

# 28 Burghley Road Lower ground floor back extension Internal alterations



# Structural Design Calculation Package

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# **Description of works**

28 Burghley Road is a four story mid-terrace property in the Kentish Town area of London, UK.

The existing building is built of traditional solid brickwork and timber stud walls and timber suspended floors. The original structure has been altered over the years with the addition of a loft space and of an outrigger extension from ground floor above.

The current structure comprises timber floor joists spanning onto load bearing walls from ground floor to the loft level. The span direction of the timber floors is predominantly front to back in all the levels. The loft conversion undertaken introduced steel elements to support the roof and dormers. The outrigger extension is supported on load bearing walls and steps outwards from first floor and above. This step is supported on a cantilever RC slab.

The proposed works comprise the extension of the lower ground floor area to the rear to accommodate more living space, the removal of the internal timber stud wall that supports the stairs wall and the alteration of the configuration of the external stairs to the front of the house. The new flat roof the rear extension will also be used as a terrace and therefore was designed to accommodate living space loads.

The new rear extension involves the removal of the original back wall of the house and the outrigger walls at lower ground floor level. To reinstate the vertical support and lateral stability previously provided by the masonry walls, two new steel box-frames are proposed to be installed in the alignments of the original back wall of the house and the new rear of the back extension. A series of new steel beams it is also proposed to be installed to support the outrigger walls at ground floor level and to frame the new flat roof floor. Where new steel beams frame onto existing brickwork walls, new concrete padstones were designed considering a reduced bearing capacity of the existing brickwork. The new extension roof will be constructed using timber joists. The vertical support of the stairway wall at ground floor level is provided by a new steel beam that will span between the new box-frame and a new steel post hidden within the thickness of the spine wall. A new mass concrete pad footing has been designed to account for the reaction on the base of the steel post. The re-configuration of the external stairs to the front involves the construction of new retaining walls to accommodate the change in retaining levels. These retaining walls are proposed to be propped by the new the ground bearing slab and the half landing of the stairs. They have therefore been designed as cantilevers or fixed-pinned walls. It is of note that these fall onto the Category 0 of the "Design manual for roads and bridges" and therefore an Approval in Principle (AIP) document is not required.

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# Loads

GENERAL LOADS CONSIDERED ON THE STRU	CTURAL DESIGN
Roof Loading - pitched roof	
Roof slope;	$\theta$ = <b>30.0</b> °
Dead load	
Tiles:	Roof <sub>D1</sub> = <b>0.45</b> kN/m <sup>2</sup>
Battens;	Roof <sub>D2</sub> = <b>0.05</b> kN/m <sup>2</sup>
Felt;	Roof <sub>D3</sub> = <b>0.05</b> kN/m <sup>2</sup>
Rafters;	Roof <sub>D4</sub> = <b>0.10</b> kN/m <sup>2</sup>
Dead load on slope;	$Roof_{DL\_sroof} = sum(Roof_{D1}, Roof_{D2}, Roof_{D3}, Roof_{D4}) = 0.65 \text{ kN/m}^2$
Ceiling joists;	Roof <sub>D5</sub> = <b>0.05</b> kN/m <sup>2</sup>
Insulation;	Roof <sub>D6</sub> = <b>0.05</b> kN/m <sup>2</sup>
Plasterboard and skim;	Roof <sub>D7</sub> = <b>0.14</b> kN/m <sup>2</sup>
Services;	Roof <sub>D8</sub> = <b>0.05</b> kN/m <sup>2</sup>
Dead load on plan;	$Roof_{DL_proof} = sum(Roof_{D5}, Roof_{D6}, Roof_{D7}, Roof_{D8}) = 0.29 \text{ kN/m}^2$
Total dead load on plan;	$Roof_{DL} = Roof_{DL\_sroof} / cos(\theta) + Roof_{DL\_proof} = 1.04 \text{ kN/m}^2$
Imposed load	
Sloped roof imposed load;	Roof <sub>IL</sub> = <b>0.60</b> kN/m <sup>2</sup> ; on plan
Flat roof imposed load;	Roof <sub>IL</sub> = <b>0.75</b> kN/m <sup>2</sup> ; on plan
Timber floor Loading	
Dead load	
Boards;	$Floor_{grnd_D1} = 0.15 \text{ kN/m}^2$
Joists;	$Floor_{grnd_D2} = 0.15 \text{ kN/m}^2$
Ceiling;	Floor <sub>grnd_D3</sub> = <b>0.20</b> kN/m <sup>2</sup>
Total dead load;	$Floor_{grnd_DL} = sum(Floor_{load}) = 0.50 \text{ kN/m}^2$
Imposed load	
Imposed load;	Floor <sub>grnd_11</sub> = <b>1.50</b> kN/m <sup>2</sup>
Partitions;	$Floor_{grnd\_l2} = 0.00 \text{ kN/m}^2$
Total imposed load;	$Floor_{grnd\_1L} = sum(Floor_{grnd\_11}, Floor_{grnd\_12}) = 1.50 \text{ kN/m}^2$
Wall Loading	
Timber wall - Dead load	
Timber;	IW <sub>D1</sub> = <b>0.20</b> kN/m <sup>2</sup>
Plaster (2 sides, lath and plaster);	IW <sub>D2</sub> = <b>0.30</b> kN/m <sup>2</sup>
Total dead load;	$IW_{DL} = sum(IW_{D1}, IW_{D2}) = 0.50 \text{ kN/m}^2$
Solid Brickwork wall - Dead load	
t – Thickness of Solid Brick;	$IW_{D4} = 18kN/m^3 x t = kN/m^2$
Plaster (2 sides, lath and plaster);	IW <sub>D2</sub> = <b>0.50</b> kN/m <sup>2</sup>
Total dead load;	$IW_{DL} = sum(IW_{D4}, IW_{D5}, IW_{D2}) = IW_{D4} + 0.5 \text{ kN/m}^2$
Dense Blockwork wall - Dead load	
t <sub>b</sub> – Thickness of Solid Block;	$IW_{D4} = 20kN/m^3 x t_b = 2 kN/m^2$
Plaster (2 sides, lath and plaster);	IW <sub>D2</sub> = <b>0.50</b> kN/m <sup>2</sup>
Total dead load;	$IW_{DL} = sum(IW_{D4}, IW_{D5}, IW_{D2}) = IW_{D4} + 0.5 \text{ kN/m}^2$

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# Steelwork

#### BOX-FRAME UNDER ORIGINAL BACK WALL

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex
Tedds calculation version 4.0.02

Analysis

Tedds calculation version 1.0.13

#### Geometry



#### Nodes

Node	Co-ord	linates	Freedom			Coordinate system		Spring		
	Х	Z	X	Z	Rot.	Name	Angle	Х	Z	Rot.
	(m)	(m)					(°)	(kN/m)	(kN/m)	kNm/°
1	0	0	Fixed	Fixed	Free		0	0	0	0
2	0	3	Free	Free	Free		0	0	0	0
3	5.6	3	Free	Free	Free		0	0	0	0
4	5.6	0	Free	Fixed	Free		0	0	0	0

#### Materials

Name	Density	Youngs Modulus	Shear Modulus	Thermal Coefficient
	(kg/m³)	kN/mm²	kN/mm²	°C <sup>-1</sup>
Steel (EC3)	7850	210	80.8	0.000012

#### Sections

Name	Area	Moment of inertia		Shea	r area
		Major Minor		Ay	Az
	(cm²)	(cm4)	(cm4)	(cm²)	(cm²)
UC 254x254x73	93	11407	3908	65	22
UC 254x254x89	113	14268	4857	80	27

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Elements

Element	Length	No	des	Section	Material	Releases		Rotated	
	(m)	Start	End			Start moment	End moment	Axial	
1	3	1	2	UC 254x254x73	Steel (EC3)	Fixed	Fixed	Fixed	
2	5.6	2	3	UC 254x254x89	Steel (EC3)	Fixed	Fixed	Fixed	
3	3	3	4	UC 254x254x73	Steel (EC3)	Fixed	Fixed	Fixed	
4	5.6	1	4	UC 254x254x73	Steel (EC3)	Fixed	Fixed	Fixed	

#### Members

Name	Elements				
	Start	End			
Member1	2	2			
Member2	4	4			

Loading

















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#### Load cases

Name	Enabled	Self weight factor	Patternable
Self Weight	yes	1	no
Solid 330mm brick wall above - 18kN/m3 x 0.33m x 9m	yes	0	no
Floor dead load - 3 x 0.8kN/m2 x 2.15m	yes	0	no
Floor live load - 3 x 1.5kN/m2 x 2.15m	yes	0	no
Terrace roof dead load - 1kN/m2 x 2m	yes	0	no
Terrace roof live load - 1.5kN/m2 x 2m	yes	0	no
Roof dead load - 1kN/m2 x 2.5m	yes	0	no
Roof live load - 0.6kN/m2 x 2.5m	yes	0	no
Roof vertical wind load - 0.6kPa x sen(30) x 2.5m	yes	0	no
SB1 Dead Reaction	yes	0	no
SB1 Live Reaction	yes	0	no
SB3 Dead Reaction	yes	0	no
SB3 Live Reaction	yes	0	no
Wind	yes	0	no
Pre-Deflection	yes	0	no
	yes	0	no

#### Load combinations

Load combination	Туре	Enabled	Patterned
1.35G + 1.5Q + 1.5RQ	Strength	yes	no
1.0G + 1.0Q + 1.0RQ	Service	yes	no
1.0G + 1.0ψ	Quasi	yes	no
1.35G + 1.5Q + 1.5ψ	Strength	yes	no
1.0G + 1.0Q + 0.5S	Service	yes	no
1.35G + 1.5ψ	Strength	yes	no
1.35G + 1.5Q + 1.5ψ	Strength	yes	no
1.0G + 1.0Q + 0.5S + 0.5W	Service	yes	no
1.35G + 1.5ψ	Strength	yes	no
1.0G + 1.0ψ	Service	yes	no
1.35G + 1.5ψ	Strength	yes	no
1.0G + 1.0ψ	Service	yes	no
1.0G + 1.5W	Strength	yes	no
1.0G + 1.0W	Service	yes	no
1.35G + 1.5ψ	Strength	yes	no
1.35ξG + 1.5Q + 1.5RQ	Strength	yes	no
DEAD	Strength	yes	no
LIVE	Strength	yes	no

Load combination: 1.35G + 1.5Q + 1.5RQ (Strength)

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Load case	Factor
Self Weight	1.35
Solid 330mm brick wall above - 18kN/m3 x 0.33m x 9m	1.35
Floor dead load - 3 x 0.8kN/m2 x 2.15m	1.35
Floor live load - 3 x 1.5kN/m2 x 2.15m	1.5
Terrace roof dead load - 1kN/m2 x 2m	1.35
Terrace roof live load - 1.5kN/m2 x 2m	1.5
Roof dead load - 1kN/m2 x 2.5m	1.35
Roof live load - 0.6kN/m2 x 2.5m	1.5
Roof vertical wind load - 0.6kPa x sen(30) x 2.5m	0.75
SB1 Dead Reaction	1.35
SB1 Live Reaction	1.5
SB3 Dead Reaction	1.35
SB3 Live Reaction	1.5
Wind	0.75

# Load combination: 1.0G + 1.0Q + 1.0RQ (Service)

Load case	Factor
Self Weight	1
Solid 330mm brick wall above - 18kN/m3 x 0.33m x 9m	1
Floor dead load - 3 x 0.8kN/m2 x 2.15m	1
Floor live load - 3 x 1.5kN/m2 x 2.15m	1
Terrace roof dead load - 1kN/m2 x 2m	1
Terrace roof live load - 1.5kN/m2 x 2m	1
Roof dead load - 1kN/m2 x 2.5m	1
Roof live load - 0.6kN/m2 x 2.5m	1
Roof vertical wind load - 0.6kPa x sen(30) x 2.5m	0.5
SB1 Dead Reaction	1
SB1 Live Reaction	1
SB3 Dead Reaction	1
SB3 Live Reaction	1
Pre-Deflection	1

Load combination: 1.0G + 1.0y2Q (Quasi)

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Load case	Factor
Self Weight	1
Solid 330mm brick wall above - 18kN/m3 x 0.33m x 9m	1
Floor dead load - 3 x 0.8kN/m2 x 2.15m	1
Floor live load - 3 x 1.5kN/m2 x 2.15m	0.3
Terrace roof dead load - 1kN/m2 x 2m	1
Terrace roof live load - 1.5kN/m2 x 2m	0.3
Roof dead load - 1kN/m2 x 2.5m	1
SB1 Dead Reaction	1
SB1 Live Reaction	0.3
SB3 Dead Reaction	1
SB3 Live Reaction	0.3
Pre-Deflection	1

# Load combination: 1.35G + 1.5Q + 1.5 $\psi_0$ S (Strength)

Load case	Factor
Self Weight	1.35
Solid 330mm brick wall above - 18kN/m3 x 0.33m x 9m	1.35
Floor dead load - 3 x 0.8kN/m2 x 2.15m	1.35
Floor live load - 3 x 1.5kN/m2 x 2.15m	1.5
Terrace roof dead load - 1kN/m2 x 2m	1.35
Terrace roof live load - 1.5kN/m2 x 2m	1.5
Roof dead load - 1kN/m2 x 2.5m	1.35
SB1 Dead Reaction	1.35
SB1 Live Reaction	1.5
SB3 Dead Reaction	1.35
SB3 Live Reaction	1.5

Load combination: 1.0G + 1.0Q + 0.5S (Service)

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Load case	Factor
Self Weight	1
Solid 330mm brick wall above - 18kN/m3 x 0.33m x 9m	1
Floor dead load - 3 x 0.8kN/m2 x 2.15m	1
Floor live load - 3 x 1.5kN/m2 x 2.15m	1
Terrace roof dead load - 1kN/m2 x 2m	1
Terrace roof live load - 1.5kN/m2 x 2m	1
Roof dead load - 1kN/m2 x 2.5m	1
SB1 Dead Reaction	1
SB1 Live Reaction	1
SB3 Dead Reaction	1
SB3 Live Reaction	1
Pre-Deflection	1

# Load combination: $1.35G + 1.5\psi_0Q + 1.5S$ (Strength)

Load case	Factor
Self Weight	1.35
Solid 330mm brick wall above - 18kN/m3 x 0.33m x 9m	1.35
Floor dead load - 3 x 0.8kN/m2 x 2.15m	1.35
Floor live load - 3 x 1.5kN/m2 x 2.15m	1.05
Terrace roof dead load - 1kN/m2 x 2m	1.35
Terrace roof live load - 1.5kN/m2 x 2m	1.05
Roof dead load - 1kN/m2 x 2.5m	1.35
SB1 Dead Reaction	1.35
SB1 Live Reaction	1.05
SB3 Dead Reaction	1.35
SB3 Live Reaction	1.05

Load combination:  $1.35G + 1.5Q + 1.5\psi_0S + 1.5\psi_0W$  (Strength)

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Load case	Factor
Self Weight	1.35
Solid 330mm brick wall above - 18kN/m3 x 0.33m x 9m	1.35
Floor dead load - 3 x 0.8kN/m2 x 2.15m	1.35
Floor live load - 3 x 1.5kN/m2 x 2.15m	1.5
Terrace roof dead load - 1kN/m2 x 2m	1.35
Terrace roof live load - 1.5kN/m2 x 2m	1.5
Roof dead load - 1kN/m2 x 2.5m	1.35
SB1 Dead Reaction	1.35
SB1 Live Reaction	1.5
SB3 Dead Reaction	1.35
SB3 Live Reaction	1.5
Wind	0.75

### Load combination: 1.0G + 1.0Q + 0.5S + 0.5W (Service)

Load case	Factor
Self Weight	1
Solid 330mm brick wall above - 18kN/m3 x 0.33m x 9m	1
Floor dead load - 3 x 0.8kN/m2 x 2.15m	1
Floor live load - 3 x 1.5kN/m2 x 2.15m	1
Terrace roof dead load - 1kN/m2 x 2m	1
Terrace roof live load - 1.5kN/m2 x 2m	1
Roof dead load - 1kN/m2 x 2.5m	1
SB1 Dead Reaction	1
SB1 Live Reaction	1
SB3 Dead Reaction	1
SB3 Live Reaction	1
Wind	0.5
Pre-Deflection	1

Load combination:  $1.35G + 1.5\psi_0Q + 1.5S + 1.5\psi_0W$  (Strength)

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Load case	Factor
Self Weight	1.35
Solid 330mm brick wall above - 18kN/m3 x 0.33m x 9m	1.35
Floor dead load - 3 x 0.8kN/m2 x 2.15m	1.35
Floor live load - 3 x 1.5kN/m2 x 2.15m	1.05
Terrace roof dead load - 1kN/m2 x 2m	1.35
Terrace roof live load - 1.5kN/m2 x 2m	1.05
Roof dead load - 1kN/m2 x 2.5m	1.35
SB1 Dead Reaction	1.35
SB1 Live Reaction	1.05
SB3 Dead Reaction	1.35
SB3 Live Reaction	1.05
Wind	0.75

#### Load combination: $1.0G + 1.0\psi_0Q + 1.0S + 0.5W$ (Service)

Load case	Factor
Self Weight	1
Solid 330mm brick wall above - 18kN/m3 x 0.33m x 9m	1
Floor dead load - 3 x 0.8kN/m2 x 2.15m	1
Floor live load - 3 x 1.5kN/m2 x 2.15m	0.7
Terrace roof dead load - 1kN/m2 x 2m	1
Terrace roof live load - 1.5kN/m2 x 2m	0.7
Roof dead load - 1kN/m2 x 2.5m	1
SB1 Dead Reaction	1
SB1 Live Reaction	0.7
SB3 Dead Reaction	1
SB3 Live Reaction	0.7
Wind	0.5
Pre-Deflection	1

Load combination:  $1.35G + 1.5\psi_0Q + 1.5\psi_0S + 1.5W$  (Strength)

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Load case	Factor
Self Weight	1.35
Solid 330mm brick wall above - 18kN/m3 x 0.33m x 9m	1.35
Floor dead load - 3 x 0.8kN/m2 x 2.15m	1.35
Floor live load - 3 x 1.5kN/m2 x 2.15m	1.05
Terrace roof dead load - 1kN/m2 x 2m	1.35
Terrace roof live load - 1.5kN/m2 x 2m	1.05
Roof dead load - 1kN/m2 x 2.5m	1.35
SB1 Dead Reaction	1.35
SB1 Live Reaction	1.05
SB3 Dead Reaction	1.35
SB3 Live Reaction	1.05
Wind	1.5

#### Load combination: $1.0G + 1.0\psi_0Q + 0.5S + 1.0W$ (Service)

Load case	Factor
Self Weight	1
Solid 330mm brick wall above - 18kN/m3 x 0.33m x 9m	1
Floor dead load - 3 x 0.8kN/m2 x 2.15m	1
Floor live load - 3 x 1.5kN/m2 x 2.15m	0.7
Terrace roof dead load - 1kN/m2 x 2m	1
Terrace roof live load - 1.5kN/m2 x 2m	0.7
Roof dead load - 1kN/m2 x 2.5m	1
SB1 Dead Reaction	1
SB1 Live Reaction	0.7
SB3 Dead Reaction	1
SB3 Live Reaction	0.7
Wind	1
Pre-Deflection	1

Load combination: 1.0G + 1.5W (Strength)

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Load case	Factor
Self Weight	1
Solid 330mm brick wall above - 18kN/m3 x 0.33m x 9m	1
Floor dead load - 3 x 0.8kN/m2 x 2.15m	1
Terrace roof dead load - 1kN/m2 x 2m	1
Roof dead load - 1kN/m2 x 2.5m	1
SB1 Dead Reaction	1
SB3 Dead Reaction	1
Wind	1.5

# Load combination: 1.0G + 1.0W (Service)

Load case	Factor
Self Weight	1
Solid 330mm brick wall above - 18kN/m3 x 0.33m x 9m	1
Floor dead load - 3 x 0.8kN/m2 x 2.15m	1
Terrace roof dead load - 1kN/m2 x 2m	1
Roof dead load - 1kN/m2 x 2.5m	1
SB1 Dead Reaction	1
SB3 Dead Reaction	1
Wind	1

#### Load combination: 1.35G + 1.5\v0Q + 1.5\v0RQ (Strength)

Load case	Factor
Self Weight	1.35
Solid 330mm brick wall above - 18kN/m3 x 0.33m x 9m	1.35
Floor dead load - 3 x 0.8kN/m2 x 2.15m	1.35
Floor live load - 3 x 1.5kN/m2 x 2.15m	1.05
Terrace roof dead load - 1kN/m2 x 2m	1.35
Terrace roof live load - 1.5kN/m2 x 2m	1.05
Roof dead load - 1kN/m2 x 2.5m	1.35
Roof live load - 0.6kN/m2 x 2.5m	1.05
Roof vertical wind load - 0.6kPa x sen(30) x 2.5m	0.5
SB1 Dead Reaction	1.35
SB1 Live Reaction	1.05
SB3 Dead Reaction	1.35
SB3 Live Reaction	1.05

