SUM UDL	Live				Live Point =	12.91	kN	1.m from LHS
	Point	Point	Dead				Dead Point =	3.35	kN	0.8m from LHS
UC 203x203x52	Point	Point	Live				Live Point =	6.69	kN	0.8m from LHS

Beam effective length is taken as 1.2 L + 1.2 D to allow for support by bearing on padstone/metal plate.

Beam deflection is checked for L/250.

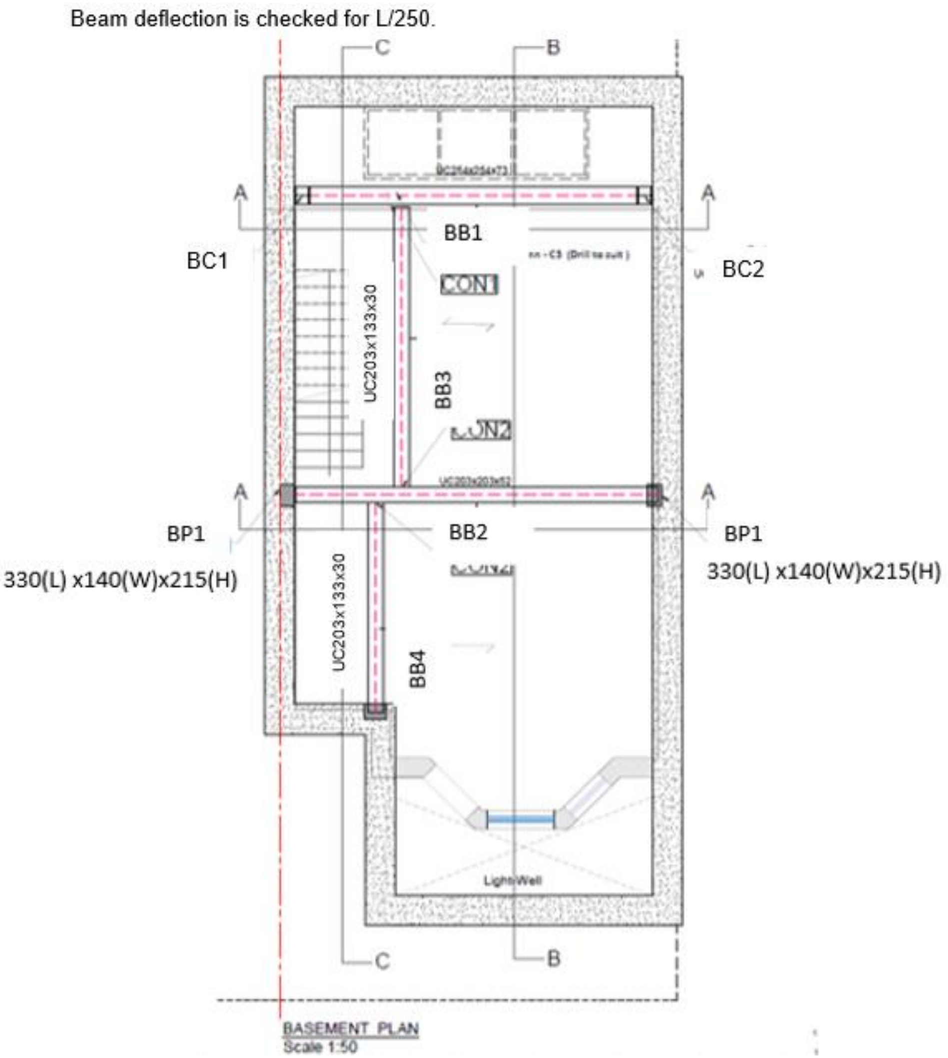


Figure 5: Location of Steel Beams on Plan showing BB2

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

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

Tedds calculation version 4.3.04

ANALYSIS

Tedds calculation version 1.0.27

Geometry

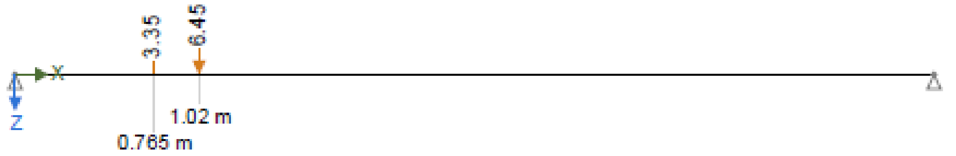
Geometry (m) - Steel (EC3) - UC 203x203x52

Span	Length (m)	Section	Start Support	End Support
1	5.1	UC 203x203x52	Pinned	Pinned
UC 203x203x52: Area 66 cm², Inertia Major 5259 cm⁴, Inertia Minor 1778 cm⁴, Shear area parallel to Minor 16 cm², Shear area parallel to Major = 46 cm²				
Steel (EC3): Density 7850 kg/m³, Youngs 210 kN/mm², Shear 80.8 kN/mm², Thermal 0.000012 °C⁻¹				

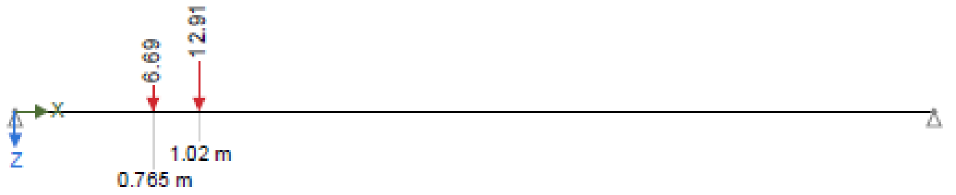
Loading

Self weight included

Permanent - Loading (kN)



Imposed - Loading (kN)



Load combination factors

Load combination	Self Weight	Permanent	Imposed
1.35G + 1.5Q + 1.5RQ (Strength)	1.35	1.35	1.50
1.35G + 1.5Q + 1.5ψ_0S (Strength)	1.35	1.35	1.50
1.35G + 1.5ψ_0Q + 1.5S (Strength)	1.35	1.35	1.05
1.35G + 1.5Q + 1.5ψ_0S + 1.5ψ_0W (Strength)	1.35	1.35	1.50

Load combination	Self Weight	Permanent	Imposed
1.35G + 1.5ψ_0Q + 1.5S + 1.5ψ_0W (Strength)	1.35	1.35	1.05
1.35G + 1.5ψ_0Q + 1.5ψ_0S + 1.5W (Strength)	1.35	1.35	1.05
1.0G + 1.5W (Strength)	1.00	1.00	
1.35G + 1.5ψ_0Q + 1.5ψ_0RQ (Strength)	1.35	1.35	1.05
1.35G + 1.5ψ_0Q + 1.5ψ_0S (Strength)	1.35	1.35	1.05
1.35ξG + 1.5Q + 1.5RQ (Strength)	1.25	1.25	1.50
1.35ξG + 1.5Q + 1.5ψ_0S (Strength)	1.25	1.25	1.50
1.35ξG + 1.5ψ_0Q + 1.5S (Strength)	1.25	1.25	1.05
1.35G + 1.5ψ_0Q + 1.5ψ_0S + 1.5ψ_0W (Strength)	1.35	1.35	1.05
1.35ξG + 1.5Q + 1.5ψ_0S + 1.5ψ_0W (Strength)	1.25	1.25	1.50
1.35ξG + 1.5ψ_0Q + 1.5S + 1.5ψ_0W (Strength)	1.25	1.25	1.05
1.35ξG + 1.5ψ_0Q + 1.5ψ_0S + 1.5W (Strength)	1.25	1.25	1.05
SERVICE (Service)	1.00	1.00	1.00

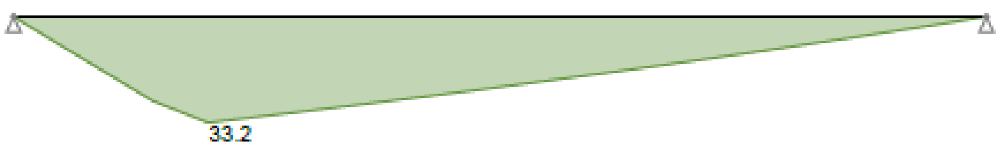
Member Loads

Member	Load case	Load Type	Orientation	Description
Member1	Permanent	Point load	GlobalZ	6.45 kN at 1.02 m
Member1	Permanent	Point load	GlobalZ	3.35 kN at 0.765 m
Member1	Imposed	Point load	GlobalZ	12.91 kN at 1.02 m
Member1	Imposed	Point load	GlobalZ	6.69 kN at 0.765 m

Results

Forces

Strength combinations - Moment envelope (kNm)



Strength combinations - Shear envelope (kN)

Service combinations - Deflection envelope (mm)

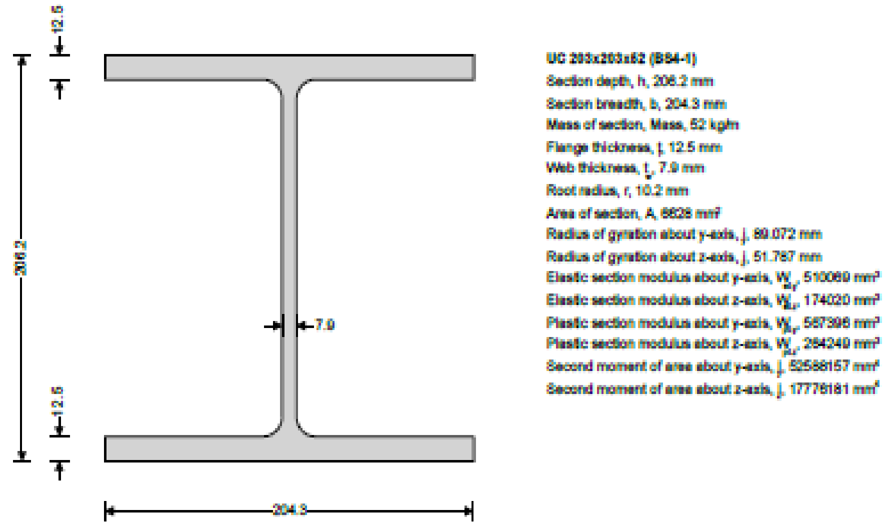
Partial factors - Section 6.1

Resistance of cross-sections	$\gamma_{M0} = 1$
Resistance of members to instability	$\gamma_{M1} = 1$
Resistance of tensile members to fracture	$\gamma_{M2} = 1.1$

Member1 design

Section details

Section type	UC 203x203x52 (BS4-1)
Steel grade - EN 10025-2:2004	S355
Nominal thickness of element	$t_{nom} = max(t_f, t_w) = 12.5$ mm
Nominal yield strength	$f_y = 355$ N/mm^2
Nominal ultimate tensile strength	$f_u = 470$ N/mm^2
Modulus of elasticity	$E = 210000$ N/mm^2



Lateral restraint

Both flanges have lateral restraint at supports only

Consider Combination 6 - 1.35G + 1.5Q + 1.5ψ₀S + 1.5ψ₀W (Strength)

Classification of cross sections - Section 5.5

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

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

Width of section	$c = d = 160.8$ mm
	$c / t_w = 20.4 = 25 \times \varepsilon <= 72 \times \varepsilon$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

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

$c / t_f = 7 = 8.7 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Section is class 1

<u>Check design 1020 mm along span</u>

Check shear - Section 6.2.6

Height of web — $h_w = h - 2 \times t_f = 181.2$ mm $\eta = 1.000$

$h_w / t_w = 22.9 = 28.2 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force — $V_{y,Ed} = 21.3$ kN

Shear area - cl 6.2.6(3) — $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 1875$ mm^2

Design shear resistance - cl 6.2.6(2) — $V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0} = 384.2$ kN

$V_{y,Ed} / V_{c,y,Rd} = 0.056$

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment — $M_{y,Ed} = 33.2$ kNm

Design bending resistance moment - eq 6.13 — $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = 201.4$ kNm

$M_{y,Ed} / M_{c,y,Rd} = 0.165$

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 — $k_c = 0.94$

$C_1 = 1 / k_c^2 = 1.132$

Poissons ratio — $\nu = 0.3$

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

Unrestrained effective length — $L = 1.2 \times L_{m1\_s1\_seg1\_T} + 2 \times h = 6532$ mm

Elastic critical buckling moment — $M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))} = 193.2$ kNm

Slenderness ratio for lateral torsional buckling — $\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} = 1.021$

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

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

Check buckling resistance - Section 6.3.2.1

Buckling curve - Table 6.5 — b

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

Correction factor for rolled sections — $\beta = 0.75$

LTB reduction determination factor — $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = 0.997$

LTB reduction factor - eq 6.57 — $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = 0.687$

Modification factor — $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = 0.973$

Modified LTB reduction factor - eq 6.58 — $\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2) = 0.706$

Design buckling resistance moment - eq 6.55 — $M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl,y} \times f_y / \gamma_{M1} = 142.2$ kNm

$M_{y,Ed} / M_{b,y,Rd} = 0.234$

PASS - Design buckling resistance moment exceeds design bending moment

<u>Consider Combination 23 - SERVICE (Service)</u>

<u>Check design 2215 mm along span</u>

Check y-y axis deflection - Section 7.2.1

Maximum deflection $\delta_y = 4.5$ mm

Allowable deflection $\delta_{y,Allowable} = Min(L_{m1\_s1} / 250, 15 mm) = 15$ mm

$\delta_y / \delta_{y,Allowable} = 0.297$

PASS - Allowable deflection exceeds design deflection

11. BB3: UB203x133x30 S355 Span 3.79M

Steel Beam GF BB3		Section	Load	UDL (kN/m2)	Length (m)	Contribution Factor	Calculation	Loads	Units	Position
4500mm		Column reaction GF UB152	Dead(kN/m)	1.9125	4.5	0.5	1.9125 x 4.5 x 0.5	4.30	kN	0.5m from LHS
		Column reaction GF UB152	Live(kN/m)	3.825	4.5	0.5	3.825 x 4.5 x 0.5	8.61	kN	0.5m from LHS
		Ground floor Joist	Dead	0.75	5.1	0.5	0.75 x 5.1 x 0.5	1.91	kN/m	Full UDL
		Ground floor Joist	Live	1.5	5.1	0.5	1.5 x 5.1 x 0.5	3.83	kN/m	Full UDL
	UDL									
	UDL									
		SUM UDL	Dead				Dead UDL =	1.91	kN/m	Full UDL
UC 203x133x30		SUM UDL	Live				Live UDL =	3.83	kN/m	Full UDL

Beam deflection is checked for L/250.

*Design is done for beam span = 4.5m, conservative , so OK

Beam deflection is checked for L/250.

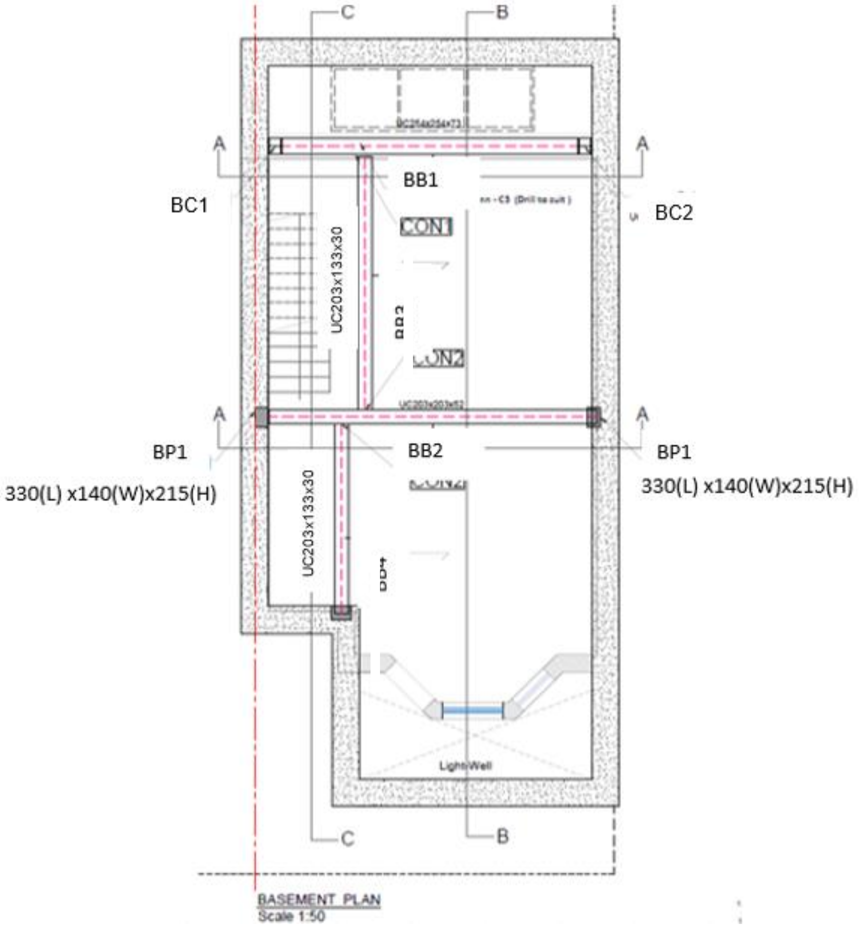


Figure 6: Location of Steel Beams on Plan showing BB3

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

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

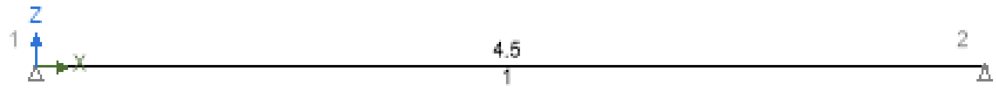
Tedds calculation version 4.3.04

ANALYSIS

Tedds calculation version 1.0.27

Geometry

Geometry (m) - Steel (EC3) - UB 203x133x30

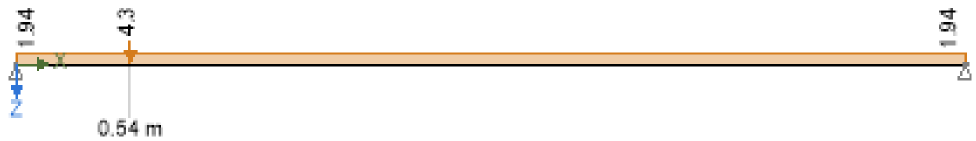


Span	Length (m)	Section	Start Support	End Support
1	4.5	UB 203x133x30	Pinned	Pinned
UB 203x133x30: Area 38 cm^2, Inertia Major 2896 cm^4, Inertia Minor 385 cm^4, Shear area parallel to Minor 13 cm^2, Shear area parallel to Major = 23 cm^2				
Steel (EC3): Density 7850 kg/m^3, Youngs 210 kN/mm^2, Shear 80.8 kN/mm^2, Thermal 0.000012 °C^{-1}				

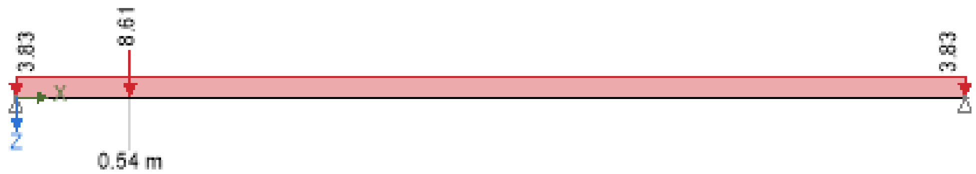
Loading

Self weight included

Permanent - Loading (kN/m,kN)



Imposed - Loading (kN/m,kN)



Load combination factors

Load combination	Self Weight	Permanent	Imposed
1.35G + 1.5Q + 1.5RQ (Strength)	1.35	1.35	1.50
1.35G + 1.5Q + 1.5ψ_0S (Strength)	1.35	1.35	1.50
1.35G + 1.5ψ_0Q + 1.5S (Strength)	1.35	1.35	1.05
1.35G + 1.5Q + 1.5ψ_0S + 1.5ψ_0W (Strength)	1.35	1.35	1.50

Load combination	Self Weight	Permanent	Imposed
$1.35G + 1.5\psi_0Q + 1.5S + 1.5\psi_0W$ (Strength)	1.35	1.35	1.05
$1.35G + 1.5\psi_0Q + 1.5\psi_0S + 1.5W$ (Strength)	1.35	1.35	1.05
$1.0G + 1.5W$ (Strength)	1.00	1.00	
$1.35G + 1.5\psi_0Q + 1.5\psi_0RQ$ (Strength)	1.35	1.35	1.05
$1.35G + 1.5\psi_0Q + 1.5\psi_0S$ (Strength)	1.35	1.35	1.05
$1.35\xi G + 1.5Q + 1.5RQ$ (Strength)	1.25	1.25	1.50
$1.35\xi G + 1.5Q + 1.5\psi_0S$ (Strength)	1.25	1.25	1.50
$1.35\xi G + 1.5\psi_0Q + 1.5S$ (Strength)	1.25	1.25	1.05
$1.35G + 1.5\psi_0Q + 1.5\psi_0S + 1.5\psi_0W$ (Strength)	1.35	1.35	1.05
$1.35\xi G + 1.5Q + 1.5\psi_0S + 1.5\psi_0W$ (Strength)	1.25	1.25	1.50
$1.35\xi G + 1.5\psi_0Q + 1.5S + 1.5\psi_0W$ (Strength)	1.25	1.25	1.05
$1.35\xi G + 1.5\psi_0Q + 1.5\psi_0S + 1.5W$ (Strength)	1.25	1.25	1.05
SERVICE LOAD (Service)	1.00	1.00	1.00

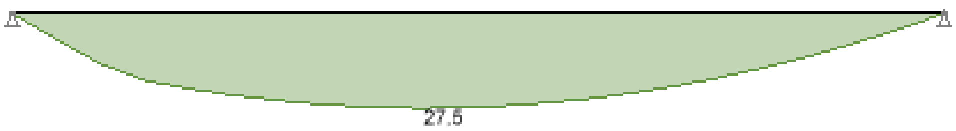
Member Loads

Member	Load case	Load Type	Orientation	Description
Member1	Permanent	UDL	GlobalZ	1.94 kN/m
Member1	Permanent	Point load	GlobalZ	4.3 kN at 0.54 m
Member1	Imposed	Point load	GlobalZ	8.61 kN at 0.54 m
Member1	Imposed	UDL	GlobalZ	3.83 kN/m

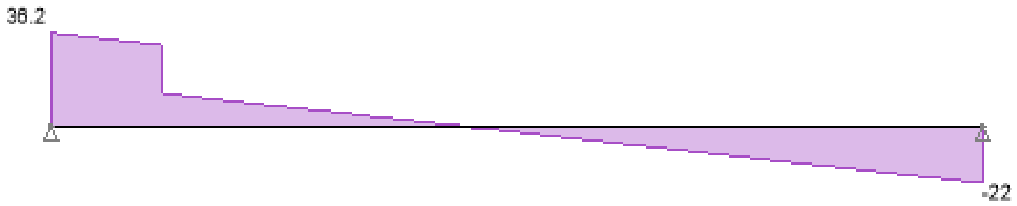
Results

Forces

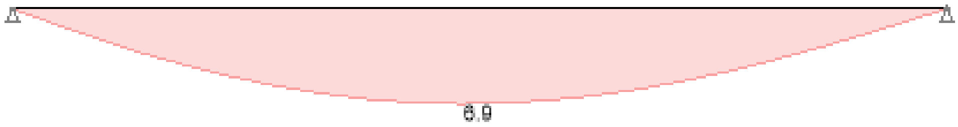
Strength combinations - Moment envelope (kNm)



Strength combinations - Shear envelope (kN)



Service combinations - Deflection envelope (mm)



Partial factors - Section 6.1

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

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

Resistance of tensile members to fracture γ_{M2} = **1.1**

Member1 design

Section details

Section type UB 203x133x30 (BS4-1)

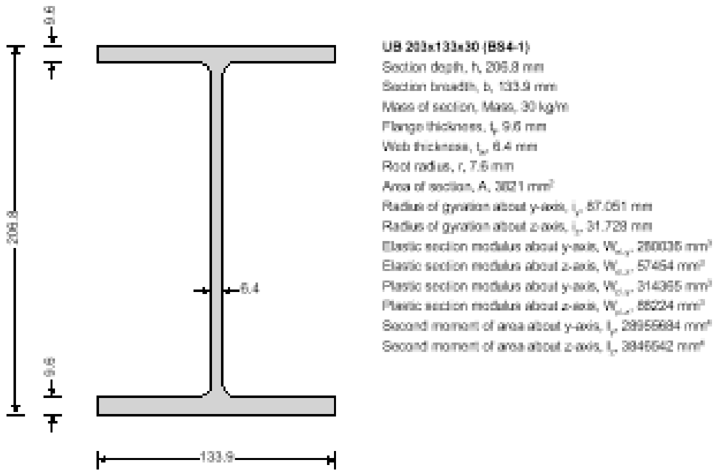
Steel grade - EN 10025-2:2004 S355

Nominal thickness of element t_{nom} = max(t_f, t_w) = **9.6** mm

Nominal yield strength f_y = **355** N/mm²

Nominal ultimate tensile strength f_u = **470** N/mm²

Modulus of elasticity E = **210000** N/mm²



Lateral restraint

Both flanges have lateral restraint at supports only

Consider Combination 6 - 1.35G + 1.5Q + 1.5ψ₀S + 1.5ψ₀W (Strength)

Classification of cross sections - Section 5.5

ε = √[235 N/mm² / f_y] = **0.81**

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

Width of section c = d = **172.4** mm

c / t_w = 26.9 = 33.1 × ε <= 72 × ε **Class 1**

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section $c = (b - t_w - 2 \times r) / 2 = 56.2$ mm

$c / t_f = 5.8 = 7.2 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Section is class 1

Check design 1994 mm along span

Check bending moment - Section 6.2.5

Design bending moment $M_{y,Ed} = 27.5$ kNm

Design bending resistance moment - eq 6.13 $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = 111.6$ kNm

$M_{y,Ed} / M_{c,y,Rd} = 0.247$

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 $k_c = 0.94$

$C_1 = 1 / k_c^2 = 1.132$

Poissons ratio $\nu = 0.3$

Shear modulus $G = E / [2 \times (1 + \nu)] = 80769$ N/mm²

Unrestrained effective length $L = 1.2 \times L_{m1\_s1\_seg1\_t} + 2 \times h = 5814$ mm

Elastic critical buckling moment $M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))} = 56.6$ kNm

Slenderness ratio for lateral torsional buckling $\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} = 1.404$

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

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

Check buckling resistance - Section 6.3.2.1

Buckling curve - Table 6.5 b

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

Correction factor for rolled sections $\beta = 0.75$

LTB reduction determination factor $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = 1.410$

LTB reduction factor - eq 6.57 $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = 0.471$

Modification factor $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = 0.992$

Modified LTB reduction factor - eq 6.58 $\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2) = 0.475$

Design buckling resistance moment - eq 6.55 $M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl,y} \times f_y / \gamma_{M1} = 53$ kNm

$M_{y,Ed} / M_{b,y,Rd} = 0.519$

PASS - Design buckling resistance moment exceeds design bending moment

Consider Combination 23 - SERVICE LOAD (Service)

Check design 2181 mm along span

Check y-y axis deflection - Section 7.2.1

Maximum deflection $\delta_y = 6.9$ mm

Allowable deflection $\delta_{y,Allowable} = \mathrm{Min}(L_{m1\_s1} / 250, 15 \text{ mm}) = 15$ mm

$\delta_y / \delta_{y,Allowable} = 0.462$

PASS - Allowable deflection exceeds design deflection

12. BB4: UB203x133x30 S355 Span 2.75M

Steel Beam GF BB4										
3500mm		Section	Load	UDL (kN/m2)	Length (m)	Contribution Factor	Calculation	Loads	Units	Position
		Ground floor Joist	Dead	0.75	5.1	0.5	0.75 x 5.1 x 0.5	1.91	kN/m	Full UDL
		Ground floor Joist	Live	1.5	5.1	0.5	1.5 x 5.1 x 0.5	3.83	kN/m	Full UDL
	UDL									
	UDL									
		SUM UDL	Dead				Dead UDL =	1.91	kN/m	Full UDL
UC 203x133x30		SUM UDL	Live				Live UDL =	3.83	kN/m	Full UDL

Beam effective length is taken as 1.2 L + 1.2 D to allow for support by bearing on padstone/metal plate.

Beam deflection is checked for L/250.