Load combination: 1.35&G + 1.5Q + 1.5RQ (Strength)

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Load case	Factor
Self Weight	1.249
Solid 330mm brick wall above - 18kN/m3 x 0.33m x 9m	1.249
Floor dead load - 3 x 0.8kN/m2 x 2.15m	1.249
Floor live load - 3 x 1.5kN/m2 x 2.15m	1.5
Terrace roof dead load - 1kN/m2 x 2m	1.249
Terrace roof live load - 1.5kN/m2 x 2m	1.5
Roof dead load - 1kN/m2 x 2.5m	1.249
Roof live load - 0.6kN/m2 x 2.5m	1.5
Roof vertical wind load - 0.6kPa x sen(30) x 2.5m	0.5
SB1 Dead Reaction	1.249
SB1 Live Reaction	1.5
SB3 Dead Reaction	1.249
SB3 Live Reaction	1.5
Wind	0.75

# Load combination: DEAD (Strength)

Load case	Factor
Self Weight	1
Solid 330mm brick wall above - 18kN/m3 x 0.33m x 9m	1
Floor dead load - 3 x 0.8kN/m2 x 2.15m	1
Terrace roof dead load - 1kN/m2 x 2m	1
Roof dead load - 1kN/m2 x 2.5m	1
SB1 Dead Reaction	1
SB3 Dead Reaction	1

# Load combination: LIVE (Strength)

Load case	Factor
Floor live load - 3 x 1.5kN/m2 x 2.15m	1
Terrace roof live load - 1.5kN/m2 x 2m	1
Roof live load - 0.6kN/m2 x 2.5m	1
Roof vertical wind load - 0.6kPa x sen(30) x 2.5m	0.5
SB1 Live Reaction	1
SB3 Live Reaction	1

Element point loads

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Element	Load case	Position		Load	Orientation
		Туре	Type Start		
				(kN)	
2	SB1 Dead Reaction	Ratio	0.37	12.2	GlobalZ
2	SB1 Live Reaction	Ratio	0.37	2.9	GlobalZ
2	SB3 Dead Reaction	Ratio	0.37	41.6	GlobalZ
2	SB3 Live Reaction	Ratio	0.37	4.2	GlobalZ
1	Wind	Ratio	1	24.3	GlobalX
2	Pre-Deflection	Ratio	0.5	-100	GlobalZ
4	Pre-Deflection	Ratio	0.5	100	GlobalZ

#### **Element UDL loads**

Element	Load case	Position			Load	Orientation
		Туре	Start	End		
					(kN/m)	
2	Solid 330mm brick wall above - 18kN/m3 x 0.33m x 9m	Ratio	0	1	45	GlobalZ
2	Floor dead load - 3 x 0.8kN/m2 x 2.15m	Ratio	0	1	5.2	GlobalZ
2	Floor live load - 3 x 1.5kN/m2 x 2.15m	Ratio	0	1	7.8	GlobalZ
2	Terrace roof dead load - 1kN/m2 x 2m	Ratio	0.37	1	2	GlobalZ
2	Terrace roof live load - 1.5kN/m2 x 2m	Ratio	0.37	1	2.4	GlobalZ
2	Roof dead load - 1kN/m2 x 2.5m	Ratio	0	1	2.5	GlobalZ
2	Roof live load - 0.6kN/m2 x 2.5m	Ratio	0	1	1.5	GlobalZ
2	Roof vertical wind load - 0.6kPa x sen(30) x 2.5m	Ratio	0	1	0.75	GlobalZ
1	Wind	Ratio	0	1	2.5	GlobalX

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**Results** 

Forces





#### Partial factors - Section 6.1

Resistance of cross-sections;	γмо = <b>1</b>
Resistance of members to instability;	$\gamma_{M1} = 1$
Resistance of tensile members to fracture;	$\gamma_{M2} = 1.1$

#### Member1 - Span 1

Section details

Section type; Steel grade - EN 10025-2:2004; Nominal thickness of element; Nominal yield strength; Nominal ultimate tensile strength; Modulus of elasticity; UC 254x254x89 (BS4-1) S275 t<sub>nom</sub> = max(t<sub>f</sub>, t<sub>w</sub>) = **17.3** mm

f<sub>y</sub> = **265** N/mm<sup>2</sup>

f<sub>u</sub> = **410** N/mm<sup>2</sup>

E = **210000** N/mm<sup>2</sup>

17.3 ¥ UC 254x254x89 (BS4-1) Section depth, h, 260.3 mm 1 Section breadth, b, 256.3 mm Mass of section, Mass, 88.9 kg/m Flange thickness, t<sub>p</sub> 17.3 mm Web thickness,  $\rm t_{\rm w}^{},\,10.3~\rm mm$ Root radius, r, 12.7 mm Area of section, A, 11331 mm<sup>2</sup> Radius of gyration about y-axis,  $\mathrm{i_y},\,112~\mathrm{mm}$ Radius of gyration about z-axis,  $i_z$ , 65 mm -10.3 260. Elastic section modulus about y-axis,  $W_{el,y}$ , 1096254 mm<sup>3</sup> Elastic section modulus about z-axis, W<sub>el.z</sub>, 379046 mm<sup>3</sup> Plastic section modulus about y-axis,  $W_{\mu,\nu}^{(a)}$  1223863 mm<sup>3</sup> Plastic section modulus about z-axis,  $W_{\mu,\nu}^{(a)}$  1223873 mm<sup>3</sup> Second moment of area about y-axis,  $l_{\nu}$  142677465 mm<sup>4</sup> Second moment of area about z-axis, I, 48574741 mm<sup>4</sup> **4**−17.3 1 256.3-

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Lateral restraint Both flanges have lateral restra Consider Combination 1 - 1.3 Classification of cross sectio Internal compression parts se Width of section;	int at supports o <u>5G + 1.5Q + 1.5</u> ns - Section 5.4 ubject to bendi	$\overline{\mathbf{k}}$ <b>RQ (Strength)</b> $\overline{\mathbf{k}}$ <b>ε</b> = √[235 Ν <b>ng and compre</b> $\mathbf{c}$ = d = <b>200</b> $\mathbf{c}$ = min([h	l/mm² / fy] = 0.94 ssion - Table 5. ).3 mm	1 2 (sheet 1 of 3)	o 1\ 0.590					
Outstand flanges - Table 5.2 (	sheet 2 of 3)	$c / t_w = 19.4$	$4 = 20.7 \times \varepsilon <= 3$	$\frac{1}{3}$ $\frac{1}$	- 1); Class	1				
width of section;		$c = (b - t_w - t_w)$	$2 \times r$ ) / 2 = <b>110</b> .	<b>3</b> mm						
		c / t <sub>f</sub> = 6.4 :	$= 6.8 \times \varepsilon \le 9 \times \varepsilon$	ε; Class 1						
					Sect	ion is class 1				
Check compression - Section	6.2.4									
Design compression force;	- / -	N <sub>Ed</sub> = <b>88</b> kl	N							
Design resistance of section - e	q 6.10;	$N_{c,Rd} = N_{pl,l}$	$N_{c,Rd} = N_{pl,Rd} = (A - \rho_{y,v} \times h \times t_w) \times f_y / \gamma_{M0} = 2953.8 \text{ kN}$							
		NEd / Nc,Rd	= 0.03							
		PASS - Desig	in compressior	resistance exc	ceeds design	compression				
Slenderness ratio for y-y axis	flexural buckli	ing - Section 6.	3.1.3							
Critical buckling length;		$L_{cr,y} = L_{m1_s}$	a1 = <b>5600</b> mm							
Critical buckling force;		$N_{cr,y} = \pi^2 \times$	$E \times I_y / L_{cr,y^2} = 9$	429.7 kN						
Slenderness ratio for buckling -	eq 6.50;	$\lambda_y = \sqrt{A \times A}$	$t_{y} / N_{cr,y}) = 0.56$	4						
Check y-y axis flexural buckli	ng resistance ·	Section 6.3.1.1	l							
Buckling curve - Table 6.2;		b								
Imperfection factor - Table 6.1;		$\alpha_y = 0.34$	$\alpha_{\rm y}=0.34$							
Buckling reduction determination	on factor;	$\phi_{\rm y}=0.5\times($	$\phi_{y} = 0.5 \times (1 + \alpha_{y} \times (\overline{\lambda}_{y} - 0.2) + \overline{\lambda}_{y}^{2}) = 0.721$							
Buckling reduction factor - eq 6	.49;	$\chi_y = min(1)$	$\chi_{y} = \min(1 / (\phi_{y} + \sqrt{(\phi_{y}^{2} - \lambda_{y}^{2})}), 1) = 0.855$							
Design buckling resistance - eq	6.47;	$N_{b,y,Rd} = \chi_y$	$N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1} = 2566.1 \text{ kN}$							
		NEd / Nb,y,Ro	$N_{Ed} / N_{b,y,Rd} = 0.034$							
		PA55 - L	esign buckling	resistance exc	eeus aesign	compression				
Slenderness ratio for z-z axis	flexural buckli	ng - Section 6.3	3.1.3							
Critical buckling length;		$L_{cr,z} = L_{m1_s}$	$s_{1\_seg1} = 5600 \text{ mm}$	n 010 4 kN						
Critical buckling force;		$N_{cr,z} = \pi^2 \times$	$E \times I_z / L_{cr,z^2} = 3$	210.4 KN						
Sienderness ratio for buckling -	eq 6.50;	$\lambda_z = \mathcal{N}(\mathbf{A} \times$	$(T_y / N_{cr,z}) = 0.96$	/						
Check z-z axis flexural buckli	ng resistance -	Section 6.3.1.1								
Buckling curve - Table 6.2;		C								
Imperfection factor - Table 6.1;	<i>.</i> .	$\alpha_z = 0.49$		$\sim$ $$ $\sim$						
Buckling reduction determination	in factor;	$\phi_z = 0.5 \times ($	$\phi_{z} = 0.5 \times (1 + \alpha_{z} \times (\lambda_{z} - 0.2) + \lambda_{z}^{2}) = 1.156$							
Design buckling resistance	.49,	$\chi_z = \min(1)$	$\sqrt{(\phi_z + \sqrt{(\phi_z^2 - \lambda_z^2)})}$	$(2^{-})), 1) = 0.559$						
Design buckling resistance - eq	0.47,	$N_{b,z,Rd} = \chi_z$	$\times \mathbf{A} \times \mathbf{I}_{\mathbf{y}} / \gamma_{M1} = \mathbf{I}$	079.2 KIN						
			a – 0.052 Desian huckling	i resistance evo	eeds desian	compression				
Check torgional and target	flow		0 4 A		ucoigii					
Torsional buckling longth:	I-IIEXURAI DUCKI	ing - Section 6.	J. 1.4	mm						
Distance from shear centre to a	entroid in v avia	$\mathbf{L}_{\mathrm{Cr},\mathrm{I}} = \mathbf{L}_{\mathrm{m1}}$	ы_segi_к = סססס I n							
Distance nom snear centre to C	entiola in y axis	$y_0 = 0.0 \text{ mm}$								

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Distance from the state									
Distance from snear centre to co	entroid in z axis;	$Z_0 = 0.0 \text{ mm}$	11 2) <b>100 0</b>						
Radius of gyralion;	f	$I_0 = V(I_y^2 + I_y^2)$	(2-) = 129.9 mm		7705 0 1.11				
	force;	$N_{cr,T} = 1 / I_0$	$f^2 \times (G \times I_t + \pi^2)$	$\times$ E $\times$ I <sub>w</sub> / L <sub>cr,T<sup>2</sup></sub> ) =	7705.2 KIN				
lorsion factor;		β⊤ = 1 - (y <sub>0</sub>	$(10)^2 = 1$						
Elastic critical torsional-flexural	DUCKIING FORCE	F4 N1 / N1		<u>)</u>	2 NL / NL 1				
Ncr,TF =	$\sim N_{cr,y} / (2 \times \beta_T) \times$	<[1 + Ncr,T / Ncr,y	$- \sqrt{(1 - N_{cr,T} / N_{cr,T})}$	cr,y) <sup>2</sup> + 4 × ( <b>y</b> <sub>0</sub> / I <sub>0</sub> )	$^{2} \times \mathbf{N}_{cr,T} / \mathbf{N}_{cr,y}$	I] = 7705.2 KIN			
Elastic critical buckling force;		$N_{cr} = min(r)$	$N_{cr,T}$ , $N_{cr,TF}$ ) = 77	<b>05.2</b> KN					
Sienderness ratio for torsional b	uckling - eq 6.52	$2;  \lambda_{T} = \sqrt{ A } \times$	$[T_y / N_{cr}] = 0.624$	ł					
Design resistance for torsiona	al and torsional	-flexural buckl	ing - Section 6	.3.1.1					
Buckling curve - Table 6.2;		С							
Imperfection factor - Table 6.1;		α <sub>T</sub> = <b>0.49</b>		_					
Buckling reduction determination	n factor;	φ <sub>T</sub> = 0.5 × (	$(1 + \alpha_T \times (\overline{\lambda}_T - 0))$	$(0.2) + \overline{\lambda}T^2) = 0.79$	99				
Buckling reduction factor - eq 6.	49;	χ⊤ = min(1	/ (φт + √(φт² - 7	ū⊤²)), 1) = <b>0.771</b>					
Design buckling resistance - eq	6.47;	$N_{b,T,Rd} = \chi_T$	$\times$ A $\times$ f <sub>y</sub> / $\gamma_{M1}$ =	<b>2314.9</b> kN					
		$N_{Ed} / N_{b,T,R}$	d = <b>0.038</b>						
		PASS - L	Design buckling	g resistance exc	eeds design	compression			
Check design at start of span									
Check shear - Section 6.2.6									
Height of web:		hw = h - 2 >	$h_w = h - 2 \times t_f = 225.7 \text{ mm};$ $\eta = 1.000$						
,		h <sub>w</sub> / t <sub>w</sub> = 21	$.9 = 23.3 \times \epsilon / n$	< 72 × ε / η					
				Shear buckling	resistance ca	n be ignored			
Design shear force;		V <sub>y,Ed</sub> = <b>297</b>	<b>.5</b> kN	-		•			
Shear area - cl 6.2.6(3);		$A_v = max(A)$	$-2 \times b \times t_{\rm f} + (t_{\rm w})$	$_{v}$ + 2 × r) × t <sub>f</sub> , $\eta$ ×	h <sub>w</sub> × t <sub>w</sub> ) = <b>308</b>	<b>1</b> mm²			
Design shear resistance - cl 6.2	.6(2);	$V_{c,y,Rd} = V_p$	$V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0} = 471.4 \text{ kN}$						
		V <sub>y,Ed</sub> / V <sub>c,y,F</sub>	Rd = <b>0.631</b>						
		PAS	SS - Design she	ear resistance ex	ceeds desig	n shear force			
Shear reduction factor - cl.6.2.8	(3);	$\rho_{y,v} = (2 \times V)$	$V_{y,Ed} / V_{pl,y,Rd} - 1$	) <sup>2</sup> = <b>0.069</b>					
Check bending moment - Sec	tion 6.2.5 and 6	.2.8							
Design bending moment;		$M_{y,Ed} = 191$	kNm						
Design bending resistance morr	ent - eq 6.13;	$M_{c,y,Rd} = M_{l}$	$p_{\text{J},y,\text{Rd}} = (W_{\text{pl},y} - \rho)$	$_{y,v} \times t_w \times h^2 / 4) \times 10^{-1}$	f <sub>y</sub> / γ <sub>M0</sub> = <b>321.1</b>	l kNm			
		$M_{y,Ed} / M_{c,y}$	<sub>Rd</sub> = <b>0.595</b>						
	PASS	- Design bendi	ing resistance	moment exceed	s design ben	ding moment			
Slenderness ratio for lateral to	orsional bucklii	ng							
Correction factor - Table 6.6;		kc = <b>0.897</b>							
		$C_1 = 1 / k_c^2$	= 1.244						
Poissons ratio;		v = <b>0.3</b>							
Shear modulus;		G = E / [2 >	< (1 + v)] = 8076	<b>59</b> N/mm²					
Unrestrained effective length;		L = 1.0 × L	m1_s1_seg1_B = <b>56</b>	<b>00</b> mm					
Elastic critical buckling moment;		$M_{cr} = C_1 \times C_1$	$\pi^2 \times E \times I_z / L^2 \times$	$\sqrt{(I_w / I_z + L^2 \times G)}$	$ imes$ I <sub>t</sub> / ( $\pi^2 \times E \times$	l <sub>z</sub> )) = <b>803.6</b>			
		kNm							
Slenderness ratio for lateral tors	ional buckling;	$\lambda_{LT} = \sqrt{W}$	$p_{l,y} \times f_y / M_{cr}) = 0$	.635					
Limiting slenderness ratio;		$\lambda_{LT,0} = 0.4$				_			
			$\lambda_{LT} > \lambda_{LT,0} - La$	teral torsional b	uckling canne	ot be ignored			
Check buckling resistance - S	ection 6.3.2.1								
Buckling curve - Table 6.5;		b							
Imperfection factor - Table 6.3;		$\alpha_{\text{LT}} = 0.34$							

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Correction factor for rolled secti	ons;	β = <b>0.75</b>						
LTB reduction determination fac	ctor;	$\phi_{LT} = 0.5 \times  $	$[1 + \alpha_{LT} \times (\overline{\lambda}_{LT})]$	$\Gamma - \overline{\lambda}_{LT,0} + \beta \times \overline{\lambda}_{LT}$	<sup>2</sup> ] = <b>0.691</b>			
LTB reduction factor - eq 6.57;		$\chi_{LT} = min(1)$	/ [ $\phi$ LT + $√(\phi$ LT <sup>2</sup>	- $\beta \times \overline{\lambda}_{LT^2}$ ], 1, 1 /	$\overline{\lambda}_{LT^2}$ ) = <b>0.90</b>	)1		
Modification factor;		f = min(1 - 0	0.5 imes (1 - k <sub>c</sub> ) $ imes$	$[1 - 2 \times (\overline{\lambda}_{LT} - 0.8)]$	3) <sup>2</sup> ], 1) = <b>0.9</b>	51		
Modified LTB reduction factor -	eq 6.58;	$\chi_{\text{LT,mod}} = mi$	n(χ <sub>LT</sub> / f, 1, 1 /	$\overline{\lambda}_{LT}^2) = 0.947$				
Design buckling resistance mor	nent - eq 6.55;	$M_{b,y,Rd} = \chi_{LT}$	$_{,mod} \times W_{pl.y} \times f_y$	, / γ <sub>M1</sub> = <b>307.2</b> kNr	n			
		M <sub>y,Ed</sub> / M <sub>b,y,I</sub>	Rd = 0.622					
	PASS	- Design buckli	ng resistance	moment exceed	ls design be	ending moment		
Check bending and axial forc	e - Section 6.2.	9						
Bending and axial force check -	eq.6.33 & eq.6.	.34; $N_{y,lim} = min($	$0.25 \times N_{\text{pl,Rd}}$ , (	$0.5 \times h_w \times t_w \times (1 - 1)$	$\rho_{y,v}$ ) × f <sub>y</sub> / $\gamma_N$	no) = <b>286.8</b> kN		
Allowerse need not be ma	ada far tha affa	NEd / Ny,lim =	= 0.307	atia vasiatavas				
Allowance need not be ma	aue for the effe	ci of the axial fo	rce on the pla	astic resistance	moment ab	out the y-y axis		
Check combined bending and	d compression	- Section 6.3.3		las 0.071				
Equivalent uniform moment fact	iors - Table B.3;	$\psi_y = -191 \text{ k}$	NM / -219.3 kM	VIII = U.8/1				
		$\alpha_y = -219.3$	KINM / 239.5 K	NM = -0.916				
		$C_{my} = 0.95$	$+ 0.05 \times \alpha_y = 0$	Nm - 0 871				
3		$\psi_{L1} = -1911$	$\psi_{LT} = -191 \text{ KNm} / -219.3 \text{ KNm} = 0.871$ $\alpha_{LT} = -219.3 \text{ KNm} / 239.5 \text{ KNm} = -0.916$					
		$C_{mlT} = 0.95$	$C_{mLT} = 0.95 + 0.05 \times \alpha_{LT} = 0.904$					
Interaction factors ky for mom	hara ayaaantik	la to toroional	defermetione					
Characteristic moment resistant	ibers susceptit		r = 324.3  k					
Characteristic moment resistant	се, се.	$M_{z,Rk} = W_{pl.}$	y ∧ iy = 324.3 k z × fy = 152.5 k	Nm				
Characteristic resistance to nor	mal force:	$N_{Rk} = A \times f_{V}$	$N_{Rk} = A \times f_y = 3002.8 \text{ kN}$					
Interaction factors;	,	$k_{vv} = C_{mv} \times$	$k_{yy} = C_{my} \times (1 + min(\overline{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times 1)^2)$					
,		N <sub>Rk</sub> / γ <sub>M1</sub> )) =	0.916	, , (	.,			
		k <sub>zy</sub> = 1 - 0.1	$\times \min(1, \overline{\lambda}_z) >$	< N <sub>Ed</sub> / ((C <sub>mLT</sub> - 0.2	$(25) \times \chi_z \times N_{\rm R}$	<sub>kk</sub> / γ <sub>M1</sub> ) = <b>0.992</b>		
Interaction formulae - eq 6.61 &	eq 6.62;	$N_{Ed}$ / ( $\chi_y  imes N$	J <sub>Rk</sub> / γ <sub>M1</sub> ) + k <sub>yy</sub>	×				
		$M_{y,Ed}$ / ( $\chi_{LT}$ :	$\times M_{y,Rk} / \gamma_{M1}) =$	0.573				
		$N_{Ed}$ / ( $\chi_z  imes N$	$I_{\rm Rk} / \gamma_{\rm M1}) + K_{zy}$	$ imes$ M <sub>y,Ed</sub> / ( $\chi_{LT}  imes$ M <sub>y</sub>	<sub>,Rk</sub> / γ <sub>M1</sub> ) = <b>0</b>	.637		
		PASS - Co	ombined bend	ling and compre	ssion chec	ks are satisfied		
Check design 2441 mm along	span							
Check bending moment - Sec	tion 6.2.5 and 6	5.2.8						
Design bending moment;		$M_{y,Ed} = 245$	. <b>5</b> kNm					
Design bending resistance mon	nent - eq 6.13;	$M_{c,y,Rd} = M_p$	$I_{y,Rd} = W_{pl.y} \times f_{y}$	y / γ <sub>M0</sub> = <b>324.3</b> kN	m			
		$M_{y,Ed} \ / \ M_{c,y,F}$	Rd = <b>0.757</b>					
	PASS	- Design bendi	ng resistance	moment exceed	ls design be	ending moment		
Slenderness ratio for lateral t	orsional buckli	ng						
Correction factor - Table 6.6;		k <sub>c</sub> = <b>0.897</b>	4.044					
Deigeone ratio		$C_1 = 1 / k_c^2$	= 1.244					
Poissons ratio;		$V = \mathbf{U}.3$	(1)] 007	$60 \text{ N/mm}^2$				
Unrestrained effective length:		$\mathbf{G} = \mathbf{E} / [\mathbf{Z} \times \mathbf{I}]$	(1 + V) = 807					
Flastic critical buckling moment		$L = 1.0 \times Ln$ $M_{er} = C_1 \times \pi$	$\frac{11}{2} \times \mathbf{F} \times \mathbf{I}_{\tau} / \mathbf{I}_{\tau}^2$	×√(l <sub>w</sub> / l₂ + l ² × G	$\times$ I <sub>t</sub> / ( $\pi^2$ $\vee$ $\vdash$	×   <sub>7</sub> )) = 803 6		
	,	kNm	· ^ L ^ 12 / L /		~ (// <b>~</b> L	. ^ 12// - 003.0		
Slenderness ratio for lateral tors	sional buckling;	$\overline{\lambda}_{LT} = \sqrt{W_{p}}$	$f_{\text{l.y}} \times f_{\text{y}} / M_{\text{cr}}) = 0$	0.635				
	5,	× F						

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Limiting slenderness ratio;		$\overline{\lambda}_{LT,0} = 0.4$	Ļ						
			$\overline{\lambda}_{LT} > \overline{\lambda}_{LT,0} - Lat$	eral torsional b	uckling canno	ot be ignored			
Check buckling resistance - S	ection 6.3.2.1								
Buckling curve - Table 6 5	0.0.2.1	b							
Imperfection factor - Table 6.3:		αιτ = <b>0.34</b>							
Correction factor for rolled section	2015	β <b>– 0 75</b>							
LTB reduction determination fac	tor:	ρ= <b>0.15</b> φτ=0.5×	$[1 + \alpha_{1} + \gamma_{1} + \overline{\lambda}_{1} +$	$-\overline{\lambda}$ $+\beta \times \overline{\lambda}$ $+$	<sup>2</sup> 1 – <b>0 601</b>				
LTB reduction factor og 6 57:		$\psi_{LT} = 0.5 \times$	$1 + \alpha c < \sqrt{\alpha - 2}$	$\beta \propto \overline{\lambda} + 2 (1 + 1 + 1)$	$\frac{1}{2} = 0.031$				
Modification factor:		$\chi_{LI} = IIIIII(I)$	□ / [ψ∟] + \(ψ∟] - Ο Ε γ (1 − k ) γ [-	$p \times \pi_{[}$ , $n, n, n$	$\lambda_{L1} = 0.901$				
Modified LTD reduction factor	6 E 9 .	1 = 11111(1 -	$0.3 \times (1 - K_c) \times [$	$1 - 2 \times (\lambda L) - 0.0$	)-], 1) = <b>0.951</b>				
	eq 6.56,	$\chi$ LT,mod = III	ITI(XLT / I, I, I /	(1,, 0.07, 0, 1.0)	_				
Design buckling resistance mon	ient - eq 6.55;	$M_{b,y,Rd} = \chi_L$	T, mod $\times$ VV pl.y $\times$ Ty	$\gamma_{M1} = 307.2 \text{ kinn}$	n				
	DACC	IVIy,Ed / IVIb,y	,Rd = <b>0.799</b>		a daalam baa				
	PASS	- Design buckli	ing resistance i	noment exceed	s aesign bend	aing moment			
Check bending and axial force	e - Section 6.2.	9							
Check combined bending and	compression	- Section 6.3.3							
Equivalent uniform moment fact	ors - Table B.3;	$\psi_{y} = -191 \ k$	xNm / -219.3 kNr	m = <b>0.871</b>					
		α <sub>y</sub> = -219.3	8 kNm / 239.5 kN	lm = <b>-0.916</b>					
		C <sub>my</sub> = 0.95	+ $0.05 \times \alpha_y = 0.00$	904					
		ψ <sub>LT</sub> = -191 kNm / -219.3 kNm = <b>0.871</b>							
		α <sub>LT</sub> = -219.	α <sub>LT</sub> = -219.3 kNm / 239.5 kNm = <b>-0.916</b>						
		C <sub>mLT</sub> = 0.95	$5 + 0.05 \times \alpha_{LT} =$	0.904					
Interaction factors ke for mem	bers susceptib	le to torsional	deformations -	Table B.2					
Characteristic moment resistance	يم.	$M_{\rm V}  {\rm Pk} = {\rm W}_{\rm Pl}$	$v \times f_{v} = 324.3 \text{ kN}$	Im					
Characteristic moment resistance	, 	My,rik – Wpi	.y × fy = <b>324.6</b> kN	Im					
Characteristic resistance to norr	nal force:	$N_{Dk} - \mathbf{A} \times \mathbf{f}$	<b>3002 8</b> kN						
Interaction factors:	narioree,	$k_{\rm HK} = C_{\rm HK} \times 1$	$(1 + \min(\overline{\lambda}) - 0)$	2 0 8) $\times$ N <sub>E4</sub> / ( $\times$	~ ~				
interaction factors,		$\mathbf{x}_{yy} = \mathbf{U}_{my} \times (\mathbf{I} + \mathbf{IIIII}(\Lambda_y - \mathbf{U}, \mathbf{Z}, \mathbf{U}, \mathbf{S}) \times \mathbf{N}_{Ed} / (\chi_y \times \mathbf{N}_{Ed} + \mathbf{U}, \mathbf{U}, \mathbf{S}) = \mathbf{O} \mathbf{O} \mathbf{O} \mathbf{O} \mathbf{O} \mathbf{O} \mathbf{O} \mathbf{O}$							
		$N_{Rk} / \gamma_{M1}) = 0.910$							
Interaction formulae on C C1 9		$K_{ZY} = 1 - 0.$	$1 \times 11111(1, \Lambda z) \times$	INEd / ((OmL1 - 0.2	$(3) \times \chi_z \times \mathrm{INRk}$	γM1) = <b>0.992</b>			
Interaction formulae - eq 6.61 &	eq 6.62,	$NEd / (\chi_y \times 1)$	$NRk / \gamma M1) + Kyy \times$	207					
		IVI <sub>y,Ed</sub> / (XLT	$\times$ IVI <sub>y,Rk</sub> / $\gamma$ M1) = U		(	<b>NO</b>			
		$N_{Ed} / (\chi_z \times I)$	NRk / γM1) + Kzy ×	$IVI_{y,Ed} / (\chi_{LT} \times IVI_{y,Ed})$	$_{\rm Rk}$ / $\gamma_{\rm M1}$ ) = <b>0.80</b>	13 ana antiotic d			
		PA55 - C	ombinea benai	ing and compre-	SSION CHECKS	are satistied			
Check design at and of an ar									
Check shear - Section 6.2.6									
Height of web;		h <sub>w</sub> = h - 2 >	< t <sub>f</sub> = <b>225.7</b> mm;	η = <b>1.00</b>	00				
		h <sub>w</sub> / t <sub>w</sub> = 21	$.9 = 23.3 \times \epsilon / \eta$	< 72 × ε / η					
				Shear buckling	resistance ca	n be ignored			
Design shear force;		V <sub>y,Ed</sub> = <b>294</b>	.2 kN						
Shear area - cl 6.2.6(3);		$A_v = max(A)$	$A - 2 \times b \times t_f + (t_w)$	$(+ 2 \times r) \times t_f, \eta \times$	$h_w \times t_w) = 308^2$	1 mm <sup>2</sup>			
Design shear resistance - cl 6.2	.6(2);	$V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = 471.4 \text{ kN}$							
		$V_{y,Ed} / V_{c,y,Rd} = 0.624$							
		PAS	SS - Design she	ear resistance ex	xceeds desigi	n shear force			
Shear reduction factor - cl.6.2.8	(3);	$\rho_{y,v} = (2 \times N)$	$V_{y,Ed} / V_{pl,y,Rd} - 1$	<sup>2</sup> = <b>0.062</b>					

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Check bending moment - Sec	tion 6.2.5 and 6	6.2.8								
Design bending moment;		$M_{y,Ed} = 219$	<b>9.3</b> kNm							
Design bending resistance mor	ent - eq 6.13;	$M_{c,y,Rd} = M$	$p_{y,Rd} = (W_{pl.y} - \rho_y)$	$_{\rm A,v} \times t_{\rm w} \times h^2 / 4) \times$	$f_y / \gamma_{M0} = 321.5$	5 kNm				
		M <sub>y,Ed</sub> / M <sub>c,y</sub>	,Rd = <b>0.682</b>							
	PASS	- Design bend	ing resistance i	moment exceed	s design ben	ding moment				
Slenderness ratio for lateral to	orsional buckli	ng								
Correction factor - Table 6.6;		kc = <b>0.897</b>								
		$C_1 = 1 / k_c^2$	<sup>2</sup> = <b>1.244</b>							
Poissons ratio;		$\nu = 0.3$								
Shear modulus;		G = E / [2 :	× (1 + v)] = <b>8076</b>	<b>9</b> N/mm²						
Unrestrained effective length;		L = 1.0 × L	.m1_s1_seg1_B = <b>56</b>	<b>00</b> mm						
Elastic critical buckling moment;		$M_{cr} = C_1 \times$	$\pi^2 \times E \times I_z / L^2 \times$	$\sqrt{(I_w / I_z + L^2 \times G)}$	$ imes$ I <sub>t</sub> / ( $\pi^2$ $ imes$ E $ imes$	l <sub>z</sub> )) = <b>803.6</b>				
-		kNm								
Slenderness ratio for lateral tors	ional buckling;	$\overline{\lambda}_{LT} = \sqrt{W}$	$f_{\text{pl.y}} \times f_{\text{y}} / M_{\text{cr}}) = 0$	.635						
Limiting slenderness ratio;		$\overline{\lambda}_{LT,0} = 0.4$								
			$\overline{\lambda}_{LT} > \overline{\lambda}_{LT,0} - Lat$	teral torsional b	uckling cann	ot be ignored				
Check buckling resistance - S	action 6 3 2 1				0	U U				
Buckling curve - Table 6.5:	ection 0.5.2.1	h								
Imperfection factor - Table 6.3:		α <sub>1</sub> τ – <b>0 34</b>	$\alpha_{1.7} = 0.34$							
Correction factor for rolled section		α <sub>L1</sub> = <b>0.0</b> 4	$\beta = 0.75$							
LTR reduction determination fac	tor:	p = 0.75	$\phi_{\text{LT}} = 0.5 \times [1 + \alpha_{\text{LT}} \times (\overline{\lambda}_{\text{LT}} - \overline{\lambda}_{\text{LT},0}) + \beta \times \overline{\lambda}_{\text{LT}}^2] = 0.691$							
LTB reduction factor og 6 57:	.01,	$\psi_{LT} = 0.5 \times$	$\gamma_{\rm LT} = \min(1 / [\phi_{\rm LT} + \sqrt{(\phi_{\rm LT}^2 - \beta \times \overline{\lambda_{\rm L}} \tau^2)}] = 0.001$							
LTB reduction factor:		$\chi_{L1} = \Pi \Pi \Pi$	$f = \min(1 - 0.5 \times (1 - k_0) \times [1 - 2 \times (\lambda_1 - 0.8)^2] = 0.951$							
Modified LTP reduction factor	6 E 9 ·	1 = 11111(1 -	$0.5 \times (1 - K_c) \times [$	$\overline{1} - 2 \times (\Lambda_{LT} - 0.0)$	) <sup>-</sup> ], 1) = <b>0.951</b>					
Design hughling resistance man	eq 0.00,	$\chi$ LT,mod = 11	$= \dots \times \mathbf{M} + \mathbf{x} + \mathbf{f}$	$(\Lambda_{LT}^{-}) = 0.947$	~					
Design buckling resistance mon	ieni - eq 6.55,	$IVIb,y,Rd = \chi L$	$T, mod \times VV pl.y \times Iy$	$\gamma \gamma M1 = 307.2 \text{ KINI}$	[]					
	PASS	- Design buckl	,Rd = 0.714 ing resistance (	moment exceed	e desian hen	dina moment				
	7 400	- Design bucki	ing resistance i		s design ben	ung moment				
Check bending and axial force	e - Section 6.2.	9								
Check combined bending and	compression	- Section 6.3.3								
Equivalent uniform moment fact	ors - Table B.3;	ψ <sub>y</sub> = -191 k	kNm / -219.3 kNi	m = <b>0.871</b>						
		α <sub>y</sub> = -219.3	3 kNm / 239.5 kN	lm = <b>-0.916</b>						
		C <sub>my</sub> = 0.95	+ $0.05 \times \alpha_y = 0$ .	904						
;		ψ∟⊤ = -191	kNm / -219.3 kN	lm = <b>0.871</b>						
		α <sub>LT</sub> = -219	.3 kNm / 239.5 k	Nm = <b>-0.916</b>						
		$C_{mLT} = 0.9$	$5 + 0.05  imes lpha_{LT} =$	0.904						
Interaction factors k <sub>ij</sub> for mem	bers susceptib	le to torsional	deformations -	Table B.2						
Characteristic moment resistance	e;	$M_{y,Rk} = W_p$	y × fy = <b>324.3</b> k№	١m						
Characteristic moment resistance	e;	$M_{z,Rk} = W_p$	z × fy = <b>152.5</b> k№	١m						
Characteristic resistance to norr	nal force;	$N_{Rk} = A \times f$	y = <b>3002.8</b> kN							
Interaction factors;		$k_{yy} = C_{my} \times$	$(1 + \min(\overline{\lambda}_y - 0))$	.2, 0.8) $\times$ N <sub>Ed</sub> / ( $\chi$	/y ×					
·		N <sub>Rk</sub> / γ <sub>M1</sub> ))	= 0.916							
		k <sub>zv</sub> = 1 - 0.	$1 \times \min(1, \overline{\lambda}_z) \times$	NEd / ((CmLT - 0.2	(5) $\times \chi_z \times N_{Rk}$	γ <sub>M1</sub> ) = <b>0.992</b>				
Interaction formulae - ea 6.61 &	eq 6.62;	$N_{Ed} / (\gamma_v \times$	N <sub>Rk</sub> / γ <sub>M1</sub> ) + k <sub>vv</sub> ×		, ,,	• • •				
		 Му Еd / (У) т	$\times$ M <sub>V.Rk</sub> / $\gamma_{M1}$ ) = (	0.653						
		$N_{Ed} / (\gamma_z \times$	N <sub>Rk</sub> / γ <sub>M1</sub> ) + k <sub>zv</sub> ×	$M_{y,Ed} / (\gamma_{LT} \times M_{v})$	. <sub>Rk</sub> / γ <sub>M1</sub> ) = <b>0.7</b> 2	23				
		PASS - C	combined bend	ing and compre	ssion checks	are satisfied				

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### Consider Combination 2 - 1.0G + 1.0Q + 1.0RQ (Service)

#### Check design 2629 mm along span

#### Check y-y axis deflection - Section 7.2.1

Maximum deflection;

Allowable deflection;

 $\delta_{\text{y}}=\textbf{9.4}~\text{mm}$ 

 $\delta_{\text{y,Allowable}} = Min(L_{\text{m1\_s1}} \ / \ 500, \ 10 \ mm) = \textbf{10} \ mm$ 

 $\delta_{\text{y}} \; / \; \delta_{\text{y,Allowable}} = \textbf{0.936}$ 

PASS - Allowable deflection exceeds design deflection

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#### BOX-FRAME ON THE ALLIGMENT OF THE NEW REAR

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

#### Analysis

Tedds calculation version 4.0.02 Tedds calculation version 1.0.13



#### Nodes

Node	Co-ord	linates	Freedom			Coordina	te system	Spring			
	Х	Z	Х	Z	Rot.	Name	Angle	Х	Z	Rot.	
	(m)	(m)					(°)	(kN/m)	(kN/m)	kNm/°	
1	0	0	Fixed	Fixed	Free		0	0	0	0	
2	0	3	Free	Free	Free		0	0	0	0	
3	5.4	3	Free	Free	Free		0	0	0	0	
4	5.4	0	Fixed	Fixed	Free		0	0	0	0	

Materials

Name	Density	Youngs Modulus	Shear Modulus	Thermal Coefficient	
	(kg/m³)	kN/mm²	kN/mm²	°C <sup>-1</sup>	
Steel (EC3)	7850	210	80.8	0.000012	

#### Sections

Name	Area	Moment of inertia Sh		Shea	r area
	(cm²)	Major (cm⁴)	Minor (cm <sup>4</sup> )	A <sub>y</sub> (cm²)	A <sub>z</sub> (cm²)
UC 152x152x30	38	1748	560	26	10
UB 254x146x37	47	5537	571	29	16

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Elements

Element	Length	No	des	Section	Material	Releases			Rotated
	(m)	Start	End			Start moment	End moment	Axial	
1	3	1	2	UC 152x152x30	Steel (EC3)	Fixed	Fixed	Fixed	
2	5.4	2	3	UB 254x146x37	Steel (EC3)	Fixed	Fixed	Fixed	
3	3	3	4	UC 152x152x30	Steel (EC3)	Fixed	Fixed	Fixed	
4	5.4	1	4	UC 152x152x30	Steel (EC3)	Fixed	Fixed	Fixed	

#### Members

Name	Elements				
	Start	End			
Member1	2	2			

Loading



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#### Load cases

Name	Enabled	Self weight factor	Patternable
Self Weight	yes	1	no
Terrace roof dead load - 1kN/m2 x 2m	yes	0	no
Terrace roof live load - 1.5kN/m2 x 2m	yes	0	no
Glass balustrade - 25kN/m2 x 0.022m x 1.5m	yes	0	no
SB3 Dead Reaction	yes	0	no
SB3 Live Reaction	yes	0	no
Wind	yes	0	no

## Load combinations

Load combination	Туре	Enabled	Patterned
1.35G + 1.5Q + 1.5RQ	Strength	yes	no
1.0G + 1.0Q + 1.0RQ	Service	yes	no
1.0G + 1.0ψ	Quasi	yes	no
1.35G + 1.5Q + 1.5ψ	Strength	yes	no
1.0G + 1.0Q + 0.5S	Service	yes	no
1.35G + 1.5ψ	Strength	yes	no
1.35G + 1.5Q + 1.5ψ	Strength	yes	no
1.0G + 1.0Q + 0.5S + 0.5W	Service	yes	no
1.35G + 1.5ψ	Strength	yes	no
1.0G + 1.0ψ	Service	yes	no
1.35G + 1.5ψ	Strength	yes	no
1.0G + 1.0ψ	Service	yes	no
1.0G + 1.5W	Strength	yes	no
1.0G + 1.0W	Service	yes	no

## Load combination: 1.35G + 1.5Q + 1.5RQ (Strength)

Load case	Factor
Self Weight	1.35
Terrace roof dead load - 1kN/m2 x 2m	1.35
Terrace roof live load - 1.5kN/m2 x 2m	1.5
Glass balustrade - 25kN/m2 x 0.022m x 1.5m	1.35
SB3 Dead Reaction	1.35
SB3 Live Reaction	1.5

Load combination: 1.0G + 1.0Q + 1.0RQ (Service)