*Design is done for beam span = 3.5m, conservative, so OK

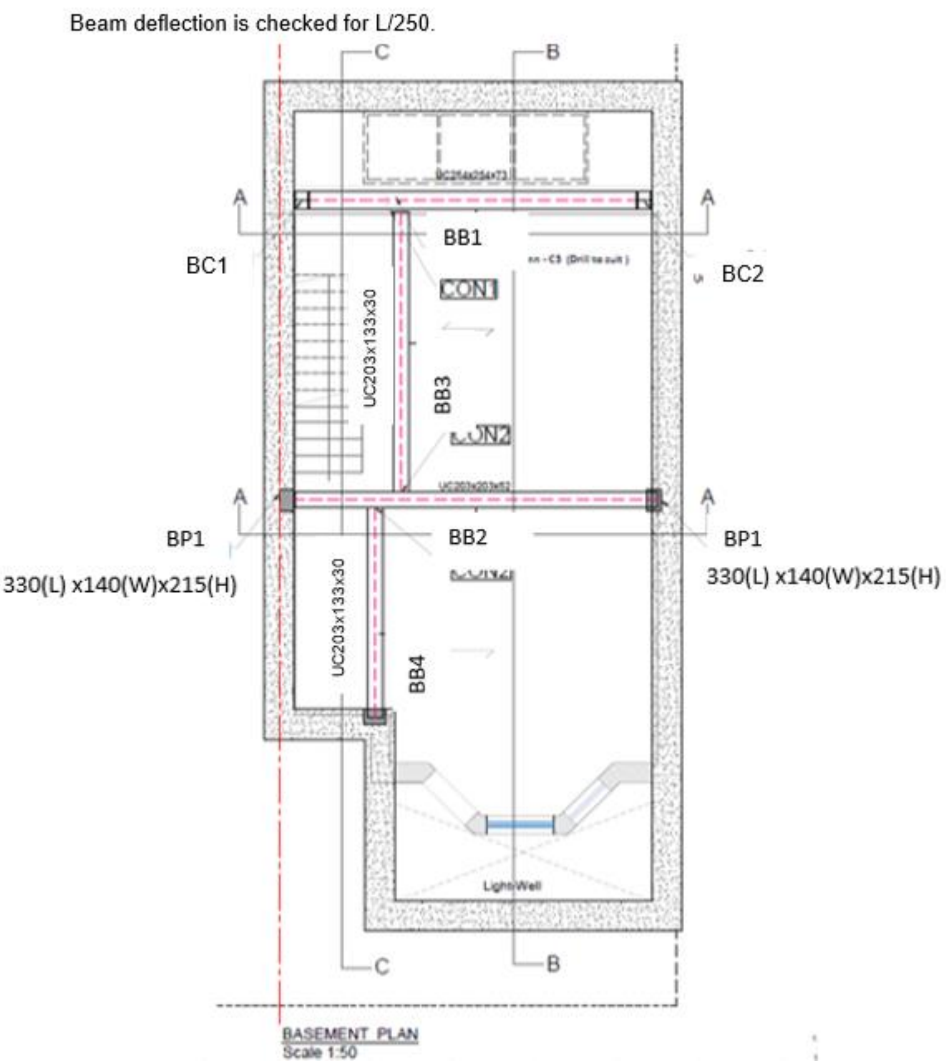


Figure 7: Location of Steel Beams on Plan showing BB4

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

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

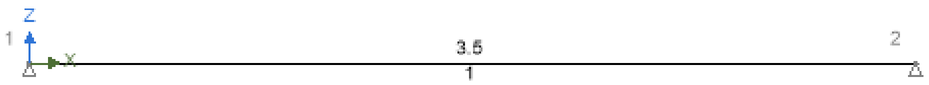
Tedds calculation version 4.3.04

ANALYSIS

Tedds calculation version 1.0.27

Geometry

Geometry (m) - Steel (EC3) - UB 203x133x30

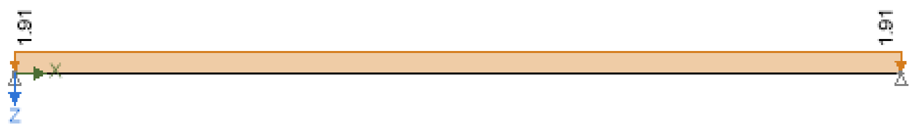


Span	Length (m)	Section	Start Support	End Support
1	3.5	UB 203x133x30	Pinned	Pinned
UB 203x133x30: Area 38 cm^2, Inertia Major 2896 cm^4, Inertia Minor 385 cm^4, Shear area parallel to Minor 13 cm^2, Shear area parallel to Major = 23 cm^2				
Steel (EC3): Density 7850 kg/m^3, Youngs 210 kN/mm^2, Shear 80.8 kN/mm^2, Thermal 0.000012 °C^{-1}				

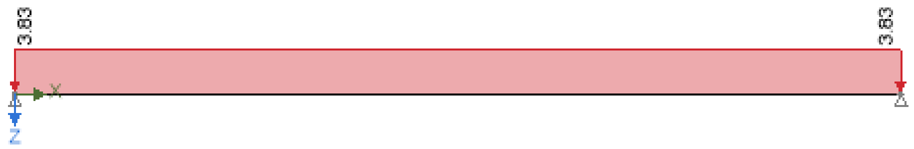
Loading

Self weight included

Permanent - Loading (kN/m)



Imposed - Loading (kN/m)



Load combination factors

Load combination	Self Weight	Permanent	Imposed
1.35G + 1.5Q + 1.5RQ (Strength)	1.35	1.35	1.50
1.35G + 1.5Q + 1.5ψ_0S (Strength)	1.35	1.35	1.50
1.35G + 1.5ψ_0Q + 1.5S (Strength)	1.35	1.35	1.05
1.35G + 1.5Q + 1.5ψ_0S + 1.5ψ_0W (Strength)	1.35	1.35	1.50
1.35G + 1.5ψ_0Q + 1.5S + 1.5ψ_0W (Strength)	1.35	1.35	1.05

Load combination	Self Weight	Permanent	Imposed
1.35G + 1.5ψ_0Q + 1.5ψ_0S + 1.5W (Strength)	1.35	1.35	1.05
1.0G + 1.5W (Strength)	1.00	1.00	
1.35G + 1.5ψ_0Q + 1.5ψ_0RQ (Strength)	1.35	1.35	1.05
1.35G + 1.5ψ_0Q + 1.5ψ_0S (Strength)	1.35	1.35	1.05
1.35ξG + 1.5Q + 1.5RQ (Strength)	1.25	1.25	1.50
1.35ξG + 1.5Q + 1.5ψ_0S (Strength)	1.25	1.25	1.50
1.35ξG + 1.5ψ_0Q + 1.5S (Strength)	1.25	1.25	1.05
1.35G + 1.5ψ_0Q + 1.5ψ_0S + 1.5ψ_0W (Strength)	1.35	1.35	1.05
1.35ξG + 1.5Q + 1.5ψ_0S + 1.5ψ_0W (Strength)	1.25	1.25	1.50
1.35ξG + 1.5ψ_0Q + 1.5S + 1.5ψ_0W (Strength)	1.25	1.25	1.05
1.35ξG + 1.5ψ_0Q + 1.5ψ_0S + 1.5W (Strength)	1.25	1.25	1.05
SERVICE (Service)	1.00	1.00	1.00

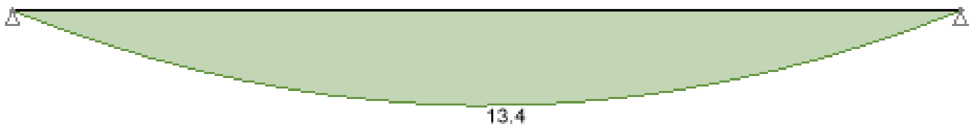
Member Loads

Member	Load case	Load Type	Orientation	Description
Member1	Permanent	UDL	GlobalZ	1.91 kN/m
Member1	Imposed	UDL	GlobalZ	3.83 kN/m

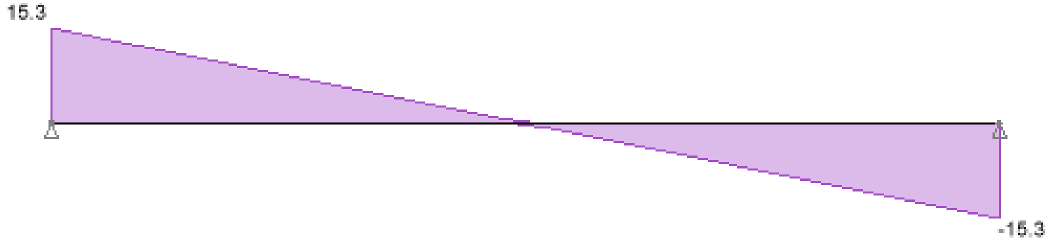
Results

Forces

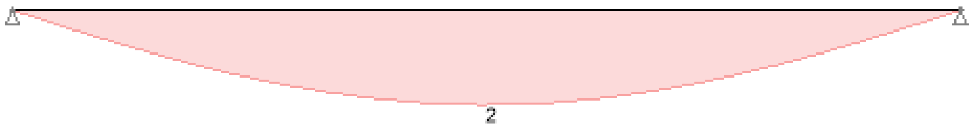
Strength combinations - Moment envelope (kNm)



Strength combinations - Shear envelope (kN)



Service combinations - Deflection envelope (mm)



Partial factors - Section 6.1

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

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

Resistance of tensile members to fracture γ_{M2} = **1.1**

Member1 design

Section details

Section type UB 203x133x30 (BS4-1)

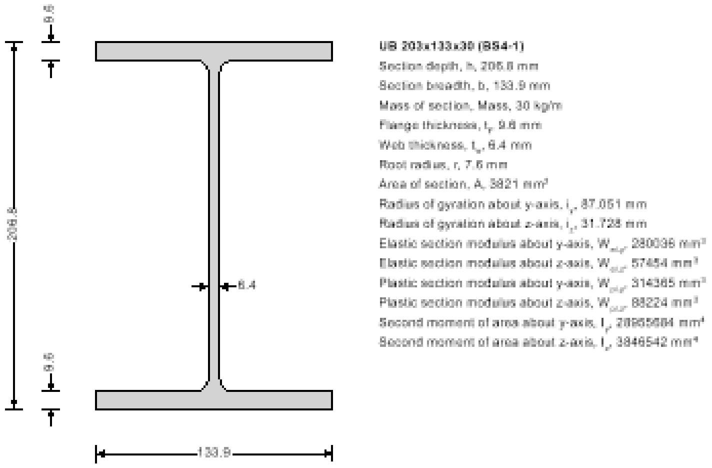
Steel grade - EN 10025-2:2004 S355

Nominal thickness of element t_{nom} = max(t_f, t_w) = **9.6** mm

Nominal yield strength f_y = **355** N/mm²

Nominal ultimate tensile strength f_u = **470** N/mm²

Modulus of elasticity E = **210000** N/mm²



Lateral restraint

Both flanges have lateral restraint at supports only

Consider Combination 6 - 1.35G + 1.5Q + 1.5ψ_0S + 1.5ψ_0W (Strength)

Classification of cross sections - Section 5.5

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

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

Width of section c = d = **172.4** mm

$c / t_w = 26.9 = 33.1 \times \varepsilon <= 72 \times \varepsilon$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section c = (b - t_w - 2 × r) / 2 = **56.2** mm

$c / t_f = 5.8 = 7.2 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Section is class 1

Check design 1750 mm along span

Check bending moment - Section 6.2.5

Design bending moment — $M_{y,Ed}$ = **13.4** kNm

Design bending resistance moment - eq 6.13 — $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0}$ = **111.6** kNm

$M_{y,Ed} / M_{c,y,Rd}$ = **0.12**

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 — k_c = **0.94**

$C_1 = 1 / k_c^2$ = **1.132**

Poissons ratio — ν = **0.3**

Shear modulus — $G = E / [2 \times (1 + \nu)]$ = **80769** N/mm²

Unrestrained effective length — $L = 1.2 \times L_{m1\_s1\_seg1\_1} + 2 \times h$ = **4614** mm

Elastic critical buckling moment — $M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **75.8** kNm

Slenderness ratio for lateral torsional buckling — $\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})}$ = **1.214**

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

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

Check buckling resistance - Section 6.3.2.1

Buckling curve - Table 6.5 — b

Imperfection factor - Table 6.3 — α_{LT} = **0.34**

Correction factor for rolled sections — β = **0.75**

LTB reduction determination factor — $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **1.191**

LTB reduction factor - eq 6.57 — $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.571**

Modification factor — $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.980**

Modified LTB reduction factor - eq 6.58 — $\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.583**

Design buckling resistance moment - eq 6.55 — $M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl,y} \times f_y / \gamma_{M1}$ = **65** kNm

$M_{y,Ed} / M_{b,y,Rd}$ = **0.205**

PASS - Design buckling resistance moment exceeds design bending moment

Consider Combination 23 - SERVICE (Service)

Check design 1750 mm along span

Check y-y axis deflection - Section 7.2.1

Maximum deflection — δ_y = **2** mm

Allowable deflection — $\delta_{y,Allowable} = \mathrm{Min}(L_{m1\_s1} / 250, 15\ \mathrm{mm})$ = **14** mm

$\delta_y / \delta_{y,Allowable}$ = **0.145**

PASS - Allowable deflection exceeds design deflection

13. BC1 and BC2: UC203x203x52 S355 Span 2.4M

Steel Column GF C1										
2400mm		Section	Load	UDL (kN/m2)	Length (m)	Contribution Factor	Calculation	Loads	Units	Position
		Beam Reaction B1	Dead					103.00	kN	
		Beam Reaction B1	Live					41.50	kN	
	UDL									
	UDL									
		SUM UDL	Dead				Dead UDL =	103.00	kN	
UC 203x203x52		SUM UDL	Live				Live UDL =	41.50	kN	

Beam deflection is checked for L/250.

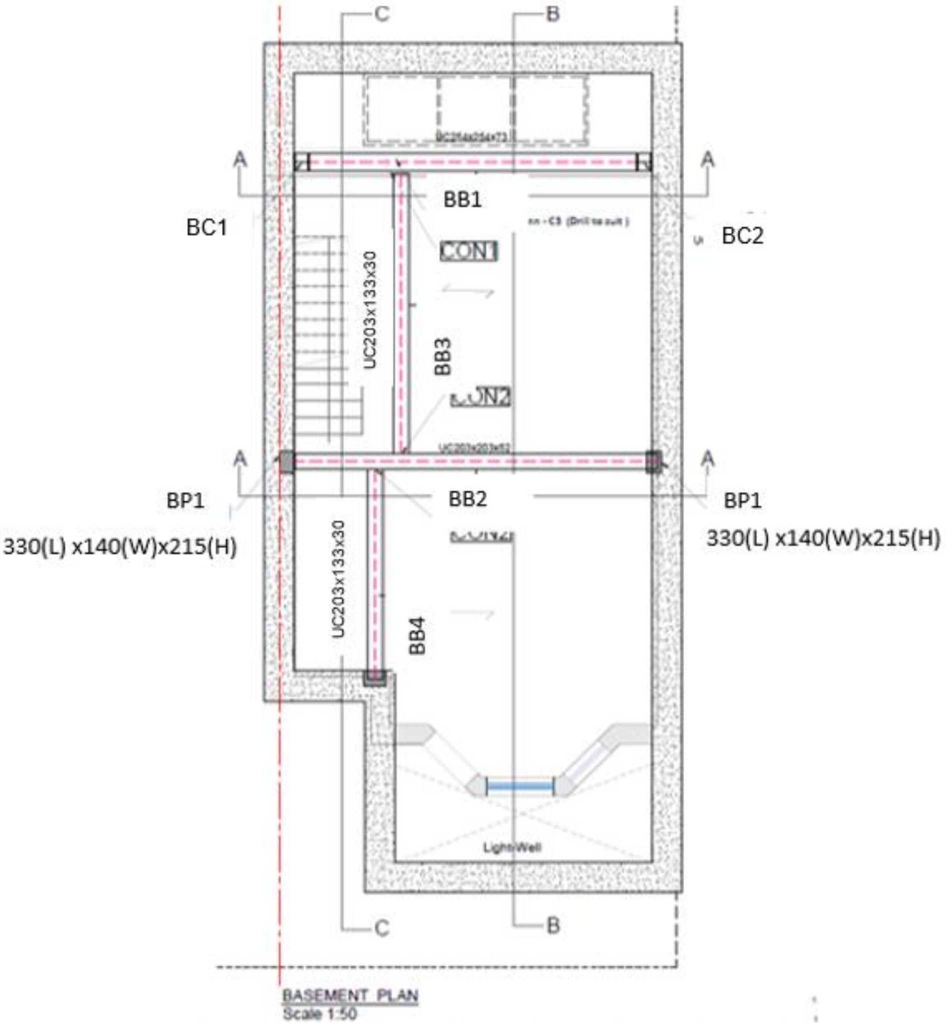


Figure 8: Location of Steel Beams on Plan showing C1

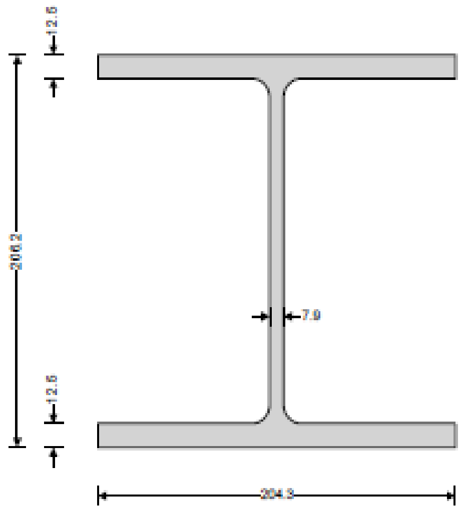
STEEL COLUMN DESIGN

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

Tedds calculation version 1.1.04

Partial factors - Section 6.1

Resistance of cross-sections	γ_{M0} = **1**
Resistance of members to instability	γ_{M1} = **1**
Resistance of cross-sections in tension to fracture	γ_{M2} = **1.1**



UKC 203x203x52 (Tata Steel Advance)
Section depth, h, 206.2 mm
Section breadth, b, 204.3 mm
Mass of section, Mass, 52 kg/m
Flange thickness, t_f, 12.5 mm
Web thickness, t_w, 7.9 mm
Root radius, r, 10.2 mm
Area of section, A, 6628 mm²
Radius of gyration about y-axis, i_y, 89.072 mm
Radius of gyration about z-axis, i_z, 51.787 mm
Elastic section modulus about y-axis, $W_{el,y}$, 510069 mm³
Elastic section modulus about z-axis, $W_{el,z}$, 174020 mm³
Plastic section modulus about y-axis, $W_{pl,y}$, 567396 mm³
Plastic section modulus about z-axis, $W_{pl,z}$, 264249 mm³
Second moment of area about y-axis, I_y, 52588157 mm⁴
Second moment of area about z-axis, I_z, 17776181 mm⁴

Column details

Column section	**UKC 203x203x52**
Steel grade	**S355**
Yield strength	f_y = 355 N/mm^2
Ultimate strength	f_u = 470 N/mm^2
Modulus of elasticity	E = 210 kN/mm^2
Poisson's ratio	ν = 0.3
Shear modulus	$G = E / [2 \times (1 + \nu)]$ = **80.8** kN/mm^2

Column geometry

System length for buckling - Major axis	L_y = **2400** mm
System length for buckling - Minor axis	L_z = **2400** mm

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

The column is part of a sway frame in the direction of the major axis

Column loading

Axial load	N_{Ed} = **210** kN (Compression)
Major axis moment at end 1 - Bottom	$M_{y,Ed1}$ = **118.0** kNm
Major axis moment at end 2 - Top	$M_{y,Ed2}$ = **0.0** kNm
	Major axis bending is single curvature
Minor axis moment at end 1 - Bottom	$M_{z,Ed1}$ = **0.0** kNm

Minor axis moment at end 2 - Top $M_{z,Ed2}$ = **0.0 kNm**

Major axis shear force $V_{y,Ed}$ = **0 kN**
Minor axis shear force $V_{z,Ed}$ = **0 kN**

Buckling length for flexural buckling - Major axis
End restraint factor K_y = **1.000**
Buckling length $L_{cr\_y} = L_y \times K_y$ = **2400 mm**

Buckling length for flexural buckling - Minor axis
End restraint factor K_z = **1.000**
Buckling length $L_{cr\_z} = L_z \times K_z$ = **2400 mm**

Web section classification (Table 5.2)
Coefficient depending on f_y $\varepsilon = \sqrt{(235\ N/mm^2 / f_y)}$ = **0.814**
Depth between fillets $c_w = h - 2 \times (t_f + r)$ = **160.8 mm**
Ratio of c/t $ratio_w = c_w / t_w$ = **20.35**
Length of web taken by axial load $l_w = \min(N_{Ed} / (f_y \times t_w), c_w)$ = **74.9 mm**
For class 1 & 2 proportion in compression $\alpha = (c_w/2 + l_w/2) / c_w$ = **0.733**
Limit for class 1 web $Limit_{1w} = (396 \times \varepsilon) / (13 \times \alpha - 1)$ = **37.79**

The web is class 1

Flange section classification (Table 5.2)
Outstand length $c_f = (b - t_w) / 2 - r$ = **88.0** mm
Ratio of c/t $ratio_f = c_f / t_f$ = **7.04**

Limit for class 1 flange $Limit_{1f} = 9 \times \varepsilon$ = **7.32**
Limit for class 2 flange $Limit_{2f} = 10 \times \varepsilon$ = **8.14**
Limit for class 3 flange $Limit_{3f} = 14 \times \varepsilon$ = **11.39**

The flange is class 1

Overall section classification

The section is class 1

Resistance of cross section (cl. 6.2)

Compression (cl. 6.2.4)
Design force N_{Ed} = **210 kN**
Design resistance $N_{c,Rd} = N_{pl,Rd} = A \times f_y / \gamma_{M0}$ = **2353 kN**
$N_{Ed} / N_{c,Rd}$ = **0.089**

PASS - The compression design resistance exceeds the design force

Bending - Major axis (cl. 6.2.5)
Design bending moment $M_{y,Ed} = \max(\text{abs}(M_{y,Ed1}), \text{abs}(M_{y,Ed2}))$ = **118.0 kNm**
Section modulus $W_y = W_{pl.y}$ = **567.4** cm^3
Design resistance $M_{c,y,Rd} = W_y \times f_y / \gamma_{M0}$ = **201.4 kNm**
$M_{y,Ed} / M_{c,y,Rd}$ = **0.586**

PASS - The bending design resistance exceeds the design moment

Combined bending and axial force (cl. 6.2.9)
Ratio design axial to design plastic resistance $n = \text{abs}(N_{Ed}) / N_{pl,Rd}$ = **0.089**

Ratio web area to gross area $a = \min(0.5, (A - 2 \times b \times t_f) / A) = 0.229$

Bending - Major axis (cl. 6.2.9.1)

Design bending moment $M_{y,Ed} = \max(\text{abs}(M_{y,Ed1}), \text{abs}(M_{y,Ed2})) = 118.0$ kNm

Plastic design resistance $M_{pl,y,Rd} = W_{pl.y} \times f_y / \gamma_{M0} = 201.4$ kNm

Modified design resistance $M_{N,y,Rd} = M_{pl,y,Rd} \times \min(1, (1 - n) / (1 - 0.5 \times a)) = 201.4$ kNm

$M_{y,Ed} / M_{N,y,Rd} = 0.586$

PASS - Bending resistance in presence of axial load exceeds design moment

Buckling resistance (cl. 6.3)

Yield strength for buckling resistance $f_y = 355$ N/mm^2

Flexural buckling - Major axis

Elastic critical buckling force $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr\_y}^2 = 18923$ kN

Non-dimensional slenderness $\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})} = 0.353$

Buckling curve (Table 6.2) b

Imperfection factor (Table 6.1) $\alpha_y = 0.34$

Parameter Φ $\Phi_y = 0.5 \times [1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2] = 0.588$

Reduction factor $\chi_y = \min(1.0, 1 / [\Phi_y + \sqrt{(\Phi_y^2 - \bar{\lambda}_y^2)}]) = 0.944$

Design buckling resistance $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1} = 2222.4$ kN

$N_{Ed} / N_{b,y,Rd} = 0.094$

PASS - The flexural buckling resistance exceeds the design axial load

Flexural buckling - Minor axis

Elastic critical buckling force $N_{cr,z} = \pi^2 \times E \times I_z / L_{cr\_z}^2 = 6396$ kN

Non-dimensional slenderness $\bar{\lambda}_z = \sqrt{(A \times f_y / N_{cr,z})} = 0.607$

Buckling curve (Table 6.2) c

Imperfection factor (Table 6.1) $\alpha_z = 0.49$

Parameter Φ $\Phi_z = 0.5 \times [1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2] = 0.784$

Reduction factor $\chi_z = \min(1.0, 1 / [\Phi_z + \sqrt{(\Phi_z^2 - \bar{\lambda}_z^2)}]) = 0.782$

Design buckling resistance $N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1} = 1838.9$ kN

$N_{Ed} / N_{b,z,Rd} = 0.114$

PASS - The flexural buckling resistance exceeds the design axial load

Torsional and torsional-flexural buckling (cl. 6.3.1.4)

Torsional buckling length factor $K_T = 1.00$

Effective buckling length $L_{cr\_T} = K_T \times \max(L_y, L_z) = 2400$ mm

Distance from shear ctr to centroid along major axis $y_0 = 0.0$ mm

Distance from shear ctr to centroid along minor axis $z_0 = 0.0$ mm

$i_0 = \sqrt{(i_y^2 + i_z^2 + y_0^2 + z_0^2)} = 103.0$ mm

$\beta_T = 1 - (y_0 / i_0)^2 = 1.000$

Elastic critical torsional buckling force $N_{cr,T} = 1 / i_0^2 \times (G \times I_t + \pi^2 \times E \times I_w / L_{cr\_T}^2) = 8068$ kN

Elastic critical torsional-flexural buckling force $N_{cr,TF} = N_{cr,y}/(2\times\beta_T)\times[1+N_{cr,T}/N_{cr,y}-\sqrt{[(1-N_{cr,T}/N_{cr,y})^2 + 4\times(y_0/i_0)^2\times N_{cr,T}/N_{cr,y}]}]$

$N_{cr,TF} = 8068$ kN

Non-dimensional slenderness $\bar{\lambda}_T = \sqrt{(A \times f_y / \min(N_{cr,T}, N_{cr,TF}))} = 0.540$

Buckling curve (Table 6.2) c

Imperfection factor (Table 6.1)	$\alpha_T = 0.49$
Parameter Φ	$\Phi_T = 0.5 \times [1 + \alpha_T \times (\bar{\lambda}_T - 0.2) + \bar{\lambda}_T^2] = 0.729$
Reduction factor	$\chi_T = \min(1.0, 1 / [\Phi_T + \sqrt{(\Phi_T^2 - \bar{\lambda}_T^2)}]) = 0.820$
Design buckling resistance	$N_{b,T,Rd} = \chi_T \times A \times f_y / \gamma_{M1} = 1930.3$ kN
	$N_{Ed} / N_{b,T,Rd} = 0.109$

PASS - The torsional/torsional-flexural buckling resistance exceeds the design axial load

Minimum buckling resistance

Minimum buckling resistance	$N_{b,Rd} = \min(N_{b,y,Rd}, N_{b,z,Rd}, N_{b,T,Rd}) = 1838.9$ kN
	$N_{Ed} / N_{b,Rd} = 0.114$

PASS - The axial load buckling resistance exceeds the design axial load

Buckling resistance moment (cl.6.3.2.1)

Lateral torsional buckling length factor	$K_{LT} = 1.00$
Effective buckling length	$L_{cr\_LT} = K_{LT} \times L_z = 2400$ mm
End moment factor	$\psi = M_{y,Ed2} / M_{y,Ed1} = 0.000$
Moment distribution correction factor (Table 6.6)	$k_c = 1 / (1.33 - 0.33 \times \psi) = 0.752$
	$C_1 = 1 / k_c^2 = 1.769$
Curvature factor	$g = \sqrt{[1 - (I_z / I_y)]} = 0.814$
Poissons ratio	$\nu = 0.3$
Shear modulus	$G = E / [2 \times (1 + \nu)] = 80769$ N/mm^2
Elastic critical buckling moment	$M_{cr} = C_1 \times \pi^2 \times E \times I_z \times \sqrt{[I_w / I_z + L_{cr\_LT}^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} / (L_{cr\_LT}^2 \times g)$
	$M_{cr} = 1609.2$ kNm
Slenderness ratio for lateral torsional buckling	$\bar{\lambda}_{LT} = \sqrt{[W_y \times f_y / M_{cr}]} = 0.354$
Limiting slenderness ratio	$\bar{\lambda}_{LT,0} = 0.40$
Correction factor for rolled sections	$\beta_r = 0.75$
Buckling curve (Table 6.5)	b
Imperfection factor (Table 6.1)	$\alpha_{LT} = 0.34$
Parameter Φ_{LT}	$\Phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta_r \times \bar{\lambda}_{LT}^2] = 0.539$
Reduction factor	$\chi_{LT} = \min(1.0, 1/ \bar{\lambda}_{LT}^2, 1 / [\Phi_{LT} + \sqrt{(\Phi_{LT}^2 - \beta_r \times \bar{\lambda}_{LT}^2)}]) = 1.000$
Modification factor	$f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = 0.925$
Modified LTB reduction factor - eq 6.58	$\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1/ \bar{\lambda}_{LT}^2) = 1.000$
Design buckling resistance moment	$M_{b,Rd} = \chi_{LT,mod} \times W_y \times f_y / \gamma_{M1} = 201.4$ kNm
Design bending moment	$M_{y,Ed} = \max(\text{abs}(M_{y,Ed1}), \text{abs}(M_{y,Ed2})) = 118.0$ kNm
	$M_{y,Ed} / M_{b,Rd} = 0.586$

PASS - The design buckling resistance moment exceeds the maximum design moment

Combined bending and axial compression (cl. 6.3.3)

Characteristic resistance to normal force	$N_{Rk} = A \times f_y = 2353$ kN
Characteristic moment resistance - Major axis	$M_{y,Rk} = W_{pl.y} \times f_y = 201.4$ kNm
Characteristic moment resistance - Minor axis	$M_{z,Rk} = W_{pl.z} \times f_y = 93.8$ kNm
Moment distribution factor - Major axis	$\psi_y = M_{y,Ed2} / M_{y,Ed1} = 0.000$
Moment factor - Major axis	$C_{my} = \max(0.4, 0.6 + 0.4 \times \psi_y) = 0.600$
Moment factor - Minor axis	$C_{mz} = 0.9$
Moment distribution factor for LTB	$\psi_{LT} = M_{y,Ed2} / M_{y,Ed1} = 0.000$

Moment factor for LTB $C_{mLT} = \max(0.4,\ 0.6 + 0.4 \times \psi_{LT}) = 0.600$

Interaction factor k_{yy} $k_{yy} = C_{my} \times [1 + \min(0.8,\ \bar{\lambda}_y - 0.2) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1})] = 0.609$

Interaction factor k_{zy} $k_{zy} = 1 - \min(0.1,\ 0.1 \times \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times (\chi_z \times N_{Rk}/\gamma_{M1})) = 0.980$

Interaction factor k_{zz} $k_{zz} = C_{mz} \times [1 + \min(1.4,\ 2 \times \bar{\lambda}_z - 0.6) \times N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1})] = 0.963$

Interaction factor k_{yz} $k_{yz} = 0.6 \times k_{zz} = 0.578$

Section utilisation $UR_{B\_1} = N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) + k_{yz} \times M_{z,Ed} / (M_{z,Rk} / \gamma_{M1})$

$UR_{B\_1} = 0.451$

$UR_{B\_2} = N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) + k_{zz} \times M_{z,Ed} / (M_{z,Rk} / \gamma_{M1})$

$UR_{B\_2} = 0.688$

PASS - The buckling resistance is adequate

14. Connection Design CON1

Beam deflection is checked for L/250.

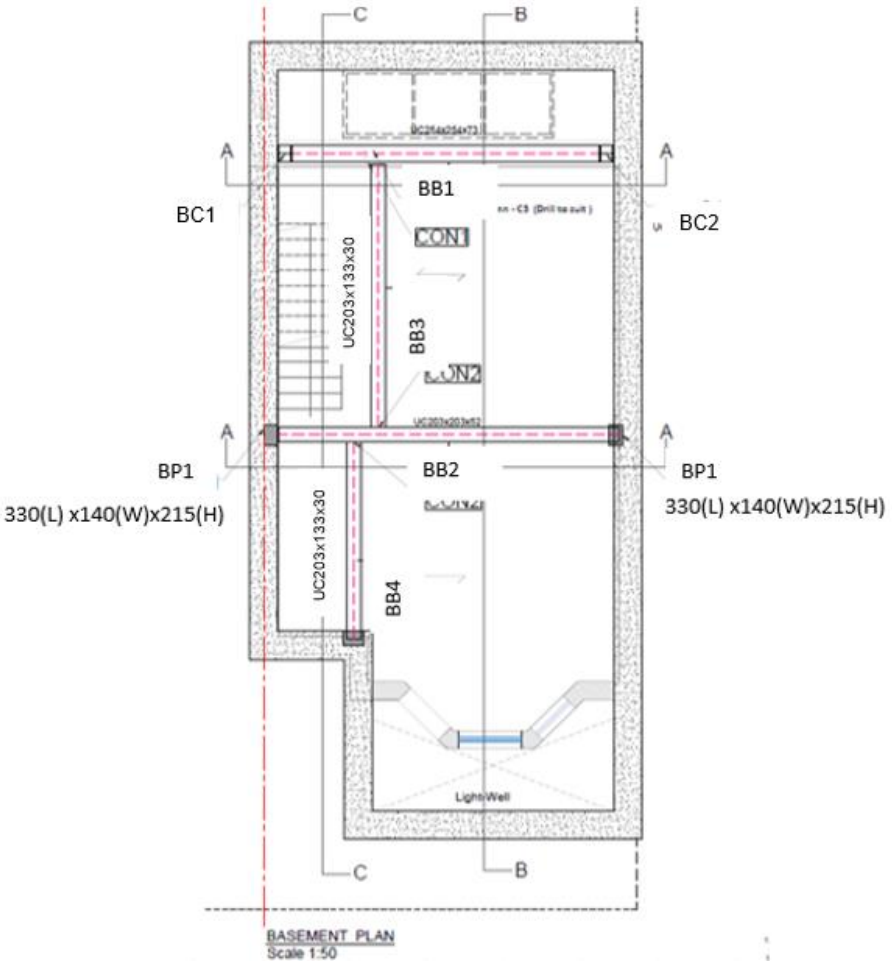


Figure 9: Location of Steel Beams to Beam connection CON1.