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Load case	Factor
Self Weight	1
Terrace roof dead load - 1kN/m2 x 2m	1
Terrace roof live load - 1.5kN/m2 x 2m	1
Glass balustrade - 25kN/m2 x 0.022m x 1.5m	1
SB3 Dead Reaction	1
SB3 Live Reaction	1

## Load combination: 1.0G + 1.0ψ<sub>2</sub>Q (Quasi)

Load case	Factor
Self Weight	1
Terrace roof dead load - 1kN/m2 x 2m	1
Terrace roof live load - 1.5kN/m2 x 2m	0.3
Glass balustrade - 25kN/m2 x 0.022m x 1.5m	1
SB3 Dead Reaction	1
SB3 Live Reaction	0.3

## Load combination: 1.35G + 1.5Q + 1.5ψ<sub>0</sub>S (Strength)

Load case	Factor
Self Weight	1.35
Terrace roof dead load - 1kN/m2 x 2m	1.35
Terrace roof live load - 1.5kN/m2 x 2m	1.5
Glass balustrade - 25kN/m2 x 0.022m x 1.5m	1.35
SB3 Dead Reaction	1.35
SB3 Live Reaction	1.5

## Load combination: 1.0G + 1.0Q + 0.5S (Service)

Load case	Factor
Self Weight	1
Terrace roof dead load - 1kN/m2 x 2m	1
Terrace roof live load - 1.5kN/m2 x 2m	1
Glass balustrade - 25kN/m2 x 0.022m x 1.5m	1
SB3 Dead Reaction	1
SB3 Live Reaction	1

Load combination:  $1.35G + 1.5\psi_0Q + 1.5S$  (Strength)

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Load case	Factor
Self Weight	1.35
Terrace roof dead load - 1kN/m2 x 2m	1.35
Terrace roof live load - 1.5kN/m2 x 2m	1.05
Glass balustrade - 25kN/m2 x 0.022m x 1.5m	1.35
SB3 Dead Reaction	1.35
SB3 Live Reaction	1.05

# Load combination: 1.35G + 1.5Q + 1.5 $\psi_0$ S + 1.5 $\psi_0$ W (Strength)

Load case	Factor
Self Weight	1.35
Terrace roof dead load - 1kN/m2 x 2m	1.35
Terrace roof live load - 1.5kN/m2 x 2m	1.5
Glass balustrade - 25kN/m2 x 0.022m x 1.5m	1.35
SB3 Dead Reaction	1.35
SB3 Live Reaction	1.5
Wind	0.75

# Load combination: 1.0G + 1.0Q + 0.5S + 0.5W (Service)

Load case	Factor
Self Weight	1
Terrace roof dead load - 1kN/m2 x 2m	1
Terrace roof live load - 1.5kN/m2 x 2m	1
Glass balustrade - 25kN/m2 x 0.022m x 1.5m	1
SB3 Dead Reaction	1
SB3 Live Reaction	1
Wind	0.5

## Load combination: $1.35G + 1.5\psi_0Q + 1.5S + 1.5\psi_0W$ (Strength)

Load case	Factor
Self Weight	1.35
Terrace roof dead load - 1kN/m2 x 2m	1.35
Terrace roof live load - 1.5kN/m2 x 2m	1.05
Glass balustrade - 25kN/m2 x 0.022m x 1.5m	1.35
SB3 Dead Reaction	1.35
SB3 Live Reaction	1.05
Wind	0.75

Load combination:  $1.0G + 1.0\psi_0Q + 1.0S + 0.5W$  (Service)

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enstructures	28 Burghley Road				SDS187	
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Load case	Factor
Self Weight	1
Terrace roof dead load - 1kN/m2 x 2m	1
Terrace roof live load - 1.5kN/m2 x 2m	0.7
Glass balustrade - 25kN/m2 x 0.022m x 1.5m	1
SB3 Dead Reaction	1
SB3 Live Reaction	0.7
Wind	0.5

# Load combination: 1.35G + 1.5\v0Q + 1.5\v0S + 1.5W (Strength)

Load case	Factor
Self Weight	1.35
Terrace roof dead load - 1kN/m2 x 2m	1.35
Terrace roof live load - 1.5kN/m2 x 2m	1.05
Glass balustrade - 25kN/m2 x 0.022m x 1.5m	1.35
SB3 Dead Reaction	1.35
SB3 Live Reaction	1.05
Wind	1.5

## Load combination: $1.0G + 1.0\psi_0Q + 0.5S + 1.0W$ (Service)

Load case	Factor
Self Weight	1
Terrace roof dead load - 1kN/m2 x 2m	1
Terrace roof live load - 1.5kN/m2 x 2m	0.7
Glass balustrade - 25kN/m2 x 0.022m x 1.5m	1
SB3 Dead Reaction	1
SB3 Live Reaction	0.7
Wind	1

# Load combination: 1.0G + 1.5W (Strength)

Load case	Factor
Self Weight	1
Terrace roof dead load - 1kN/m2 x 2m	1
Glass balustrade - 25kN/m2 x 0.022m x 1.5m	1
SB3 Dead Reaction	1
Wind	1.5

Load combination: 1.0G + 1.0W (Service)

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Load case	Factor
Self Weight	1
Terrace roof dead load - 1kN/m2 x 2m	1
Glass balustrade - 25kN/m2 x 0.022m x 1.5m	1
SB3 Dead Reaction	1
Wind	1

## **Element point loads**

Element	Load case	Position		Load	Orientation
		Type Start			
				(kN)	
2	SB3 Dead Reaction	Ratio	0.37	46	GlobalZ
2	SB3 Live Reaction	Ratio	0.37	3	GlobalZ

## Element UDL loads

Element	Load case	Position			Load	Orientation
		Туре	Start	End		
					(kN/m)	
2	Terrace roof dead load - 1kN/m2 x 2m	Ratio	0	1	2	GlobalZ
2	Terrace roof live load - 1.5kN/m2 x 2m	Ratio	0	1	3	GlobalZ
2	Glass balustrade - 25kN/m2 x 0.022m x 1.5m	Ratio	0	1	1	GlobalZ
1	Wind	Ratio	0	1	1.2	GlobalX

# **Results**

## **Total deflection**



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Member		Defle	ction		Axial deflection			
	Pos (m)	Max (mm)	Pos (m)	Min (mm)	Pos (m)	Max (mm)	Pos (m)	Min (mm)
Member1	2.497	16.9	5.4	0.2	0	1.5	5.4	1.4

## Load combination: 1.35G + 1.5Q + 1.5RQ (Strength)

1.0G + 1.0Q + 1.0RQ (Service) - Total deflection @ 10x



#### Member results

## Load combination: 1.0G + 1.0Q + 1.0RQ (Service)

Member		Defle	ction		Axial deflection			
	Pos	Pos Max Pos Min				Max	Pos	Min
	(m)	(mm)	(m)	(mm)	(m)	(mm)	(m)	(mm)
Member1	2.494	12.2	5.4	0.1	0	1.1	5.4	1



Load combination:  $1.0G + 1.0\psi_2Q$  (Quasi)

Member		Defle	ction		Axial deflection			
	Pos	Pos Max Pos Min				Max	Pos	Min
	(m)	(mm)	(m)	(mm)	(m)	(mm)	(m)	(mm)
Member1	2.474	10.6	5.4	0.1	0	1	5.4	1

1.35G + 1.5Q + 1.5 $\psi_0$ S (Strength) - Total deflection @ 10x



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Member	Deflection				Axial deflection			
	Pos	Pos Max Pos Min				Max	Pos	Min
	(m)	(mm)	(m)	(mm)	(m)	(mm)	(m)	(mm)
Member1	2.497	16.9	5.4	0.2	0	1.5	5.4	1.4

## Load combination: 1.35G + 1.5Q + 1.5\v0S (Strength)

1.0G + 1.0Q + 0.5S (Service) - Total deflection @ 10x



### Member results

## Load combination: 1.0G + 1.0Q + 0.5S (Service)

Member		Defle	ction		Axial deflection				
	Pos	Pos Max Pos Min				Max	Pos	Min	
	(m)	(mm)	(m)	(mm)	(m)	(mm)	(m)	(mm)	
Member1	2.494	12.2	5.4	0.1	0	1.1	5.4	1	



Load combination:  $1.35G + 1.5\psi_0Q + 1.5S$  (Strength)

Member		Defle	ction		Axial deflection			
	Pos	Pos Max Pos Min				Max	Pos	Min
	(m)	(mm)	(m)	(mm)	(m)	(mm)	(m)	(mm)
Member1	2.488	15.8	5.4	0.2	0	1.4	5.4	1.4

1.35G + 1.5Q + 1.5 $\psi_0 S$  + 1.5 $\psi_0 W$  (Strength) - Total deflection @ 10x



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## Load combination: $1.35G + 1.5Q + 1.5\psi_0S + 1.5\psi_0W$ (Strength)

Member		Defle	ction		Axial deflection			
	Pos	Pos Max Pos Min				Max	Pos	Min
	(m)	(mm)	(m)	(mm)	(m)	(mm)	(m)	(mm)
Member1	2.491	16.8	5.4	0.2	0	2.4	5.4	2.3

1.0G + 1.0Q + 0.5S + 0.5W (Service) - Total deflection @ 10x



### Member results

## Load combination: 1.0G + 1.0Q + 0.5S + 0.5W (Service)

Member		Defle	ction		Axial deflection			
	Pos	Pos Max Pos Min				Max	Pos	Min
	(m)	(mm)	(m)	(mm)	(m)	(mm)	(m)	(mm)
Member1	2.488	12.2	5.4	0.1	0	1.7	5.4	1.7



Load combination:  $1.35G + 1.5\psi_0Q + 1.5S + 1.5\psi_0W$  (Strength)

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Member		Defle	ction		Axial deflection			
	Pos	Pos Max Pos Min				Max	Pos	Min
	(m)	(mm)	(m)	(mm)	(m)	(mm)	(m)	(mm)
Member1	2.482	15.7	5.4	0.2	0	2.4	5.4	2.3

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 $1.0G + 1.0\psi_0Q + 1.0S + 0.5W$  (Service) - Total deflection @ 10x



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## Load combination: $1.0G + 1.0\psi_0Q + 1.0S + 0.5W$ (Service)

Member	Deflection				Axial deflection			
	Pos	Max	Pos	Min	Pos	Max	Pos	Min
	(m)	(mm)	(m)	(mm)	(m)	(mm)	(m)	(mm)
Member1	2.48	11.5	5.4	0.1	0	1.7	5.4	1.6

1.35G + 1.5 $\psi_0$ Q + 1.5 $\psi_0$ S + 1.5W (Strength) - Total deflection @ 10x



### Member results

# Load combination: $1.35G + 1.5\psi_0Q + 1.5\psi_0S + 1.5W$ (Strength)

Member	Deflection				Axial deflection			
	Pos	Max	Pos	Min	Pos	Max	Pos	Min
	(m)	(mm)	(m)	(mm)	(m)	(mm)	(m)	(mm)
Member1	2.475	15.7	5.4	0.2	0	3.3	5.4	3.3





Load combination:  $1.0G + 1.0\psi_0Q + 0.5S + 1.0W$  (Service)

Member		Defle	ction		Axial deflection			
	Pos	Max	Pos	Min	Pos	Max	Pos	Min
	(m)	(mm)	(m)	(mm)	(m)	(mm)	(m)	(mm)
Member1	2.474	11.4	5.4	0.1	0	2.3	5.4	2.3





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Eoua combinatio										
Member	Deflection				Axial deflection					
	Pos	Max	Pos	Min	Pos	Max	Pos	Min		
	(m)	(mm)	(m)	(mm)	(m)	(mm)	(m)	(mm)		
Member1	2.443	9.7	5.4	0.1	0	2.9	5.4	2.9		
	•		-	•	•			-		



1.0G + 1.0W	(Service) -	Total deflection	@ 10x



### Member results

## Load combination: 1.0G + 1.0W (Service)

Member	Deflection				Axial deflection			
	Pos	Max	Pos	Min	Pos	Max	Pos	Min
	(m)	(mm)	(m)	(mm)	(m)	(mm)	(m)	(mm)
Member1	2.449	9.8	5.4	0.1	0	2.3	5.4	2.2

Node deflections

## Load combination: 1.35G + 1.5Q + 1.5RQ (Strength)

Node	Deflection		Rotation	Co-ordinate system
	Х	Z		
	(mm)	(mm)	(°)	
1	0	0	-0.14989	
2	1.5	0.3	0.48782	
3	1.4	0.2	-0.39915	
4	0	0	0.17046	

## Load combination: 1.0G + 1.0Q + 1.0RQ (Service)

Node	Deflection		Rotation	Co-ordinate system
	Х	Z		
	(mm)	(mm)	(°)	
1	0	0	-0.10836	

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Node	Deflection		Rotation	Co-ordinate system
	X	Z		
	(mm)	(mm)	(°)	
2	1.1	0.2	0.354	
3	1	0.1	-0.28877	
4	0	0	0.1235	

## Load combination: $1.0G + 1.0\psi_2Q$ (Quasi)

Node	Deflection		Rotation	Co-ordinate system
	Х	Z		
	(mm)	(mm)	(°)	
1	0	0	-0.09158	
2	1	0.2	0.30775	
3	1	0.1	-0.24531	
4	0	0	0.10607	

### Load combination: 1.35G + 1.5Q + 1.5ψ<sub>0</sub>S (Strength)

		-		
Node	Deflection		Rotation	Co-ordinate system
	Х	Z		
	(mm)	(mm)	(°)	
1	0	0	-0.14989	
2	1.5	0.3	0.48782	
3	1.4	0.2	-0.39915	
4	0	0	0.17046	

## Load combination: 1.0G + 1.0Q + 0.5S (Service)

Node	Deflection		Rotation	Co-ordinate system
	х	Z		
	(mm)	(mm)	(°)	
1	0	0	-0.10836	
2	1.1	0.2	0.354	
3	1	0.1	-0.28877	
4	0	0	0.1235	

Load combination: 1.35G + 1.5y<sub>0</sub>Q + 1.5S (Strength)

Node	Deflection		Rotation	Co-ordinate system
	х	Z		
	(mm)	(mm)	(°)	
1	0	0	-0.1391	
2	1.4	0.2	0.45808	
3	1.4	0.2	-0.37121	
4	0	0	0.15926	

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#### Load combination: $1.35G + 1.5Q + 1.5\psi_0S + 1.5\psi_0W$ (Strength)

Node	Deflection		Rotation	Co-ordinate system
	х	Z		
	(mm)	(mm)	(°)	
1	0	0	-0.12983	
2	2.4	0.2	0.48877	
3	2.3	0.2	-0.39172	
4	0	0	0.18169	

#### Load combination: 1.0G + 1.0Q + 0.5S + 0.5W (Service)

Node	Deflection		Rotation	Co-ordinate system
	х	Z		
	(mm)	(mm)	(°)	
1	0	0	-0.09499	
2	1.7	0.2	0.35464	
3	1.7	0.1	-0.28382	
4	0	0	0.13098	

#### Load combination: $1.35G + 1.5\psi_0Q + 1.5S + 1.5\psi_0W$ (Strength)

Node	Deflection		Rotation	Co-ordinate system
	х	Z		
	(mm)	(mm)	(°)	
1	0	0	-0.11904	
2	2.4	0.2	0.45903	
3	2.3	0.2	-0.36378	
4	0	0	0.17048	

#### Load combination: $1.0G + 1.0\psi_0Q + 1.0S + 0.5W$ (Service)

Node	Deflection		Rotation	Co-ordinate system
	х	Z		
	(mm)	(mm)	(°)	
1	0	0	-0.0878	
2	1.7	0.2	0.33481	
3	1.6	0.1	-0.26519	
4	0	0	0.12351	

## Load combination: $1.35G + 1.5\psi_0Q + 1.5\psi_0S + 1.5W$ (Strength)

Node	Deflection		Rotation	Co-ordinate system
	Х	Z		
	(mm)	(mm)	(°)	
1	0	0	-0.09898	
2	3.3	0.2	0.45999	
3	3.3	0.2	-0.35636	
4	0	0	0.1817	

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## Load combination: $1.0G + 1.0\psi_0Q + 0.5S + 1.0W$ (Service)

Node	Deflection		Rotation	Co-ordinate system
	х	Z		
	(mm)	(mm)	(°)	
1	0	0	-0.07443	
2	2.3	0.2	0.33545	
3	2.3	0.1	-0.26024	
4	0	0	0.13099	

# Load combination: 1.0G + 1.5W (Strength)

Node	Deflection		Rotation	Co-ordinate system
	х	Z		
	(mm)	(mm)	(°)	
1	0	0	-0.04427	
2	2.9	0.1	0.28983	
3	2.9	0.1	-0.21183	
4	0	0	0.12104	

## Load combination: 1.0G + 1.0W (Service)

Node	Deflection		Rotation	Co-ordinate system
	х	Z		
	(mm)	(mm)	(°)	
1	0	0	-0.05764	
2	2.3	0.1	0.28919	
3	2.2	0.1	-0.21678	
4	0	0	0.11356	

#### **Total base reactions**

Load case/combination	Fo	rce
	FX	FZ
	(kN)	(kN)
1.35G + 1.5Q + 1.5RQ (Strength)	0	120
1.0G + 1.0Q + 1.0RQ (Service)	0	86.7
1.0G + 1.0ψ2Q (Quasi)	0	73.3
1.35G + 1.5Q + 1.5ψ₀S (Strength)	0	120
1.0G + 1.0Q + 0.5S (Service)	0	86.7
1.35G + 1.5ψ <sub>0</sub> Q + 1.5S (Strength)	0	111.3
1.35G + 1.5Q + 1.5ψ₀S + 1.5ψ₀W (Strength)	-2.7	120
1.0G + 1.0Q + 0.5S + 0.5W (Service)	-1.8	86.7
1.35G + 1.5ψ₀Q + 1.5S + 1.5ψ₀W (Strength)	-2.7	111.3
1.0G + 1.0ψ₀Q + 1.0S + 0.5W (Service)	-1.8	81
1.35G + 1.5ψ₀Q + 1.5ψ₀S + 1.5W (Strength)	-5.4	111.3
1.0G + 1.0ψ₀Q + 0.5S + 1.0W (Service)	-3.6	81
1.0G + 1.5W (Strength)	-5.4	67.5

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Load case/combinat	tion	For	се				
		FX	FZ				
		(kN)	(kN)				
1.0G + 1.0W (Servic	ce)	-3.6	67.5				
Reactions							
1: (11.{	1.35G + 1.5Q + 1.5RQ (Strer Node: (Horiz (kN), Vert (kN), Memt				3 		

# Load combination: 1.35G + 1.5Q + 1.5RQ (Strength)

Node	Fo	Moment	
	Fx Fz		Му
	(kN)	(kN)	(kNm)
1	11.8	68.6	0
4	-11.8	51.3	0



### Load combination: 1.0G + 1.0Q + 1.0RQ (Service)

Node	Fo	Moment	
	Fx Fz		Му
	(kN)	(kN)	(kNm)
1	8.5	49.7	0
4	-8.5	37	0



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### Load combination: 1.0G + 1.0y2Q (Quasi)

Node	Fo	Moment	
	Fx (kN)	Fz (kN)	My (kNm)
1	7.4	42.7	0
4	-7.4	30.5	0



### Load combination: 1.35G + 1.5Q + 1.5 $\psi_0$ S (Strength)

Node	Fo	Moment	
	Fx Fz		Му
	(kN)	(kN)	(kNm)
1	11.8	68.6	0
4	-11.8	51.3	0



## Load combination: 1.0G + 1.0Q + 0.5S (Service)

Node	Fo	Moment	
	Fx Fz		Му
	(kN)	(kN)	(kNm)
1	8.5	49.7	0
4	-8.5	37	0

 $1.35G + 1.5_{\psi_0}Q + 1.5S$  (Strength) - Local node reactions Node: (Horiz (kN), Vert (kN), Mom (kNm))



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### Load combination: $1.35G + 1.5\psi_0Q + 1.5S$ (Strength)

Node	Fo	Moment	
	Fx (kN)	Fz (kN)	My (kNm)
1	11	64.1	0
4	-11	47.2	0



#### Load combination: $1.35G + 1.5Q + 1.5\psi_0S + 1.5\psi_0W$ (Strength)

Node	Fo	Moment	
	Fx Fz		Му
	(kN)	(kN)	(kNm)
1	9.8	67.9	0
4	-12.5	52.1	0



#### Load combination: 1.0G + 1.0Q + 0.5S + 0.5W (Service)

Node	Force		Moment
	Fx Fz		Му
	(kN)	(kN)	(kNm)
1	7.2	49.2	0
4	-9	37.5	0





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#### Load combination: $1.35G + 1.5\psi_0Q + 1.5S + 1.5\psi_0W$ (Strength)

Node	Fo	Moment	
	Fx	Fz	Му
	(kN)	(kN)	(kNm)
1	9.1	63.4	0
4	-11.8	47.9	0



#### Load combination: $1.0G + 1.0\psi_0Q + 1.0S + 0.5W$ (Service)

Node	Fo	Moment	
	Fx Fz		Му
	(kN)	(kN)	(kNm)
1	6.7	46.2	0
4	-8.5	34.7	0



#### Load combination: $1.35G + 1.5\psi_0Q + 1.5\psi_0S + 1.5W$ (Strength)

Node	Fo	Moment	
	Fx Fz		Му
	(kN)	(kN)	(kNm)
1	7.1	62.6	0
4	-12.5	48.7	0

 $1.0G + 1.0_{\psi_0}Q + 0.5S + 1.0W$  (Service) - Local node reactions Node: (Horiz (kN), Vert (kN), Mom (kNm))



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## Load combination: $1.0G + 1.0\psi_0Q + 0.5S + 1.0W$ (Service)

Node	Fo	Moment	
	Fx	Fz	Му
	(kN)	(kN)	(kNm)
1	5.4	45.7	0
4	-9	35.2	0



## Load combination: 1.0G + 1.5W (Strength)

Node	Fo	Moment	
	Fx (kN)	Fz (kN)	My (kNm)
1	2.9	38.2	0
4	-8.3	29.3	0



Load combination: 1.0G + 1.0W (Service)

Node	Fo	Moment	
	Fx Fz		Му
	(kN)	(kN)	(kNm)
1	4.3	38.7	0
4	-7.9	28.8	0

#### **Element end forces**

#### Load combination: 1.35G + 1.5Q + 1.5RQ (Strength)

Element	Length	Nodes	Axial force	Shear force	Moment
	(m)	Start/End	(kN)	(kN)	(kNm)
1	3	1	-67.8	11.8	-4
		2	66.6	-11.8	-31.3
2	5.4	2	-11.8	-66.6	31.3
		3	11.8	-48.8	-29.8
3	3	3	-48.8	-11.8	29.8
		4	50	11.8	5.5
4	5.4	1	0	-0.8	4
		4	0	-1.3	-5.5

#### Load combination: 1.0G + 1.0Q + 1.0RQ (Service)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	3	1	-49.1	8.5	-2.9
		2	48.2	-8.5	-22.7
2	5.4	2	-8.5	-48.2	22.7
		3	8.5	-35.1	-21.6
3	3	3	-35.1	-8.5	21.6
		4	36	8.5	4
4	5.4	1	0	-0.6	2.9

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Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
		4	0	-1	-4

#### Load combination: $1.0G + 1.0\psi_2Q$ (Quasi)

Element	Length	Nodes	Axial force	Shear force	Moment
	(m)	Start/End	(kN)	(kN)	(kNm)
1	3	1	-42.1	7.4	-2.5
		2	41.2	-7.4	-19.6
2	5.4	2	-7.4	-41.2	19.6
		3	7.4	-28.7	-18.6
3	3	3	-28.7	-7.4	18.6
		4	29.6	7.4	3.6
4	5.4	1	0	-0.6	2.5
		4	0	-1	-3.6

## Load combination: $1.35G + 1.5Q + 1.5\psi_0S$ (Strength)

Element	Length	Nodes	Axial force	Shear force	Moment
	(m)	Start/End	(kN)	(kN)	(kNm)
1	3	1	-67.8	11.8	-4
		2	66.6	-11.8	-31.3
2	5.4	2	-11.8	-66.6	31.3
		3	11.8	-48.8	-29.8
3	3	3	-48.8	-11.8	29.8
		4	50	11.8	5.5
4	5.4	1	0	-0.8	4
		4	0	-1.3	-5.5

# Load combination: 1.0G + 1.0Q + 0.5S (Service)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	3	1	-49.1	8.5	-2.9
		2	48.2	-8.5	-22.7
2	5.4	2	-8.5	-48.2	22.7
		3	8.5	-35.1	-21.6
3	3	3	-35.1	-8.5	21.6
		4	36	8.5	4
4	5.4	1	0	-0.6	2.9
		4	0	-1	-4

### Load combination: 1.35G + 1.5\v0Q + 1.5S (Strength)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	3	1	-63.3	11	-3.8
		2	62.1	-11	-29.3
2	5.4	2	-11	-62.1	29.3
		3	11	-44.6	-27.9
3	3	3	-44.6	-11	27.9
		4	45.8	11	5.2
4	5.4	1	0	-0.8	3.8
		4	0	-1.3	-5.2

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## Load combination: $1.35G + 1.5Q + 1.5\psi_0S + 1.5\psi_0W$ (Strength)

Element	Length	Nodes	Axial force	Shear force	Moment
	(m)	Start/End	(kN)	(kN)	(kNm)
1	3	1	-67.5	9.8	-2.8
		2	66.3	-12.5	-30.6
2	5.4	2	-12.5	-66.3	30.6
		3	12.5	-49.1	-31
3	3	3	-49.1	-12.5	31
		4	50.3	12.5	6.5
4	5.4	1	0	-0.4	2.8
		4	0	-1.8	-6.5

### Load combination: 1.0G + 1.0Q + 0.5S + 0.5W (Service)

Element	Length	Nodes	Axial force	Shear force	Moment
	(m)	Start/End	(kN)	(kN)	(kNm)
1	3	1	-48.9	7.2	-2.1
		2	48	-9	-22.2
2	5.4	2	-9	-48	22.2
		3	9	-35.3	-22.4
3	3	3	-35.3	-9	22.4
		4	36.2	9	4.7
4	5.4	1	0	-0.3	2.1
		4	0	-1.3	-4.7

## Load combination: $1.35G + 1.5\psi_0Q + 1.5S + 1.5\psi_0W$ (Strength)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	3	1	-63	9.1	-2.6
		2	61.8	-11.8	-28.6
2	5.4	2	-11.8	-61.8	28.6
		3	11.8	-45	-29
3	3	3	-45	-11.8	29
		4	46.2	11.8	6.2
4	5.4	1	0	-0.4	2.6
		4	0	-1.7	-6.2

Load combination:  $1.0G + 1.0\psi_0Q + 1.0S + 0.5W$  (Service)

Element	Length	Nodes	Axial force	Shear force	Moment
	(m)	Start/End	(kN)	(kN)	(kNm)
1	3	1	-45.9	6.7	-2
		2	45	-8.5	-20.9
2	5.4	2	-8.5	-45	20.9
		3	8.5	-32.6	-21.1
3	3	3	-32.6	-8.5	21.1
		4	33.5	8.5	4.5
4	5.4	1	0	-0.3	2
		4	0	-1.3	-4.5

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## Load combination: $1.35G + 1.5\psi_0Q + 1.5\psi_0S + 1.5W$ (Strength)

Element	Length	Nodes	Axial force	Shear force	Moment
	(m)	Start/End	(kN)	(kN)	(kNm)
1	3	1	-62.6	7.1	-1.4
		2	61.5	-12.5	-27.9
2	5.4	2	-12.5	-61.5	27.9
		3	12.5	-45.3	-30.2
3	3	3	-45.3	-12.5	30.2
		4	46.5	12.5	7.2
4	5.4	1	0	0	1.4
		4	0	-2.2	-7.2

## Load combination: 1.0G + 1.0y0Q + 0.5S + 1.0W (Service)

Element	Length	Nodes	Axial force	Shear force	Moment
	(m)	Start/End	(kN)	(kN)	(kNm)
1	3	1	-45.7	5.4	-1.2
		2	44.8	-9	-20.5
2	5.4	2	-9	-44.8	20.5
		3	9	-32.8	-21.9
3	3	3	-32.8	-9	21.9
		4	33.7	9	5.2
4	5.4	1	0	-0.1	1.2
		4	0	-1.5	-5.2

## Load combination: 1.0G + 1.5W (Strength)

Element	Length	Nodes	Axial force	Shear force	Moment
	(m)	Start/End	(kN)	(kN)	(kNm)
1	3	1	-38.4	2.9	0
		2	37.6	-8.3	-16.9
2	5.4	2	-8.3	-37.6	16.9
		3	8.3	-26.6	-19.6
3	3	3	-26.6	-8.3	19.6
		4	27.5	8.3	5.4
4	5.4	1	0	0.2	0
		4	0	-1.8	-5.4

Load combination: 1.0G + 1.0W (Service)

Element	Length	Nodes	Axial force	Shear force	Moment
	(m)	Start/End	(kN)	(kN)	(kNm)
1	3	1	-38.7	4.3	-0.8
		2	37.8	-7.9	-17.4
2	5.4	2	-7.9	-37.8	17.4
		3	7.9	-26.4	-18.8
3	3	3	-26.4	-7.9	18.8
		4	27.3	7.9	4.7
4	5.4	1	0	-0.1	0.8
		4	0	-1.5	-4.7

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Forces











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## **Envelope - Strength combinations**

Member	Shear force		Moment				
	Pos	Max abs	Pos	Max	Pos	Min	
	(m)	(kN)	(m)	(kNm)	(m)	(kNm)	
Member1	0	66.6	1.998	83.8	0	-31.3	

**Envelope - All combinations** 

Member	Axial force					
	Pos	Max	Pos	Min		
	(m)	(kN)	(m)	(kN)		
Member1	0	12.5	0	7.4		

**Envelope - Service combinations** 

Member	Deflection					
	Pos	Max	Pos	Min		
	(m)	(mm)	(m)	(mm)		
Member1	2.494	12.2	5.4	0.1		

**Envelope - All combinations** 

Member	Axial deflection					
	Pos	Max	Pos	Min		
	(m)	(mm)	(m)	(mm)		
Member1	0	3.3	5.4	1		

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Load combination: 1.35G + 1.5Q + 1.5RQ (Strength)

Member	Shear	ar force Moment				
	Pos	Max abs	Pos	Max	Pos	Min
	(m)	(kN)	(m)	(kNm)	(m)	(kNm)
Member1	0	66.6	1.998	83.8	0	-31.3

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### Load combination: 1.35G + 1.5Q + 1.5RQ (Strength)

Member	Axial force					
	Pos	Pos Max Pos		Min		
	(m)	(kN)	(m)	(kN)		
Member1	0	11.8	0	11.8		

# Load combination: 1.35G + 1.5Q + 1.5RQ (Strength)

Member	Axial deflection					
	Pos Max Pos		Min			
	(m)	(mm)	(m)	(mm)		
Member1	0	1.5	5.4	1.4		

1.0G + 1.0Q + 1.0RQ (Service) - Axial force (kN)





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## Load combination: 1.0G + 1.0Q + 1.0RQ (Service)

Member	Axial force				
	Pos	Max	Pos	Min	
	(m)	(kN)	(m)	(kN)	
Member1	0	8.5	0	8.5	

Load combination: 1.0G + 1.0Q + 1.0RQ (Service)

Member	Deflection						
	Pos	Max	Pos	Min			
	(m)	(mm)	(m)	(mm)			
Member1	2.494	12.2	5.4	0.1			

### Load combination: 1.0G + 1.0Q + 1.0RQ (Service)

Member		Axial deflection					
	Pos	Pos Max Pos		Min			
	(m)	(mm)	(m)	(mm)			
Member1	0	1.1	5.4	1			





29.6

Member results

Load combination: 1.0G + 1.0y2Q (Quasi)

42.1

Member	Axial force					
	Pos	Max	Pos	Min		
	(m)	(kN)	(m)	(kN)		
Member1	0	7.4	0	7.4		

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# Load combination: $1.0G + 1.0\psi_2Q$ (Quasi)

Member	Axial deflection					
	Pos Max Pos		Min			
	(m)	(mm)	(m)	(mm)		
Member1	0	1	5.4	1		







1.35G + 1.5Q + 1.5 $\psi_0 S$  (Strength) - Axial deflection

50

1.4 0.2

67.8

1.5

-0.3



Member	Shea	r force	Moment				
	Pos	Max abs	Pos	Max	Pos	Min	
	(m)	(kN)	(m)	(kNm)	(m)	(kNm)	
Member1	0	66.6	1.998	83.8	0	-31.3	

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# Load combination: 1.35G + 1.5Q + 1.5 $\psi_0$ S (Strength)

Member	Axial force						
	Pos	Max	Pos	Min			
	(m)	(kN)	(m)	(kN)			
Member1	0	11.8	0	11.8			

# Load combination: $1.35G + 1.5Q + 1.5\psi_0S$ (Strength)

49.1

Member		Axial deflection						
	Pos	Max	Pos	Min				
	(m)	(mm)	(m)	(mm)				
Member1	0	1.5	5.4	1.4				



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Load combination: 1.0G + 1.0Q + 0.5S (Service)

Member		Axial force							
	Pos Max		Pos	Min					
	(m)	(kN)	(m)	(kN)					
Member1	0	8.5	0	8.5					

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## Load combination: 1.0G + 1.0Q + 0.5S (Service)

Deflection					
Pos	Max	Pos	Min		
(m)	(mm)	(m)	(mm)		
2.494	12.2	5.4	0.1		
	<b>Pos</b> (m) 2.494	Defie           Pos         Max           (m)         (mm)           2.494         12.2	Deflection           Pos         Max         Pos           (m)         (m)         (m)           2.494         12.2         5.4		

# Load combination: 1.0G + 1.0Q + 0.5S (Service)

Member	Axial deflection						
	Pos	Max	Pos	Min			
	(m)	(mm)	(m)	(mm)			
Member1	0	1.1	5.4	1			

1.35G + 1.5 $\psi_0$ Q + 1.5S (Strength) - Moment (kNm)











Load combination:  $1.35G + 1.5\psi_0Q + 1.5S$  (Strength)

Member	Shear force		Moment					
	Pos	Max abs	Pos	Max	Pos	Min		
	(m)	(kN)	(m)	(kNm)	(m)	(kNm)		
Member1	0	62.1	1.998	79.5	0	-29.3		

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# Load combination: $1.35G + 1.5\psi_0Q + 1.5S$ (Strength)

Member	Axial force						
	Pos	Max	Pos	Min			
	(m)	(kN)	(m)	(kN)			
Member1	0	11	0	11			

# Load combination: 1.35G + 1.5y0Q + 1.5S (Strength)

Member	Axial deflection						
	Pos	Max	Pos	Min			
	(m)	(mm)	(m)	(mm)			
Member1	0	1.4	5.4	1.4			

 $1.35G + 1.5Q + 1.5\psi_0 S + 1.5\psi_0 W$  (Strength) - Moment (kNm)



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### Load combination: $1.35G + 1.5Q + 1.5\psi_0S + 1.5\psi_0W$ (Strength)

Member	Shear force		Moment					
	Pos	Max abs	Pos	Max	Pos	Min		
	(m)	(kN)	(m)	(kNm)	(m)	(kNm)		
Member1	0	66.3	1.998	83.8	5.4	-31		

Load combination:  $1.35G + 1.5Q + 1.5\psi_0S + 1.5\psi_0W$  (Strength)

Member	Axial force							
	Pos	Pos Max Pos						
	(m)	(kN)	(m)	(kN)				
Member1	0	12.5	0	12.5				

### Load combination: $1.35G + 1.5Q + 1.5\psi_0S + 1.5\psi_0W$ (Strength)

Member	Axial deflection							
	Pos	Pos Max Pos Mir						
	(m)	(mm)	(m)	(mm)				
Member1	0	2.4	5.4	2.3				

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### Load combination: 1.0G + 1.0Q + 0.5S + 0.5W (Service)

Member	Axial force						
	Pos Max Pos (m) (kN) (m)		Min (kN)				
Member1	0	9	0	9			

Load combination: 1.0G + 1.0Q + 0.5S + 0.5W (Service)

Member	Deflection							
	Pos	Max	Pos	Min				
	(m) (mm)		(m)	(mm)				
Member1	2.488	12.2	5.4	0.1				

Load combination: 1.0G + 1.0Q + 0.5S + 0.5W (Service)

Member	Axial deflection						
	Pos	Pos Max Pos Min					
	(m)	(mm)	(m)	(mm)			
Member1	0	1.7	5.4	1.7			

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## Load combination: $1.35G + 1.5\psi_0Q + 1.5S + 1.5\psi_0W$ (Strength)

Member	Shear force			Moment					
	Pos	Max abs	Pos	Max	Pos	Min			
	(m)	(kN)	(m)	(kNm)	(m)	(kNm)			
Member1	0	61.8	1.998	79.5	5.4	-29			

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# Load combination: 1.35G + 1.5 $\psi_0$ Q + 1.5S + 1.5 $\psi_0$ W (Strength)

Member	Axial force						
	Pos	Max	Pos	Min			
	(m)	(kN)	(m)	(kN)			
Member1	0	11.8	0	11.8			

# Load combination: $1.35G + 1.5\psi_0Q + 1.5S + 1.5\psi_0W$ (Strength)

Member	Axial deflection						
	Pos	Max	Pos	Min			
	(m)	(mm)	(m)	(mm)			
Member1	0	2.4	5.4	2.3			

1.0G +  $1.0_{\psi_0}\text{Q}$  + 1.0S + 0.5W (Service) - Axial force (kN)









Load combination: 1.0G + 1.0 $\psi_0$ Q + 1.0S + 0.5W (Service)

Member	Axial force						
	Pos	Max	Pos	Min			
	(m)	(kN)	(m)	(kN)			
Member1	0	8.5	0	8.5			

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# Load combination: $1.0G + 1.0\psi_0Q + 1.0S + 0.5W$ (Service)

Deflection						
Pos	Max	Pos	Min			
(m)	(mm)	(m)	(mm)			
2.48	11.5	5.4	0.1			
	Pos (m) 2.48	Defle           Pos         Max           (m)         (mm)           2.48         11.5	Deflection           Pos         Max         Pos           (m)         (mm)         (m)           2.48         11.5         5.4			

## Load combination: $1.0G + 1.0\psi_0Q + 1.0S + 0.5W$ (Service)

Member		Axial deflection							
	Pos	Max	Pos	Min					
	(m)	(mm)	(m)	(mm)					
Member1	0	1.7	5.4	1.6					





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### Load combination: $1.35G + 1.5\psi_0Q + 1.5\psi_0S + 1.5W$ (Strength)

Member	Shear force		Moment					
	Pos	Max abs	Pos	Max	Pos	Min		
	(m)	(kN)	(m)	(kNm)	(m)	(kNm)		
Member1	0	61.5	1.998	79.5	5.4	-30.2		

Load combination:  $1.35G + 1.5\psi_0Q + 1.5\psi_0S + 1.5W$  (Strength)

Member	Axial force						
	Pos	Max	Pos	Min			
	(m)	(kN)	(m)	(kN)			
Member1	0	12.5	0	12.5			

## Load combination: $1.35G + 1.5\psi_0Q + 1.5\psi_0S + 1.5W$ (Strength)

Member	Axial deflection							
	Pos	Max	Pos	Min				
	(m)	(mm)	(m)	(mm)				
Member1	0	3.3	5.4	3.3				

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## Load combination: $1.0G + 1.0\psi_0Q + 0.5S + 1.0W$ (Service)

Member	Axial force					
	Pos (m)	Max (kN)	Pos (m)	Min (kN)		
Member1	0	9	0	9		

Load combination:  $1.0G + 1.0\psi_0Q + 0.5S + 1.0W$  (Service)

Member	Deflection						
	Pos	Max	Pos	Min			
	(m)	(mm)	(m)	(mm)			
Member1	2.474	11.4	5.4	0.1			

### Load combination: $1.0G + 1.0\psi_0Q + 0.5S + 1.0W$ (Service)

Member		Axial deflection						
	Pos Max Pos		Pos	Min				
	(m)	(mm)	(m)	(mm)				
Member1	0	2.3	5.4	2.3				

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Member results

Load combination: 1.0G + 1.5W (Strength)

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Member	Shear	force		Mor	nent	
	Pos	Max abs	Pos	Max	Pos	Min
	(m)	(kN)	(m)	(kNm)	(m)	(kNm)
Member1	0	37.6	1.998	51.4	5.4	-19.6

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# Load combination: 1.0G + 1.5W (Strength)

Member	Axial force					
	Pos	Max	Pos	Min		
	(m)	(kN)	(m)	(kN)		
Member1	0	8.3	0	8.3		

# Load combination: 1.0G + 1.5W (Strength)

Member	Axial deflection					
	Pos Max Pos		Min			
	(m)	(mm)	(m)	(mm)		
Member1	0	2.9	5.4	2.9		





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Load combination: 1.0G + 1.0W (Service)

Member	Axial force					
	Pos (m)	Max (kN)	Pos (m)	Min (kN)		
Member1	0	7.9	0	7.9		

Load combination: 1.0G + 1.0W (Service)

Member	Deflection						
	Pos	Max	Pos	Min			
	(m)	(mm)	(m)	(mm)			
Member1	2.449	9.8	5.4	0.1			

Load combination: 1.0G + 1.0W (Service)

Member	Axial deflection					
	Pos (m)	Max (mm)	Pos (m)	Min (mm)		
Member1	0	2.3	5.4	2.2		
;						

Partial factors - Section 6.	Partial	factors	- Section	6.1
------------------------------	---------	---------	-----------	-----

Resistance of cross-sections;	$\gamma_{M0} = 1$
Resistance of members to instability;	γ <sub>M1</sub> = <b>1</b>
Resistance of tensile members to fracture;	γ <sub>M2</sub> = <b>1.1</b>