Project:
Project no:
Author:

IDEA StatiCa

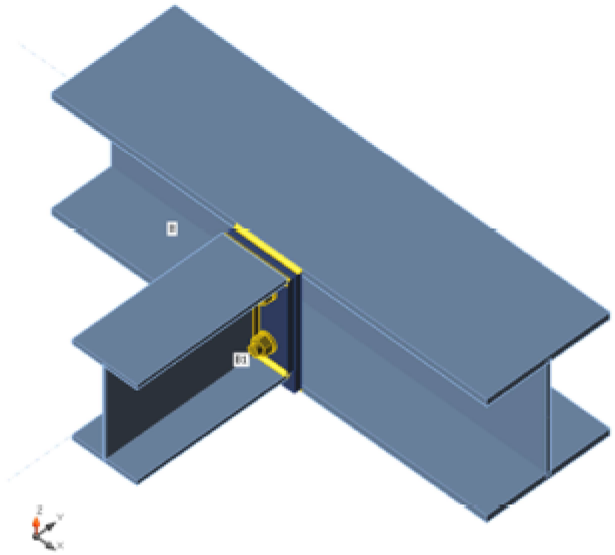
Project item Connection Type C1

Design

Name	Connection Type C1
Description	
Analysis	Stress, strain/ simplified loading

Beams and columns

Name	Cross-section	β – Direction [°]	γ - Pitch [°]	α - Rotation [°]	Offset ex [mm]	Offset ey [mm]	Offset ez [mm]	Forces in
B	1 - CON1(UC 254 x 254 x 73)	0,0	0,0	0,0	0	0	0	Node
B1	2 - CON1(UB 203 x 133 x 30)	-90,0	0,0	0,0	0	0	24	Node



Cross-sections

Name	Material
1 - CON1(UC 254 x 254 x 73)	S 355
2 - CON1(UB 203 x 133 x 30)	S 355

Bolts

Name	Bolt assembly	Diameter [mm]	fu [MPa]	Gross area [mm^2]
M20 8,8	M20 8,8	20	800,0	314

Project:
Project no:
Author:

IDEA StatiCa®
Calculate yesterday's estimates

Load effects (equilibrium not required)

Name	Member	N [kN]	Vy [kN]	Vz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
LE1	B1	0,0	0,0	-37,0	0,0	0,0	0,0

Check

Summary

Name	Value	Status
Analysis	100,0%	OK
Plates	0,1 < 5%	OK
Bolts	31,8 < 100%	OK
Welds	35,4 < 100%	OK
Buckling	62,24	

Plates

Name	Thickness [mm]	Loads	σ_{Ed} [MPa]	ε_{Pl} [%]	Status
B-bfl 1	14,2	LE1	176,1	0,0	OK
B-tfl 1	14,2	LE1	193,1	0,0	OK
B-w 1	8,6	LE1	106,7	0,0	OK
B1-bfl 1	9,6	LE1	74,2	0,0	OK
B1-tfl 1	9,6	LE1	74,1	0,0	OK
B1-w 1	6,4	LE1	96,5	0,0	OK
SEP1a	15,0	LE1	355,1	0,1	OK
SEP1b	15,0	LE1	138,3	0,0	OK
STIFF1	14,2	LE1	34,8	0,0	OK

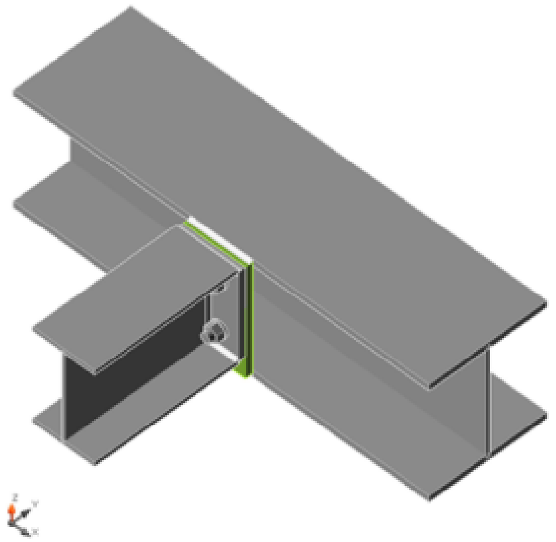
Design data

Material	f_y [MPa]	ε_{lim} [%]
S 355	355,0	5,0

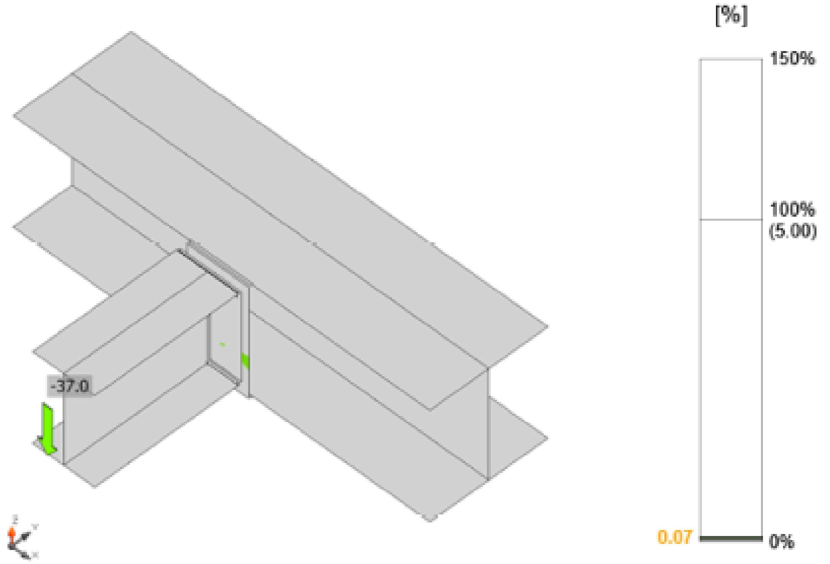
Symbol explanation

ε_{Pl}	Strain
σ_{Ed}	Eq. stress
f_y	Yield strength
ε_{lim}	Limit of plastic strain

Project:
Project no:
Author:



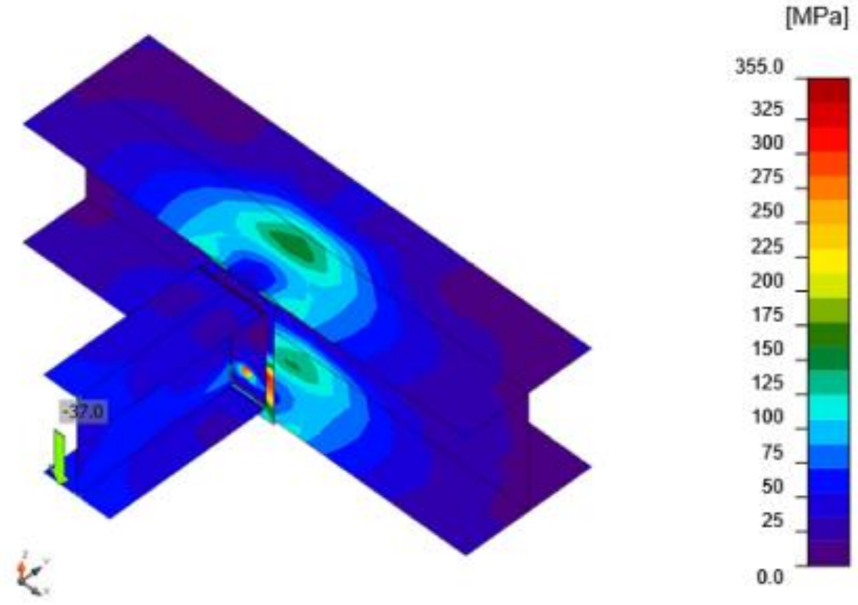
Overall check, LE1



Strain check, LE1

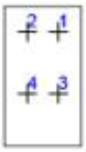
Project:
Project no:
Author:

IDEA StatiCa
Calculate yesterday's estimates



Equivalent stress, LE1

Bolts

	Name	Loads	$F_{t,Ed}$ [kN]	V [kN]	Ut_t [%]	$F_{b,Rd}$ [kN]	Ut_s [%]	Ut_{ts} [%]	Status
	B1	LE1	1.7	10.9	1.2	132.6	11.6	12.5	OK
	B2	LE1	1.7	10.9	1.2	132.6	11.6	12.5	OK
	B3	LE1	44.8	7.6	31.8	203.4	8.1	30.8	OK
	B4	LE1	44.9	7.6	31.8	203.4	8.1	30.8	OK

Design data

Name	$F_{t,Rd}$ [kN]	$B_{p,Rd}$ [kN]	$F_{v,Rd}$ [kN]
M20 8.8 - 1	141.1	349.1	94.1

Symbol explanation

$F_{t,Rd}$	Bolt tension resistance EN 1993-1-8 tab. 3.4
$F_{t,Ed}$	Tension force
$B_{p,Rd}$	Punching shear resistance
V	Resultant of shear forces Vy, Vz in bolt
$F_{v,Rd}$	Bolt shear resistance EN_1993-1-8 table 3.4
$F_{b,Rd}$	Plate bearing resistance EN 1993-1-8 tab. 3.4
Ut_t	Utilization in tension
Ut_s	Utilization in shear
Ut_{ts}	Utilization in tension and shear EN 1993-1-8 table 3.4

Project:
Project no:
Author:

IDEA StatiCa®
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Welds (Plastic redistribution)

Item	Edge	Throat th. [mm]	Length [mm]	Loads	$\sigma_{w,Ed}$ [MPa]	ε_{Pl} [%]	$\sigma_{\perp}$ [MPa]	$\tau_{\parallel}$ [MPa]	$\tau_{\perp}$ [MPa]	Ut [%]	Ut_c [%]	Status
SEP1a	B-tfl 1	◢4.3	134	LE1	141.0	0.0	33.8	-20.3	76.4	32.4	22.4	OK
SEP1a	B-bfl 1	◢4.3	134	LE1	135.8	0.0	-17.0	24.0	74.0	31.2	21.3	OK
SEP1b	B1-bfl 1	◢4.3◣	134	LE1	57.3	0.0	48.8	0.2	17.4	13.8	9.7	OK
		◢4.3◣	134	LE1	19.6	0.0	-7.4	-7.5	7.3	4.5	3.3	OK
SEP1b	B1-tfl 1	◢4.3◣	134	LE1	38.0	0.0	-20.1	-7.0	17.3	8.7	6.8	OK
		◢4.3◣	134	LE1	39.1	0.0	-30.2	0.2	14.3	9.0	7.0	OK
SEP1b	B1-w 1	◢4.3◣	193	LE1	154.1	0.0	76.5	-18.0	75.1	35.4	13.0	OK
		◢4.3◣	193	LE1	153.0	0.0	73.9	17.3	-75.4	35.1	13.0	OK
B-bfl 1	STIFF1	◢4.3◣	110	LE1	91.8	0.0	-35.0	34.2	-35.0	21.1	9.5	OK
		◢4.3◣	110	LE1	91.8	0.0	-35.0	-34.2	35.0	21.1	9.5	OK
B-w 1	STIFF1	◢4.3◣	200	LE1	96.6	0.0	-22.4	-49.4	-22.4	22.2	11.3	OK
		◢4.3◣	200	LE1	96.6	0.0	-22.4	49.4	22.4	22.2	11.3	OK
B-tfl 1	STIFF1	◢4.3◣	110	LE1	98.5	0.0	38.8	35.0	38.8	22.6	10.4	OK
		◢4.3◣	110	LE1	98.5	0.0	38.8	-35.0	-38.8	22.6	10.4	OK

Design data

	β_w [-]	$\sigma_{w,Rd}$ [MPa]	0.9 σ [MPa]
S 355	0.90	435.6	352.8

Symbol explanation

ε_{Pl}	Strain
$\sigma_{w,Ed}$	Equivalent stress
$\sigma_{w,Rd}$	Equivalent stress resistance
$\sigma_{\perp}$	Perpendicular stress
$\tau_{\parallel}$	Shear stress parallel to weld axis
$\tau_{\perp}$	Shear stress perpendicular to weld axis
0.9 σ	Perpendicular stress resistance - 0.9*fu/γM2
β_w	Corelation factor EN 1993-1-8 tab. 4.1
Ut	Utilization
Utc	Weld capacity utilization

Buckling

Loads	Shape	Factor [-]
LE1	1	62.24
	2	70.09
	3	85.98
	4	98.14
	5	124.18
	6	127.11

Project:
Project no:
Author:

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Code settings

Item	Value	Unit	Reference
γ_{M0}	1,00	-	EN 1993-1-1: 6.1
γ_{M1}	1,00	-	EN 1993-1-1: 6.1
γ_{M2}	1,25	-	EN 1993-1-1: 6.1
γ_{M3}	1,25	-	EN 1993-1-8: 2.2
γ_C	1,50	-	EN 1992-1-1: 2.4.2.4
γ_{Inst}	1,20	-	ETAG 001-C: 3.2.1
Joint coefficient βj	0,67	-	EN 1993-1-8: 6.2.5
Effective area - influence of mesh size	0,10	-	
Friction coefficient - concrete	0,25	-	EN 1993-1-8
Friction coefficient in slip-resistance	0,30	-	EN 1993-1-8 tab 3.7
Limit plastic strain	0,05	-	EN 1993-1-5
Weld stress evaluation	Plastic redistribution		
Detailing	No		
Distance between bolts [d]	2,20	-	EN 1993-1-8: tab 3.3
Distance between bolts and edge [d]	1,20	-	EN 1993-1-8: tab 3.3
Concrete breakout resistance	Yes		ETAG 001-C
Use calculated αb in bearing check.	Yes		EN 1993-1-8: tab 3.4
Cracked concrete	Yes		
Local deformation check	No		
Local deformation limit	0,03	-	CIDECT DG 1, 3 - 1.1
Geometrical nonlinearity (GMNA)	Yes		Large deformations for hollow sections

15. Connection Design CON2

Beam deflection is checked for L/250.

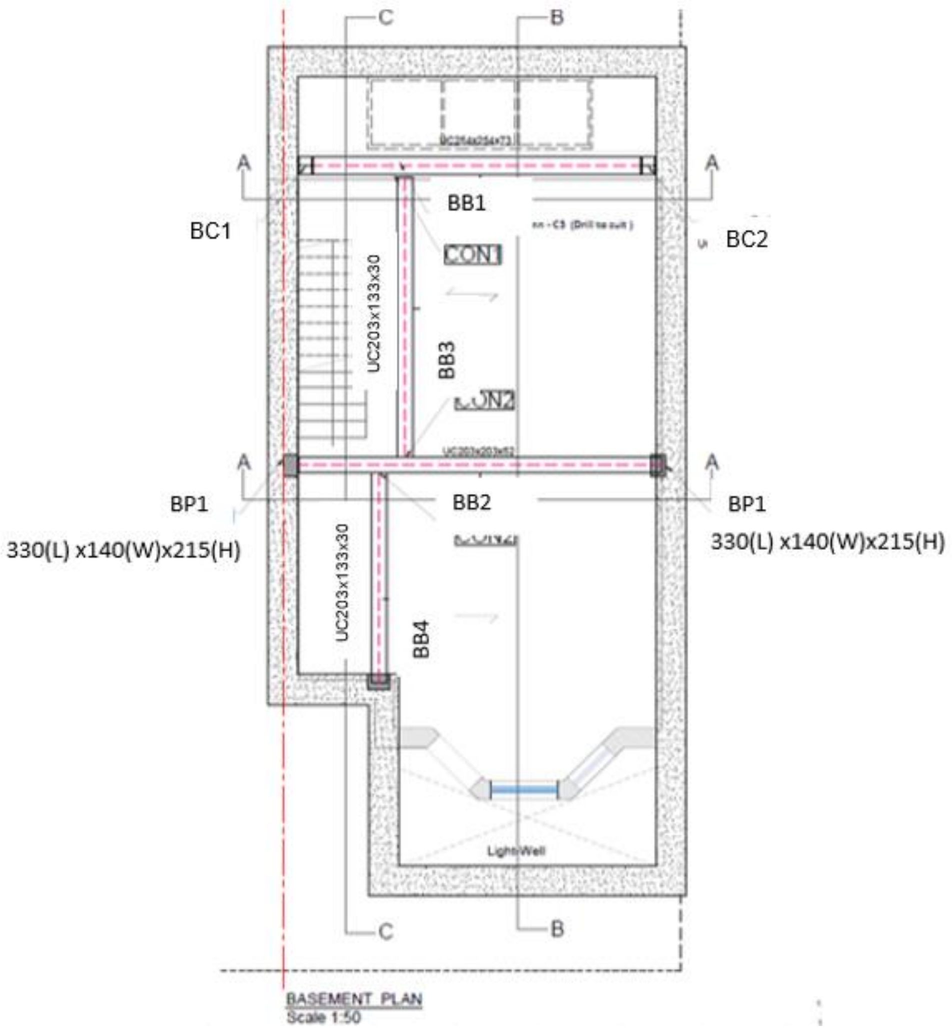


Figure 10: Location of Steel Beams to Beam connection CON2.

Project:
Project no:
Author: AJ

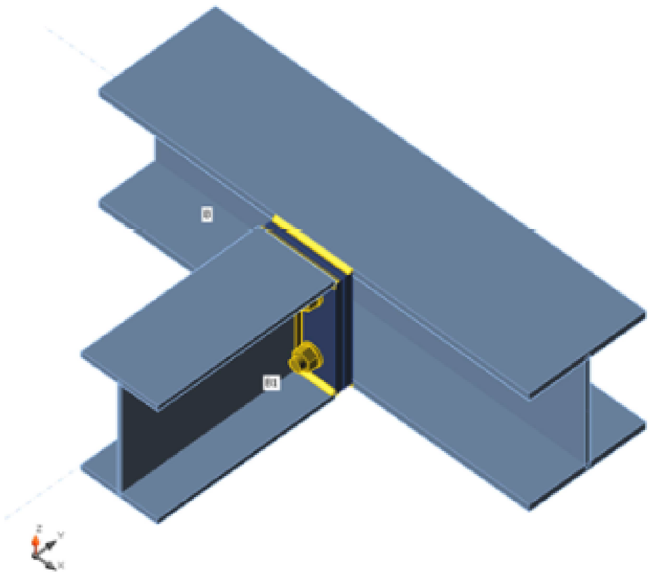
Project item Connection type C2

Design

Name	Connection type C2
Description	15 Landor Road
Analysis	Stress, strain/ simplified loading

Beams and columns

Name	Cross-section	β – Direction [°]	γ - Pitch [°]	α - Rotation [°]	Offset ex [mm]	Offset ey [mm]	Offset ez [mm]	Forces in
B	1 - CON1(UC 203 x 203 x 52)	0,0	0,0	0,0	0	0	0	Node
B1	2 - CON1(UB 203 x 133 x 30)	-90,0	0,0	0,0	0	0	0	Node



Cross-sections

Name	Material
1 - CON1(UC 203 x 203 x 52)	S 355
2 - CON1(UB 203 x 133 x 30)	S 355

Bolts

Name	Bolt assembly	Diameter [mm]	fu [MPa]	Gross area [mm^2]
M20 8.8	M20 8.8	20	800,0	314

Project:
Project no:
Author:

IDEA StatiCa®
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Load effects (equilibrium not required)

Name	Member	N [kN]	Vy [kN]	Vz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
LE1	B1	0,0	0,0	-37,0	0,0	0,0	0,0

Check

Summary

Name	Value	Status
Analysis	100,0%	OK
Plates	0,0 < 5%	OK
Bolts	22,9 < 100%	OK
Welds	42,5 < 100%	OK
Buckling	64,99	

Plates

Name	Thickness [mm]	Loads	σ_{Ed} [MPa]	ε_{Pl} [%]	Status
B-bfl 1	12,5	LE1	227,5	0,0	OK
B-tfl 1	12,5	LE1	233,9	0,0	OK
B-w 1	7,9	LE1	130,1	0,0	OK
B1-bfl 1	9,6	LE1	71,2	0,0	OK
B1-tfl 1	9,6	LE1	71,1	0,0	OK
B1-w 1	6,4	LE1	81,1	0,0	OK
SEP1a	15,0	LE1	239,4	0,0	OK
SEP1b	15,0	LE1	94,4	0,0	OK
STIFF1	12,5	LE1	42,7	0,0	OK

Design data

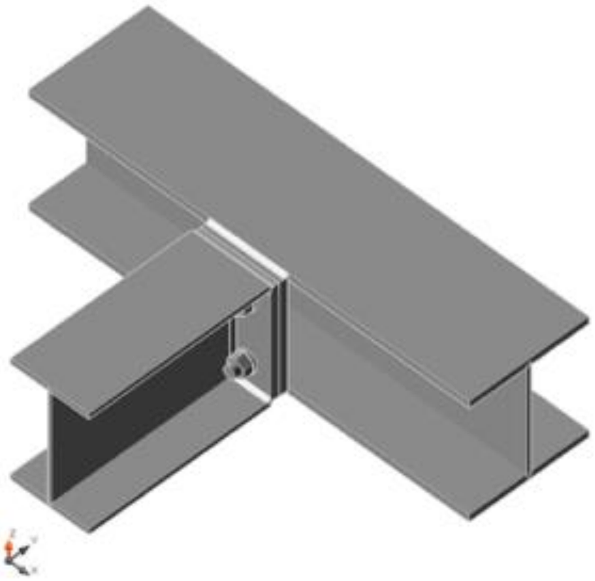
Material	f_y [MPa]	ε_{lim} [%]
S 355	355,0	5,0

Symbol explanation

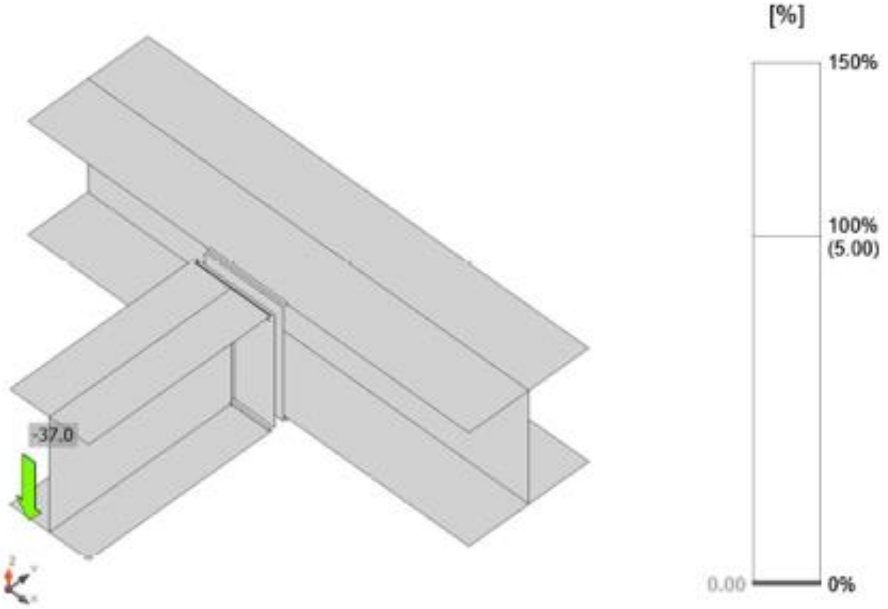
ε_{Pl}	Strain
σ_{Ed}	Eq. stress
f_y	Yield strength
ε_{lim}	Limit of plastic strain

Project:
Project no:
Author:

IDEA StatiCa
Calculate yesterday's estimates

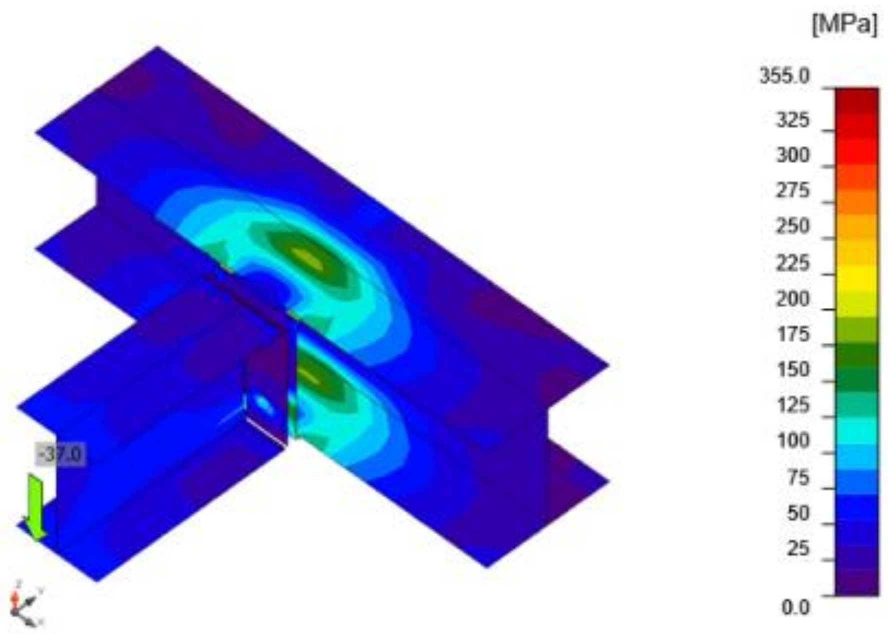


Overall check, LE1



Strain check, LE1

Project:
Project no:
Author:



Equivalent stress, LE1

Bolts

	Name	Loads	$F_{t,Ed}$ [kN]	V [kN]	Ut_t [%]	$F_{b,Rd}$ [kN]	Ut_s [%]	Ut_{ts} [%]	Status
	B1	LE1	2.1	10.1	1.5	132.6	10.7	11.8	OK
	B2	LE1	2.1	10.1	1.5	132.6	10.7	11.8	OK
	B3	LE1	27.6	8.4	19.5	134.0	9.0	22.9	OK
	B4	LE1	27.5	8.4	19.5	134.0	9.0	22.9	OK

Design data

Name	$F_{t,Rd}$ [kN]	$B_{p,Rd}$ [kN]	$F_{v,Rd}$ [kN]
M20 8.8 - 1	141.1	349.1	94.1

Symbol explanation

$F_{t,Rd}$	Bolt tension resistance EN 1993-1-8 tab. 3.4
$F_{t,Ed}$	Tension force
$B_{p,Rd}$	Punching shear resistance
V	Resultant of shear forces Vy, Vz in bolt
$F_{v,Rd}$	Bolt shear resistance EN_1993-1-8 table 3.4
$F_{b,Rd}$	Plate bearing resistance EN 1993-1-8 tab. 3.4
Ut_t	Utilization in tension
Ut_s	Utilization in shear
Ut_{ts}	Utilization in tension and shear EN 1993-1-8 table 3.4

Project:
Project no:
Author:

IDEA StatiCa®
Calculate yesterday's estimates

Welds (Plastic redistribution)

Item	Edge	Throat th. [mm]	Length [mm]	Loads	$\sigma_{w,Ed}$ [MPa]	ε_{Pl} [%]	$\sigma_{\perp}$ [MPa]	$\tau_{\parallel}$ [MPa]	$\tau_{\perp}$ [MPa]	Ut [%]	Ut_c [%]	Status
SEP1a	B-tfl 1	◢4.3	134	LE1	185.2	0.0	55.4	22.5	99.5	42.5	23.0	OK
SEP1a	B-bfl 1	◢4.3	134	LE1	178.4	0.0	-64.0	23.5	93.2	41.0	23.4	OK
SEP1b	B1-bfl 1	◢4.3◣	134	LE1	45.9	0.0	40.8	0.2	12.1	11.6	8.1	OK
		◢4.3◣	134	LE1	16.4	0.0	-7.0	7.0	5.0	3.8	2.5	OK
SEP1b	B1-tfl 1	◢4.3◣	134	LE1	39.8	0.0	-22.6	5.0	18.2	9.1	7.3	OK
		◢4.3◣	134	LE1	36.6	0.0	-27.2	0.3	14.1	8.4	6.2	OK
SEP1b	B1-w 1	◢4.3◣	193	LE1	111.0	0.0	52.7	-20.7	52.5	25.5	10.3	OK
		◢4.3◣	193	LE1	109.8	0.0	51.9	19.9	-52.2	25.2	10.3	OK
B-bfl 1	STIFF1	◢4.3◣	88	LE1	100.3	0.0	-38.4	37.2	-38.4	23.0	11.1	OK
		◢4.3◣	88	LE1	100.3	0.0	-38.4	-37.2	38.4	23.0	11.1	OK
B-w 1	STIFF1	◢4.3◣	161	LE1	108.3	0.0	-28.2	-53.4	-28.2	24.9	13.2	OK
		◢4.3◣	161	LE1	108.3	0.0	-28.3	53.4	28.2	24.9	13.2	OK
B-tfl 1	STIFF1	◢4.3◣	88	LE1	102.4	0.0	39.8	37.3	39.8	23.5	11.4	OK
		◢4.3◣	88	LE1	102.4	0.0	39.8	-37.3	-39.8	23.5	11.4	OK

Design data

	β_w [-]	$\sigma_{w,Rd}$ [MPa]	0.9 σ [MPa]
S 355	0.90	435.6	352.8

Symbol explanation

ε_{Pl}	Strain
$\sigma_{w,Ed}$	Equivalent stress
$\sigma_{w,Rd}$	Equivalent stress resistance
$\sigma_{\perp}$	Perpendicular stress
$\tau_{\parallel}$	Shear stress parallel to weld axis
$\tau_{\perp}$	Shear stress perpendicular to weld axis
0.9 σ	Perpendicular stress resistance - 0.9*fu/γM2
β_w	Corelation factor EN 1993-1-8 tab. 4.1
Ut	Utilization
Utc	Weld capacity utilization

Buckling

Loads	Shape	Factor [-]
LE1	1	64.99
	2	72.70
	3	90.63
	4	103.64
	5	132.15
	6	136.12

Project:
Project no:
Author:

Code settings

Item	Value	Unit	Reference
γ_{M0}	1,00	-	EN 1993-1-1: 6,1
γ_{M1}	1,00	-	EN 1993-1-1: 6,1
γ_{M2}	1.25	-	EN 1993-1-1: 6.1
γ_{M3}	1.25	-	EN 1993-1-8: 2.2
γ_{C}	1,50	-	EN 1992-1-1: 2,4,2,4
γ_{Inst}	1,20	-	ETAG 001-C: 3,2,1
Joint coefficient βj	0,67	-	EN 1993-1-8: 6,2,5
Effective area - influence of mesh size	0,10	-	
Friction coefficient - concrete	0,25	-	EN 1993-1-8
Friction coefficient in slip-resistance	0.30	-	EN 1993-1-8 tab 3.7
Limit plastic strain	0,05	-	EN 1993-1-5
Weld stress evaluation	Plastic redistribution		
Detailing	No		
Distance between bolts [d]	2.20	-	EN 1993-1-8: tab 3.3
Distance between bolts and edge [d]	1.20	-	EN 1993-1-8: tab 3.3
Concrete breakout resistance	Yes		ETAG 001-C
Use calculated αb in bearing check.	Yes		EN 1993-1-8: tab 3,4
Cracked concrete	Yes		
Local deformation check	No		
Local deformation limit	0,03	-	CIDECT DG 1, 3 - 1,1
Geometrical nonlinearity (GMNA)	Yes		Large deformations for hollow sections

16. Connection Design CON3

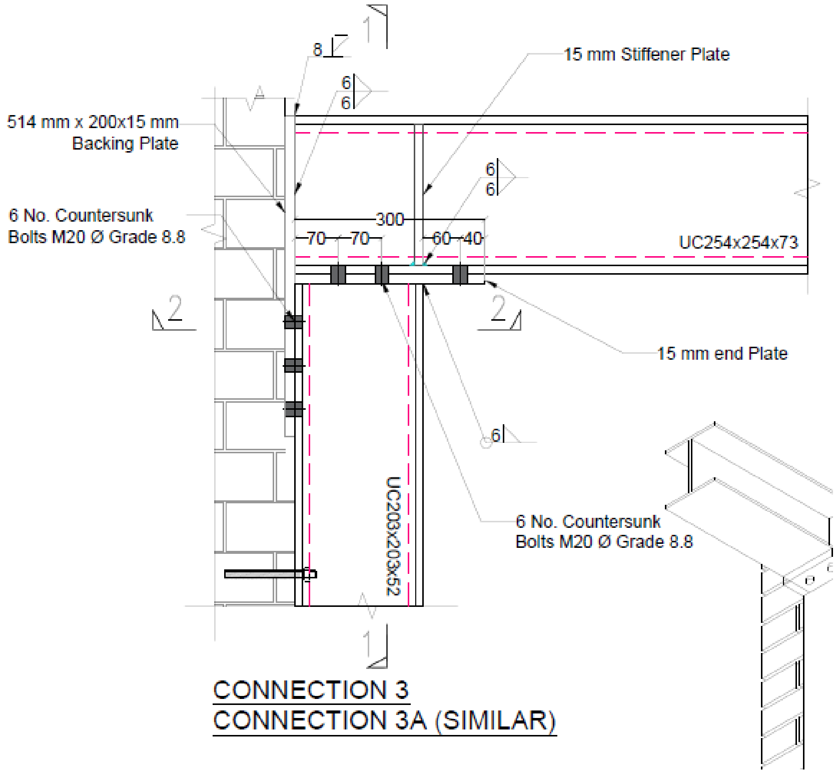


Figure 11: Location of Steel Beams to Column connection CON3.