## <u>Member1 - Span 1</u>

Section details	
Section type;	UB 254x146x37 (BS4-1)
Steel grade - EN 10025-2:2004;	S275
Nominal thickness of element;	$t_{nom} = max(t_{f}, t_{w}) = 10.9 mm$
Nominal yield strength;	f <sub>y</sub> = <b>275</b> N/mm <sup>2</sup>

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Nominal ultimate tensile strength	ı.	fu = <b>410</b> N/	mm <sup>2</sup>			
Modulus of elasticity;	.,	E = 21000	<b>0</b> N/mm²			
.05 0.9						
<u></u>		UB	254x146x37 (BS4-1)			
↑		Sec Sec	tion depth, h, 256 mm tion breadth, b, 146.4 mm			
		Mas Flar	s of section, Mass, 37 kg/ı nge thickness, t <sub>r</sub> , 10.9 mm	m		
		Wel	o thickness, t <sub>w</sub> , 6.3 mm ot radius, r, 7.6 mm			
		Area Rac	a of section, A, 4717 mm² lius of gyration about v-axis	s. i . 108 mm		
- 256-		-6.3 Rad	lius of gyration about z-axis	y s, i <sub>2</sub> , 35 mm v-axis W, 432562 mm <sup>3</sup>		
		Elas Elas	stic section modulus about	z-axis, W <sub>el.z</sub> , 77957 mm <sup>3</sup>		
		Plas	stic section modulus about	z-axis, W <sub>pl.y</sub> , 119374 mm <sup>3</sup>		
0		Sec Sec	ond moment of area about ond moment of area about	: y-axis, l <sub>y</sub> , 55367902 mm <sup>4</sup> : z-axis, l <sub>z</sub> , 5706456 mm <sup>4</sup>		
↓ 10						
$\downarrow$ $\stackrel{\sim}{\uparrow}$						
	4	<b>──</b> ►				
Classification of cross sections - Section 5.5 $\epsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]} = 0.92$ Internal compression parts subject to bending and compression - Table 5.2 (sheet 1 of 3)Width of section; $c = d = 219 \text{ mm}$ $\alpha = \min([h / 2 + N_{Ed} / (2 \times t_w \times f_v) - (t_f + r)] / c. 1) = 0.516$						
		c / t <sub>w</sub> = 34.8	$8 = 37.6 \times \varepsilon <= 3$	$396 \times \epsilon / (13 \times \alpha -$	1); Class	1
Outstand flanges - Table 5.2 (s	sheet 2 of 3)					
Width of section;		c = (b - t <sub>w</sub> -	2 × r) / 2 = <b>62.5</b>	mm		
		c / t <sub>f</sub> = 5.7 :	$= 6.2 \times \varepsilon \le 9 \times \varepsilon$	ε; Class 1		
					Sect	ion is class 1
Check compression - Section	6.2.4					
Design compression force;	0.40	N <sub>Ed</sub> = <b>12.5</b>	kN			
Design resistance of section - ec	q 6.10;	$N_{c,Rd} = N_{pl,l}$	$Rd = A \times f_y / \gamma_{M0} =$	= <b>1297.1</b> kN		
		NEd / Nc,Rd	= U.U1	racistanca ava	oode dooian	comprossion
<b>0</b>		FASS - Desig	n compression		eeus uesign	compression
Slenderness ratio for y-y axis	flexural buckli	ng - Section 6.	3.1.3			
Critical buckling length;		$L_{cr,y} = L_{m1_s}$	s1 = <b>3400</b> mm	025 A LN		
Slenderness ratio for buckling	ea 6 50.	$\overline{\lambda}_{i} = \sqrt{\Delta}$	$L \times Iy / Lcr, y^- = 3$	4 AIN		
Obeels was said flow with the	oq 0.00,	$Ay = v(A \times A)$	(v, v) = 0.37	7		
Check y-y axis flexural bucklin	ng resistance -	Section 6.3.1.1	I			
Imperfection factor - Table 6.2;		a ~_^_1				
Ruckling reduction determination	a factor:	$u_y = 0.21$	$1 + \alpha_{\rm v} \times (\overline{2} - \alpha_{\rm v})$	$2) + \overline{2} + 2 = 0.70$	4	
Buckling reduction factor - eq.6	19.	$\psi y = 0.5 \times (0.5 \times 10^{-1})$	$1 + uy \times (\Lambda y - 0)$ / ( $\Delta u + \sqrt{(\Delta x^2 - \overline{\lambda})}$	) + λy ) = 0.70 ( <sup>2</sup> )) 1) - <b>ΛΟ</b>	•	
Design buckling resistance - eq.	, 6.47:	$\lambda y = mm(1)$ Nb y Bd = $\gamma y$		166.8 kN		
	,	NEd / Nb.v Br	d = 0.011			

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		PASS - L	Design buckling	g resistance exc	eeds design	compression			
Slenderness ratio for z-z axis	flexural buckli	na - Section 6.3	3.1.3						
Critical buckling length:		$L_{cr,z} = L_{m1}$	s1_seg1 = <b>5400</b> mr	m					
Critical buckling force:		$N_{\rm or  z} = \pi^2 \times$	$F \times I_7 / I_{or} z^2 = 4$	05.6 kN					
Slenderness ratio for buckling -	ea 6 50:	$\overline{\lambda}_{7} = \sqrt{(A)}$	$(f_v / N_{or}) = 1.78$	8					
	• •			•					
Check z-z axis flexural bucklin	ng resistance -	Section 6.3.1.1							
Buckling curve - Table 6.2;		D							
Imperfection factor - Table 6.1;		$\alpha_z = 0.34$			_				
Buckling reduction determinatio	n factor;	$\phi_z = 0.5 \times ($	$1 + \alpha_z \times (\lambda_z - 0)$	$(.2) + \lambda_z^2) = 2.36$	9				
Buckling reduction factor - eq 6.	49;	$\chi_z = \min(1)$	$/(\phi_z + \sqrt{\phi_z^2} - \lambda_z)$	z <sup>2</sup> )), 1) = <b>0.255</b>					
Design buckling resistance - eq	6.47;	$N_{b,z,Rd} = \chi_z$	$\times A \times f_y / \gamma_{M1} = 3$	<b>330.7</b> kN					
		N <sub>Ed</sub> / N <sub>b,z,R</sub>	d = <b>0.038</b>	_					
		PASS - L	Design buckling	g resistance exc	eeds design	compression			
Check torsional and torsional	-flexural buckli	ng - Section 6.	3.1.4						
Torsional buckling length;		$L_{cr,T} = L_{m1}$	s1_seg1_R = <b>5400</b> I	mm					
Distance from shear centre to c	entroid in y axis	y <sub>0</sub> = <b>0.0</b> mi	y <sub>0</sub> = <b>0.0</b> mm						
Distance from shear centre to c	entroid in z axis	z <sub>0</sub> = <b>0.0</b> mi	z <sub>0</sub> = <b>0.0</b> mm						
Radius of gyration;	Radius of gyration;								
Elastic critical torsional buckling	Elastic critical torsional buckling force;			$\times E \times I_w / L_{cr,T^2}) =$	1426.8 kN				
Torsion factor;	Torsion factor; $\beta_T = 1 - (y_0 / i_0)^2 = 1$								
Elastic critical torsional-flexural	ouckling force								
N <sub>cr,TF</sub> =	$N_{cr,y}$ / (2 $ imes$ $\beta_T$ ) >	< [1 + N <sub>cr,T</sub> / N <sub>cr,y</sub>	- √[(1 - N <sub>cr,T</sub> / N <sub>c</sub>	$(x_{cr,y})^2 + 4 \times (y_0 / i_0)$	$^{2} \times N_{cr,T} / N_{cr,y}$	] = <b>1426.8</b> kN			
Elastic critical buckling force;		N <sub>cr</sub> = min(N	$N_{cr,T}$ , $N_{cr,TF}$ ) = 142	2 <b>6.8</b> kN					
Slenderness ratio for torsional b	uckling - eq 6.5	$2; \qquad \overline{\lambda}_{T} = \sqrt{[A>]}$	$(f_y / N_{cr}] = 0.953$	i i i i i i i i i i i i i i i i i i i					
Design resistance for torsion	al and torsiona	I-flexural buck	ing - Section 6.	3.1.1					
Buckling curve - Table 6.2;		b	U						
Imperfection factor - Table 6.1;		α <sub>T</sub> = <b>0.34</b>							
Buckling reduction determinatio	n factor;	$\phi_{\rm T} = 0.5 \times$	$(1 + \alpha_T \times (\overline{\lambda}_T - 0))$	$(0.2) + \overline{\lambda}T^2 = 1.08$	33				
Buckling reduction factor - eq 6.	49:	$\dot{\chi}_{T} = \min(1)$	/(φ <sub>T</sub> + √(φ <sub>T</sub> <sup>2</sup> - λ̄	$(\bar{L}_{T}^{2})), 1) = 0.627$					
Design buckling resistance - eg	6.47:	$N_{b,T,Bd} = \chi_T$	$\times \mathbf{A} \times \mathbf{f}_{v} / \gamma_{M1} = \mathbf{i}$	813 kN					
	,	N <sub>Ed</sub> / N <sub>b.T.B</sub>	d = 0.015						
		PASS - L	Design buckling	resistance exc	eeds design	compression			
			0 0		Ū	•			
Uneck design at start of span									
Check shear - Section 6.2.6									
Height of web;		h <sub>w</sub> = h - 2 >	< t <sub>f</sub> = <b>234.2</b> mm;	η = <b>1.00</b>	00				
		h <sub>w</sub> / t <sub>w</sub> = 37	$1.2 = 40.2 \times \epsilon / \eta$	< 72 × ε / η					
				Shear buckling	resistance ca	n be ignored			
Design shear force;		$V_{y,Ed} = 66.3$	<b>3</b> kN						
Shear area - cl 6.2.6(3);		$A_v = max(A)$	A - 2 $\times$ b $\times$ t <sub>f</sub> + (t <sub>w</sub>	$(+ 2 \times r) \times t_f, \eta \times$	h <sub>w</sub> × t <sub>w</sub> ) = <b>175</b>	<b>9</b> mm <sup>2</sup>			
Design shear resistance - cl 6.2	.6(2);	$V_{c,y,Rd} = V_p$	$I_{y,Rd} = A_v \times (f_y / \gamma)$	/(3)) / γ <sub>M0</sub> = <b>279.3</b>	8 kN				
		V <sub>y,Ed</sub> / V <sub>c,y,I</sub>	Rd = <b>0.237</b>						
		PAS	SS - Design she	ear resistance ex	xceeds desig	n shear force			
Check bending moment - Sec	tion 6.2.5								
Design bending moment;		$M_{y,Ed} = 30.$	<b>6</b> kNm						
Design bending resistance mor	ent - eq 6.13;	$M_{c,y,Rd} = M$	$p_{\text{I},y,\text{Rd}} = W_{\text{PI},y} \times f_y$	/ γ <sub>M0</sub> = <b>132.9</b> kNr	n				
-	-	M <sub>y,Ed</sub> / M <sub>c,y</sub>	,Rd = <b>0.23</b>						

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#### PASS - Design bending resistance moment exceeds design bending moment

#### Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6;	kc = <b>0.909</b>
	$C_1 = 1 / k_c^2 = 1.211$
Poissons ratio;	v = <b>0.3</b>
Shear modulus;	$G = E / [2 \times (1 + v)] = 80769 \text{ N/mm}^2$
Unrestrained effective length;	$L = 1.0 \times L_{m1\_s1\_seg1\_B} = 5400 \text{ mm}$
Elastic critical buckling moment;	$M_{cr} = C_1 \times \pi^2 \times E \times I_z \ / \ L^2 \times \sqrt{(I_w \ / \ I_z + L^2 \times G \times I_t \ / \ (\pi^2 \times E \times I_z))} = \textbf{104.8}$
	kNm
Slenderness ratio for lateral torsional buckling;	$\overline{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})} = 1.126$
Limiting slenderness ratio;	$\overline{\lambda}_{LT,0} = 0.4$
	$\overline{\lambda}_{LT} > \overline{\lambda}_{LT,0}$ - 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 <sub>LT</sub> = <b>0.34</b>
Correction factor for rolled sections;	$\beta = 0.75$
LTB reduction determination factor;	$\phi_{\text{LT}} = 0.5 \times [1 + \alpha_{\text{LT}} \times (\overline{\lambda}_{\text{LT}} - \overline{\lambda}_{\text{LT},0}) + \beta \times \overline{\lambda}_{\text{LT}}^2] = 1.099$
LTB reduction factor - eq 6.57;	$\chi_{\text{LT}} = min(1 / [\phi_{\text{LT}} + \sqrt{(\phi_{\text{LT}}^2 - \beta \times \overline{\lambda}_{\text{LT}}^2)}], 1, 1 / \overline{\lambda}_{\text{LT}}^2) = 0.623$
Modification factor;	f = min(1 - 0.5 × (1 - k <sub>c</sub> ) × [1 - 2 × ( $\overline{\lambda}_{LT}$ - 0.8) <sup>2</sup> ], 1) = <b>0.964</b>
Modified LTB reduction factor - eq 6.58;	$\chi_{\text{LT,mod}} = min(\chi_{\text{LT}} / f, 1, 1 / \overline{\lambda}_{\text{LT}}^2) = 0.646$
Design buckling resistance moment - eq 6.55;	$M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl,y} \times f_y \ / \ \gamma_{M1} = \textbf{85.8 kNm}$
	M <sub>y,Ed</sub> / M <sub>b,y,Rd</sub> = 0.356

#### PASS - Design buckling resistance moment exceeds design bending moment

### Check bending and axial force - Section 6.2.9

 $Bending \ and \ axial \ force \ check \ - \ eq. 6.33 \ \& \ eq. 6.34; \quad N_{y,lim} = min(0.25 \times N_{pl,Rd}, \ 0.5 \times h_w \times t_w \times f_y \ / \ \gamma_{M0}) = \textbf{202.9} \ kN = 1000 \ km^2 \$ 

#### $N_{Ed} / N_{y,lim} = 0.062$

Allowance need not be made for the effect of the axial force on the plastic resistance moment about the y-y axis

#### Check combined bending and compression - Section 6.3.3

Equivalent uniform moment factors - Table B.3;	ψ <sub>y</sub> = -30.6 kNm / -31 kNm = <b>0.987</b>
	α <sub>y</sub> = -31 kNm / 68.7 kNm = <b>-0.451</b>
	$C_{my} = 0.95 + 0.05 \times \alpha_y = 0.927$
,	ψ <sub>LT</sub> = -30.6 kNm / -31 kNm = <b>0.987</b>
	α <sub>LT</sub> = -31 kNm / 68.7 kNm = <b>-0.451</b>
	$C_{mLT} = 0.95 + 0.05 \times \alpha_{LT} = \textbf{0.927}$
Interaction factors k <sub>ij</sub> for members susceptible	to torsional deformations - Table B.2
Characteristic moment resistance;	$M_{y,Rk} = W_{pl.y} \times f_y = \textbf{132.9} \text{ kNm}$
Characteristic moment resistance;	$M_{z,Rk} = W_{pl.z} \times f_y = \textbf{32.8 kNm}$
Characteristic resistance to normal force;	$N_{Rk} = A \times f_y = $ <b>1297.1</b> kN
Interaction factors;	$k_{yy}$ = $C_{my}$ $ imes$ (1 + min( $\overline{\lambda}_y$ - 0.2, 0.8) $ imes$ $N_{Ed}$ / ( $\chi_y$ $ imes$
	N <sub>Rk</sub> / γ <sub>M1</sub> )) = <b>0.931</b>
	$k_{zy} = 1 - 0.1 \times min(1, \ \overline{\lambda}_z) \times N_{Ed} \ / \ ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} \ / \ \gamma_{M1}) = \textbf{0.994}$
Interaction formulae - eq 6.61 & eq 6.62;	N <sub>Ed</sub> / ( $\chi_y  imes N_{Rk}$ / $\gamma_{M1}$ ) + k <sub>yy</sub> $ imes$
	$M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) = 0.225$
	$N_{Ed}$ / ( $\chi_z \times N_{Rk}$ / $\gamma_{M1}$ ) + $k_{zy} \times M_{y,Ed}$ / ( $\chi_{LT} \times M_{y,Rk}$ / $\gamma_{M1}$ ) = 0.267
	PASS - Combined bending and compression checks are satisfied

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www.sd-structures.com	RC	26/06/2017	MD	26/06/2017				
Check design 1998 mm along s	nan							
Chock shoar - Section 6.2.6	pun							
Height of web:		h h - 2 \	∕ t₄ – <b>234 2</b> mn	n: n – <b>1 O</b>	00			
neight of web,		hw / tw - 37	/ u = <b>234.2</b> mm	$n < 72 \times s/n$	00			
		10% $10%$ $-07$	.2 = 40.2 × 07	Shear buckling	resistance	can be ignored		
Design shear force:		V <sub>v.Ed</sub> = <b>48.</b>	<b>2</b> kN	enear buening		can be ignored		
Shear area - cl 6.2.6(3);		$A_v = max(A)$	$-2 \times b \times t_{f} + 0$	$(t_w + 2 \times r) \times t_f, \eta \times$	$h_w \times t_w) = 17$	<b>759</b> mm²		
Design shear resistance - cl 6.2.6	(2);	$V_{c,v,Rd} = V_p$	$I_{v,Rd} = A_v \times (f_v)$	$\sqrt{\sqrt{(3)}} / \gamma_{M0} = 279.3$	3 kN			
-		V <sub>y,Ed</sub> / V <sub>c,y,F</sub>	Rd = <b>0.173</b>					
		PAS	SS - Design sl	hear resistance e	xceeds des	ign shear force		
Check bending moment - Section	on 6.2.5							
Design bending moment;		$M_{y,Ed} = 83.$	<b>8</b> kNm					
Design bending resistance mome	nt - eq 6.13;	$M_{c,y,Rd} = M$	$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = 132.9 \text{ kNm}$					
		$M_{y,Ed} / M_{c,y}$	M <sub>y,Ed</sub> / M <sub>c,y,Rd</sub> = <b>0.631</b>					
	PASS	- Design bend	ing resistance	e moment exceed	ls design be	ending moment		
Slenderness ratio for lateral tor	sional buckli	ng						
Correction factor - Table 6.6; $k_c = 0.909$								
		$C_1 = 1 / k_c^2$	<sup>2</sup> = 1.211					
Poissons ratio;		v = 0.3						
Shear modulus;		G = E / [2 :	(1 + v) = 80	<b>769</b> N/mm²				
Unrestrained effective length;		$L = 1.0 \times L$	m1_s1_seg1_T = 5	<b>400</b> mm				
Elastic critical buckling moment;		$M_{cr}=C_1 \times$	$\pi^2 \times E \times I_z / L^2$	$\times \sqrt{(I_w / I_z + L^2 \times G)}$	$i \times I_t / (\pi^2 \times E)$	× lz)) = <b>104.8</b>		
		kNm						
Slenderness ratio for lateral torsio	nal buckling;	$\lambda_{LT} = \sqrt{W}$	$p_{I,y} \times f_y / M_{cr}) =$	1.126				
Limiting slenderness ratio;		λ <sub>LT,0</sub> = <b>0.4</b>						
			$\lambda_{LT} > \lambda_{LT,0} - L$	ateral torsional b	ouckling car	not be ignored		
Check buckling resistance - Se	ction 6.3.2.1							
Buckling curve - Table 6.5;		b						
Imperfection factor - Table 6.3;		$\alpha_{LT} = 0.34$						
Correction factor for rolled section	IS;	$\beta = 0.75$		<del>.</del>	0			
LTB reduction determination facto	or;	$\phi_{LT} = 0.5 \times$	$[1 + \alpha_{LT} \times (\lambda_{I})]$	$T - \lambda_{LT,0} + \beta \times \lambda_{L}$	<sup>2</sup> ] = <b>1.099</b>			
LTB reduction factor - eq 6.57;		χ <sub>LT</sub> = min(1	I / [фlт + √(фlт²	$-\beta \times \lambda_{LT^2}$ ], 1, 1 /	λ <sub>LT</sub> <sup>2</sup> ) = <b>0.62</b>	23		
Modification factor;		f = min(1 -	0.5 × (1 - k <sub>c</sub> ) ×	[1 - 2 × (λ <sub>LT</sub> - 0.8	3) <sup>2</sup> ], 1) = <b>0.96</b>	64		
Modified LTB reduction factor - ec	1 6.58; 	$\chi_{LT,mod} = m$	$\ln(\chi_{LT} / f, 1, 1)$	$\lambda_{LT}^{2}$ ) = <b>0.646</b>				
Design buckling resistance mome	ent - eq 6.55;	5; $M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl,y} \times f_y / \gamma_{M1} = 85.8 \text{ kNm}$						
	DACC	My,Ed / Mb,y	,Rd = <b>0.976</b>	momentexees	la daaign h	nding momon		
Observations and excitat former	PASS	- Design bucki	ing resistance	e moment exceed	is design be	enaing moment		
	- Section 6.2.	<b>7</b>						
Check combined bending and c	compression	- Section 6.3.3		0.00-				
Equivalent uniform moment factor	s - Table B.3;	$\psi_y = -30.6$	кіNm / -31 kNn	n = 0.987				
		$\alpha_y = -31 \text{ kM}$	NTT / 68.7 KNM	= -U.451				
		Cmy = 0.95	+ $0.05 \times \alpha_{\rm V} =$	0.927				

$$\begin{split} \psi_{\text{LT}} &= -30.6 \ \text{kNm} \ / \ \text{-31} \ \text{kNm} = \textbf{0.987} \\ \alpha_{\text{LT}} &= -31 \ \text{kNm} \ / \ 68.7 \ \text{kNm} = \textbf{-0.451} \\ C_{\text{mLT}} &= 0.95 + 0.05 \times \alpha_{\text{LT}} = \textbf{0.927} \end{split}$$

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Interaction factors k <sub>ij</sub> for men	nbers susceptit	ole to torsional	deformations ·	- Table B.2					
Characteristic moment resistan	ce;	$M_{y,Rk}=W_{pl}$	.y × fy = <b>132.9</b> kl	Nm					
Characteristic moment resistance;		$M_{z,Rk} = W_{pl}$	.z × f <sub>y</sub> = <b>32.8</b> kN	m					
Characteristic resistance to nor	$N_{Rk} = A \times f_{f}$	y = <b>1297.1</b> kN							
Interaction factors;		$k_{yy} = C_{my} \times$	$(1 + min(\overline{\lambda}_y - 0))$	$0.2, 0.8)  imes N_{Ed} / (2)$	ζy×				
		N <sub>Rk</sub> / γ <sub>M1</sub> )) :	= 0.931						
		$k_{zy} = 1 - 0.7$	$1 \times \min(1, \overline{\lambda}_z) \times$	$N_{Ed}$ / (( $C_{mLT}$ - 0.2	25) $\times \chi_z \times N_{RI}$	<sub>κ</sub> / γ <sub>M1</sub> ) = <b>0.994</b>			
Interaction formulae - eq 6.61 & eq 6.62;		$N_{Ed}$ / ( $\chi_y$ $ imes$	N <sub>Rk</sub> / γ <sub>M1</sub> ) + k <sub>yy</sub> >	<					
		$M_{y,Ed}$ / ( $\chi_{LT}$	$\times$ M <sub>y,Rk</sub> / $\gamma$ M1) =	0.598					
		$N_{Ed}$ / ( $\chi_z$ $ imes$	N <sub>Rk</sub> / γ <sub>M1</sub> ) + k <sub>zy</sub> >	$ imes$ M <sub>y,Ed</sub> / ( $\chi_{LT}  imes$ M <sub>y</sub>	, <sub>Rk</sub> / γ <sub>M1</sub> ) = <b>0.</b>	665			
		PASS - C	ombined bend	ling and compre	ession checl	ks are satisfied			
Check design at end of span									
Check shear - Section 6.2.6									
Height of web;		h <sub>w</sub> = h - 2 >	< t <sub>f</sub> = <b>234.2</b> mm;	η = <b>1.0</b>	00				
		$h_w \ / \ t_w = 37.2 = 40.2 \times \epsilon \ / \ \eta < 72 \times \epsilon \ / \ \eta$							
				Shear buckling	resistance	can be ignored			
Design shear force;		$V_{y,Ed} = 49.1 \text{ KIN}$ $\Delta_{y} = \max(\Delta_{z} 2 \times h \times t_{0.1} + (t_{0.1} + 2 \times r) \times t_{0.1} n \times h_{0.1} \times t_{0.1}) = 1750 \text{ mm}^{2}$							
Shear area - cl 6.2.6(3);		$A_v = \max(A_v)$	$V_{0} = Ha_{N}(7 - 2 \wedge 0 \wedge u + (w + 2 \wedge 1) \wedge u, \eta \wedge 1w \wedge w) = 17.55$ HHF $V_{0} = V_{0} = V_{0} = A_{v} \times (f_{v} / \sqrt{(3)}) / \gamma_{0} = 279.3$ kN						
Design shear resistance - ci 6.2	0(2),	$V_{c,y,Rd} = V_{pl}$	$I_{y,Rd} = A_v \times (I_y / I_{z,Rd})$	$V(3)) / \gamma_{M0} = 219.$	<b>3</b> KIN				
		Vy,Ed / Vc,y,F	Rd = 0.170 SS - Design shi	ear resistance e	vreeds des	ian shear force			
Chook bonding moment - Sec	tion 6 2 5					.g			
Design bending moment:	;0011 0.2.5	M	kNm						
Design bending resistance mor	nent - ea 6 13 <sup>.</sup>	$M_{y,Ed} = O M$	oly Rd = Woly X fy	/ vwo = <b>132 9</b> kN	m				
Sesign bending resistance mor		My, Ed / Mc y	$M_{y,Ed} / M_{c,y,Rd} = 0.233$						
	PASS	- Design bendi	ing resistance	moment exceed	ls design be	ending moment			
Slenderness ratio for lateral t		•	•		•	0			
Correction factor - Table 6.6:	orsional buckli	na							
	orsional buckli	ng kc = 0.909							
,	orsional buckli	ng $k_c = 0.909$ $C_1 = 1 / k_c^2$	= <b>1.211</b>						
Poissons ratio;	orsional buckli	ng $k_c = 0.909$ $C_1 = 1 / k_c^2$ $v = 0.3$	e = 1.211						
Poissons ratio; Shear modulus;	orsional buckli	ng $k_c = 0.909$ $C_1 = 1 / k_c^2$ v = 0.3 G = E / [2 > 2]	<sup>2</sup> = 1.211 < (1 + ν)] = 8076	<b>69</b> N/mm²					
Poissons ratio; Shear modulus; Unrestrained effective length;	orsional buckli	ng $k_c = 0.909$ $C_1 = 1 / k_c^2$ v = 0.3 G = E / [2 > L = 1.0 × L	<sup>2</sup> = <b>1.211</b> < (1 + ν)] = <b>8076</b> m1_s1_seg1_B = <b>54</b>	<b>69</b> N/mm² <b>00</b> mm					
Poissons ratio; Shear modulus; Unrestrained effective length; Elastic critical buckling moment	orsional buckli	ng $k_c = 0.909$ $C_1 = 1 / k_c^2$ v = 0.3 G = E / [2 > L = 1.0 × L $M_{cr} = C_1 × 1$	f = 1.211 < (1 + v)] = 8076 $m_1s_1seg_1B = 54$ $\pi^2 \times E \times I_z / L^2 \times I_z$	<b>69</b> N/mm² <b>00</b> mm < √(Iw / Iz + L² × G	$1 \times I_t / (\pi^2 \times E)$	× lz)) = <b>104.8</b>			
Poissons ratio; Shear modulus; Unrestrained effective length; Elastic critical buckling moment	orsional buckli	ng $k_c = 0.909$ $C_1 = 1 / k_c^2$ v = 0.3 G = E / [2 > 2 > 2 > 2 > 2 > 2 > 2 > 2 > 2 > 2	f = 1.211 × (1 + v)] = 8076 m1_s1_seg1_B = 54 $\pi^2 \times E \times I_z / L^2 \times I_z$	<b>69</b> N/mm² <b>00</b> mm ⊲ √(I <sub>w</sub> / I <sub>z</sub> + L² × G	$ imes$ It / ( $\pi^2$ × E	× lz)) = <b>104.8</b>			
Poissons ratio; Shear modulus; Unrestrained effective length; Elastic critical buckling moment Slenderness ratio for lateral tors	orsional buckli ;; sional buckling;	ng $k_c = 0.909$ $C_1 = 1 / k_c^2$ v = 0.3 G = E / [2 > 2 > 2 = 1.0 × L $M_{cr} = C_1 × 1 = 1.0 × L$ $k_{Nm}$ $\overline{\lambda}_{LT} = \sqrt{W}$	f = 1.211 (1 + v)] = 8076 $m_{1_{s1_{seg1_B}}} = 54$ $\pi^2 \times E \times I_z / L^2 \times I_z$ $p_{l,y} \times f_y / M_{cr}) = 1$	<b>59</b> N/mm² 00 mm ⊲ √(Iw / Iz + L² × G I <b>.126</b>	$ imes$ It / ( $\pi^2$ × E	× lz)) = <b>104.8</b>			
Poissons ratio; Shear modulus; Unrestrained effective length; Elastic critical buckling moment Slenderness ratio for lateral tors _imiting slenderness ratio;	orsional buckli ;; sional buckling;	ng $k_c = 0.909$ $C_1 = 1 / k_c^2$ v = 0.3 G = E / [2 > 2 > 2 > 2 > 2 > 2 > 2 > 2 > 2 > 2	f = 1.211 (1 + v)] = 8076 $m_{s1_seg1_B} = 54$ $\pi^2 \times E \times I_z / L^2 \times I_z$ $p_{s1_s} \times f_y / M_{cr}) = 1$	<b>69</b> N/mm² <b>00</b> mm ⊲ √(I <sub>w</sub> / I <sub>z</sub> + L² × G I <b>.126</b>	$ imes$ It / ( $\pi^2$ × E	× lz)) = <b>104.8</b>			
Poissons ratio; Shear modulus; Jnrestrained effective length; Elastic critical buckling moment Slenderness ratio for lateral tors Limiting slenderness ratio;	orsional buckli ;; sional buckling;	$\begin{array}{l} \text{hg} \\ \text{k}_{\text{c}} = \textbf{0.909} \\ \text{C}_{1} = 1 \ / \ \text{k}_{\text{c}}^{2} \\ \text{v} = \textbf{0.3} \\ \text{G} = \text{E} \ / \ [2 \ > \\ \text{L} = 1.0 \ \times \text{L} \\ \text{M}_{\text{cr}} = \text{C}_{1} \ \times \text{s} \\ \text{kNm} \\ \overline{\lambda}_{\text{LT}} = \sqrt{(W)} \\ \overline{\lambda}_{\text{LT},0} = \textbf{0.4} \end{array}$	f = 1.211 (1 + v)] = 8076 $m_{1_{s1_{seg1_B}}} = 54$ $\pi^2 \times E \times I_z / L^2 \times I_z$ $p_{Ly} \times f_y / M_{cr}) = 1$ $J_{Ly}$ $\overline{\lambda_L}\tau > \overline{\lambda_L}\tau_r_o - La$	<b>59</b> N/mm² 00 mm < √(Iw / Iz + L² × G I.126 teral torsional b	$ imes$ I <sub>t</sub> / ( $\pi^2$ × E	× l <sub>z</sub> )) = <b>104.8</b> Inot be ignored			
Poissons ratio; Shear modulus; Unrestrained effective length; Elastic critical buckling moment Slenderness ratio for lateral tors Limiting slenderness ratio; Check buckling resistance - S	orsional buckli ;; sional buckling; Section 6.3.2.1	ng $k_c = 0.909$ $C_1 = 1 / k_c^2$ v = 0.3 G = E / [2 > L = 1.0 × L $M_{cr} = C_1 × T$ kNm $\overline{\lambda}_{LT} = \sqrt{(W)}$ $\overline{\lambda}_{LT,0} = 0.4$	f = 1.211 (1 + v)] = 8076 $m_{s1_{seg1_B}} = 54$ $\pi^2 \times E \times I_z / L^2 \times I_z / L^2 \times I_z$ $p_{sy} \times f_y / M_{cr}) = 1$ $\lambda_{LT} > \lambda_{LT,o} - La$	<b>69</b> N/mm² <b>00</b> mm ⊲ √(l <sub>w</sub> / l <sub>z</sub> + L² × G I <b>.126</b> teral torsional b	$1  imes I_t / (\pi^2  imes E$	× lz)) = 104.8 Inot be ignored			
Poissons ratio; Shear modulus; Unrestrained effective length; Elastic critical buckling moment Slenderness ratio for lateral tors Limiting slenderness ratio; <b>Check buckling resistance - \$</b> Buckling curve - Table 6.5;	sional buckling; sional buckling;	ng $k_c = 0.909$ $C_1 = 1 / k_c^2$ v = 0.3 G = E / [2 > 2 > 2 = 1.0 × L $M_{cr} = C_1 × 1 = 1.0 × L$ kNm $\overline{\lambda}_{LT} = \sqrt{W}$ $\overline{\lambda}_{LT,0} = 0.4$	f = 1.211 (1 + v)] = 8076 $m_{1_s1\_seg1\_B} = 54$ $\pi^2 \times E \times I_z / L^2 \times I_z / L^2 \times I_z$ $p_{1,y} \times f_y / M_{cr}) = 1$ $\lambda_{LT} > \overline{\lambda_{LT}} - La$	<b>69</b> N/mm² 00 mm ⊲ √(Iw / Iz + L² × G I.126 teral torsional b	$ imes$ I <sub>t</sub> / ( $\pi^2$ × E	× lz)) = 104.8 Inot be ignored			
Poissons ratio; Shear modulus; Unrestrained effective length; Elastic critical buckling moment Slenderness ratio for lateral tors Limiting slenderness ratio; <b>Check buckling resistance - S</b> Buckling curve - Table 6.5; Imperfection factor - Table 6.3;	orsional buckli ;; sional buckling; Section 6.3.2.1	ng $k_c = 0.909$ $C_1 = 1 / k_c^2$ v = 0.3 G = E / [2 > L = 1.0 × L $M_{cr} = C_1 × \pi$ kNm $\overline{\lambda}_{LT} = \sqrt{(W)}$ $\overline{\lambda}_{LT,0} = 0.4$ b $\alpha_{LT} = 0.34$	f = 1.211 < (1 + ν)] = 8076 m1_s1_seg1_B = 54 $π^2 × E × I_z / L^2 ×$ pl.y × fy / Mcr) = 1 $\overline{λ_L τ} > \overline{λ_L τ_r o} - La$	<b>59</b> N/mm² <b>00</b> mm ⊲√(Iw / Iz + L² × G I <b>.126</b> teral torsional b	× It / (π² × Ε buckling can	× lz)) = 104.8 mot be ignored			
Poissons ratio; Shear modulus; Unrestrained effective length; Elastic critical buckling moment Slenderness ratio for lateral tors Limiting slenderness ratio; <b>Check buckling resistance - S</b> Buckling curve - Table 6.5; Imperfection factor - Table 6.3; Correction factor for rolled secti	orsional buckli ;; sional buckling; Section 6.3.2.1 ons;	ng $k_c = 0.909$ $C_1 = 1 / k_c^2$ v = 0.3 G = E / [2 > L = 1.0 × L $M_{cr} = C_1 × T$ kNm $\overline{\lambda}_{LT} = \sqrt{(W)}$ $\overline{\lambda}_{LT,0} = 0.4$ b $\alpha_{LT} = 0.34$ $\beta = 0.75$	f = 1.211 (1 + v)] = 8076 $m_{1_s1\_seg1\_B} = 54$ $\pi^2 \times E \times I_z / L^2 \times I_z / L^2 \times I_z$ $p_{Ly} \times f_y / M_{cr}) = 1$ $\lambda_L \tau > \lambda_L \tau, o - La$	<b>69</b> N/mm² <b>00</b> mm ⊲ √(l <sub>w</sub> / l <sub>z</sub> + L² × G I. <b>126</b> teral torsional b	× It / (π² × Ε buckling can	× lz)) = 104.8 mot be ignored			
Poissons ratio; Shear modulus; Unrestrained effective length; Elastic critical buckling moment Slenderness ratio for lateral tors Limiting slenderness ratio; <b>Check buckling resistance - \$</b> Buckling curve - Table 6.5; Imperfection factor - Table 6.3; Correction factor for rolled secti LTB reduction determination fac	orsional buckli ;; sional buckling; Section 6.3.2.1 ons; ctor;	ng $k_{c} = 0.909$ $C_{1} = 1 / k_{c}^{2}$ $v = 0.3$ $G = E / [2 > 2 + 1.0 \times L$ $M_{cr} = C_{1} \times \pi$ $kNm$ $\overline{\lambda}_{LT} = \sqrt{(W)}$ $\overline{\lambda}_{LT,0} = 0.4$ $b$ $\alpha_{LT} = 0.34$ $\beta = 0.75$ $\phi_{LT} = 0.5 \times$	$f = 1.211$ $< (1 + v)] = 8076$ m1_s1_seg1_B = 54 $\pi^2 \times E \times I_z / L^2 \times I_z$	59 N/mm² 00 mm < √(Iw / Iz + L² × G I.126 teral torsional b	$1 \times l_t / (\pi^2 \times E)$ buckling can	× l <sub>z</sub> )) = 104.8 Inot be ignored			
Poissons ratio; Shear modulus; Unrestrained effective length; Elastic critical buckling moment Slenderness ratio for lateral tors Limiting slenderness ratio; <b>Check buckling resistance - \$</b> Buckling curve - Table 6.5; Imperfection factor - Table 6.3; Correction factor for rolled secti LTB reduction determination fac	orsional buckli ;; sional buckling; Section 6.3.2.1 ons; ctor;	ng $k_c = 0.909$ $C_1 = 1 / k_c^2$ v = 0.3 G = E / [2 > L = 1.0 × L $M_{cr} = C_1 × \pi$ kNm $\overline{\lambda}_{LT} = \sqrt{(W)}$ $\overline{\lambda}_{LT,0} = 0.4$ b $\alpha_{LT} = 0.34$ $\beta = 0.75$ $\phi_{LT} = 0.5 \times \chi_{LT} = min(1)$	$F = 1.211$ $((1 + v)] = 8076$ $m1_s1_seg1_B = 54$ $\pi^2 \times E \times I_Z / L^2 \times$ $pl.y \times fy / M_{cr}) = 1$ $\overline{\lambda_L \tau} > \overline{\lambda_L \tau}, o - La$ $[1 + \alpha_L \tau \times (\overline{\lambda_L \tau})]$ $[1 + \alpha_L \tau \times (\overline{\lambda_L \tau})]$	<b>69</b> N/mm <sup>2</sup> <b>00</b> mm $< \sqrt{(I_w / I_z + L^2 × G)}$ <b>1.126</b> <i>teral torsional b</i> teral torsional b teral torsional 1 b	$r^{2}] = 1.099$ $\overline{\lambda}_{LT^{2}}) = 0.62$	× l <sub>z</sub> )) = 104.8 not be ignored			
Poissons ratio; Shear modulus; Unrestrained effective length; Elastic critical buckling moment Slenderness ratio for lateral tors Limiting slenderness ratio; <b>Check buckling resistance - S</b> Buckling curve - Table 6.5; Imperfection factor - Table 6.3; Correction factor for rolled secti LTB reduction determination fac LTB reduction factor - eq 6.57; Modification factor;	orsional buckli ;; sional buckling; Section 6.3.2.1 ons; ctor;	ng $k_c = 0.909$ $C_1 = 1 / k_c^2$ v = 0.3 G = E / [2 > 2 + 2] $L = 1.0 \times L$ $M_{cr} = C_1 \times 1$ kNm $\overline{\lambda}_{LT} = \sqrt{W}$ $\overline{\lambda}_{LT,0} = 0.4$ b $\alpha_{LT} = 0.34$ $\beta = 0.75$ $\phi_{LT} = 0.5 \times 12$ $\chi_{LT} = min(1 + 2)$	$f = 1.211$ $< (1 + v)] = 8076$ $m_{1_{s1_{seg1_B}}} = 54$ $\pi^2 \times E \times I_z / L^2 \times I_z / L$	<b>69</b> N/mm <sup>2</sup> <b>00</b> mm $< \sqrt{(I_w / I_z + L^2 × G)}$ <b>1.126</b> <i>teral torsional b</i> $(-\overline{\lambda}_{LT,0}) + \beta × \overline{\lambda}_L$ $(-\overline{\beta} × \overline{\lambda}_L T^2)], 1, 1 / (1 - 2 × (\overline{\lambda}_L T - 0.8))$	$1 \times I_t / (\pi^2 \times E)$ buckling can $\bar{\lambda}_{LT^2} = 1.099$ $\bar{\lambda}_{LT^2} = 0.62$ $3)^2], 1) = 0.962$	× lz)) = 104.8 not be ignored			