Project:
Project no:
Author:

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Project item Con N3

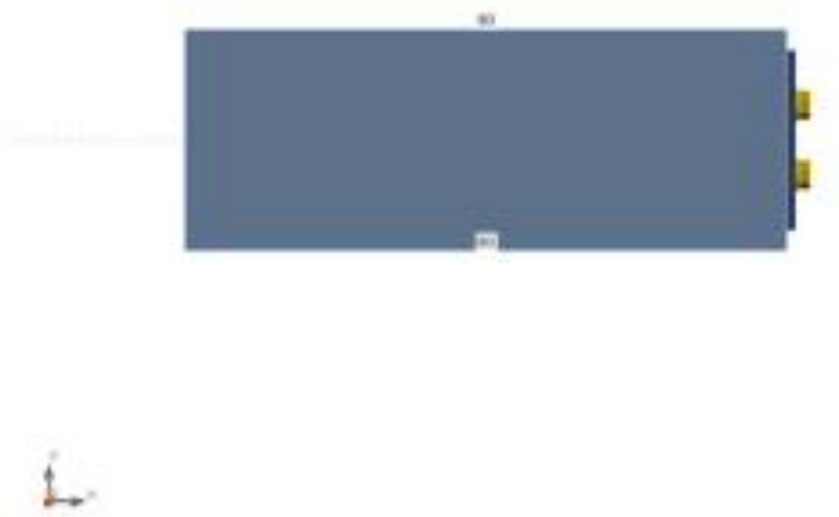
Design

Name	Con N3
Description	
Analysis	Stress, strain/ loads in equilibrium

Beams and columns

Name	Cross-section	β – Direction [°]	γ - Pitch [°]	α - Rotation [°]	Offset ex [mm]	Offset ey [mm]	Offset ez [mm]	Forces in
B2	3 - UC 203 x 203 x 52	0.0	90.0	0.0	0	0	0	Position
B3	2 - UC 254 x 254 x 73	180.0	0.0	0.0	-108	0	0	Position

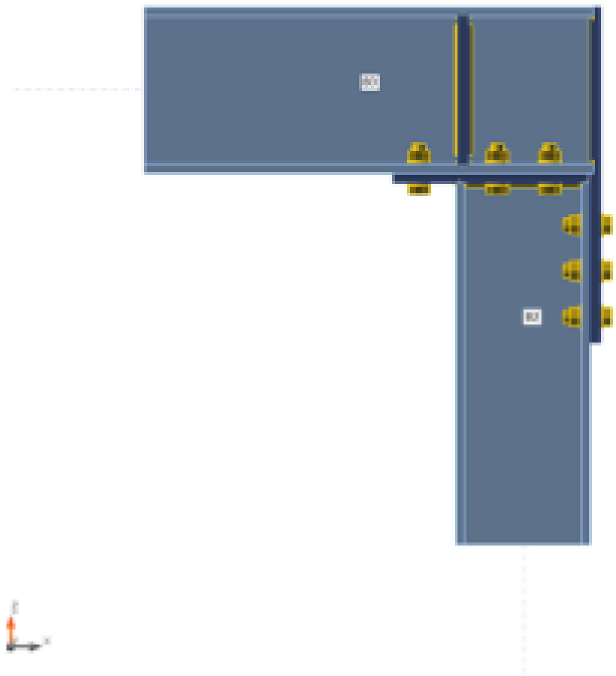
BETA DESIGN CONSULTANTS	Project: 15 Landor Road, London SW9 9RX			Job Ref.: 20.052
	Section: Basement Underpinning Structural Design Report			Sheet no.: 52
	Calc. by: AJ	Date: 10/03/2021	Chk'd by: FM	Date: 10/03/2021	Doc No.: REP-ST-20-052-01 A0

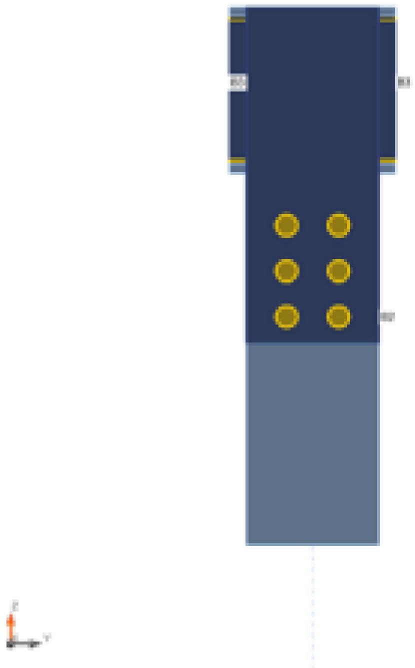


Project:

Project no:

Author:





Cross-sections

Name	Material
3 - UC 203 x 203 x 52	S 355
2 - UC 254 x 254 x 73	S 355

Project:
Project no:
Author:

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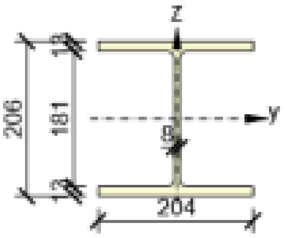
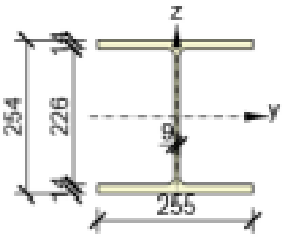
Cross-sections

Name	Material	Drawing
3 - UC 203 x 203 x 52	S 355	
2 - UC 254 x 254 x 73	S 355	

Bolts

Name	Bolt assembly	Diameter [mm]	fu [MPa]	Gross area [mm^2]
M20 8.8	M20 8.8	20	800.0	314

Load effects (forces in equilibrium)

Name	Member	N [kN]	Vy [kN]	Vz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
LE3	B2	-210.0	0.0	0.0	0.0	-118.0	0.0
	B3	0.0	0.0	-210.0	0.0	118.0	0.0

Check

Summary

Name	Value	Status
Analysis	100.0%	OK
Plates	0.0 < 5%	OK
Bolts	56.9 < 100%	OK
Welds	99.5 < 100%	OK
Buckling	9.86	

Project:
Project no:
Author:

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Plates

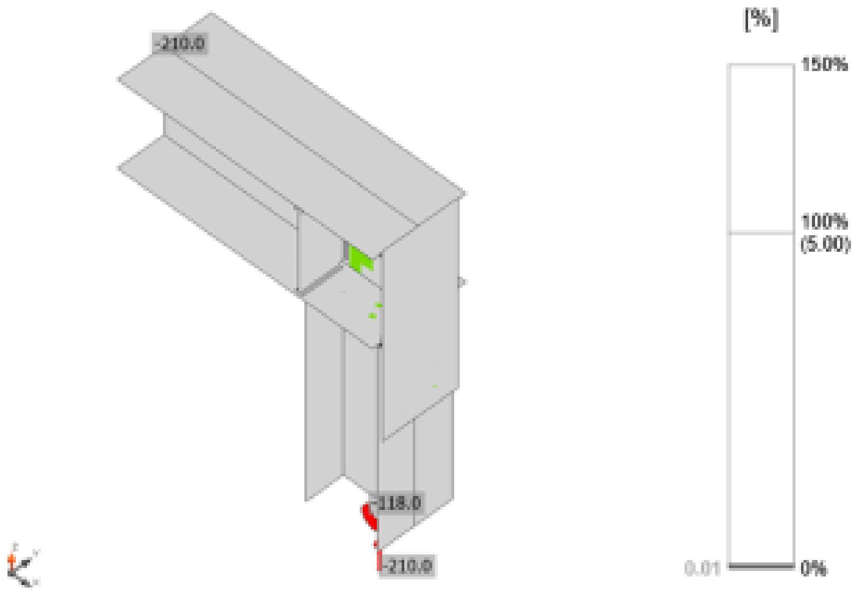
Name	Thickness [mm]	Loads	σ_{Ed} [MPa]	ε_{Pl} [%]	Status
B2-bfl 1	12.5	LE3	248.1	0.0	OK
B2-tfl 1	12.5	LE3	305.0	0.0	OK
B2-w 1	7.9	LE3	230.0	0.0	OK
B3-bfl 1	14.2	LE3	317.7	0.0	OK
B3-tfl 1	14,2	LE3	193,9	0,0	OK
B3-w 1	8,6	LE3	343,5	0,0	OK
EP1	15,0	LE3	268,1	0,0	OK
SPL1	15,0	LE3	257,6	0,0	OK
STIFF1a	15,0	LE3	212,6	0,0	OK
STIFF1b	15,0	LE3	212,6	0,0	OK

Design data

Material	f_y [MPa]	ε_{lim} [%]
S 355	355,0	5,0

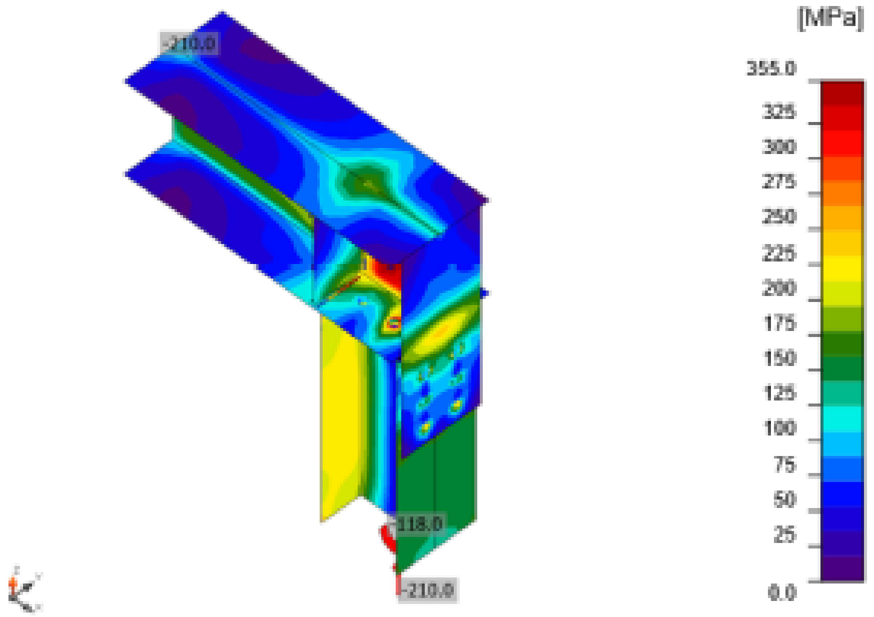
Symbol explanation

ε_{Pl}	Strain
σ_{Ed}	Eq. stress
f_y	Yield strength
ε_{lim}	Limit of plastic strain



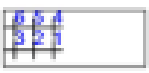
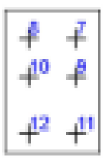
Strain check, LE3

Project:
Project no:
Author:



Equivalent stress, LE3

Bolts

	Name	Grade	Loads	$F_{t,Ed}$ [kN]	V [kN]	Ut_t [%]	$F_{b,Rd}$ [kN]	Ut_s [%]	Ut_{ts} [%]	Status
	B1	M20 8,8 - 1	LE3	13,0	46,5	9,2	236,3	49,4	56,0	OK
	B2	M20 8,8 - 1	LE3	1,0	46,9	0,7	198,6	49,8	50,3	OK
	B3	M20 8,8 - 1	LE3	10,3	48,7	7,3	178,2	51,7	56,9	OK
	B4	M20 8,8 - 1	LE3	13,0	46,5	9,2	236,3	49,4	56,0	OK
	B5	M20 8,8 - 1	LE3	1,0	46,9	0,7	198,6	49,8	50,3	OK
	B6	M20 8,8 - 1	LE3	10,2	48,6	7,3	178,2	51,7	56,9	OK
	B7	M20 8,8 - 2	LE3	69,3	5,1	49,1	278,3	5,4	40,5	OK
	B8	M20 8,8 - 2	LE3	69,1	5,1	49,0	278,3	5,4	40,4	OK
	B9	M20 8,8 - 2	LE3	6,4	2,7	4,5	278,3	2,9	6,1	OK
	B10	M20 8,8 - 2	LE3	6,4	2,7	4,6	278,3	2,9	6,1	OK
	B11	M20 8,8 - 2	LE3	2,8	9,1	2,0	278,3	9,7	11,1	OK
	B12	M20 8,8 - 2	LE3	2,9	9,1	2,1	278,3	9,6	11,1	OK

Design data

Name	$F_{t,Rd}$ [kN]	$B_{p,Rd}$ [kN]	$F_{v,Rd}$ [kN]
M20 8,8 - 1	141,1	290,9	94,1
M20 8,8 - 2	141,1	330,5	94,1

Project:
Project no:
Author:

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Detailed result for B3

Tension resistance check (EN 1993-1-8 tab 3.4)

$F_{t,Rd} = \frac{k_2 f_{ub} A_s}{\gamma_{M2}} = 141{,}1$ kN $\geq$ $F_t = 10{,}3$ kN

where:

$k_2 = 0{,}90$ – Factor

$f_{ub} = 800{,}0$ MPa – Ultimate tensile strength of the bolt

$A_s = 245$ mm^2 – Tensile stress area of the bolt

$\gamma_{M2} = 1{,}25$ – Safety factor

Punching resistance check (EN 1993-1-8 tab 3.4)

$B_{p,Rd} = \frac{0.6 \pi d_m t_p f_u}{\gamma_{M2}} = 290.9$ kN $\geq$ $F_t = 10.3$ kN

where:

$d_m = 32$ mm – The mean of the across points and across flats dimensions of the bolt head or the nut, whichever is smaller

$t_p = 13$ mm – Thickness

$f_u = 490{,}0$ MPa – Ultimate strength

$\gamma_{M2} = 1.25$ – Safety factor

Shear resistance check (EN 1993-1-8 tab 3.4)

$F_{v,Rd} = \frac{\beta_p \alpha_v f_{ub} A}{\gamma_{M2}} = 94{,}1$ kN $\geq$ $V = 48{,}7$ kN

where:

$\beta_p = 1{,}00$ – Reducing factor

$\alpha_v = 0{,}60$ – Reducing factor

$f_{ub} = 800{,}0$ MPa – Ultimate tensile strength of the bolt

$A = 245$ mm^2 – Tensile stress area of the bolt

$\gamma_{M2} = 1{,}25$ – Safety factor

Bearing resistance check (EN 1993-1-8 tab 3.4)

$F_{b,Rd} = \frac{k_1 \alpha_b f_u d t}{\gamma_{M2}} = 178.2$ kN $\geq$ $V = 48.7$ kN

where:

$k_1 = 2.50$ – Factor for edge distance and bolt spacing perpendicular to the direction of load transfer

$\alpha_b = 0{,}61$ – Factor

$f_u = 490{,}0$ MPa – Ultimate strength

$d = 20$ mm – Nominal diameter of the fastener

$t = 15$ mm – Thickness

$\gamma_{M2} = 1{,}25$ – Safety factor

Project:
Project no:
Author:

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Interaction of tension and shear (EN 1993-1-8 tab 3,4)

$$U_{tts} = \frac{F_{v,Ed}}{F_{v,Rd}} + \frac{F_{t,Ed}}{1.4F_{t,Rd}} = \quad 56,9 \quad \%$$

Utilization in tension

$$U_{tt} = \frac{F_{t,Ed}}{\min(F_{t,Rd};B_{p,Rd})} = \quad 7,3 \quad \%$$

Utilization in shear

$$U_{ts} = \frac{V_{Ed}}{\min(F_{v,Rd};F_{b,Rd})} = \quad 51,7 \quad \%$$

Symbol explanation

$F_{t,Rd}$	Bolt tension resistance EN 1993-1-8 tab. 3.4
$F_{t,Ed}$	Tension force
$B_{p,Rd}$	Punching shear resistance
V	Resultant of shear forces Vy, Vz in bolt
$F_{v,Rd}$	Bolt shear resistance EN_1993-1-8 table 3.4
$F_{b,Rd}$	Plate bearing resistance EN 1993-1-8 tab. 3.4
Ut_t	Utilization in tension
Ut_s	Utilization in shear
Ut_{ts}	Utilization in tension and shear EN 1993-1-8 table 3.4

Project:
Project no:
Author:

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Welds (Plastic redistribution)

Item	Edge	Throat th. [mm]	Length [mm]	Loads	$\sigma_{w,Ed}$ [MPa]	ε_{Pl} [%]	$\sigma_{\perp}$ [MPa]	$\tau_{\parallel}$ [MPa]	$\tau_{\perp}$ [MPa]	Ut [%]	Ut_c [%]	Status
EP1	B2-bfl 1	◢4,3◣	200	LE3	430,8	2,2	-156,8	0,2	231,6	98,9	96,7	OK
		◢4,3◣	200	LE3	433,5	3,8	-237,2	-6,4	-209,4	99,5	99,5	OK
EP1	B2-tfl 1	◢4,3◣	200	LE3	102,7	0,0	-76,6	39,4	2,4	23,6	18,7	OK
		◢4,3◣	200	LE3	157,9	0,0	89,2	48,4	57,5	36,2	30,0	OK
EP1	B2-w 1	◢4.3◣	194	LE3	429.6	1.6	-208.0	-61.8	-208.0	98.6	39.2	OK
		◢4,3◣	194	LE3	429,6	1,6	-208,0	61,9	208,0	98,6	39,2	OK
SPL1	B3-w 1	◢4,3	240	LE3	428,5	1,0	3,5	247,4	3,5	98,4	98,3	OK
SPL1	B3-tfl 1	◢4,3	204	LE3	97,9	0,0	65,4	-30,2	29,2	22,5	12,9	OK
SPL1	B3-bfl 1	◢4,3	204	LE3	170,9	0,0	-124,0	-34,7	58,4	39,2	29,6	OK
B3-tfl 1	STIFF1a	◢4.3◣	110	LE3	209.9	0.0	-24.6	-113.3	40.7	48.2	15.5	OK
		◢4,3◣	110	LE3	227,7	0,0	-71,4	112,0	-55,2	52,3	20,7	OK
B3-w 1	STIFF1a	◢4,3◣	200	LE3	210,4	0,0	26,2	118,9	-19,8	48,3	37,4	OK
		◢4.3◣	200	LE3	202.4	0.0	-6.9	-116.8	-0.6	46.5	37.0	OK
B3-bfl 1	STIFF1a	◢4,3◣	110	LE3	426,9	0,1	-197,3	-106,2	191,0	98,0	77,4	OK
		◢4.3◣	110	LE3	354.8	0.0	-150.4	97.0	-158.2	81.5	55.5	OK
B3-tfl 1	STIFF1b	◢4,3◣	110	LE3	227,5	0,0	-71,3	-111,9	55,1	52,2	20,7	OK
		◢4,3◣	110	LE3	209,7	0,0	-24,5	113,2	-40,7	48,2	15,5	OK
B3-w 1	STIFF1b	◢4.3◣	200	LE3	202.1	0.0	-6.8	116.6	0.4	46.4	37.0	OK
		◢4,3◣	200	LE3	210,2	0,0	26,3	-118,7	20,0	48,2	37,3	OK
B3-bfl 1	STIFF1b	◢4.3◣	110	LE3	352.9	0.0	-149.4	-96.9	157.1	81.0	55.4	OK
		◢4,3◣	110	LE3	426,9	0,0	-197,0	106,8	-190,8	98,0	77,4	OK

Design data

	β_w [-]	$\sigma_{w,Rd}$ [MPa]	0,9 σ [MPa]
S 355	0,90	435,6	352,8

Symbol explanation

ε_{Pl}	Strain
$\sigma_{w,Ed}$	Equivalent stress
$\sigma_{w,Rd}$	Equivalent stress resistance
$\sigma_{\perp}$	Perpendicular stress
$\tau_{\parallel}$	Shear stress parallel to weld axis
$\tau_{\perp}$	Shear stress perpendicular to weld axis
0,9 σ	Perpendicular stress resistance - 0,9*fu/γM2
β_w	Corelation factor EN 1993-1-8 tab. 4.1
Ut	Utilization
Utc	Weld capacity utilization

Project:
Project no:
Author:

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Detailed result for EP1 B2-bfl 1

Weld resistance check (EN 1993-1-8 4.5.3.2)

$\sigma_{w,Rd} = f_u/(\beta_w \gamma_{M2}) =$ 435,6 MPa ≥ $\sigma_{w,Ed} = [\sigma_\perp^2 + 3(\tau_\perp^2 + \tau_\parallel^2)]^{0.5} =$ 433,5 MPa

$\sigma_{\perp,Rd} = 0.9 f_u/\gamma_{M2} =$ 352,8 MPa ≥ $|\sigma_\perp| =$ 237,2 MPa

where:

$f_u = 490{,}0$ MPa – Ultimate strength

$\beta_w = 0{,}90$ – appropriate correlation factor taken from Table 4.1

$\gamma_{M2} = 1{,}25$ – Safety factor

Stress utilization

$U_t = \max(\frac{\sigma_{w,Ed}}{\sigma_{w,Rd}}; \frac{|\sigma_\perp|}{\sigma_{\perp,Rd}}) =$ 99,5 %

Buckling

Loads	Shape	Factor [-]
LE3	1	9,86
	2	10,20
	3	14,10
	4	17,34
	5	17,44
	6	17,85

Project:
Project no:
Author:

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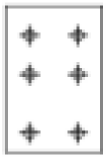
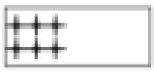
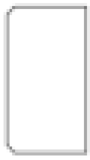
Bill of material

Manufacturing operations

Name	Plates [mm]	Shape	Nr.	Welds [mm]	Length [mm]	Bolts	Nr.
EP1	P15,0x200,0-300,0 (S 355)		1	Double fillet: a = 4,3	593,7	M20 8,8	6
SPL1	P15,0x514,0-204,3 (S 355)		1			M20 8,8	6
STIFF1	P15,0x123,0-225,7 (S 355)		2	Double fillet: a = 4,3	841,8		

Welds

Type	Material	Throat thickness [mm]	Leg size [mm]	Length [mm]
Double fillet	S 355	4,3	6,1	1435,5
Fillet	S 355	4,3	6,1	444,2
Fillet	S 355	4,3	6,1	204,3

Bolts

Name	Grip length [mm]	Count
M20 8.8	27	6
M20 8.8	29	6

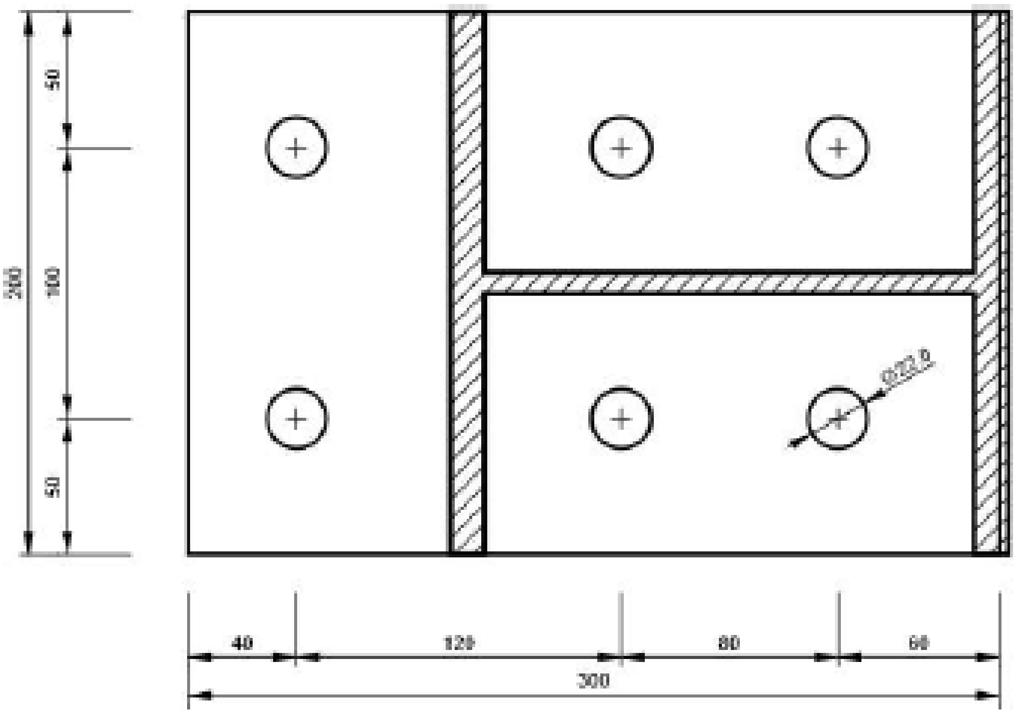
Drawing

EP1

Project:
Project no:
Author:

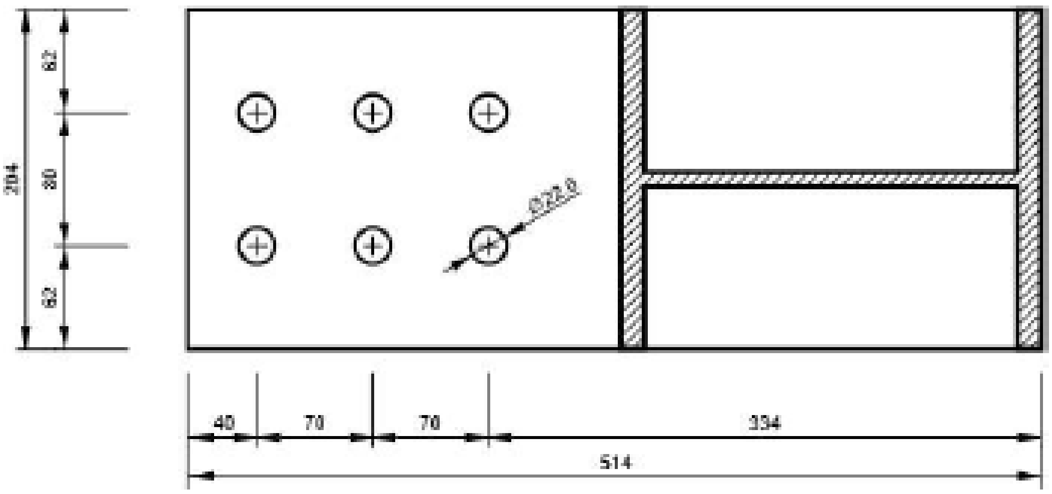
IDEA StatiCa

P15,0x300-200 (S 355)



SPL1

P15,0x204-514 (S 355)

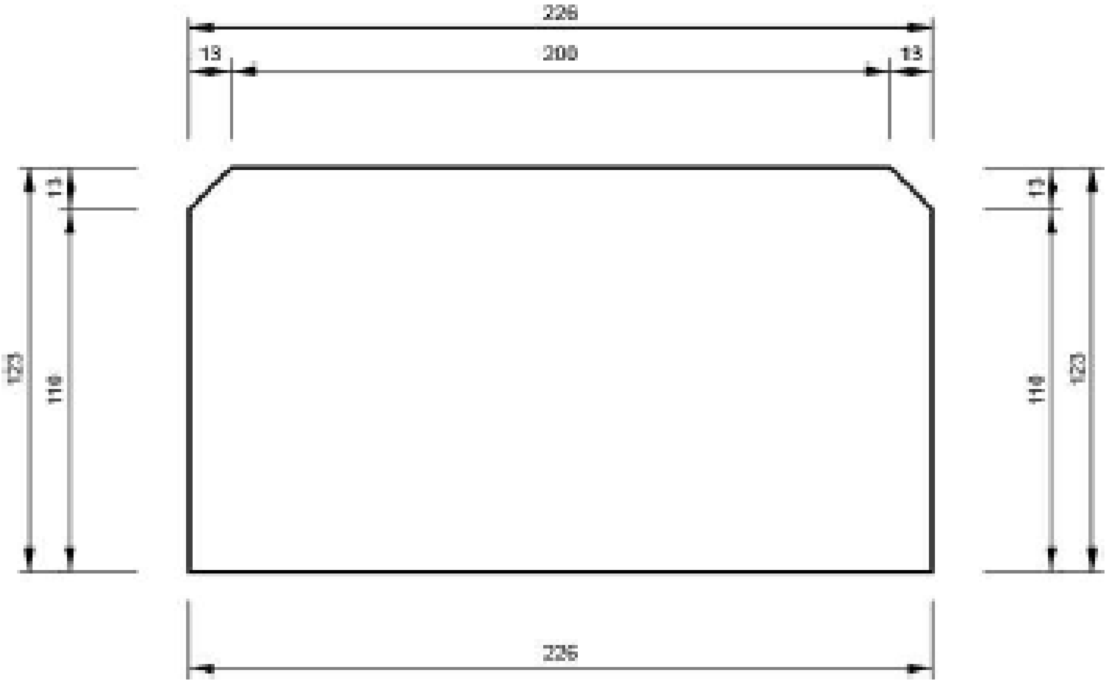


Project:
Project no:
Author:

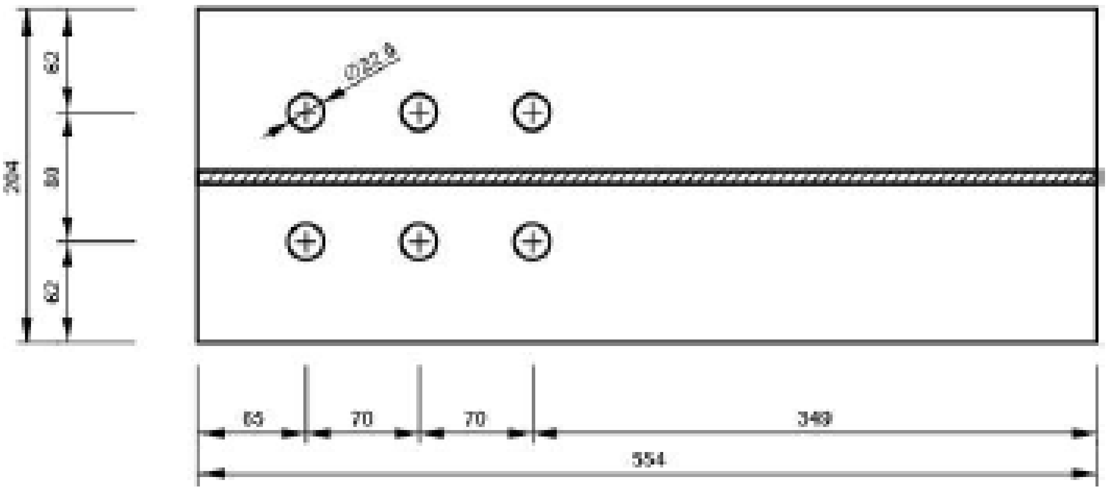
IDEA StatiCa

STIFF1

P15,0x226-123 (S 355)



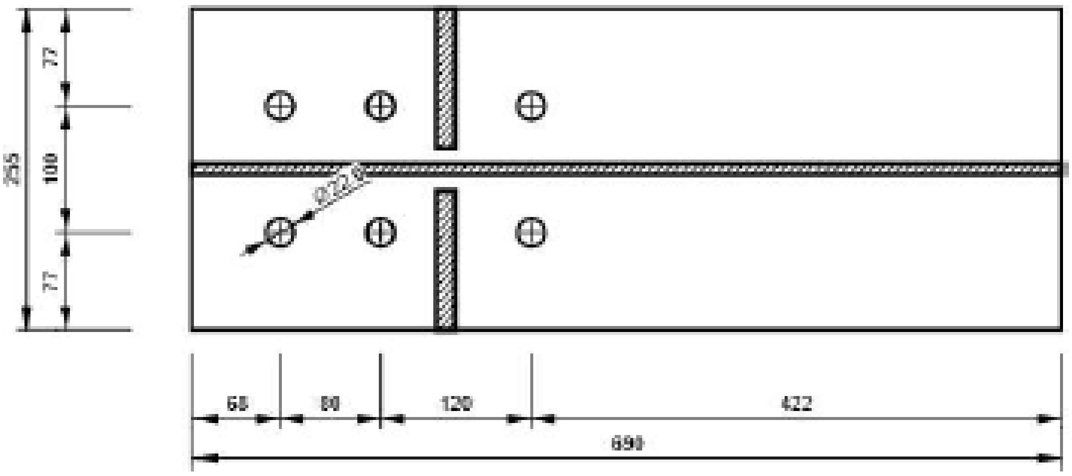
B2, UC 203 x 203 x 52 - Top flange 1:



Project:
Project no:
Author:

IDEA StatiCa

B3, UC 254 x 254 x 73 - Bottom flange 1:



Code settings

Item	Value	Unit	Reference
γ_{M0}	1.00	-	EN 1993-1-1: 6.1
γ_{M1}	1.00	-	EN 1993-1-1: 6.1
γ_{M2}	1.25	-	EN 1993-1-1: 6.1
γ_{M3}	1.25	-	EN 1993-1-8: 2.2
γ_C	1.50	-	EN 1992-1-1: 2.4.2.4
γ_{Inst}	1.20	-	ETAG 001-C: 3.2.1
Joint coefficient βj	0.67	-	EN 1993-1-8: 6.2.5
Effective area - influence of mesh size	0.10	-	
Friction coefficient - concrete	0.25	-	EN 1993-1-8
Friction coefficient in slip-resistance	0.30	-	EN 1993-1-8 tab 3.7
Limit plastic strain	0.05	-	EN 1993-1-5
Weld stress evaluation	Plastic redistribution		
Detailing	No		
Distance between bolts [d]	2.20	-	EN 1993-1-8: tab 3.3
Distance between bolts and edge [d]	1.20	-	EN 1993-1-8: tab 3.3
Concrete breakout resistance	Yes		ETAG 001-C
Use calculated αb in bearing check.	Yes		EN 1993-1-8: tab 3.4
Cracked concrete	Yes		
Local deformation check	No		
Local deformation limit	0.03	-	CIDECT DG 1, 3 - 1.1
Geometrical nonlinearity (GMNA)	Yes		Large deformations for hollow sections

17. Connection Design connection CON4: Base Plate and Anchor Bolts

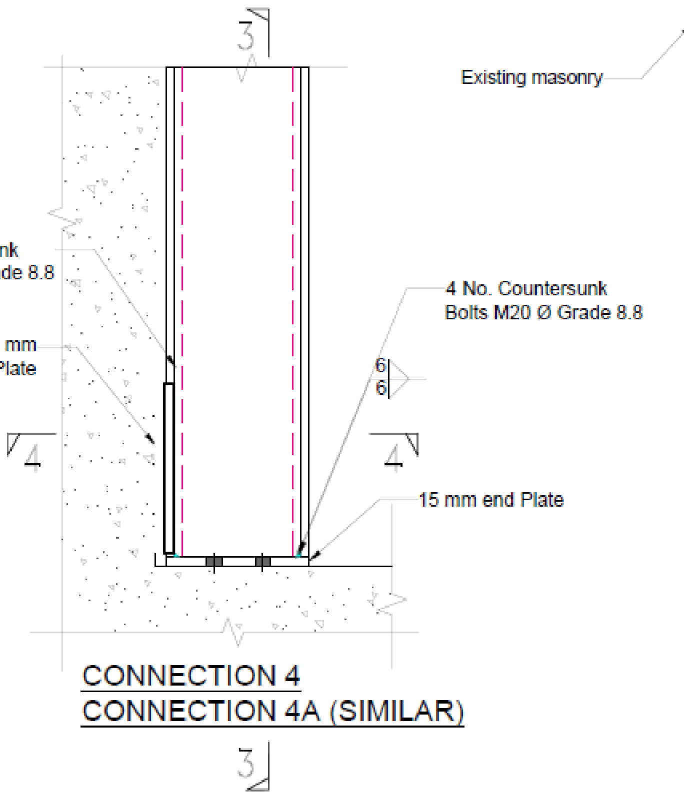


Figure 12: Location of Steel Base Plate and Anchor Bolts

Project:
Project no:
Author:

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Calculate yesterday's estimates

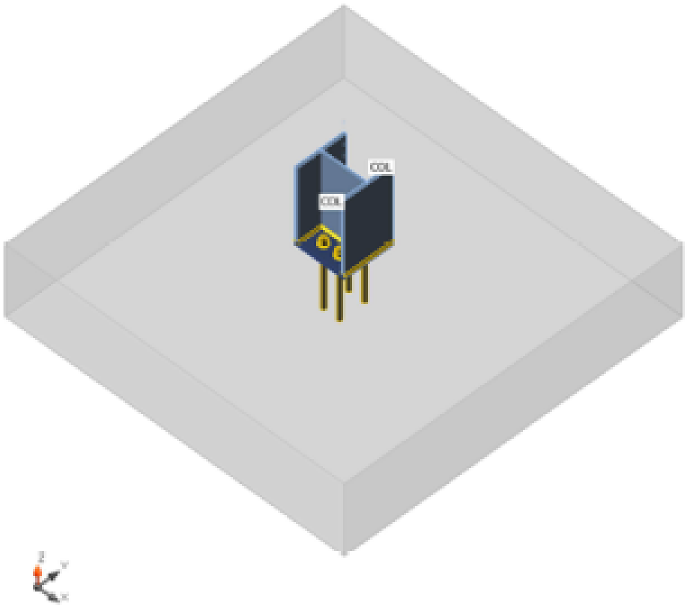
Project item Base plate

Design

Name	Base plate
Description	15 Landoor Road
Analysis	Stress, strain/ simplified loading

Beams and columns

Name	Cross-section	β – Direction [°]	γ - Pitch [°]	α - Rotation [°]	Offset ex [mm]	Offset ey [mm]	Offset ez [mm]	Forces in
COL	1 - CON1(UC 203 x 203 x 52)	0,0	-90,0	0,0	0	0	0	Node



Cross-sections

Name	Material
1 - CON1(UC 203 x 203 x 52)	S 355

Anchors

Name	Bolt assembly	Diameter [mm]	fu [MPa]	Gross area [mm^2]
M20 8,8	M20 8,8	20	800,0	314

Project:
Project no:
Author:

IDEA StatiCa
Calculate yesterday's estimates

Load effects (equilibrium not required)

Name	Member	N [kN]	Vy [kN]	Vz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
LE1	COL	-210.0	0.0	0.0	0.0	0.0	0.0

Foundation block

Item	Value	Unit
CB 1		
Dimensions	1404 x 1406	mm
Depth	300	mm
Anchor	M20 8.8	
Anchoring length	250	mm
Shear force transfer	Friction	

Check

Summary

Name	Value	Status
Analysis	100.0%	OK
Plates	0.0 < 5%	OK
Anchors	0.0 < 100%	OK
Welds	23.9 < 100%	OK
Concrete block	25.4 < 100%	OK
Shear	0.0 < 100%	OK
Buckling	57.64	

Plates

Name	Thickness [mm]	Loads	σ_{Ed} [MPa]	ε_{Pl} [%]	Status
COL-bfl 1	12.5	LE1	80.6	0.0	OK
COL-tfl 1	12.5	LE1	80.6	0.0	OK
COL-w 1	7.9	LE1	72.1	0.0	OK
BP1	15.0	LE1	87.8	0.0	OK

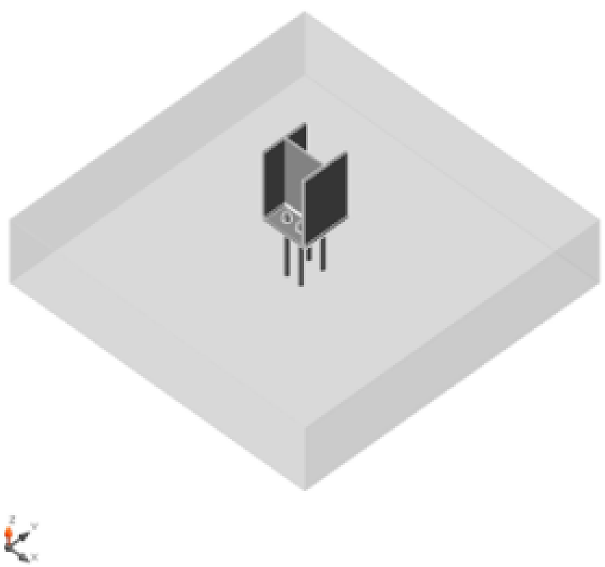
Design data

Material	f_y [MPa]	ε_{lim} [%]
S 355	355.0	5.0

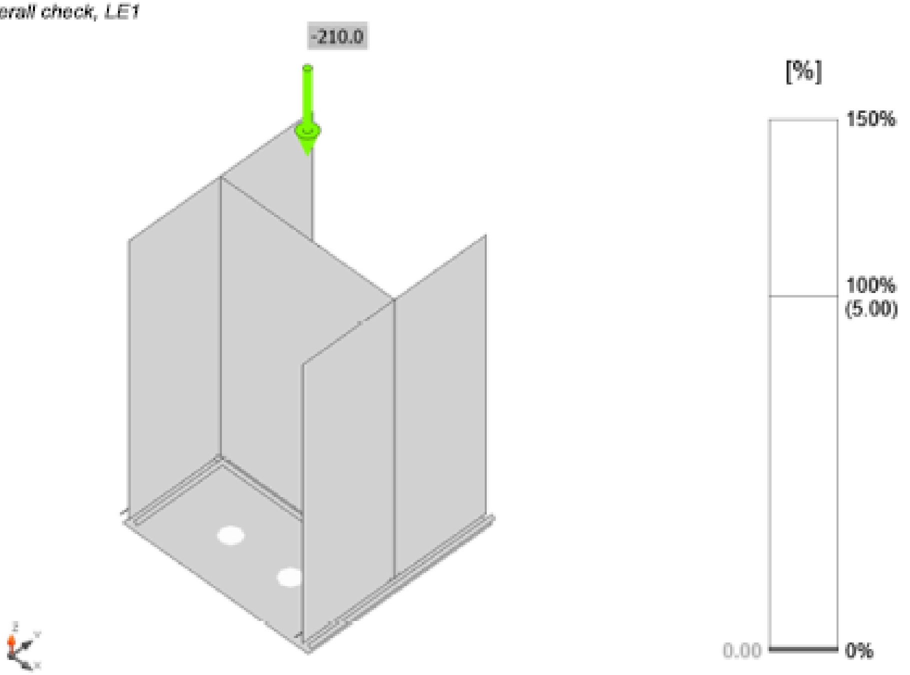
Symbol explanation

ε_{Pl}	Strain
σ_{Ed}	Eq. stress
f_y	Yield strength
ε_{lim}	Limit of plastic strain

Project:
Project no:
Author:

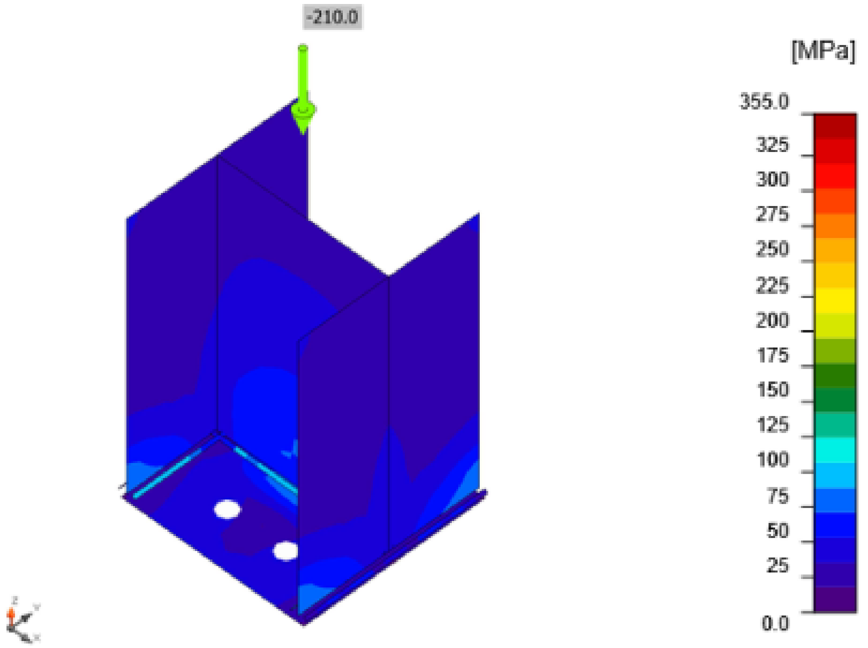


Overall check, LE1



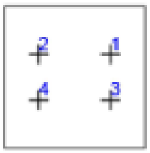
Strain check, LE1

Project:
Project no:
Author:



Equivalent stress, LE1

Anchors

	Name	Loads	$F_{t,Ed}$ [kN]	V [kN]	N_{rdc} [kN]	N_{rdp} [kN]	Ut_t [%]	$F_{b,Rd}$ [kN]	Ut_s [%]	Ut_{ts} [%]	$V_{Rd,cp}$ [kN]	$V_{Rd,c}$ [kN]	Status
	A1	LE1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	OK
	A2	LE1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	OK
	A3	LE1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	OK
	A4	LE1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	OK

Design data

Name	$F_{t,Rd}$ [kN]	$B_{p,Rd}$ [kN]	$F_{v,Rd}$ [kN]	V_{rds} [kN]	S_{tf} [MN/m]
M20 8.8 - 1	120.0	349.1	120.6	0.0	412

Project:
Project no:
Author:

Symbol explanation

$F_{t,Rd}$	Bolt tension resistance EN 1993-1-8 tab. 3.4
$F_{t,Ed}$	Tension force
$B_{p,Rd}$	Punching shear resistance
V	Resultant of shear forces Vy, Vz in bolt
$F_{v,Rd}$	Bolt shear resistance EN_1993-1-8 table 3.4
V_{rds}	Characteristic anchor resistance ETAG 001 Annex C (5.2.3.2)
S_{lf}	Anchor longitudinal stiffness
$F_{b,Rd}$	Plate bearing resistance EN 1993-1-8 tab. 3.4
$N_{rd,c}$	Concrete breakout resistance
$N_{rd,p}$	Pull-out resistance
Ut_t	Utilization in tension
Ut_s	Utilization in shear
Ut_{ts}	Utilization in tension and shear EN 1993-1-8 table 3.4
$V_{rd,cp}$	Concrete pry-out failure ETAG 001 Annex C (5.2.3.3)
$V_{rd,c}$	Concrete edge failure ETAG 001 Annex C (5.2.3.4)
C_{pf}	Concrete pry-out failure ETAG 001 Annex C (5.2.3.3)
C_{ef}	Concrete edge failure ETAG 001 Annex C (5.2.3.4)

Welds (Plastic redistribution)

Item	Edge	Throat th. [mm]	Length [mm]	Loads	$\sigma_{w,Ed}$ [MPa]	ε_{Pl} [%]	$\sigma_\perp$ [MPa]	$\tau_{\|\|}$ [MPa]	$\tau_\perp$ [MPa]	Ut [%]	Ut_c [%]	Status
BP1	COL-bfl 1	◢4.3◣	204	LE1	103.8	0.0	-51.7	32.6	-40.5	23.8	20.1	OK
		◢4.3◣	204	LE1	55.9	0.0	-4.9	29.5	12.8	12.8	10.4	OK
BP1	COL-tfl 1	◢4.3◣	204	LE1	55.9	0.0	-4.9	-29.5	-12.8	12.8	10.4	OK
		◢4.3◣	204	LE1	103.8	0.0	-51.7	-32.6	40.5	23.8	20.1	OK
BP1	COL-w 1	◢4.3◣	194	LE1	104.2	0.0	-51.9	-2.2	-52.1	23.9	19.1	OK
		◢4.3◣	194	LE1	104.2	0.0	-52.2	2.4	52.0	23.9	19.1	OK

Design data

	β_w [-]	$\sigma_{w,Rd}$ [MPa]	0.9 σ [MPa]
S 355	0.90	435.6	352.8

Project:
Project no:
Author:

IDEA StatiCa
Calculate yesterday's estimates

Symbol explanation

ε_{Pl}	Strain
$\sigma_{w,Ed}$	Equivalent stress
$\sigma_{w,Rd}$	Equivalent stress resistance
$\sigma_{\perp}$	Perpendicular stress
$\tau_{\parallel}$	Shear stress parallel to weld axis
$\tau_{\perp}$	Shear stress perpendicular to weld axis
0,9 σ	Perpendicular stress resistance - 0,9*fu/γM2
β_w	Corelation factor EN 1993-1-8 tab. 4.1
Ut	Utilization
Utc	Weld capacity utilization

Concrete block

Item	Loads	c [mm]	A_{eff} [mm^2]	σ [MPa]	k_j [-]	F_{jd} [MPa]	Ut [%]	Status
CB 1	LE1	28	24650	8.5	3.00	33.5	25.4	OK

Symbol explanation

c	Bearing width
A_{eff}	Effective area
σ	Average stress in concrete
k_j	Concentration factor
F_{jd}	The ultimate bearing strength of the concrete block
Ut	Utilization

Shear in contact plane

Name	Loads	V_y [kN]	V_z [kN]	$V_{Rd,y}$ [kN]	$V_{Rd,z}$ [kN]	$V_{c,Rd}$ [kN]	Ut [%]	Status
BP1	LE1	0.0	0.0	52.5	52.5	0.0	0.0	OK

Symbol explanation

V_y	Shear force in base plate Vy
V_z	Shear force in base plate Vz
$V_{Rd,y}$	Shear resistance
$V_{Rd,z}$	Shear resistance
$V_{c,Rd}$	Concrete bearing resistance
Ut	Utilization

Project:
Project no:
Author:

Buckling

Loads	Shape	Factor [-]
LE1	1	57,64
	2	64,65
	3	82,02
	4	93,00
	5	103,58
	6	129,89

Code settings

Item	Value	Unit	Reference
γ_{M0}	1,00	-	EN 1993-1-1: 6,1
γ_{M1}	1,00	-	EN 1993-1-1: 6.1
γ_{M2}	1,25	-	EN 1993-1-1: 6.1
γ_{M3}	1,25	-	EN 1993-1-8: 2,2
γ_{C}	1,50	-	EN 1992-1-1: 2,4,2,4
γ_{Inst}	1,20	-	ETAG 001-C: 3.2.1
Joint coefficient βj	0,67	-	EN 1993-1-8: 6.2.5
Effective area - influence of mesh size	0,10	-	
Friction coefficient - concrete	0,25	-	EN 1993-1-8
Friction coefficient in slip-resistance	0,30	-	EN 1993-1-8 tab 3,7
Limit plastic strain	0,05	-	EN 1993-1-5
Weld stress evaluation	Plastic redistribution		
Detailing	No		
Distance between bolts [d]	2,20	-	EN 1993-1-8: tab 3,3
Distance between bolts and edge [d]	1,20	-	EN 1993-1-8: tab 3.3
Concrete breakout resistance	Yes		ETAG 001-C
Use calculated αb in bearing check,	Yes		EN 1993-1-8: tab 3,4
Cracked concrete	Yes		
Local deformation check	No		
Local deformation limit	0,03	-	CIDECT DG 1, 3 - 1.1
Geometrical nonlinearity (GMNA)	Yes		Large deformations for hollow sections

18. Padstone BP1 Design

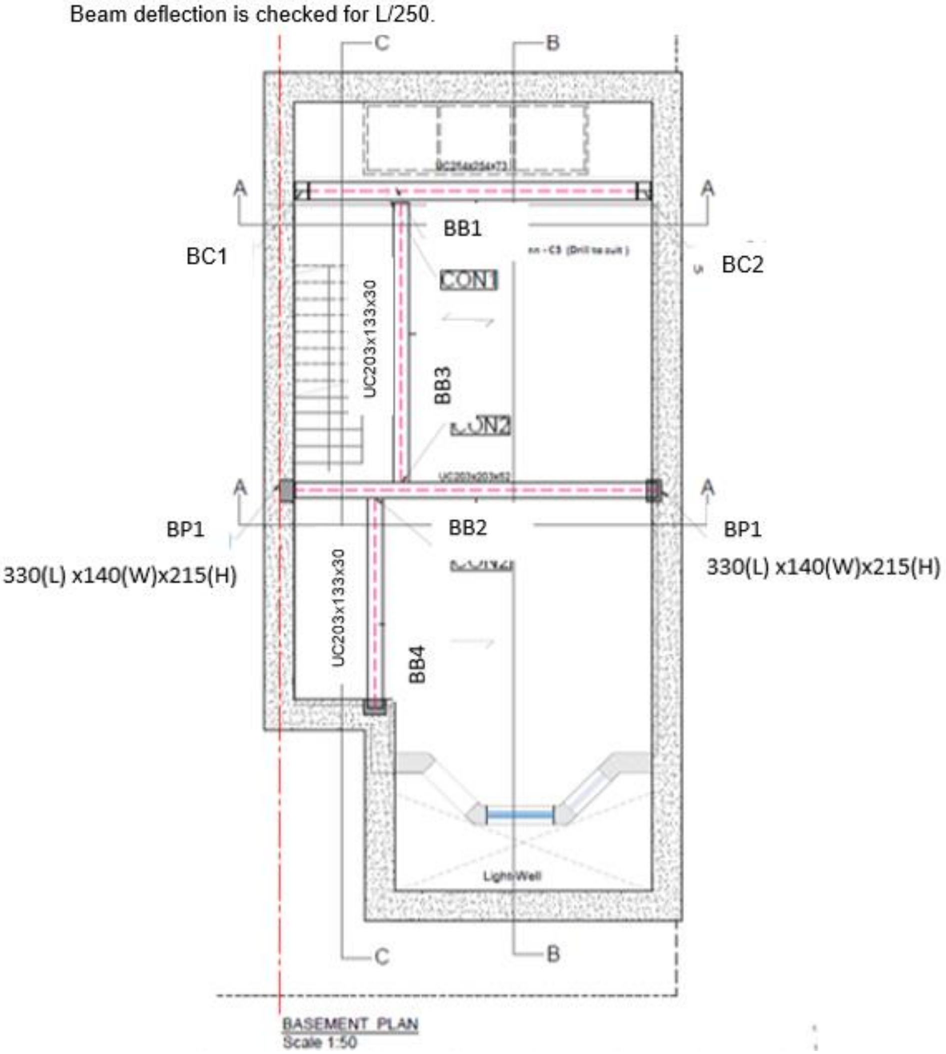


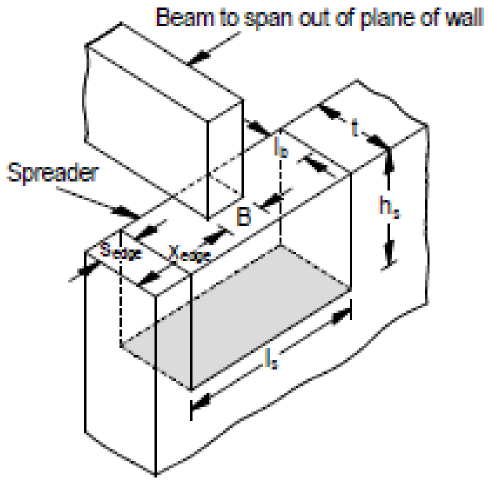
Figure 13: Location of Steel Beams on Plan showing Padstone BP1

MASONRY BEARING DESIGN TO BS5628-1:2005

TEDDS calculation version 1.0.06

Masonry details

Masonry type	**Clay or calcium silicate bricks**
Compressive strength of unit	p_{unit} = **5.0** N/mm^2
Mortar designation	**iii**
Category of masonry units	**Category II**
Category of construction control	**Normal**
Partial safety factor for material strength	γ_m = **3.5**
Thickness of load bearing leaf	t = **140** mm
Effective thickness of masonry wall	t_{ef} = **260** mm
Height of masonry wall	h = **1000** mm
Effective height of masonry wall	h_{ef} = **1000** mm



Bearing details

Beam spanning out of plane of wall

Width of bearing	B = **200** mm
Length of bearing	l_b = **140** mm
Edge distance	x_{edge} = **50** mm

Compressive strength from Table 2 BS5628:Part 1 - Clay or calcium silicate bricks

Mortar designation	Mortar = **"iii"**
Brick compressive strength	p_{unit} = **5.0** N/mm^2
Characteristic compressive strength	f_k = **2.50** N/mm^2

Loading details

Characteristic concentrated dead load	G_k = **27** kN
Characteristic concentrated imposed load	Q_k = **0** kN

Design concentrated load $F = (G_k \times 1.4) + (Q_k \times 1.6) = \mathbf{38.1}$ kN

Characteristic distributed dead load $g_k = \mathbf{0.0}$ kN/m

Characteristic distributed imposed load $q_k = \mathbf{0.0}$ kN/m

Design distributed load $f = (g_k \times 1.4) + (q_k \times 1.6) = \mathbf{0.0}$ kN/m

Masonry bearing type

Bearing type **Type 1**

Bearing safety factor $\gamma_{bear} = \mathbf{1.25}$

Check design bearing without a spreader

Design bearing stress $f_{ca} = F / (B \times l_b) + f / t = \mathbf{1.360}$ N/mm^2

Allowable bearing stress $f_{cp} = \gamma_{bear} \times f_k / \gamma_m = \mathbf{0.893}$ N/mm^2

FAIL - Design bearing stress exceeds allowable bearing stress, use a spreader

Spreader details

Length of spreader $l_s = \mathbf{330}$ mm

Depth of spreader $h_s = \mathbf{215}$ mm

Edge distance $s_{edge} = \max(0 \text{ mm}, x_{edge} - (l_s - B) / 2) = \mathbf{0}$ mm

Spreader bearing type

Bearing type **Type 3**

Bearing safety factor $\gamma_{bear} = \mathbf{2.00}$

Check design bearing with a spreader

Loading acts eccentrically within middle third – triangular stress distribution

Eccentricity of load $e = ((l_s - B) / 2) - x_{edge} = \mathbf{15}$ mm

Maximum bearing stress $f_{ca} = F \times (1 + (6 \times e / l_s)) / (l_s \times t) + f / t = \mathbf{1.049}$ N/mm^2

Allowable bearing stress $f_{cp} = \gamma_{bear} \times f_k / \gamma_m = \mathbf{1.429}$ N/mm^2

PASS - Allowable bearing stress exceeds design bearing stress

Check design bearing at 0.4 × h below the bearing level

Slenderness ratio $h_{ef} / t_{ef} = \mathbf{3.85}$

Eccentricity at top of wall $e_x = \mathbf{0.0}$ mm

From BS5628:1 Table 7

Capacity reduction factor $\beta = \mathbf{0.99}$

Length of bearing distributed at 0.4 × h $l_d = \mathbf{650}$ mm

Maximum bearing stress $f_{ca} = F / (l_d \times t) + f / t = \mathbf{0.418}$ N/mm^2

Allowable bearing stress $f_{cp} = \beta \times f_k / \gamma_m = \mathbf{0.707}$ N/mm^2

PASS - Allowable bearing stress at 0.4 × h below bearing level exceeds design bearing stress

19. Design of Underpins and Slab

19.1 Underpin Design inc Slab & Wall Stability (Side wall and party wall)

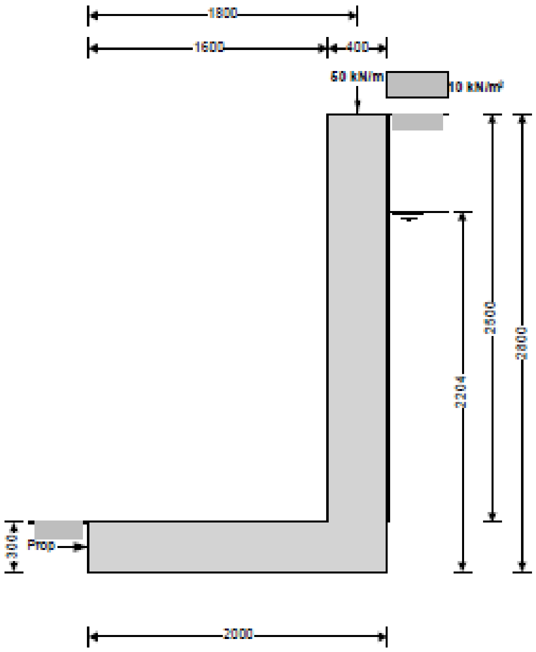
Loads are based on the self weight of the wall only as this weight gives the most onerous load case becasuse the weight of the wall will act to restore any destabilizing moment and we do not want to overestimate this beneficial restoring moment. As such, any loads on the rear wall or flank wall coming from beams supported on the wall are ignored. At the same time, it is important not to underestimate these loads as the vertical load on the wall will affect the bearing pressure under the wall. So these loads are considered when checking bearing and ingored when checking sliding and overturning.

Load on Retaining wall (Side wall /Party wall)

Walls				
Ground floor wall load	2.8 x 0.26 x 18	=	13.1	kN/m
First floor wall load	2.8 x 0.26 x 18	=	13.1	kN/m
Second floor wall load	2.8 x 0.26 x 18	=	13.1	kN/m
Timber Floors (Ground floor)				
Dead Load Ground floor (0.75kN/m^2)	0.75 x 2.55	=	1.9	kN/m
Live Load Ground floor (1.5kN/m^2)	1.5 x 2.55	=	3.8	kN/m
Pitched Roof				
Dead Load (0.75kN/m^2)	0.75 x 2.55	=	1.9	kN/m
Live Load (1.0kN/m^2)	1.5 x 2.55	=	2.6	kN/m
Total Load on wall (Basement wall Load not added)		=	49.5	kN/m

RETAINING WALL ANALYSIS (BS 8002:1994)

TEDDS calculation version 1.2.01.06



Wall details

Retaining wall type	Cantilever propped at base
Height of retaining wall stem	h_{stem} = 2500 mm
Thickness of wall stem	t_{wall} = 400 mm
Length of toe	l_{toe} = 1600 mm
Length of heel	l_{heel} = 0 mm
Overall length of base	$l_{base} = l_{toe} + l_{heel} + t_{wall}$ = 2000 mm
Thickness of base	t_{base} = 300 mm
Depth of downstand	d_{ds} = 0 mm
Position of downstand	l_{ds} = 1000 mm
Thickness of downstand	t_{ds} = 300 mm
Height of retaining wall	$h_{wall} = h_{stem} + t_{base} + d_{ds}$ = 2800 mm
Depth of cover in front of wall	d_{cover} = 0 mm
Depth of unplanned excavation	d_{exc} = 0 mm
Height of ground water behind wall	h_{water} = 2204 mm
Height of saturated fill above base	$h_{sat} = \max(h_{water} - t_{base} - d_{ds}, 0 \text{ mm})$ = 1904 mm
Density of wall construction	γ_{wall} = 23.6 kN/m^3
Density of base construction	γ_{base} = 23.6 kN/m^3
Angle of rear face of wall	α = 90.0 deg
Angle of soil surface behind wall	β = 0.0 deg
Effective height at virtual back of wall	$h_{eff} = h_{wall} + l_{heel} \times \tan(\beta)$ = 2800 mm

Retained material details

Mobilisation factor	M = 1.5

Moist density of retained material	γ_m = 18.0 kN/m³
Saturated density of retained material	γ_s = 21.0 kN/m³
Design shear strength	ϕ' = 24.2 deg
Angle of wall friction	δ = 0.0 deg

Base material details

Moist density	γ_{mb} = 18.0 kN/m³
Design shear strength	ϕ'_b = 24.2 deg
Design base friction	δ_b = 18.6 deg
Allowable bearing pressure	$P_{bearing}$ = 85 kN/m²

Using Coulomb theory

Active pressure coefficient for retained material

$$K_a = \sin(\alpha + \phi')^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta) \times [1 + \surd(\sin(\phi' + \delta) \times \sin(\phi' - \beta) / (\sin(\alpha - \delta) \times \sin(\alpha + \beta)))]^2) = 0.419$$

Passive pressure coefficient for base material

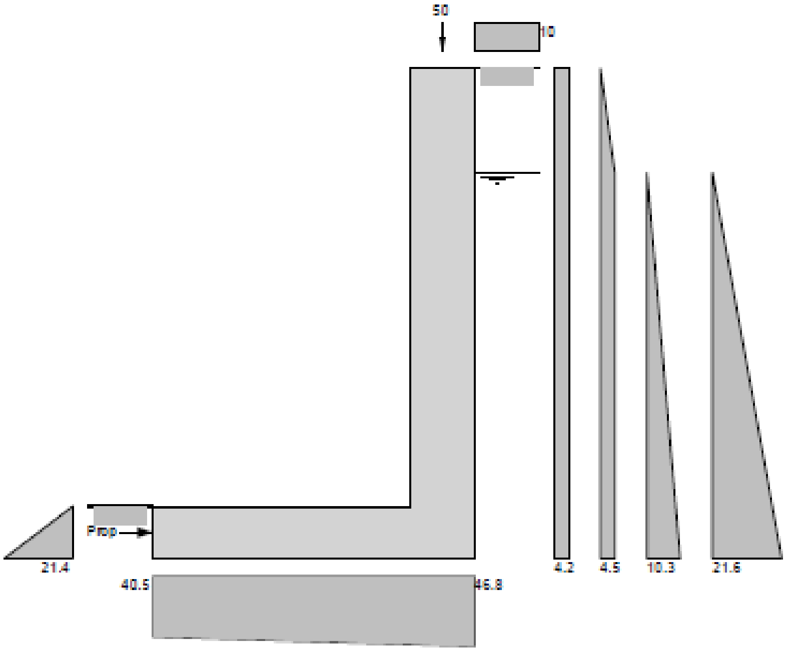
$$K_p = \sin(90 - \phi'_b)^2 / (\sin(90 - \delta_b) \times [1 - \surd(\sin(\phi'_b + \delta_b) \times \sin(\phi'_b) / (\sin(90 + \delta_b)))]^2) = 4.187$$

At-rest pressure

At-rest pressure for retained material	$K_0 = 1 - \sin(\phi') = 0.590$

Loading details

Surcharge load on plan	Surcharge = 10.0 kN/m²
Applied vertical dead load on wall	W_{dead} = 49.5 kN/m
Applied vertical live load on wall	W_{live} = 0.0 kN/m
Position of applied vertical load on wall	l_{load} = 1800 mm
Applied horizontal dead load on wall	F_{dead} = 0.0 kN/m
Applied horizontal live load on wall	F_{live} = 0.0 kN/m
Height of applied horizontal load on wall	h_{load} = 0 mm



BETA DESIGN CONSULTANTS

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

Vertical forces on wall

Wall stem $W_{wall} = h_{stem} \times t_{wall} \times \gamma_{wall} = 23.6$ kN/m

Wall base $W_{base} = l_{base} \times t_{base} \times \gamma_{base} = 14.2$ kN/m

Applied vertical load $W_v = W_{dead} + W_{live} = 49.5$ kN/m

Total vertical load $W_{total} = W_{wall} + W_{base} + W_v = 87.3$ kN/m

Horizontal forces on wall

Surcharge $F_{sur} = K_a \times \text{Surcharge} \times h_{eff} = 11.7$ kN/m

Moist backfill above water table $F_{m\_a} = 0.5 \times K_a \times \gamma_m \times (h_{eff} - h_{water})^2 = 1.3$ kN/m

Moist backfill below water table $F_{m\_b} = K_a \times \gamma_m \times (h_{eff} - h_{water}) \times h_{water} = 9.9$ kN/m

Saturated backfill $F_s = 0.5 \times K_a \times (\gamma_s - \gamma_{water}) \times h_{water}^2 = 11.4$ kN/m

Water $F_{water} = 0.5 \times h_{water}^2 \times \gamma_{water} = 23.8$ kN/m

Total horizontal load $F_{total} = F_{sur} + F_{m\_a} + F_{m\_b} + F_s + F_{water} = 58.2$ kN/m

Calculate propping force

Passive resistance of soil in front of wall $F_p = 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb} = 3.2$ kN/m

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

$F_{prop} = 25.6$ kN/m

Overturning moments

Surcharge $M_{sur} = F_{sur} \times (h_{eff} - 2 \times d_{ds}) / 2 = 16.4$ kNm/m

Moist backfill above water table $M_{m\_a} = F_{m\_a} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3 = 3.2$ kNm/m

Moist backfill below water table $M_{m\_b} = F_{m\_b} \times (h_{water} - 2 \times d_{ds}) / 2 = 10.9$ kNm/m

Saturated backfill $M_s = F_s \times (h_{water} - 3 \times d_{ds}) / 3 = 8.4$ kNm/m

Water $M_{water} = F_{water} \times (h_{water} - 3 \times d_{ds}) / 3 = 17.5$ kNm/m

Total overturning moment $M_{ot} = M_{sur} + M_{m\_a} + M_{m\_b} + M_s + M_{water} = 56.4$ kNm/m

Restoring moments

Wall stem $M_{wall} = w_{wall} \times (l_{toe} + t_{wall} / 2) = 42.5$ kNm/m

Wall base $M_{base} = w_{base} \times l_{base} / 2 = 14.2$ kNm/m

Design vertical load $M_v = W_v \times l_{load} = 89.1$ kNm/m

Total restoring moment $M_{rest} = M_{wall} + M_{base} + M_v = 145.7$ kNm/m

Check bearing pressure

Total moment for bearing $M_{total} = M_{rest} - M_{ot} = 89.4$ kNm/m

Total vertical reaction $R = W_{total} = 87.3$ kN/m

Distance to reaction $x_{bar} = M_{total} / R = 1024$ mm

Eccentricity of reaction $e = \text{abs}((l_{base} / 2) - x_{bar}) = 24$ mm

Reaction acts within middle third of base

Bearing pressure at toe $p_{toe} = (R / l_{base}) - (6 \times R \times e / l_{base}^2) = 40.5$ kN/m^2

Bearing pressure at heel $p_{heel} = (R / l_{base}) + (6 \times R \times e / l_{base}^2) = 46.8$ kN/m^2

PASS - Maximum bearing pressure is less than allowable bearing pressure

RETAINING WALL DESIGN (BS 8002:1994)

TEDDS calculation version 1.2.01.06

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	$w_{wall\_f} = \gamma_{f\_d} \times h_{stem} \times t_{wall} \times \gamma_{wall} = 33$ kN/m
Wall base	$w_{base\_f} = \gamma_{f\_d} \times l_{base} \times t_{base} \times \gamma_{base} = 19.8$ kN/m
Applied vertical load	$W_{v\_f} = \gamma_{f\_d} \times W_{dead} + \gamma_{f\_l} \times W_{live} = 69.3$ kN/m
Total vertical load	$W_{total\_f} = w_{wall\_f} + w_{base\_f} + W_{v\_f} = 122.2$ kN/m

Factored horizontal at-rest forces on wall

Surcharge	$F_{sur\_f} = \gamma_{f\_l} \times K_0 \times \text{Surcharge} \times h_{eff} = 26.4$ kN/m
Moist backfill above water table	$F_{m\_a\_f} = \gamma_{f\_e} \times 0.5 \times K_0 \times \gamma_m \times (h_{eff} - h_{water})^2 = 2.6$ kN/m
Moist backfill below water table	$F_{m\_b\_f} = \gamma_{f\_e} \times K_0 \times \gamma_m \times (h_{eff} - h_{water}) \times h_{water} = 19.5$ kN/m
Saturated backfill	$F_{s\_f} = \gamma_{f\_e} \times 0.5 \times K_0 \times (\gamma_s - \gamma_{water}) \times h_{water}^2 = 22.5$ kN/m
Water	$F_{water\_f} = \gamma_{f\_e} \times 0.5 \times h_{water}^2 \times \gamma_{water} = 33.4$ kN/m
Total horizontal load	$F_{total\_f} = F_{sur\_f} + F_{m\_a\_f} + F_{m\_b\_f} + F_{s\_f} + F_{water\_f} = 104.4$ kN/m

Calculate propping force

Passive resistance of soil in front of wall	$F_{p\_f} = \gamma_{f\_e} \times 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb} = 4.5$ kN/m
Propping force	$F_{prop\_f} = \max(F_{total\_f} - F_{p\_f} - (W_{total\_f}) \times \tan(\delta_b), 0 \text{ kN/m})$
	$F_{prop\_f} = 58.8$ kN/m

Factored overturning moments

Surcharge	$M_{sur\_f} = F_{sur\_f} \times (h_{eff} - 2 \times d_{ds}) / 2 = 37$ kNm/m
Moist backfill above water table	$M_{m\_a\_f} = F_{m\_a\_f} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3 = 6.3$ kNm/m
Moist backfill below water table	$M_{m\_b\_f} = F_{m\_b\_f} \times (h_{water} - 2 \times d_{ds}) / 2 = 21.5$ kNm/m
Saturated backfill	$M_{s\_f} = F_{s\_f} \times (h_{water} - 3 \times d_{ds}) / 3 = 16.5$ kNm/m
Water	$M_{water\_f} = F_{water\_f} \times (h_{water} - 3 \times d_{ds}) / 3 = 24.5$ kNm/m
Total overturning moment	$M_{ot\_f} = M_{sur\_f} + M_{m\_a\_f} + M_{m\_b\_f} + M_{s\_f} + M_{water\_f} = 105.9$ kNm/m

Restoring moments

Wall stem	$M_{wall\_f} = w_{wall\_f} \times (l_{toe} + t_{wall} / 2) = 59.5$ kNm/m
Wall base	$M_{base\_f} = w_{base\_f} \times l_{base} / 2 = 19.8$ kNm/m
Design vertical load	$M_{v\_f} = W_{v\_f} \times l_{load} = 124.7$ kNm/m
Total restoring moment	$M_{rest\_f} = M_{wall\_f} + M_{base\_f} + M_{v\_f} = 204$ kNm/m

Factored bearing pressure

Total moment for bearing	$M_{total\_f} = M_{rest\_f} - M_{ot\_f} = 98.2$ kNm/m
Total vertical reaction	$R_f = W_{total\_f} = 122.2$ kN/m
Distance to reaction	$x_{bar\_f} = M_{total\_f} / R_f = 803$ mm
Eccentricity of reaction	$e_f = \text{abs}((l_{base} / 2) - x_{bar\_f}) = 197$ mm
	Reaction acts within middle third of base
Bearing pressure at toe	$p_{toe\_f} = (R_f / l_{base}) + (6 \times R_f \times e_f / l_{base}^2) = 97.1$ kN/m^2

Bearing pressure at heel $p_{heel\_f} = (R_f / l_{base}) - (6 \times R_f \times e_f / l_{base}^2) = \mathbf{25.1}$ kN/m^2

Rate of change of base reaction rate $= (p_{toe\_f} - p_{heel\_f}) / l_{base} = \mathbf{36.01}$ kN/m^2/m

Bearing pressure at stem / toe $p_{stem\_toe\_f} = \max(p_{toe\_f} - (\text{rate} \times l_{toe}), 0 \text{ kN/m}^2) = \mathbf{39.5}$ kN/m^2

Bearing pressure at mid stem $p_{stem\_mid\_f} = \max(p_{toe\_f} - (\text{rate} \times (l_{toe} + t_{wall} / 2)), 0 \text{ kN/m}^2) = \mathbf{32.3}$ kN/m^2

Bearing pressure at stem / heel $p_{stem\_heel\_f} = \max(p_{toe\_f} - (\text{rate} \times (l_{toe} + t_{wall})), 0 \text{ kN/m}^2) = \mathbf{25.1}$ kN/m^2

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

Material properties

Characteristic strength of concrete $f_{cu} = 30$ N/mm^2

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

Base details

Minimum area of reinforcement k = 0.13 %

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

Calculate shear for toe design

Shear from bearing pressure $V_{toe\_bear} = (p_{toe\_f} + p_{stem\_toe\_f}) \times l_{toe} / 2 = \mathbf{109.3}$ kN/m

Shear from weight of base $V_{toe\_wt\_base} = \gamma_{f\_d} \times \gamma_{base} \times l_{toe} \times t_{base} = \mathbf{15.9}$ kN/m

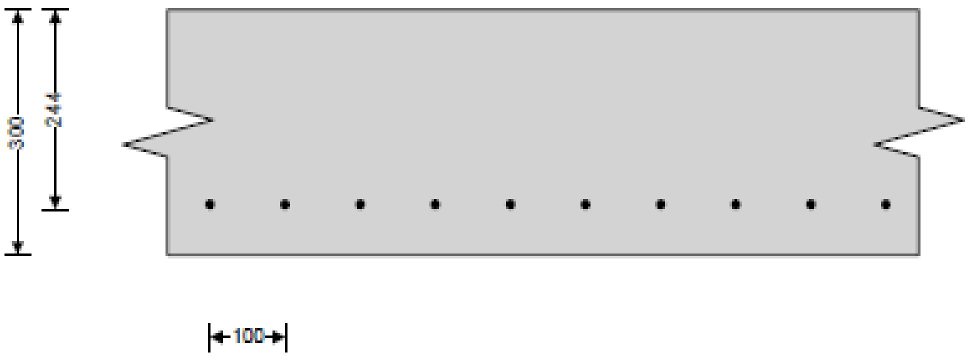
Total shear for toe design $V_{toe} = V_{toe\_bear} - V_{toe\_wt\_base} = \mathbf{93.4}$ kN/m

Calculate moment for toe design

Moment from bearing pressure $M_{toe\_bear} = (2 \times p_{toe\_f} + p_{stem\_mid\_f}) \times (l_{toe} + t_{wall} / 2)^2 / 6 = \mathbf{122.3}$ kNm/m

Moment from weight of base $M_{toe\_wt\_base} = (\gamma_{f\_d} \times \gamma_{base} \times t_{base} \times (l_{toe} + t_{wall} / 2)^2 / 2) = \mathbf{16.1}$ kNm/m

Total moment for toe design $M_{toe} = M_{toe\_bear} - M_{toe\_wt\_base} = \mathbf{106.2}$ kNm/m



Check toe in bending

Width of toe b = 1000 mm/m

Depth of reinforcement $d_{toe} = t_{base} - c_{toe} - (\phi_{toe} / 2) = \mathbf{244.0}$ mm

Constant $K_{toe} = M_{toe} / (b \times d_{toe}^2 \times f_{cu}) = \mathbf{0.059}$

Compression reinforcement is not required

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

$z_{toe} = 227$ mm

Area of tension reinforcement required $A_{s\_toe\_des} = M_{toe} / (0.87 \times f_y \times z_{toe}) = \mathbf{1078}$ mm^2/m

Minimum area of tension reinforcement $A_{s\_toe\_min} = k \times b \times t_{base} = \mathbf{390}$ mm^2/m

Area of tension reinforcement required $A_{s\_toe\_req} = \text{Max}(A_{s\_toe\_des}, A_{s\_toe\_min}) = \mathbf{1078}$ mm^2/m

Reinforcement provided **12 mm dia.bars @ 100 mm centres**

Area of reinforcement provided $A_{s\_toe\_prov} = \mathbf{1131}$ mm^2/m

PASS - Reinforcement provided at the retaining wall toe is adequate

Check shear resistance at toe

Design shear stress $v_{toe} = V_{toe} / (b \times d_{toe})$ = **0.383** N/mm^2

Allowable shear stress $v_{adm} = \min(0.8 \times \sqrt{(f_{cu} / 1\ N/mm^2)}, 5) \times 1\ N/mm^2$ = **4.382** 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} = **0.588** N/mm^2

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

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

Material properties

Characteristic strength of concrete f_{cu} = **30** N/mm^2

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

Wall details

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

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

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

Factored horizontal at-rest forces on stem

Surcharge $F_{s\_sur\_f} = \gamma_{f\_l} \times K_0 \times \text{Surcharge} \times (h_{eff} - t_{base} - d_{ds})$ = **23.6** kN/m

Moist backfill above water table $F_{s\_m\_a\_f} = 0.5 \times \gamma_{f\_e} \times K_0 \times \gamma_m \times (h_{eff} - t_{base} - d_{ds} - h_{sat})^2$ = **2.6** kN/m

Moist backfill below water table $F_{s\_m\_b\_f} = \gamma_{f\_e} \times K_0 \times \gamma_m \times (h_{eff} - t_{base} - d_{ds} - h_{sat}) \times h_{sat}$ = **16.9** kN/m

Saturated backfill $F_{s\_s\_f} = 0.5 \times \gamma_{f\_e} \times K_0 \times (\gamma_s - \gamma_{water}) \times h_{sat}^2$ = **16.8** kN/m

Water $F_{s\_water\_f} = 0.5 \times \gamma_{f\_e} \times \gamma_{water} \times h_{sat}^2$ = **24.9** kN/m

Calculate shear for stem design

Shear at base of stem $V_{stem} = F_{s\_sur\_f} + F_{s\_m\_a\_f} + F_{s\_m\_b\_f} + F_{s\_s\_f} + F_{s\_water\_f} - F_{prop\_f}$ = **26** kN/m

Calculate moment for stem design

Surcharge $M_{s\_sur} = F_{s\_sur\_f} \times (h_{stem} + t_{base}) / 2$ = **33** kNm/m

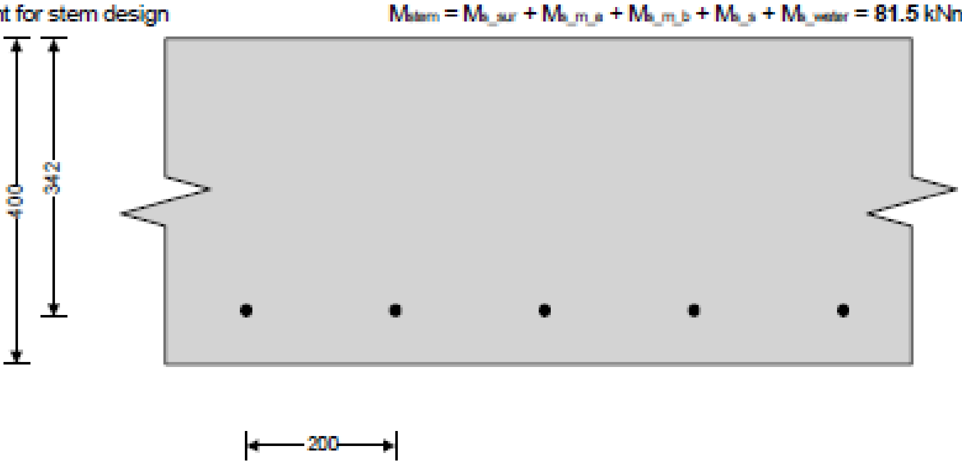
Moist backfill above water table $M_{s\_m\_a} = F_{s\_m\_a\_f} \times (2 \times h_{sat} + h_{eff} - d_{ds} + t_{base} / 2) / 3$ = **5.9** kNm/m

Moist backfill below water table $M_{s\_m\_b} = F_{s\_m\_b\_f} \times h_{sat} / 2$ = **16.1** kNm/m

Saturated backfill $M_{s\_s} = F_{s\_s\_f} \times h_{sat} / 3$ = **10.6** kNm/m

Water $M_{s\_water} = F_{s\_water\_f} \times h_{sat} / 3$ = **15.8** kNm/m

Total moment for stem design $M_{stem} = M_{s\_sur} + M_{s\_m\_a} + M_{s\_m\_b} + M_{s\_s} + M_{s\_water}$ = **81.5** kNm/m



Check wall stem in bending

Width of wall stem — $b = 1000$ mm/m

Depth of reinforcement — $d_{stem} = t_{wall} - c_{stem} - (\phi_{stem} / 2) = 342.0$ mm

Constant — $K_{stem} = M_{stem} / (b \times d_{stem}^2 \times f_{cu}) = 0.023$

Compression reinforcement is not required

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

$z_{stem} = 325$ mm

Area of tension reinforcement required — $A_{s\_stem\_des} = M_{stem} / (0.87 \times f_y \times z_{stem}) = 577$ mm^2/m

Minimum area of tension reinforcement — $A_{s\_stem\_min} = k \times b \times t_{wall} = 520$ mm^2/m

Area of tension reinforcement required — $A_{s\_stem\_req} = \mathrm{Max}(A_{s\_stem\_des}, A_{s\_stem\_min}) = 577$ mm^2/m

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

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

PASS - Reinforcement provided at the retaining wall stem is adequate

Check shear resistance at wall stem

Design shear stress — $v_{stem} = V_{stem} / (b \times d_{stem}) = 0.076$ N/mm^2

Allowable shear stress — $v_{adm} = \min(0.8 \times \sqrt{(f_{cu} / 1 \text{ N/mm}^2)}, 5) \times 1 \text{ N/mm}^2 = 4.382$ 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 — $v_{c\_stem} = 0.464$ N/mm^2

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

Check retaining wall deflection

Basic span/effective depth ratio — $ratio_{bas} = 7$

Design service stress — $f_s = 2 \times f_y \times A_{s\_stem\_req} / (3 \times A_{s\_stem\_prov}) = 191.2$ N/mm^2

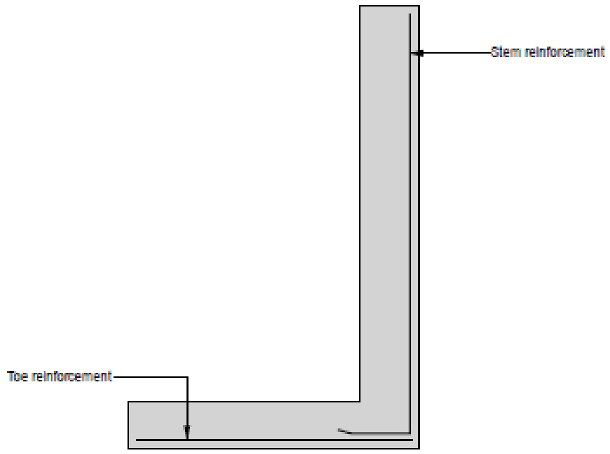
Modification factor — $factor_{tens} = \min(0.55 + (477 \text{ N/mm}^2 - f_s)/(120 \times (0.9 \text{ N/mm}^2 + (M_{stem}/(b \times d_{stem}^2)))), 2) = 2.00$

Maximum span/effective depth ratio — $ratio_{max} = ratio_{bas} \times factor_{tens} = 14.00$

Actual span/effective depth ratio — $ratio_{act} = h_{stem} / d_{stem} = 7.31$

PASS - Span to depth ratio is acceptable

<u>Indicative retaining wall reinforcement diagram</u>



Toe bars - 12 mm dia.@ 100 mm centres - (1131 mm^2/m)
Stem bars - 16 mm dia.@ 200 mm centres - (1005 mm^2/m)

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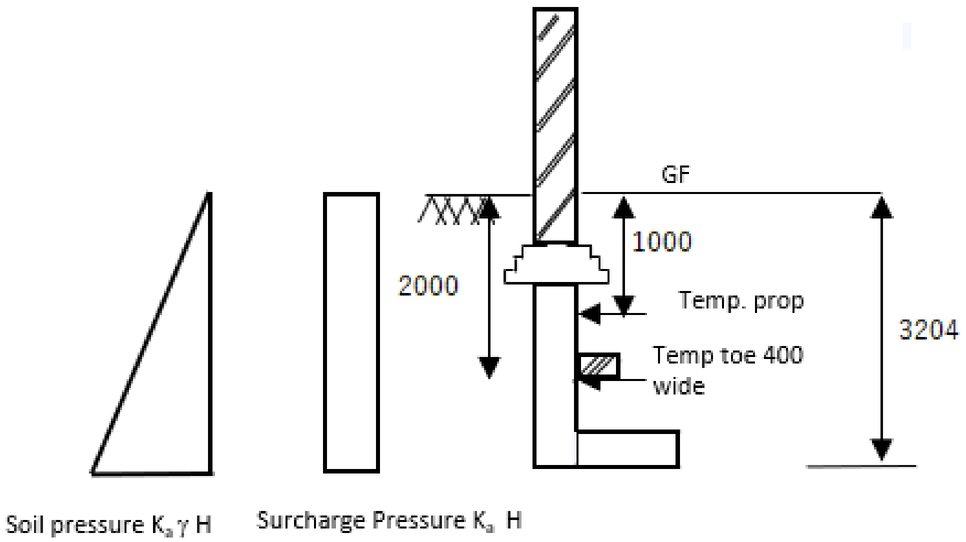
Sliding between brick wall and concrete basement check (Temporary condition)

Consider underpinning to basement with Temp. prop at 1.0m Below GL (neglect water as borehole sample did not encounter any)

Geometry Basement underpinning



Sliding between brick wall and concrete basement check (Temporary condition)

Consider underpinning to basement with Temp. prop at 1.0m Below GL (neglect water as borehole sample did not encounter any)

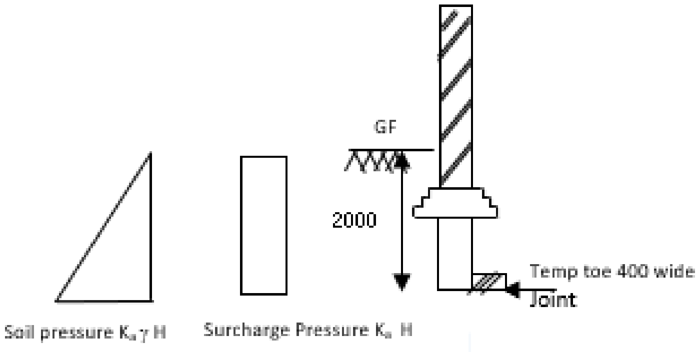
Shear at prop level - assume pinned at base

Base Shear Load due to soil + Surcharge $= K_a \gamma H (H/2) (H/3)$ + $K_a Q_s H H/2$	=	45.0	kN/m	
Load in prop	=	20.4	kN/m	
Base Load $= K_a \gamma H (H/2)$ + $K_a Q_s H$ - Load in prop	=	16.9	kN/m	
Wall Load (GF wall+ 1st floor wall + 2nd floor wall+basement wall) (Wall Load is based on the self weight of the wall only as this weight gives the most onerous load case)	=	73.9	kN/m	
Friction Load	=	36.9	kN/m	
FOS sliding	=	2.2		OK for shear load

Check bending moment in underpin at joint

Bending moment at the base = Load in prob x dist from joir $K_a \gamma H(H/2).(H/3)$ - $K_a Q_s$ H.(	=	7.2	kN-m/m	
Line of thrust 'e'	=	0.2		OK line of thrust lies within underpin

Sliding between brick wall and concrete basement check (before the prop install)



Shear at Joint level -

Shear Load due to soil + Surcharge	$= K_a \gamma (H/2)$ + $K_a Q_s H\ H/2$	=	13.8	kN/m
Wall Load (GF wall+ 1[st] floor wall + 2[nd] floor wall+basement wall) (Wall Load is based on the self weight of the wall only as this weight gives the most onerous load case)		=	62.3	kN/m
Friction Load		=	31.2	kN/m
FOS sliding		=	2.26	OK for shear load

Check bending moment in underpin at joint

OT Bending moment at the Joint =	$K_a \gamma H(H/2).(H/3) + K_a Q_s H.(H/2)$	=	13.2	kN-m/m
Restoring Moment (eccentricity of load CG of temp Toe)		=	12.5	kN-m/m
Net OT moment		=	0.7	kN-m/m
Eccentricity 'e'		=	11.8	mm — OK, by inspection

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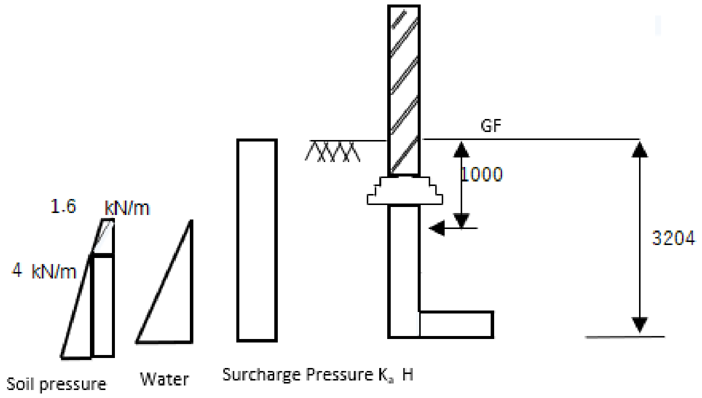
Check final case underpinning

Allow for burst water main pipe , therefore design for 1.0m water table below ground level.



Total lateral load on wall	=	51.4	kN/m
Say free BM	=	20.6	kN-m/m
Axial load	=	73.9	kN/m
Eccentricity of load 'e'	=	278.6	mm (Need to reinforce the underpin)

Design Base slab for uplift

Slab span	=	9.45	m
Basement Slab Thickness	=	0.30	m
Concrete Density	=	25.00	kN/m^3
Ground water level	=	1.00	m
Basement slab width	=	1.00	m
Dead weight of slab	=	7.50	kN/m
Uplift pressure (Ground water pressure assumed)	=	9.81	kN/m^2
Uplift UDL of slab	=	9.81	kN/m
Hogging moment due to uplift	=	2.31 x 9.45^2 / 8	
	=	25.8	kN-m/m
Grade of steel, Fy	=	500.0	Mpa
Effective depth of slab , d	=	242.0	mm
Area of steel required Ast , (at the top face of slab)	=	25.79x 10^6 / (0.783 x 1000 x 242 x 500)	
	=	0.2722	mm2/m
Minimum steel required , Ast min	=	314.6	mm2/m

Provide mesh A393 at Top of slab

19.2 Design Front and back Slab & Wall

Load on Retaining wall (back and front)

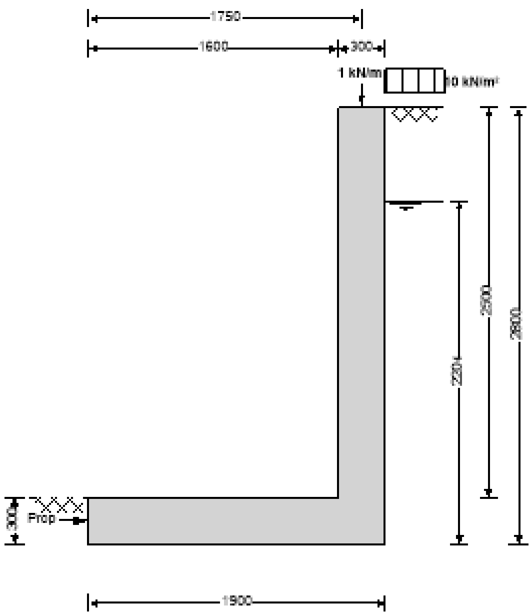
Glass Rooflight

Dead Load (0.25kN/m^2)	0.5 x 0.5	=	0.3	kN/m
Live Load (1.5kN/m^2)	1.5 x 0.5	=	0.8	kN/m
Total Load on wall		=	1	kN/m

(Basement wall Load not added)

RETAINING WALL ANALYSIS (BS 8002:1994)

TEDDS calculation version 1.2.01.06



Wall details

Retaining wall type	**Cantilever propped at base**
Height of retaining wall stem	h_{stem} = **2500** mm
Thickness of wall stem	t_{wall} = **300** mm
Length of toe	l_{toe} = **1600** mm
Length of heel	l_{heel} = **0** mm
Overall length of base	$l_{base} = l_{toe} + l_{heel} + t_{wall}$ = **1900** mm
Thickness of base	t_{base} = **300** mm
Depth of downstand	d_{ds} = **0** mm
Position of downstand	l_{ds} = **1600** mm
Thickness of downstand	t_{ds} = **300** mm
Height of retaining wall	$h_{wall} = h_{stem} + t_{base} + d_{ds}$ = **2800** mm
Depth of cover in front of wall	d_{cover} = **0** mm
Depth of unplanned excavation	d_{exc} = **0** mm
Height of ground water behind wall	h_{water} = **2204** mm
Height of saturated fill above base	$h_{sat} = \max(h_{water} - t_{base} - d_{ds}, 0\text{ mm})$ = **1904** mm
Density of wall construction	γ_{wall} = **23.6** kN/m^3
Density of base construction	γ_{base} = **23.6** kN/m^3
Angle of rear face of wall	α = **90.0** deg
Angle of soil surface behind wall	β = **0.0** deg
Effective height at virtual back of wall	$h_{eff} = h_{wall} + l_{heel} \times \tan(\beta)$ = **2800** mm

Retained material details

Mobilisation factor	M = **1.5**

Moist density of retained material γ_m = **18.0** kN/m^3

Saturated density of retained material γ_s = **21.0** kN/m^3

Design shear strength ϕ' = **24.2** deg

Angle of wall friction δ = **0.0** deg

Base material details

Moist density γ_{mb} = **18.0** kN/m^3

Design shear strength ϕ'_b = **24.2** deg

Design base friction δ_b = **18.6** deg

Allowable bearing pressure $P_{bearing}$ = **85** kN/m^2

Using Coulomb theory

Active pressure coefficient for retained material

$$K_a = \sin(\alpha + \phi')^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta) \times [1 + \sqrt{(\sin(\phi' + \delta) \times \sin(\phi' - \beta) / (\sin(\alpha - \delta) \times \sin(\alpha + \beta)))}]^2) = \mathbf{0.419}$$

Passive pressure coefficient for base material

$$K_p = \sin(90 - \phi'_b)^2 / (\sin(90 - \delta_b) \times [1 - \sqrt{(\sin(\phi'_b + \delta_b) \times \sin(\phi'_b) / (\sin(90 + \delta_b)))}]^2) = \mathbf{4.187}$$

At-rest pressure

At-rest pressure for retained material $K_0 = 1 - \sin(\phi')$ = **0.590**

Loading details

Surcharge load on plan Surcharge = **10.0** kN/m^2

Applied vertical dead load on wall W_{dead} = **1.0** kN/m

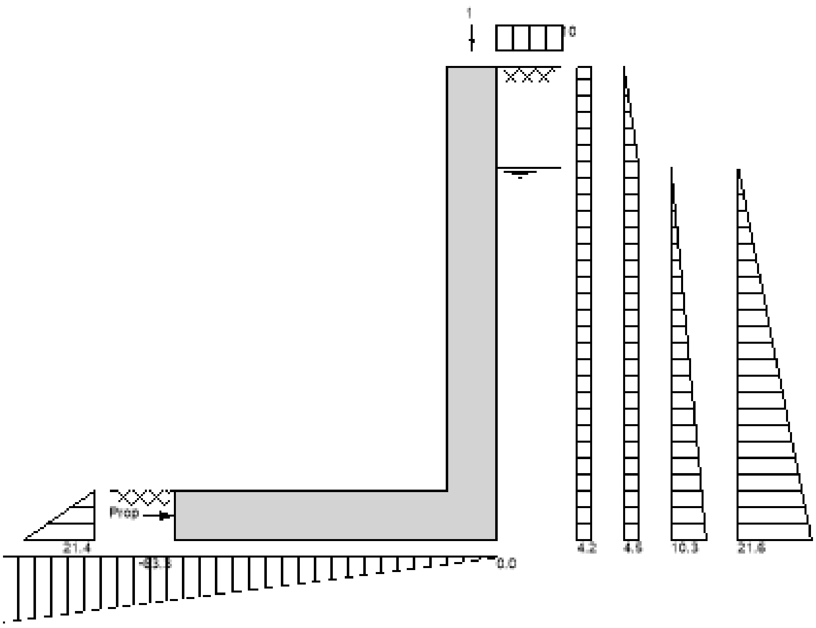
Applied vertical live load on wall W_{live} = **0.0** kN/m

Position of applied vertical load on wall l_{load} = **1750** mm

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

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

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



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

Vertical forces on wall

Wall stem	$w_{wall} = h_{stem} \times t_{wall} \times \gamma_{wall}$ = **17.7** kN/m
Wall base	$w_{base} = l_{base} \times t_{base} \times \gamma_{base}$ = **13.5** kN/m
Applied vertical load	$W_v = W_{dead} + W_{live}$ = **1** kN/m
Total vertical load	$W_{total} = w_{wall} + w_{base} + W_v$ = **32.2** kN/m

Horizontal forces on wall

Surcharge	$F_{sur} = K_a \times \text{Surcharge} \times h_{eff}$ = **11.7** kN/m
Moist backfill above water table	$F_{m\_a} = 0.5 \times K_a \times \gamma_m \times (h_{eff} - h_{water})^2$ = **1.3** kN/m
Moist backfill below water table	$F_{m\_b} = K_a \times \gamma_m \times (h_{eff} - h_{water}) \times h_{water}$ = **9.9** kN/m
Saturated backfill	$F_s = 0.5 \times K_a \times (\gamma_s - \gamma_{water}) \times h_{water}^2$ = **11.4** kN/m
Water	$F_{water} = 0.5 \times h_{water}^2 \times \gamma_{water}$ = **23.8** kN/m
Total horizontal load	$F_{total} = F_{sur} + F_{m\_a} + F_{m\_b} + F_s + F_{water}$ = **58.2** kN/m

Calculate propping force

Passive resistance of soil in front of wall	$F_p = 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **3.2** kN/m
Propping force	$F_{prop} = \max(F_{total} - F_p - (W_{total}) \times \tan(\delta_b), 0 \text{ kN/m})$
	F_{prop} = **44.1** kN/m

Overturning moments

Surcharge	$M_{sur} = F_{sur} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **16.4** kNm/m
Moist backfill above water table	$M_{m\_a} = F_{m\_a} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3$ = **3.2** kNm/m
Moist backfill below water table	$M_{m\_b} = F_{m\_b} \times (h_{water} - 2 \times d_{ds}) / 2$ = **10.9** kNm/m
Saturated backfill	$M_s = F_s \times (h_{water} - 3 \times d_{ds}) / 3$ = **8.4** kNm/m
Water	$M_{water} = F_{water} \times (h_{water} - 3 \times d_{ds}) / 3$ = **17.5** kNm/m
Total overturning moment	$M_{ot} = M_{sur} + M_{m\_a} + M_{m\_b} + M_s + M_{water}$ = **56.4** kNm/m

Restoring moments

Wall stem	$M_{wall} = w_{wall} \times (l_{toe} + t_{wall} / 2)$ = **31** kNm/m
Wall base	$M_{base} = w_{base} \times l_{base} / 2$ = **12.8** kNm/m
Design vertical load	$M_v = W_v \times l_{load}$ = **1.8** kNm/m
Total restoring moment	$M_{rest} = M_{wall} + M_{base} + M_v$ = **45.5** kNm/m

Check bearing pressure

Total moment for bearing	$M_{total} = M_{rest} - M_{ot}$ = **-10.9** kNm/m
Total vertical reaction	$R = W_{total}$ = **32.2** kN/m
Distance to reaction	$x_{bar} = M_{total} / R$ = **-338** mm
Eccentricity of reaction	$e = \text{abs}((l_{base} / 2) - x_{bar})$ = **1288** mm

WARNING - Beyond scope of calculation

Bearing pressure at toe	$p_{toe} = R / (1.5 \times x_{bar})$ = **-63.3** kN/m²
Bearing pressure at heel	p_{heel} = 0 kN/m² = **0** kN/m²

PASS - Maximum bearing pressure is less than allowable bearing pressure

RETAINING WALL DESIGN (BS 8002:1994)

TEDDS calculation version 1.2.01.06

Ultimate limit state load factors

Dead load factor	γ_{f_d} = **1.4**
Live load factor	γ_{f_l} = **1.6**
Earth and water pressure factor	γ_{f_e} = **1.4**

Factored vertical forces on wall

Wall stem	$w_{wall\_f} = \gamma_{f\_d} \times h_{stem} \times t_{wall} \times \gamma_{wall}$ = **24.8** kN/m
Wall base	$w_{base\_f} = \gamma_{f\_d} \times l_{base} \times t_{base} \times \gamma_{base}$ = **18.8** kN/m
Applied vertical load	$W_{v\_f} = \gamma_{f\_d} \times W_{dead} + \gamma_{f\_l} \times W_{live}$ = **1.4** kN/m
Total vertical load	$W_{total\_f} = w_{wall\_f} + w_{base\_f} + W_{v\_f}$ = **45** kN/m

Factored horizontal at-rest forces on wall

Surcharge	$F_{sur\_f} = \gamma_{f\_l} \times K_0 \times Surcharge \times h_{eff}$ = **26.4** kN/m
Moist backfill above water table	$F_{m\_a\_f} = \gamma_{f\_e} \times 0.5 \times K_0 \times \gamma_m \times (h_{eff} - h_{water})^2$ = **2.6** kN/m
Moist backfill below water table	$F_{m\_b\_f} = \gamma_{f\_e} \times K_0 \times \gamma_m \times (h_{eff} - h_{water}) \times h_{water}$ = **19.5** kN/m
Saturated backfill	$F_{s\_f} = \gamma_{f\_e} \times 0.5 \times K_0 \times (\gamma_s - \gamma_{water}) \times h_{water}^2$ = **22.5** kN/m
Water	$F_{water\_f} = \gamma_{f\_e} \times 0.5 \times h_{water}^2 \times \gamma_{water}$ = **33.4** kN/m
Total horizontal load	$F_{total\_f} = F_{sur\_f} + F_{m\_a\_f} + F_{m\_b\_f} + F_{s\_f} + F_{water\_f}$ = **104.4** kN/m

Calculate propping force

Passive resistance of soil in front of wall	$F_{p\_f} = \gamma_{f\_e} \times 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **4.5** kN/m
Propping force	$F_{prop\_f} = \max(F_{total\_f} - F_{p\_f} - (W_{total\_f}) \times \tan(\delta_b), 0$ kN/m)
	F_{prop_f} = **84.8** kN/m

Factored overturning moments

Surcharge	$M_{sur\_f} = F_{sur\_f} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **37** kNm/m
Moist backfill above water table	$M_{m\_a\_f} = F_{m\_a\_f} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3$ = **6.3** kNm/m
Moist backfill below water table	$M_{m\_b\_f} = F_{m\_b\_f} \times (h_{water} - 2 \times d_{ds}) / 2$ = **21.5** kNm/m
Saturated backfill	$M_{s\_f} = F_{s\_f} \times (h_{water} - 3 \times d_{ds}) / 3$ = **16.5** kNm/m
Water	$M_{water\_f} = F_{water\_f} \times (h_{water} - 3 \times d_{ds}) / 3$ = **24.5** kNm/m
Total overturning moment	$M_{ot\_f} = M_{sur\_f} + M_{m\_a\_f} + M_{m\_b\_f} + M_{s\_f} + M_{water\_f}$ = **105.9** kNm/m

Restoring moments

Wall stem	$M_{wall\_f} = w_{wall\_f} \times (l_{toe} + t_{wall} / 2)$ = **43.4** kNm/m
Wall base	$M_{base\_f} = w_{base\_f} \times l_{base} / 2$ = **17.9** kNm/m
Design vertical load	$M_{v\_f} = W_{v\_f} \times l_{load}$ = **2.5** kNm/m
Total restoring moment	$M_{rest\_f} = M_{wall\_f} + M_{base\_f} + M_{v\_f}$ = **63.7** kNm/m

Factored bearing pressure

Total moment for bearing	$M_{total\_f} = M_{rest\_f} - M_{ot\_f}$ = **-42.2** kNm/m
Total vertical reaction	$R_f = W_{total\_f}$ = **45.0** kN/m
Distance to reaction	$x_{bar\_f} = M_{total\_f} / R_f$ = **-937** mm
Eccentricity of reaction	$e_f = \text{abs}((l_{base} / 2) - x_{bar\_f})$ = **1887** mm
	WARNING - Beyond scope of calculation
Bearing pressure at toe	$p_{toe\_f} = R_f / (1.5 \times x_{bar\_f})$ = **-32** kN/m^2

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

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

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

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

Material properties

Characteristic strength of concrete $f_{cu} = \mathbf{30}\ N/mm^2$

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

Base details

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

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

Calculate shear for toe design

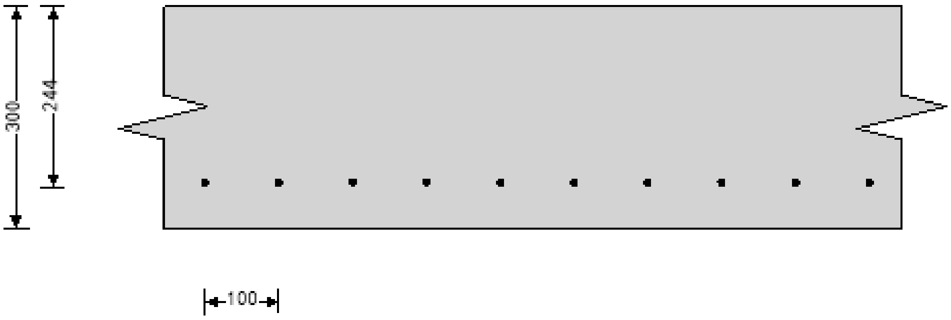
Shear from weight of base $V_{toe\_wt\_base} = \gamma_{f\_d} \times \gamma_{base} \times l_{toe} \times t_{base} = \mathbf{15.9}\ kN/m$

Total shear for toe design $V_{toe} = V_{toe\_wt\_base} = \mathbf{15.9}\ kN/m$

Calculate moment for toe design

Moment from weight of base $M_{toe\_wt\_base} = (\gamma_{f\_d} \times \gamma_{base} \times t_{base} \times (l_{toe} + t_{wall} / 2)^2 / 2) = \mathbf{15.2}\ kNm/m$

Total moment for toe design $M_{toe} = M_{toe\_wt\_base} = \mathbf{15.2}\ kNm/m$



Check toe in bending

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

Depth of reinforcement $d_{toe} = t_{base} - c_{toe} - (\phi_{toe} / 2) = \mathbf{244.0}\ mm$

Constant $K_{toe} = M_{toe} / (b \times d_{toe}^2 \times f_{cu}) = \mathbf{0.008}$

Compression reinforcement is not required

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

$z_{toe} = \mathbf{232}\ mm$

Area of tension reinforcement required $A_{s\_toe\_des} = M_{toe} / (0.87 \times f_y \times z_{toe}) = \mathbf{151}\ mm^2/m$

Minimum area of tension reinforcement $A_{s\_toe\_min} = k \times b \times t_{base} = \mathbf{390}\ mm^2/m$

Area of tension reinforcement required $A_{s\_toe\_req} = \max(A_{s\_toe\_des}, A_{s\_toe\_min}) = \mathbf{390}\ mm^2/m$

Reinforcement provided **12 mm dia.bars @ 100 mm centres**

Area of reinforcement provided $A_{s\_toe\_prov} = \mathbf{1131}\ mm^2/m$

PASS - Reinforcement provided at the retaining wall toe is adequate

Check shear resistance at toe

Design shear stress $v_{toe} = V_{toe} / (b \times d_{toe}) = \mathbf{0.065}\ N/mm^2$

Allowable shear stress $v_{adm} = \min(0.8 \times \sqrt{(f_{cu} / 1\ N/mm^2)}, 5) \times 1\ N/mm^2 = \mathbf{4.382}\ N/mm^2$

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} = **0.588** N/mm²

v_{toe} < v_{c_toe} - No shear reinforcement required

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

Material properties

Characteristic strength of concrete f_{cu} = **30** N/mm²

Characteristic strength of reinforcement f_y = **500** N/mm²

Wall details

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

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

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

Factored horizontal at-rest forces on stem

Surcharge $F_{s\_sur\_f} = \gamma_{f\_l} \times K_0 \times \text{Surcharge} \times (h_{eff} - t_{base} - d_{ds})$ = **26** kN/m

Moist backfill above water table $F_{s\_m\_a\_f} = 0.5 \times \gamma_{f\_e} \times K_0 \times \gamma_m \times (h_{eff} - t_{base} - d_{ds} - h_{sat})^2$ = **31.2** kN/m

Moist backfill below water table $F_{s\_m\_b\_f} = \gamma_{f\_e} \times K_0 \times \gamma_m \times (h_{eff} - t_{base} - d_{ds} - h_{sat}) \times h_{sat}$ = **21.3** kN/m

Saturated backfill $F_{s\_s\_f} = 0.5 \times \gamma_{f\_e} \times K_0 \times (\gamma_s - \gamma_{water}) \times h_{sat}^2$ = **2.3** kN/m

Water $F_{s\_water\_f} = 0.5 \times \gamma_{f\_e} \times \gamma_{water} \times h_{sat}^2$ = **3.4** kN/m

Calculate shear for stem design

Shear at base of stem $V_{stem} = F_{s\_sur\_f} + F_{s\_m\_a\_f} + F_{s\_m\_b\_f} + F_{s\_s\_f} + F_{s\_water\_f} - F_{prop\_f}$ = **2.2** kN/m

Calculate moment for stem design

Surcharge $M_{s\_sur} = F_{s\_sur\_f} \times (h_{stem} + t_{base}) / 2$ = **39.6** kNm/m

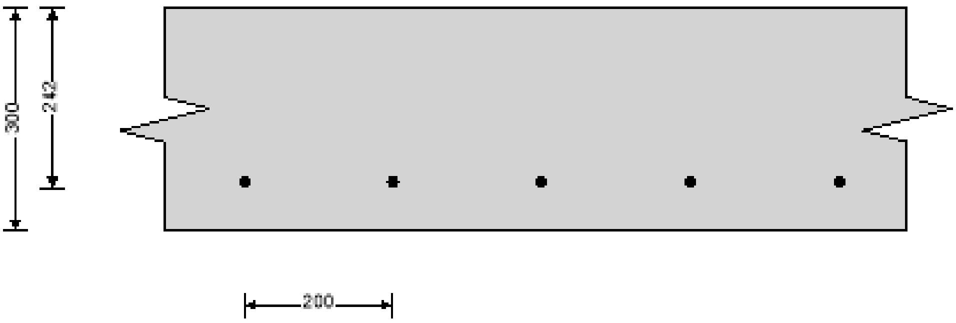
Moist backfill above water table $M_{s\_m\_a} = F_{s\_m\_a\_f} \times (2 \times h_{sat} + h_{eff} - d_{ds} + t_{base} / 2) / 3$ = **47.9** kNm/m

Moist backfill below water table $M_{s\_m\_b} = F_{s\_m\_b\_f} \times h_{sat} / 2$ = **7.5** kNm/m

Saturated backfill $M_{s\_s} = F_{s\_s\_f} \times h_{sat} / 3$ = **0.5** kNm/m

Water $M_{s\_water} = F_{s\_water\_f} \times h_{sat} / 3$ = **0.8** kNm/m

Total moment for stem design $M_{stem} = M_{s\_sur} + M_{s\_m\_a} + M_{s\_m\_b} + M_{s\_s} + M_{s\_water}$ = **96.3** kNm/m



Check wall stem in bending

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

Depth of reinforcement $d_{stem} = t_{wall} - c_{stem} - (\phi_{stem} / 2)$ = **242.0** mm

Constant $K_{stem} = M_{stem} / (b \times d_{stem}^2 \times f_{cu})$ = **0.055**

Compression reinforcement is not required

Depth of reinforcement $d_{stem} = t_{wall} - c_{stem} - (\phi_{stem} / 2) = $ **242.0** mm

Constant $K_{stem} = M_{stem} / (b \times d_{stem}^2 \times f_{cu}) = $ **0.046**

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} = $ **229** mm

Area of tension reinforcement required $A_{s_stem_des} = M_{stem} / (0.87 \times f_y \times z_{stem}) = $ **819** mm²/m

Minimum area of tension reinforcement $A_{s_stem_min} = k \times b \times t_{wall} = $ **390** mm²/m

Area of tension reinforcement required $A_{s_stem_req} = \mathrm{Max}(A_{s_stem_des}, A_{s_stem_min}) = $ **819** mm²/m

Reinforcement provided **16 mm dia.bars @ 150 mm centres**

Area of reinforcement provided $A_{s_stem_prov} = $ **1340** mm²/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}) = $ **0.000** N/mm²

Allowable shear stress $v_{adm} = \min(0.8 \times \sqrt{(f_{cu} / 1\ \mathrm{N/mm^2})}, 5) \times 1\ \mathrm{N/mm^2} = $ **4.382** N/mm²

PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress $v_{c_stem} = $ **0.625** N/mm²

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

Check retaining wall deflection

Basic span/effective depth ratio $ratio_{bas} = $ **7**

Design service stress $f_s = 2 \times f_y \times A_{s_stem_req} / (3 \times A_{s_stem_prov}) = $ **203.6** N/mm²

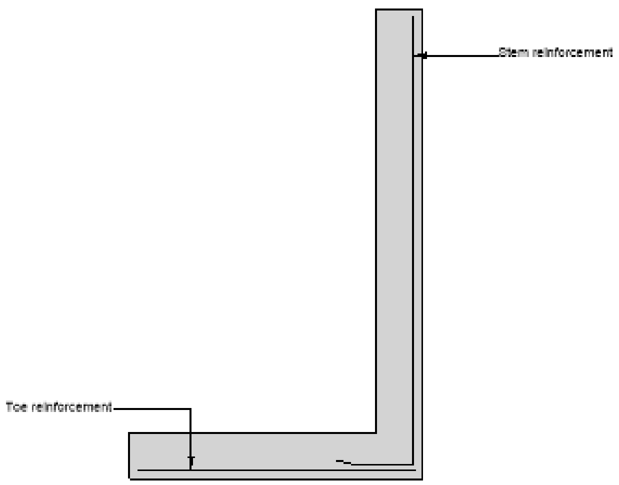
Modification factor $factor_{tens} = \min(0.55 + (477\ \mathrm{N/mm^2} - f_s)/(120 \times (0.9\ \mathrm{N/mm^2} + (M_{stem}/(b \times d_{stem}^2)))), 2) = $ **1.54**

Maximum span/effective depth ratio $ratio_{max} = ratio_{bas} \times factor_{tens} = $ **10.81**

Actual span/effective depth ratio $ratio_{act} = h_{stem} / d_{stem} = $ **10.33**

PASS - Span to depth ratio is acceptable

<u>Indicative retaining wall reinforcement diagram</u>



Toe bars - 12 mm dia.@ 100 mm centres - (1131 mm²/m)
Stem bars - 16 mm dia.@ 200 mm centres - (1005 mm²/m)

20. Design steel beams B4 and B5 supporting Chimney Breast.

Chimney brick work load

Chimney brickwork width		=	1.5	m
Chinmeybrickwork depth		=	0.4	m
Chimney brickwork height (first floor + Second floor) =	2.55 +2.55	=	5.1	m
Unit weight brickwork		=	18.0	kN/m^3
Reduced weight brickwork 30%		=	12.6	kN/m^3
Dead load of chimney brickwork		=	38.6	kN
UDL on UC beam for 1.5m width		=	25.7	kN/m
Beam Span (Max of B4 and B5)		=	3.88	m

Steel member analysis & design (EN1993-1-1:2005)

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

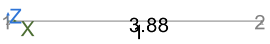
Tedds calculation version 4.3.04

Analysis

Tedds calculation version 1.0.27

Geometry

Geometry (m) - Steel (EC3) - UC 203x203x46

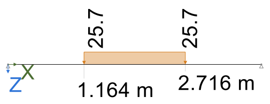


Spa	Length	Section	Start Support	End Support
1	3.88	UC 203x203x46	Pinned	Pinned
UC 203x203x46: Area 59 cm^2, Inertia Major 4568 cm^4, Inertia Minor 1548 cm^4, Shear area parallel to Minor 15 cm^2, Shear area parallel to Major = 40 cm^2				
Steel (EC3): Density 7850 kg/m^3, Youngs 210 kN/mm^2, Shear 80.8 kN/mm^2, Thermal 0.000012 $°C^{-1}$				

Loading

Self weight included

Permanent - Loading (kN/m)



Load combination factors

Load combination	**Self**	**Permane**	**Imposed**
STRENGTH DESIGN (Strength)	1.	1.	1.

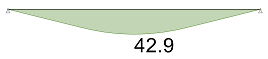
Member Loads

Member	**Load case**	**Load Type**	**Orientat**	**Description**
Member1	Permanent	UDL	GlobalZ	25.7 kN/m at 1.164 m to 2.716 m

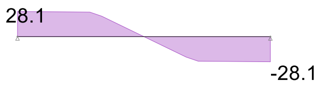
Results

Forces

Strength combinations - Moment envelope (kNm)



Strength combinations - Shear envelope (kN)



;

Partial factors - Section 6.1

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

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

Resistance of tensile members to fracture; γ_{M2} = **1.1**

Library item: Partial factors out

Member1 design

Section details

Section type; UC 203x203x46 (BS4-1)

Steel grade - EN 10025-2:2004; S355

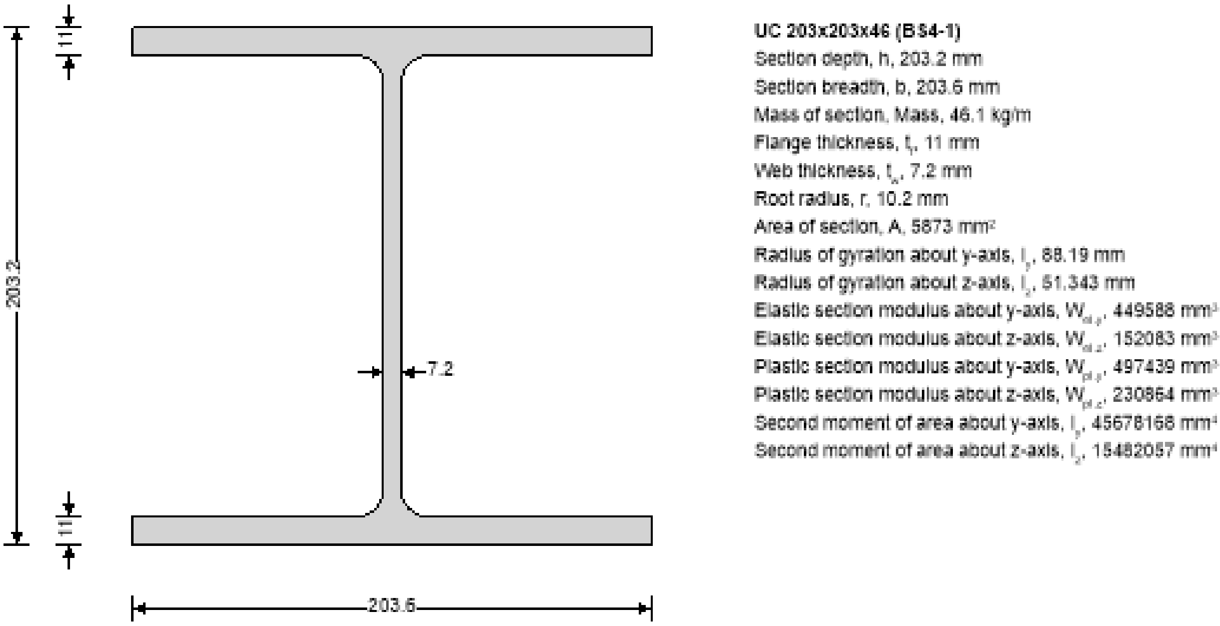
Nominal thickness of element; $t_{nom} = \max(t_f, t_w) =$ **11** mm

Nominal yield strength; f_y = **355** N/mm^2

Nominal ultimate tensile strength; f_u = **470** N/mm^2

Modulus of elasticity; E = **210000** N/mm^2

Library item: Section details out



Lateral restraint

Both flanges have lateral restraint at supports only

Classification of cross sections - Section 5.5

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

Library item: Class heading out

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

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

$c / t_w = 22.3 = 27.4 \times \varepsilon <= 72 \times \varepsilon$; Class 1

Library item: Int bend class out

Outstand flanges - Table 5.2 (sheet 2 of 3)

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

$c / t_f = 8 = 9.8 \times \varepsilon <= 10 \times \varepsilon$; Class 2

Library item: Out flange class out

Section is class 2

Check design at start of span

Check shear - Section 6.2.6

Height of web; $h_w = h - 2 \times t_f$ = **181.2** mm; η = **1.000**

$h_w / t_w = 25.2 = 30.9 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Library item: Shear slenderness out

Design shear force; $V_{y,Ed}$ = **28.1** kN

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

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

$V_{y,Ed} / V_{c,y,Rd}$ = **0.081**

PASS - Design shear resistance exceeds design shear force

Library item: Shear resistance out

Check design 1940 mm along span

Check bending moment - Section 6.2.5

Design bending moment; $M_{y,Ed}$ = **42.9** kNm

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

$M_{y,Ed} / M_{c,y,Rd}$ = **0.243**

PASS - Design bending resistance moment exceeds design bending moment

Library item: Bending resistance out

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6; k_c = **0.94**

$C_1 = 1 / k_c^2$ = **1.132**

Poissons ratio; ν = **0.3**

Shear modulus; $G = E / [2 \times (1 + \nu)]$ = **80769** N/mm^2

Unrestrained effective length; $L = 1.2 \times L_{m1\_s1\_seg1\_T} + 2 \times h$ = **5062** mm

Elastic critical buckling moment; $M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **217.3** kNm

Slenderness ratio for lateral torsional buckling; $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **0.901**

Limiting slenderness ratio; $\bar{\lambda}_{LT,0}$ = **0.4**

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

Library item: Bending LTB slenderness out

Check buckling resistance - Section 6.3.2.1

Buckling curve - Table 6.5; b

Imperfection factor - Table 6.3; α_{LT} = **0.34**

Correction factor for rolled sections; β = **0.75**

LTB reduction determination factor; $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **0.890**

LTB reduction factor - eq 6.57; $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.759**

Modification factor; $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.971**

Modified LTB reduction factor - eq 6.58; $\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.782**

Design buckling resistance moment - eq 6.55; $M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1}$ = **138.1** kNm

$M_{y,Ed} / M_{b,y,Rd}$ = **0.311**

PASS - Design buckling resistance moment exceeds design bending moment

Library item: Bending buckling out

Check design 1940 mm along span

Check y-y axis deflection - Section 7.2.1

Maximum deflection; δ_y = **6.9** mm

Allowable deflection; $\delta_{y,Allowable} = L_{m1\_s1} / 360$ = **10.8** mm

$\delta_y / \delta_{y,Allowable}$ = **0.639**

PASS - Allowable deflection exceeds design deflection

Library item: Deflection out

21. Design steel column C3.

steel column design

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

Tedds calculation version 1.1.04

Library item: Calc title

Partial factors - Section 6.1

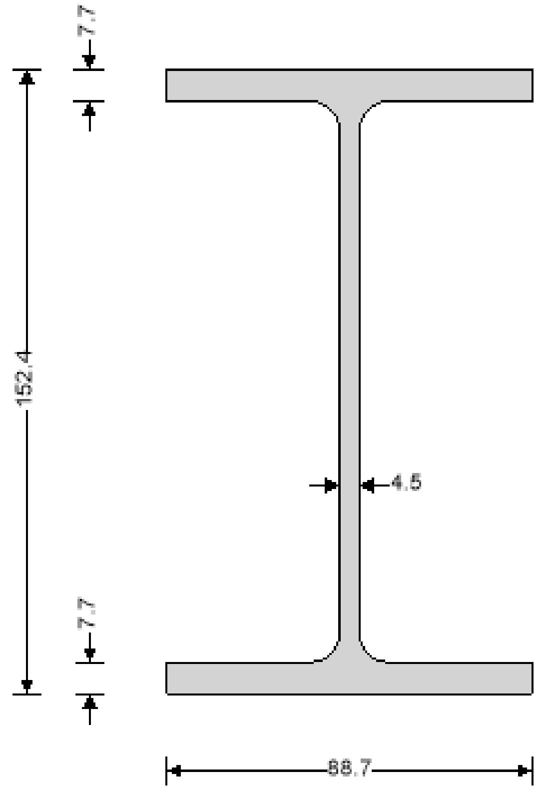
Resistance of cross-sections; γ_{M0} = **1**

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

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

Library item: Partial factors

Library item: Partial factors



UB 152x89x16 (BS4-1)

Section depth, h, 152.4 mm

Section breadth, b, 88.7 mm

Mass of section, Mass, 16 kg/m

Flange thickness, t_f, 7.7 mm

Web thickness, t_w, 4.5 mm

Root radius, r, 7.6 mm

Area of section, A, 2032 mm^2

Radius of gyration about y-axis, i_y, 64.074 mm

Radius of gyration about z-axis, i_z, 21.016 mm

Elastic section modulus about y-axis, $W_{el.y}$, 109483 mm^3

Elastic section modulus about z-axis, $W_{el.z}$, 20237 mm^3

Plastic section modulus about y-axis, $W_{pl.y}$, 123256 mm^3

Plastic section modulus about z-axis, $W_{pl.z}$, 31180 mm^3

Second moment of area about y-axis, I_y, 8342621 mm^4

Second moment of area about z-axis, I_z, 897506 mm^4

Column details

Column section **UB 152x89x16**

Steel grade **S355**

Column details

Column section **UB 152x89x16**

Steel grade **S355**

$f_y = f_{y\_basic}$ = **355.0** N/mm^2; _tmp.fy = f_y =**355.0** N/mm^2; _tmp.fyReduced = f_y =**355.0** N/mm^2

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

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

E = E_{SEC3} = **210** kN/mm^2; ν = 0.3 = **0.3**

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

Poisson's ratio ν = **0.3**

Shear modulus G = E / [2 × (1 + ν)] = **80.8** kN/mm^2

Column geometry

System length for buckling - Major axis L_y = **2550** mm

System length for buckling - Minor axis L_z = **2550** mm

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

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

Column loading

Axial load N_{Ed} = **56** kN (Compression)

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

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

Minor axis moment at end 1 - Bottom $M_{z,Ed1}$ = **0.0** kNm

Minor axis moment at end 2 - Top $M_{z,Ed2}$ = **0.0** kNm

Major axis shear force $V_{y,Ed}$ = **0** kN

Minor axis shear force $V_{z,Ed}$ = **0** kN

Library item - Column details

Buckling length for flexural buckling - Major axis

End restraint factor; K_y = **1.000**

Buckling length; $L_{cr\_y} = L_y \times K_y$ = **2550** mm

Library item - Buckling length factor y

Buckling length for flexural buckling - Minor axis

End restraint factor; K_z = **1.000**

Buckling length; $L_{cr\_z} = L_z \times K_z$ = **2550** mm

Library item - Buckling length factor z

Web section classification (Table 5.2)

f_y = _tmp.fy = **355** N/mm^2

Coefficient depending on f_y; $\varepsilon = \sqrt{(235 \text{ N/mm}^2 / f_y)}$ = **0.814**

Depth between fillets; $c_w = h - 2 \times (t_f + r)$ = **121.8** mm

Ratio of c/t; $ratio_w = c_w / t_w$ = **27.07**

Length of web taken by axial load; $l_w = \min(N_{Ed} / (f_y \times t_w), c_w)$ =**35.1** mm

For class 1 & 2 proportion in compression; $\alpha = (c_w/2 + l_w/2) / c_w$ = **0.644**

$Limit_{1w}$ = if(α>0.5,(396 × ε) / (13 × α - 1), 36 × ε / α) = **43.71**

Limit for class 1 web; $Limit_{1w} = (396 \times \varepsilon) / (13 \times \alpha - 1)$ = ;**43.71**

The web is class 1

Library item - Web class 1 I/PFC

Flange section classification (Table 5.2)

c_f = if(type=="I",(b - t_w) / 2 - r, b - t_w - r) = **34.5** mm

Outstanding length; $c_f = (b - t_w) / 2 - r$ = ;**34.5**; mm

Ratio of c/t; $ratio_f = c_f / t_f$ = **4.48**

Limit for class 1 flange; $Limit_{1f} = 9 \times \varepsilon$ = **7.32**

Limit for class 2 flange; $Limit_{2f} = 10 \times \varepsilon$ = **8.14**

Limit for class 3 flange; $Limit_{3f} = 14 \times \varepsilon$ = **11.39**

$Class_f$ = if($ratio_f$ <= $9 \times \varepsilon$, 1, if($ratio_f$ <= $10 \times \varepsilon$, 2, if($ratio_f$ <= $14 \times \varepsilon$, 3, 4))) = **1**

The flange is class 1

Library item - Flange class I/PFC

Overall section classification

Class = max($Class_w$, $Class_f$) = **1**

The section is class 1

Library item - Overall class

Resistance of cross section (cl. 6.2)

Library item - CS title

Compression (cl. 6.2.4)

Design force; N_{Ed} = **56** kN

f_y = _tmp.fy = **355** N/mm^2

Design resistance; $N_{c,Rd} = N_{pl,Rd} = A \times f_y / \gamma_{M0}$ = **721** kN

$N_{Ed} / N_{c,Rd}$ = **0.078**

PASS - The compression design resistance exceeds the design force

Library item - CS comp resistance

Buckling resistance (cl. 6.3)

f_y = max(_tmp.fyReduced, 0.001 N/mm^2) = **355** N/mm^2

Yield strength for buckling resistance; f_y = **355** N/mm^2

Library item - Buck resistance title

Flexural buckling - Major axis

Elastic critical buckling force; $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr\_y}^2$ = **2659** kN

Non-dimensional slenderness; $\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})}$ = **0.521**

Buckling curve (Table 6.2); **a**

α_y = Select(_scd.FBucklingCurveY,"a_0", 0.13, "a", 0.21, "b", 0.34, "c", 0.49, "d", 0.76) = **0.21**

Imperfection factor (Table 6.1); α_y = **0.21**

Parameter Φ;$\Phi_y = 0.5 \times [1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2]$ = **0.669**

Reduction factor; $\chi_y = \min(1.0, 1 / [\Phi_y + \sqrt{(\Phi_y^2 - \bar{\lambda}_y^2)}])$ = **0.918**

Design buckling resistance; $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1}$ = **662.0** kN

$N_{Ed} / N_{b,y,Rd}$ = **0.085**

PASS - The flexural buckling resistance exceeds the design axial load

Library item - Flexural buckling y

Flexural buckling - Minor axis

Elastic critical buckling force; $N_{cr,z} = \pi^2 \times E \times I_z / L_{cr\_z}^2$ = **286** kN

Non-dimensional slenderness; $\bar{\lambda}_z = \sqrt{(A \times f_y / N_{cr,z})}$ = **1.588**

Buckling curve (Table 6.2); **b**

α_z = Select(_scd.FBucklingCurveZ,"a_0", 0.13, "a", 0.21, "b", 0.34, "c", 0.49, "d", 0.76) = **0.34**

Imperfection factor (Table 6.1); α_z = **0.34**

Parameter Φ;$\Phi_z = 0.5 \times [1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2]$ = **1.997**

Reduction factor; $\chi_z = \min(1.0, 1 / [\Phi_z + \sqrt{(\Phi_z^2 - \bar{\lambda}_z^2)}])$ = **0.312**

Design buckling resistance; $N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1}$ = **224.9** kN

N_{Ed} / $N_{b,z,Rd}$ = **0.249**

PASS - The flexural buckling resistance exceeds the design axial load

Library item - Flexural buckling z

Torsional and torsional-flexural buckling (cl. 6.3.1.4)

Torsional buckling length factor; K_T = **1.00**

Effective buckling length; $L_{cr\_T} = K_T \times \max(L_y, L_z)$ = **2550** mm

e = if(type=="C",[(b - t_w/2)^2 × (h - t_f)^2 × t_f] / (4 × I_y), 0 mm) = **0.0** mm

; ;;

y_0 = if(type=="C",e - t_w/2 + getvar("y_s", 0 mm), 0 mm) = **0.0** mm

Distance from shear ctr to centroid along major axis; y_0 = ;**0.0**; mm

z_0 = 0 mm

Distance from shear ctr to centroid along minor axis; z_0 = **0.0** mm

$i_0 = \sqrt{(i_y^2 + i_z^2 + y_0^2 + z_0^2)}$ = **67.4** mm

$\beta_T = 1 - (y_0 / i_0)^2$ = **1.000**

Elastic critical torsional buckling force; $N_{cr,T} = 1 / i_0^2 \times (G \times I_t + \pi^2 \times E \times I_w / L_{cr\_T}^2)$ = **962** kN

Elastic critical torsional-flexural buckling force; $N_{cr,TF} = N_{cr,y}/(2\times\beta_T)\times[1+N_{cr,T}/N_{cr,y}-\sqrt{[(1-N_{cr,T}/N_{cr,y})^2 + 4\times(y_0/i_0)^2\times N_{cr,T}/N_{cr,y}]}]$

$N_{cr,TF}$ = **962** kN

Non-dimensional slenderness; $\bar{\lambda}_T = \sqrt{(A \times f_y / \min(N_{cr,T}, N_{cr,TF}))}$ = **0.866**

Buckling curve (Table 6.2); **b**

α_T = Select(_scd.TBucklingCurve,"a_0", 0.13, "a", 0.21, "b", 0.34, "c", 0.49, "d", 0.76) = **0.34**

Imperfection factor (Table 6.1); α_T = **0.34**

Parameter Φ;$\Phi_T = 0.5 \times [1 + \alpha_T \times (\bar{\lambda}_T - 0.2) + \bar{\lambda}_T^2]$ = **0.988**

Reduction factor; $\chi_T = \min(1.0, 1 / [\Phi_T + \sqrt{(\Phi_T^2 - \bar{\lambda}_T^2)}])$ = **0.683**

Design buckling resistance; $N_{b,T,Rd} = \chi_T \times A \times f_y / \gamma_{M1}$ = **492.6** kN

N_{Ed} / $N_{b,T,Rd}$ = **0.114**

PASS - The torsional/torsional-flexural buckling resistance exceeds the design axial load

Library item - Tor/tor-flex buckling

Minimum buckling resistance

Minimum buckling resistance; $N_{b,Rd} = \min(N_{b,y,Rd}, N_{b,z,Rd}, N_{b,T,Rd})$ = **224.9** kN

N_{Ed} / $N_{b,Rd}$ = **0.249**

PASS - The axial load buckling resistance exceeds the design axial load

Library item - Min buck resist I/PFC

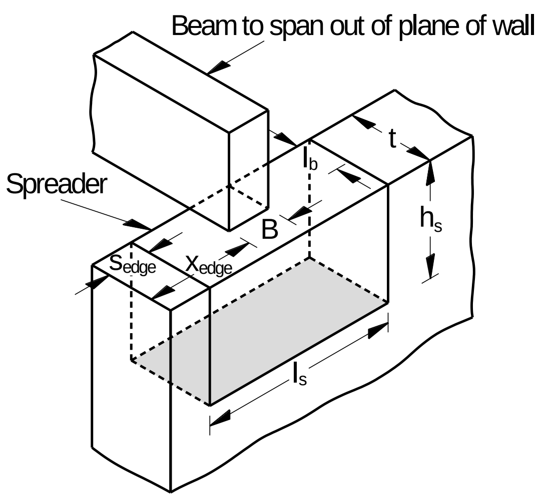
22. Design padstone P3 and P4.

MASONRY BEARING DESIGN TO BS5628-1:2005

TEDDS calculation version 1.0.06

Masonry details

Masonry type;	***Clay or calcium silicate bricks***
Compressive strength of unit;	***punit = 5.0 N/mm2***
Mortar designation;	***iii***
Category of masonry units;	***Category II***
Category of construction control ;	***Normal***
Partial safety factor for material strength;	***gm = 3.5***
Thickness of load bearing leaf;	***t = 110 mm***
Effective thickness of masonry wall;	***tef = 250 mm***
Height of masonry wall;	***h = 2550 mm***
Effective height of masonry wall;	***hef = 2550 mm***



Bearing details

Beam spanning out of plane of wall

Width of bearing;	***B = 200 mm***
Length of bearing;	***lb = 110 mm***
Edge distance;	***xedge = 200 mm***

Compressive strength from Table 2 BS5628:Part 1 - Clay or calcium silicate bricks

Mortar designation;	***Mortar = "iii"***
Brick compressive strength;	***punit = 5.0 N/mm2***
Characteristic compressive strength;	***fk = 2.50 N/mm2***

Loading details

Characteristic concentrated dead load;	***Gk = 21 kN***

Characteristic concentrated imposed load; ***Qk = 0 kN***

Design concentrated load; ***F = (Gk × 1.4) + (Qk × 1.6) = 29.4 kN***

Characteristic distributed dead load; ***gk = 0.0 kN/m***

Characteristic distributed imposed load; ***qk = 0.0 kN/m***

Design distributed load; ***f = (gk × 1.4) + (qk × 1.6) = 0.0 kN/m***

Masonry bearing type

Bearing type; ***Type 2***

Bearing safety factor; ***gbear = 1.50***

Check design bearing without a spreader

Design bearing stress; ***fca = F / (B × lb) + f / t = 1.336 N/mm2***

Allowable bearing stress; ***fcp = gbear × fk / gm = 1.071 N/mm2***

FAIL - Design bearing stress exceeds allowable bearing stress, use a spreader

Spreader details

Length of spreader; ***ls = 600 mm***

Depth of spreader; ***hs = 215 mm***

Edge distance; ***sedge = max(0 mm, xedge – (ls - B) / 2) = 0 mm***

Spreader bearing type

Bearing type; ***Type 3***

Bearing safety factor; ***gbear = 2.00***

Check design bearing with a spreader

Loading acts at midpoint of spreader

Design bearing stress; ***fca = F / (ls × t) + f / t = 0.445 N/mm2***

Allowable bearing stress; ***fcp = gbear × fk / gm = 1.429 N/mm2***

PASS - Allowable bearing stress exceeds design bearing stress

Check design bearing at 0.4 × h below the bearing level

Slenderness ratio; ***hef / tef = 10.20***

Eccentricity at top of wall; ***ex = 0.0 mm***

From BS5628:1 Table 7

Capacity reduction factor; ***b = 0.99***

Length of bearing distributed at 0.4 × h; ***ld = 1420 mm***

Maximum bearing stress; ***fca = F / (ld × t) + f / t = 0.188 N/mm2***

Allowable bearing stress; ***fcp = b × fk / gm = 0.707 N/mm2***

PASS - Allowable bearing stress at 0.4 ´ h below bearing level exceeds design bearing stress

23. Design Connection CON4

Project data

Project name	
Project number	
Author	
Description	
Date	3/16/2021
Design code	EN

Material

Steel	S 355

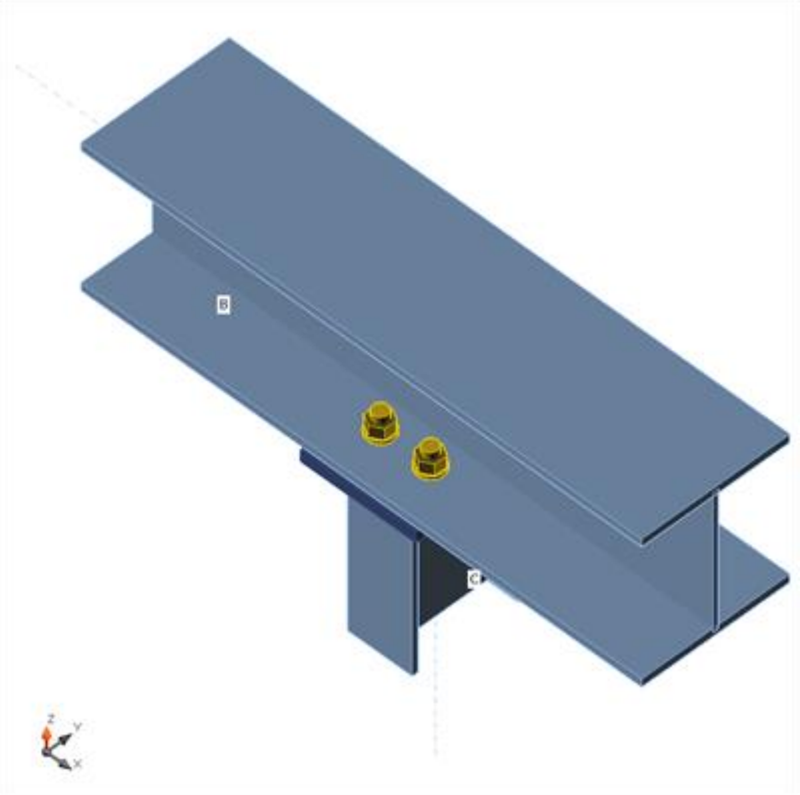
Project item connection

Design

Name	connection
Description	
Analysis	Stress, strain/ simplified loading

Beams and columns

Name	Cross-section	β – Direction [°]	γ - Pitch [°]	α - Rotation [°]	Offset ex [mm]	Offset ey [mm]	Offset ez [mm]	Forces in
C	1 - UB 152 x 89 x 16	0.0	90.0	90.0	100	0	0	Node
B	2 - UC 203 x 203 x 46	0.0	0.0	0.0	0	0	0	Node



Cross-sections

Name	Material
1 - UB 152 x 89 x 16	S 355
2 - UC 203 x 203 x 46	S 355

Bolts

Name	Bolt assembly	Diameter [mm]	fu [MPa]	Gross area [mm^2]
M20 8.8	M20 8.8	20	800.0	314

Load effects (equilibrium not required)

Name	Member	N [kN]	Vy [kN]	Vz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
LE1	B	0.0	0.0	-56.0	0.0	0.0	0.0
	B	0.0	0.0	0.0	0.0	0.0	0.0

Check

Summary

Name	Value	Status
Analysis	100.0%	OK

Plates	0.0 < 5%	OK
Bolts	1.1 < 100%	OK
Welds	24.5 < 100%	OK
Buckling	33.00	

Plates

Name	Thickness [mm]	Loads	σ_{Ed} [MPa]	ε_{Pl} [%]	Status
C-bfl 1	7.7	LE1	48.1	0.0	OK
C-tfl 1	7.7	LE1	48.0	0.0	OK
C-w 1	4.5	LE1	124.7	0.0	OK
B-bfl 1	11.0	LE1	72.8	0.0	OK
B-tfl 1	11.0	LE1	73.2	0.0	OK
B-w 1	7.2	LE1	85.0	0.0	OK
EP1	15.0	LE1	71.3	0.0	OK

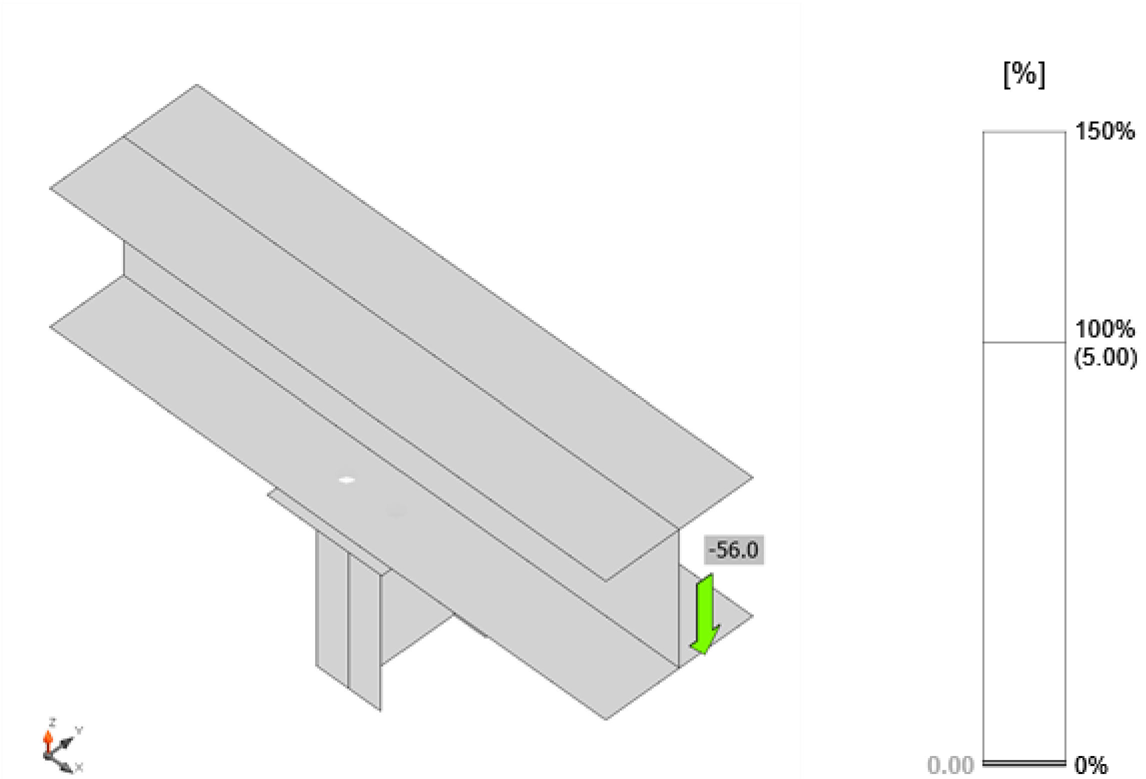
Design data

Material	f_y [MPa]	ε_{lim} [%]
S 355	355.0	5.0

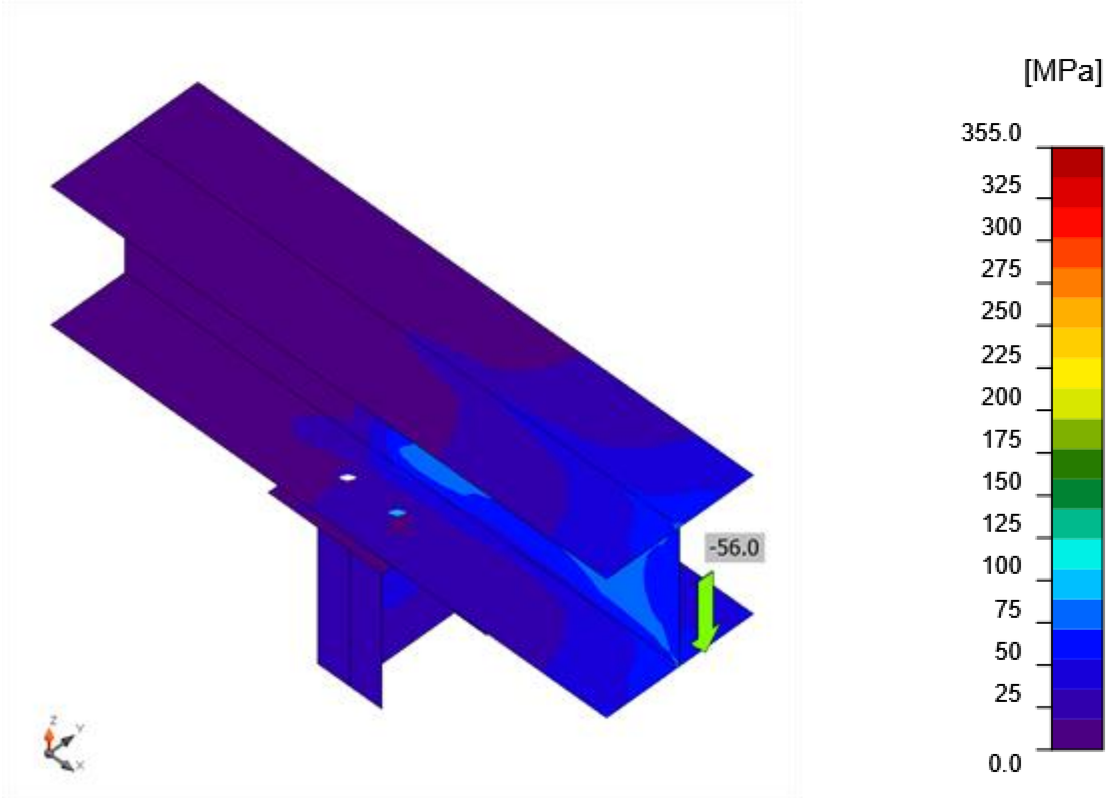
Symbol explanation

ε_{Pl}	Strain
σ_{Ed}	Eq. stress
f_y	Yield strength
ε_{lim}	Limit of plastic strain

Overall check, LE1 **Error! Bookmark not defined.**

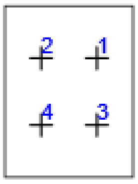


Strain check, LE1 **Error! Bookmark not defined.**



Equivalent stress, LE1 **Error! Bookmark not defined.**

Bolts

	Name	Loads	$F_{t,Ed}$ [kN]	V [kN]	Ut_t [%]	$F_{b,Rd}$ [kN]	Ut_s [%]	Ut_{ts} [%]	Status
	B1	LE1	0.1	1.0	0.1	215.6	1.1	1.1	OK
	B2	LE1	0.1	1.0	0.0	215.6	1.1	1.1	OK
	B3	LE1	0.1	1.0	0.1	215.6	1.1	1.1	OK
	B4	LE1	0.1	1.0	0.1	215.6	1.1	1.1	OK

Design data

Name	$F_{t,Rd}$ [kN]	$B_{p,Rd}$ [kN]	$F_{v,Rd}$ [kN]
M20 8.8 - 1	141.1	256.0	94.1

Symbol explanation

$F_{t,Rd}$	Bolt tension resistance EN 1993-1-8 tab. 3.4
$F_{t,Ed}$	Tension force
$B_{p,Rd}$	Punching shear resistance
V	Resultant of shear forces Vy, Vz in bolt
$F_{v,Rd}$	Bolt shear resistance EN_1993-1-8 table 3.4

$F_{b,Rd}$ Plate bearing resistance EN 1993-1-8 tab. 3.4
Ut_t Utilization in tension
Ut_s Utilization in shear
Ut_{ts} Utilization in tension and shear EN 1993-1-8 table 3.4

Welds (Plastic redistribution)

Item	Edge	Throat th. [mm]	Length [mm]	Loads	σw,Ed [MPa]	εPl [%]	σ⊥ [MPa]	τ∥ [MPa]	τ⊥ [MPa]	Ut [%]	Utc [%]	Status
EP1	C-bfl 1	◢4.3◣	89	LE1	78.7	0.0	-30.1	34.3	-24.3	18.1	7.2	OK
		◢4.3◣	89	LE1	55.3	0.0	-14.7	23.1	20.3	12.7	7.8	OK
EP1	C-tfl 1	◢4.3◣	89	LE1	54.9	0.0	-14.7	-22.9	-20.3	12.6	7.6	OK
		◢4.3◣	89	LE1	78.2	0.0	-29.7	34.1	24.1	18.0	7.1	OK
EP1	C-w 1	◢4.3◣	145	LE1	106.9	0.0	-50.1	-21.4	-50.1	24.5	13.6	OK
		◢4.3◣	145	LE1	106.8	0.0	-50.1	21.4	50.1	24.5	13.6	OK

Design data

	βw [-]	σw,Rd [MPa]	0.9 σ [MPa]
S 355	0.90	435.6	352.8

Symbol explanation

ε_{Pl} Strain
$\sigma_{w,Ed}$ Equivalent stress
$\sigma_{w,Rd}$ Equivalent stress resistance
$\sigma_{\perp}$ Perpendicular stress
$\tau_{\parallel}$ Shear stress parallel to weld axis
$\tau_{\perp}$ Shear stress perpendicular to weld axis
0.9 σ Perpendicular stress resistance - 0.9*fu/γM2
β_w Corelation factor EN 1993-1-8 tab. 4.1
Ut Utilization
Utc Weld capacity utilization

Buckling

Loads	Shape	Factor [-]
LE1	1	33.00
	2	39.66
	3	47.51
	4	55.15
	5	62.31
	6	63.31

Code settings

Item	Value	Unit	Reference
γ_{M0}	1.00	-	EN 1993-1-1: 6.1
γ_{M1}	1.00	-	EN 1993-1-1: 6.1
γ_{M2}	1.25	-	EN 1993-1-1: 6.1
γ_{M3}	1.25	-	EN 1993-1-8: 2.2
γ_{C}	1.50	-	EN 1992-1-1: 2.4.2.4
γ_{Inst}	1.20	-	ETAG 001-C: 3.2.1
Joint coefficient βj	0.67	-	EN 1993-1-8: 6.2.5
Effective area - influence of mesh size	0.10	-	
Friction coefficient - concrete	0.25	-	EN 1993-1-8
Friction coefficient in slip-resistance	0.30	-	EN 1993-1-8 tab 3.7
Limit plastic strain	0.05	-	EN 1993-1-5
Weld stress evaluation	Plastic redistribution		
Detailing	No		
Distance between bolts [d]	2.20	-	EN 1993-1-8: tab 3.3
Distance between bolts and edge [d]	1.20	-	EN 1993-1-8: tab 3.3
Concrete breakout resistance	Yes		ETAG 001-C
Use calculated αb in bearing check.	Yes		EN 1993-1-8: tab 3.4
Cracked concrete	Yes		
Local deformation check	No		
Local deformation limit	0.03	-	CIDECT DG 1, 3 - 1.1
Geometrical nonlinearity (GMNA)	Yes		Large deformations for hollow sections

24. Designer's risk assessment

24.1 Health and safety

1. All work will be carried out in accordance with the relevant health and safety legislation including the Construction (Design and Management) Regulations 2007, the Health and Safety at Work Act 1974 and the Management of Health and Safety at Work Regulations 1999.
2. To comply with the above, detailed risk assessments and method statements will be developed by the main contractor for all site activities prior to commencing the works. This will include but not be limited to measures for protecting the public, local environment and surrounding areas.
3. As vehicles arrive, unload, and leave the site they will be supervised by banksmen. During loading/unloading vehicles, barriers will be erected across the pavement to prevent access by pedestrians, which will be supervised by banksmen. At all other times, the public/pedestrians will have right of way along the pathways that surround the site. The construction site entrance will be kept closed and closely monitored to prevent unauthorized access.

24.2 Noise control

1. All noisy work on site will be carried out in accordance with guidance provided by RBKC. It is intended that site hours will be Monday to Friday – 8am to 6pm, Saturday – 8am to 1pm, Sunday and public holidays – no working.
2. The site hoarding will assist in acting as a noise barrier. All machinery, tools and equipment will be well maintained and silenced where possible for items such as compressors, generators, and power tools. Stationary noise sources will be shielded with acoustic barriers were considered necessary.
3. In all cases, the best practicable means of minimising noise will be used, and the guidance given in British Standard BS 5228: Parts 1, and 2 (1984) and Part 4 (1986) entitled 'Noise control on construction and open sites' will be used where practical.
4. In the event that a complaint or concern is raised, an immediate review will be completed to remove the problem wherever possible and to establish what levels of noise have been emitted from the site.
5. These actions may include reducing the operating hours, re-siting the equipment, changing the method of working or providing temporary barriers.

24.3 Party Wall Matters

1. The Party Wall etc Act 1996 provides owners of buildings with certain rights and obligations to adjoining owners for work in relation to party walls or similar structures. Therefore, the requirements of the act will be adhered to which will require party wall agreements with all adjoining owners prior to the works commencing. This may include the Highways Department of the city council.

24.4 Strategy for waste Disposal and management

A waste management strategy for the project will be implemented to ensure that:

- The site is kept clean and tidy.
- The collection of waste is from a central point.
- There is segregation of waste on site.
- The removal of waste products from site is minimised by recycling and re-use of excess materials wherever possible.
- Waste is disposed of legally at licensed tips and/or designated sites.
- Disruption to traffic in the surrounding area is minimised.

Prior to the works commencing the removal of waste will be covered by a site waste management plan developed by the contractors in line with this CMS, which will follow the requirements of the Environment Agency under S.34 of the Environmental Protection Act 1990 and the guidelines contained in the statutory guidance, Waste Management, The Duty of Care, A Code of Practice, 1996. This will be developed further in strict accordance with this strategy once a main contractor has been appointed for the project.

25. Outline Method Statement for the Basement Construction

Please refer to the structural drawings and Report – REP-ST-20-052-01-R1

26. Appendix 01: Designer's risk assessment form

Designers Risk Assessment

15 Landor Road, London SW9 9RX
March, 2021

Rev	Date	Comment
A0	10-03-2021	-

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Great West House E4, Great West Road, TW8 9DF, London, United Kingdom

PROJECT RISK REGISTER	JOB No. [Comments]	PROJECT: Harwell		SHEET No. 1 of	REVISION 0
	STAGE: CONSTRUCTION	PREPARED BY: AJ	APPROVED BY: FM	DATE: 19/11/2020	

Hazard	Location	Who is at Risk	Consequence	Risk before Mitigation	Mitigation measures	Residual Risk info / Notes	Residual Risk
Substrate	Beam & Slab	Occupants of the building	The FRP system not working according to the design intent	H	Surface preparation to manufacturer or Structural Engineer's recommendations Substrate to be inspected before the application Surface must be free from any contamination		**L**
CONCRETING							
Operatives coming in contact with concrete	Beams and slabs	Site operative	Sensitisation of skin & development of dermatitis	H	Long sleeves and rubberised gloves to be worn by pump man. Ensure all operatives have been briefed on the COSHH assessment and follow the manufacturer's recommendation.		L
			Damage to eyes and skin from concrete splashes	H	Eye protection to be worn at all times. Ensure sufficient eye washout facilities are available and close to the working area. Ensure there are sufficient hygiene/washing facilities available nearby.		L
Cube Making	Beam& Slab		Error in cube making could result in strengthening being condemned unnecessarily	H	Ensure person undertaking test has been trained to carry out sampling and cube making.	Error may still occur / Process to be monitored	L

[Comments]

Risk Level: Risk = severity x likelihood

(H) HIGHUnacceptable likelihood that injury or damage will be severe – further management controls are essential

(M) MEDIUMInjury or damage is very unlikely to be severe, but minor injury could occur from time to time

(L) LOWThe risk of injury is well controlled and harm is likely to be minor if it occurs

OCCUPATIONAL HEALTH & WELFARE							
Use of hazardous materials	Beams & Slab surface preparation	Site operative	Exposure could lead to a health hazard.	H	Ensure COSHH assessments on site and relevant PPE is provided.	Incorrect disposal	M
Noise	Beams & Slab surface preparation	Site operative	Excessive noise in the workplace could cause hearing loss to employees	H	Hearing protection to be provided as per noise assessment for machine & usage to be monitored. Persons in or near (within 2m) to wear ear protection Ear protection zone marked		L
Vibration	Beams & Slab surface preparation	Site operative	Excessive exposure to vibration equipment causes Hand-Arm Vibration Syndrome	H	Low vibration and noise tools and equipment will be used where possible Equipment certificated Operators to take frequent short breaks Wear working gloves or anti vibration gloves		L
Flying debris	Beams & Slab surface preparation	Site operative	Injury due to flying objects	H	Before grinding make sure no one will be affected by flying objects Operative involved to wear appropriate PPE (googles additional to mandatory PPE)		L
Reinforcement							
Lifting of Reinforcement cage	Beams	Site operative	Cage dropped causing damage. Major injury or Fatality could occur.	H	Lift Plan to be in place before commencing any lift.		L
Dowels and rebar projecting from surface	Slabs and beam	Site operative	Injury due cuts, trip and fall	H	Plastic mushroom cap to the ends of the dowels/rebar		L

[Comments]

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Working at Height							
Work at Height	Beams	Site operative	Major injury or fatality	H	Provide temporary platform		L

[Comments]

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