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Design buckling resistance mor	ent - eq 6.55;	$M_{b,y,Rd} = \chi_L$ $M_{y,Ed} / M_{b,y}$	$T_{T,mod} \times W_{pl.y} \times f_{y}$ Rd = <b>0.361</b>	<sub>y</sub> / γ <sub>M1</sub> = <b>85.8</b> kNm	s design by	anding moment		
Check bending and axial force	- Section 6.2.9	) )	ny resistance	moment exceed	s design be			
Check combined bending and	compression	- Section 6.3.3						
Equivalent uniform moment factor	$\psi_{y} = -30.6$	ψ <sub>y</sub> = -30.6 kNm / -31 kNm = <b>0.987</b>						
	α <sub>y</sub> = -31 kNm / 68.7 kNm = <b>-0.451</b>							
	C <sub>my</sub> = 0.95	$C_{my} = 0.95 + 0.05 \times \alpha_y = 0.927$						
;		ψ <sub>LT</sub> = -30.6 kNm / -31 kNm = <b>0.987</b>						
		α <sub>LT</sub> = -31 kNm / 68.7 kNm = <b>-0.451</b>						
	$C_{mLT} = 0.95 + 0.05 \times \alpha_{LT} = 0.927$							
Interaction factors kij for mem	bers susceptib	le to torsional	deformations	- Table B.2				
Characteristic moment resistance;		$M_{y,Rk} = W_{pl.y} \times f_y = \textbf{132.9 kNm}$						
Characteristic moment resistance;		$M_{z,Rk} = W_{pl}$	$M_{z,Rk} = W_{pl.z} \times f_y = \textbf{32.8 kNm}$					
Characteristic resistance to normal force;		$N_{Rk} = A \times f_y = 1297.1 \text{ kN}$						
Interaction factors;		$k_{yy} = C_{my} \times$	$k_{yy} = C_{my} \times (1 + min(\ \overline{\lambda_y} - 0.2, 0.8) \times N_{Ed} / (\chi_y \times$					
		N <sub>Rk</sub> / γ <sub>M1</sub> )) :	= 0.931					
		$k_{zy} = 1 - 0.7$	$I \times min(1, \overline{\lambda}_z) >$	< N <sub>Ed</sub> / ((C <sub>mLT</sub> - 0.2	$(5) \times \chi_z \times N_R$	<sub>k</sub> / γ <sub>M1</sub> ) = <b>0.994</b>		
Interaction formulae - eq 6.61 & eq 6.62;		N <sub>Ed</sub> / ( $\chi_y  imes N_{Rk}$ / $\gamma_{M1}$ ) + k <sub>yy</sub> $ imes$						
		M <sub>v.Ed</sub> / (γ <sub>LT</sub>	$\times$ M <sub>y,Rk</sub> / $\gamma$ M1) =	0.228				
•		<i>, , , , , , , , , ,</i>	$N_{Ed} \ / \ (\chi_z \times N_{Rk} \ / \ \gamma_{M1}) \ + \ k_{zy} \times M_{y,Ed} \ / \ (\chi_{LT} \times M_{y,Rk} \ / \ \gamma_{M1}) = \textbf{0.27}$					
		$N_{Ed}$ / ( $\chi_z$ ×	N <sub>Rk</sub> / γ <sub>M1</sub> ) + k <sub>zy</sub>	$ imes$ M <sub>y,Ed</sub> / ( $\chi_{LT}  imes$ M <sub>y</sub>	<sub>,Rk</sub> / γ <sub>M1</sub> ) = <b>0</b>	.27		

# Consider Combination 5 - 1.0G + 1.0Q + 0.5S (Service)

# Check design 2494 mm along span

Check y-y axis deflection - Section 7.2.1	
Maximum deflection;	δ <sub>y</sub> = <b>12.2</b> mm
Allowable deflection;	$\delta_{y,Allowable} = L_{m1_{s1}} / 250 = 21.6 \text{ mm}$
	$\delta_y / \delta_{y,Allowable} = 0.566$

PASS - Allowable deflection exceeds design deflection

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#### NEW STEEL BEAM THAT SUPPORTS STAIRWAY WALL

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

TEDDS calculation version 3.0.13



Load Envelope - Combination 1



Load Combination 1 (shown in proportion)





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				·			
kN 20.9		Shear Force En	velope				
0.0-					25		
-20.870					-20.9		
Ä			1		В		
Support conditions							
Support A		Vertically r	estrained				
Support B		Vertically	y liee ostrained				
Support B		Rotational	v free				
Applied los din		rotational	,				
		0					
Beam loads		Steel self v	veight - Perman	ent self weight of	beam × 1		
		Wall above	e (timber stud) -	0.5KN/m2 x heigr	nt - Permane	nt full UDL 4.5	
		Floor dead	lload - 0.8kN/m	2 v 0 85m - Porm	anont full LIC	$0.7 \mathrm{kN/m}$	
	Floor live l	noad - 1 5kN/m2	x 0.85m - Variabl		3 kN/m		
				valiabi			
Load combinations		<b>•</b> • •		-			
Load combination 1		Support A		Perman	ent $\times$ 1.35		
		<b>a</b> ,		Variable	e × 1.50		
		Span 1		Perman	ent $\times$ 1.35		
		0 1 0		Variable	e × 1.50		
		Support B	Support BPermanent × 1.35Variable × 1.50				
Analysis results							
Maximum moment;		M <sub>max</sub> = 23.	<b>5</b> kNm;	M <sub>min</sub> = <b>0</b>	kNm		
Maximum shear;		V <sub>max</sub> = <b>20.</b>	9 kN;	V <sub>min</sub> = -2	2 <b>0.9</b> kN		
Deflection;		δ <sub>max</sub> = <b>8.1</b>	mm;	$\delta_{\min} = 0$	mm		
Maximum reaction at support A		$R_{A_{max}} = 2$	<b>D.9</b> kN;	$R_{A_{min}} =$	20.9 kN		
Unfactored permanent load rea	ction at support A	; KA_Permanen	t = 12.2  kN				
Unfactored variable load reaction	on at support A;	KA_Variable =	= 2.9 KN	-	00.01.11		
Inviaximum reaction at support B	, otion of ourset F	$H_{B_{max}} = 2$	J.9 KIN;	$R_{B_{min}} =$	20.9 KN		
Unractored permanent load reaction at support B;		$B_{\text{Permanent}} = 12.2 \text{ KN}$					
	m at support B;	□B_Variable =	ε <b>2.3</b> κιν				
Section details							
Section type;		UB 203x1	J2X23 (BS4-1)				
Steel grade;		0077					
EIN 10025-2:2004 - Hot rolled I	and the state of the	S275					
Nominal thickness of clamate	products of stru	S275 ctural steels	+ ) 0.2				
Nominal thickness of element;	products of stru	S275 ctural steels t = max(t <sub>f</sub> ,	t <sub>w</sub> ) = <b>9.3</b> mm				
Nominal thickness of element; Nominal yield strength;	products of stru	S275 ctural steels t = max(t <sub>f</sub> , f <sub>y</sub> = 275 N/	tw) = <b>9.3</b> mm mm <sup>2</sup> /mm <sup>2</sup>				
Nominal thickness of element; Nominal yield strength; Nominal ultimate tensile strengt	<b>products of stru</b> h;	S275 ctural steels t = max(tr, fy = 275 N/ fu = 410 N/ E = 21000	t <sub>w</sub> ) = <b>9.3</b> mm mm <sup>2</sup> mm <sup>2</sup> <b>0</b> N/mm <sup>2</sup>				
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		┥━━━━101.8━━━━━┝					
Deutial factors - Castion 6.1							
Besistance of cross-sections:		$\infty = 1.00$					
Resistance of members to inst	tability:	$\gamma_{M1} = 1.00$					
Resistance of tensile members	s to fracture;	$\gamma_{M2} = 1.10$					
l ateral restraint	,						
		Span 1 has lateral restraint at	supports only				
Effective length factors		·					
Effective length factor in major	r axis;	K <sub>v</sub> = <b>1.000</b>					
Effective length factor in minor	r axis;	K <sub>z</sub> = <b>1.000</b>					
Effective length factor for torsi	on;	K <sub>LT.A</sub> = <b>1.200</b> ; + 2 × h					
		K <sub>LT.B</sub> = <b>1.200</b> ; + 2 × h					
Classification of cross section	ons - Section	5.5					
		$\epsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]} = 0.92$					
Internal compression parts	subject to ber	nding - Table 5.2 (sheet 1 of 3)					
Width of section;		c = d = <b>169.4</b> mm					
		c / t <sub>w</sub> = $33.9 \times \epsilon \le 72 \times \epsilon$ ;	Class 1				
Outstand flanges - Table 5.2	(sheet 2 of 3)	)					
Width of section;		$c = (b - t_w - 2 \times r) / 2 = 40.6 mm$	n Olivi				
		$c / t_f = 4.7 \times \epsilon \le 9 \times \epsilon;$	Class 1	6	tion in class 1		
				500	CIUN IS CIASS 1		
Check shear - Section 6.2.6		h h 0xt <b>1946</b> mm					
Height of web;		$m_w = 1 - 2 \times t_f = 184.6 \text{ mm}$					
		$h_{\rm m} / t_{\rm m} < 72 \times s / n$					
		She	ar buckling r	esistance d	can be ignored		
Design shear force;		$V_{Ed} = max(abs(V_{max}), abs(V_{min}))$	)) = <b>20.9</b> kN				
Shear area - cl 6.2.6(3);		$A_v = max(A - 2 \times b \times t_f + (t_w + 2))$	$2 \times r) \times t_f$ , $\eta \times h$	$w \times t_w) = 12$	<b>38</b> mm²		
Design shear resistance - cl 6	.2.6(2);	$V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y \ / \ \sqrt[]{3]}) \ / \ \gamma$	үмо = <b>196.6</b> kN				
		PASS - Design shear r	resistance ex	ceeds desi	gn shear force		
Check bending moment maj	jor (y-y) axis -	Section 6.2.5					
Design bending moment;		$M_{Ed} = max(abs(M_{s1_max}), abs(M_{s1_max}))$	/ls1_min)) = <b>23.5</b>	kNm			

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Dogian bonding registence mon									
Design bending resistance mon	ient - eq 6.13,	$W_{\rm rely} \propto f_{\rm rel} / 2$	Rd = 64.4  k  Nm						
<b>.</b>		VVpi.y × Ty /	M0 = <b>04.4</b> KINIII						
Slenderness ratio for lateral t	orsional buckli	ng							
Correction factor - Table 6.6;		$K_c = 0.94$	1 100						
Current une feieteru		$G_1 = 1 / K_c^2$	= 1.132						
Curvature factor;		$\mathbf{g} = \sqrt{1 - (1_2)}$	z / ly)] = <b>0.96</b>						
Poissons fallo,		V = 0.3							
Shear modulus,	$G = E / [2 \times (1 + v)] = 80/69 \text{ N/mm}^2$								
Clastic critical buckling moment		$L = 1.2 \times L$	$=2 \times \Gamma \times L / (12)$		2 ~ 0 ~ 1 / (-2)				
Elastic childal buckling moment	,	$M_{cr} = C_1 \times C_2$	$\pi^{-} \times E \times I_{z} / (L^{-})$	$\langle g \rangle \times \sqrt{[I_w / I_z + L]}$	$- \times G \times I_t / (\pi^{-1})$	$\times \vdash \times I_z) =$			
Slandarnass ratio for lateral tor	sional buckling:	$\overline{\lambda}$ = $\sqrt{M}$		151					
Limiting slenderness ratio	sona bucking,	$\overline{\lambda}_{1}$ = -0.4							
Limiting sienderness ratio,		7.LT,0 = <b>0.4</b>	- Arts Artorla	teral torsional h	uckling cann	ot he ianored			
			$M = \lambda E_{1,0} - E_{0,0}$			ol be ignored			
Design resistance for bucklin	g - Section 6.3.	2.1							
Buckling curve - Table 6.5;		D 							
Imperiection factor - Table 6.3;		$\alpha_{LT} = 0.34$							
Correction factor for rolled section	ons;	p = 0.75	[ <b>1</b> ]	$\overline{2}$ ) $\cdot$ $0$ $\cdot$ $\overline{2}$	21 4 470				
LTB reduction determination la	clor;	$\phi_{\text{LT}} = 0.5 \times [1 + \alpha_{\text{LT}} \times (\lambda_{\text{LT}} - \lambda_{\text{LT},0}) + \beta \times \lambda_{\text{LT}}^2] = 1.473$							
LTB reduction factor - eq 6.57;		$\chi_{LT} = \min(1 / [\phi_{LT} + v(\phi_{LT}^2 - \beta \times \lambda_{LT}^2)], 1, 1 / \lambda_{LT}^2) = 0.447$							
Modified LTP reduction factor	og 6 59.	1 = min(1 -	$0.5 \times (1 - K_c) \times [1$	- 2 × ( λlt - 0.8)	-], I) = <b>0.996</b>				
Design buckling registeres mer		$\chi_{LT,mod} = M$	$\operatorname{III}(\chi_{LT} / 1, 1) = \mathbf{U}.$	44 <del>3</del>					
Design buckling resistance mor		$IVI_{b,Rd} = \chi_{LT}$	mod × vv <sub>pl.y</sub> × ly /	$\gamma_{M1} = 20.9 \text{ kiNIII}$	le docian hon	dina momon			
	PA35	- Design Duckli	ny resistance l		s design ben	ung moment			
Check vertical deflection - Se	ction 7.2.1								

Consider deflection due to permanent and variab	le loads
Limiting deflection;	$\delta_{lim} = L_{s1} / 250 = 18 \text{ mm}$
Maximum deflection span 1;	$\delta = max(abs(\delta_{max}), abs(\delta_{min})) = 8.125 \text{ mm}$
	PASS - Maximum deflection does not exceed deflection limit

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# EXISTING RC CANTILEVER SLAB ANALYSIS

Tedds calculation version 1.0.13

#### **Results**

**Total deflection** 



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#### Node deflections

# Load case: Self Weight

Node	Deflection		Rotation	Co-ordinate system
	Х	Z		
	(mm)	(mm)	(°)	
1	0	0	0.003	
2	0	0	-0.00143	
3	0	0	-0.00017	

# Load case: Cavity wall self weight - 7.5kN/m

Node	Deflection		Rotation	Co-ordinate system
	х	Z		
	(mm)	(mm)	(°)	
1	0	0	-0.00367	
2	0	0	0.00747	
3	0	0.1	0.01206	

# Load case: Roof dead load - 1.0kN/m2

Node	Deflection		Rotation	Co-ordinate system
	х	Z		
	(mm)	(mm)	(°)	
1	0	0	-0.00056	
2	0	0	0.00115	
3	0	0	0.00185	

# Load case: Roof live load - 0.6kN/m2

Node	Deflection		Rotation	Co-ordinate system
	х	Z		
	(mm)	(mm)	(°)	
1	0	0	-0.00034	
2	0	0	0.0007	
3	0	0	0.00113	

# Load case: Bath tub filled - 10kN/m3

Node	Deflection		Rotation	Co-ordinate system
	Х	Z		
	(mm)	(mm)	(°)	
1	0	0	-0.00017	

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Node	Deflection		Rotation	Co-ordinate system
	Х	Z		
	(mm)	(mm)	(°)	
2	0	0	0.00035	
3	0	0	0.00049	

# **Total base reactions**

Load case/combination	Fo	rce
	FX	FZ
	(kN)	(kN)
Self Weight	0	21.2
Cavity wall self weight - 7.5kN/m	0	7.5
Roof dead load - 1.0kN/m2	0	1.2
Roof live load - 0.6kN/m2	0	0.7
Bath tub filled - 10kN/m3	0	0.7

# **Element end forces**

# Load case: Self Weight

Element	Length	Nodes	Axial force	Shear force	Moment
	(m)	Start/End	(kN)	(kN)	(kNm)
1	1.7	1	0	-6.2	0
		2	0	-8.8	-2.2
2	0.7	2	0	-6.2	2.2
		3	0	0	0

# Load case: Cavity wall self weight - 7.5kN/m

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	1.7	1	0	3.1	0
		2	0	-3.1	-5.2
2	0.7	2	0	-7.5	5.3
		3	0	7.5	0

# Load case: Roof dead load - 1.0kN/m2

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
	(,		()	()	()
1	1.7	1	0	0.5	0
		2	0	-0.5	-0.8
2	0.7	2	0	-1.1	0.8
		3	0	1.2	0

# Load case: Roof live load - 0.6kN/m2

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	1.7	1	0	0.3	0
		2	0	-0.3	-0.5
2	0.7	2	0	-0.7	0.5
		3	0	0.7	0

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# Load case: Bath tub filled - 10kN/m3

Element	Length	Nodes	Axial force	Shear force	Moment
	(m)	Start/End	(kN)	(kN)	(kNm)
1	1.7	1	0	0.1	0
		2	0	-0.1	-0.2
2	0.7	2	0	-0.7	0.2
		3	0	0	0

# Forces



-8.8

# **Element results**

Envelope - All load cases

Element	Shear force		Moment				
	Pos Max abs		Pos	Max	Pos	Min	
	(m)	(kN)	(m)	(kNm)	(m)	(kNm)	
1	1.7	-8.8	0.706	2.2	1.7	-5.2	
2	0	7.5	0.7	0	0	-5.2	
;							

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# STEEL BEAM THAT SUPPORTS THE BACK WALL OF THE OUTRIGGER

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

TEDDS calculation version 3.0.13





Load Envelope - Combination 1



Load Combination 1 (shown in proportion)





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kN		Shear Force En	velope			
72.484						
0.0-						
					Ť	
-72.484 J mm I			2400		-72.5	
A			1		J B	
Support conditions						
Support A		Vertically r	estrained			
		Rotational	y free			
Subbout R		Vertically r	estrained			
		notationali	упее			
Applied loading		041 1	valaht D			
Beam loads		Steel self v	veignt - Permai	nent self weight of	peam × 1	
		Dead 10ad	abuve - Perma	nieni iuli UDL 27 k ∍ full HDL 2.5 kN/r	n ////////////////////////////////////	
		Terrace ro	of dead load - 1	1kN/m2 x 1.25m -	 Permanent fi	ull UDI 1 kN/m
		Terrace ro	of live load - 1.	5kN/m2 x 1.25m -	Variable full	UDL 1.9 kN/m
		Floor dead	load - 0.8kN/m	12 x 0.75m - Perm	anent full UD	DL 0.6 kN/m
		Floor live le	oad - 1.5kN/m2	2 x 0.75m - Variabl	le full UDL 1.	2 kN/m
		Solid 215m	nm brick wall -	18kN/m3 x 0.215n	n x 3m - Pern	nanent full
		UDL 9.675	kN/m			
Load combinations						
Load combination 1		Support A		Perman	ent $ imes$ 1.35	
				Variable	e × 1.50	
		Span 1		Perman	ent × 1.35	
				Variable	e × 1.50	
		Support B		Perman	ent $ imes$ 1.35	
				Variable	e × 1.50	
Analysis results						
Maximum moment;		M <sub>max</sub> = <b>43</b> .	<b>5</b> kNm;	$M_{min} = 0$	) kNm	
Maximum shear;		V <sub>max</sub> = 72.5	5 kN;	$V_{min} = -7$	72.5 kN	
Deflection;		δ <sub>max</sub> = <b>3.9</b> I	mm;	$\delta_{\min} = 0$	mm	
Maximum reaction at support A;	tion of oursest A	$HA_{max} = 72$	2.5 KN;	$R_{A_{min}} =$	12.5 kN	
Uniaciored permanent load read	n at support A	RA_Permanent	= 40.2 KIN			
Maximum reaction at support R	n al support A;	$\Box A_{variable} = \mathbf{R}_{B_{variable}} = 7$	2.5 kN	Rp min -	72.5 kN	
Unfactored permanent load read	ction at support P	$R_{\rm B} = I_{\rm A}$	= <b>46.2</b> kN	i iB_min =		
Unfactored variable load reactio	n at support B;	RB_Variable =	6.7 kN			
Section details						
Section type;		UB 203x13	33x25 (BS4-1)			
Steel grade;		S275	. ,			
EN 10025-2:2004 - Hot rolled p	products of stru	ctural steels				
Nominal thickness of element;		$t = max(t_f, f)$	t <sub>w</sub> ) = <b>7.8</b> mm			
Nominal yield strength;		f <sub>y</sub> = <b>275</b> N/	mm <sup>2</sup>			
Nominal ultimate tensile strengt	h;	fu = <b>410</b> N/	mm <sup>2</sup>			

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www.sd-structures.com Modulus of elasticity;	RC	E = 21000	MD 0 N/mm <sup>2</sup>	26/06/2017		
<b>Partial factors - Section 6.1</b> Resistance of cross-sections; Resistance of members to insta Resistance of tensile members	bility; to fracture;	γ <sub>M0</sub> = 1.00 γ <sub>M1</sub> = 1.00 γ <sub>M2</sub> = 1.10	133.2			
Lateral restraint		Span 1 ha	s lateral restrain	t at supports only	/	
Effective length factors Effective length factor in major a Effective length factor in minor a Effective length factor for torsion	axis; axis; n;	K <sub>y</sub> = 1.000 K <sub>z</sub> = 1.000 K <sub>LT.A</sub> = 1.2 K <sub>LT.B</sub> = 1.2	00; + 2 × h 00; + 2 × h			
Classification of cross soction	ns - Soction 5	5				
Siassification of Closs Section	13 - Section 3.	י 2 – √נטעד א	V/mm <sup>2</sup> / f.1 – <b>n o</b>	2		
Internal communication of the	ible at the base of	$\mathbf{r} = \mathbf{v} [\mathbf{z} \mathbf{u} \mathbf{v}]$	$y_{1} = 0.3$	-		
Width of section:	ibject to bendi	ng - rable 5.2 (	sneet 1 of 3) 2 4 mm			
		c = u = 1/2	<b></b> μμμμ 7 × ε <= 79 × ε <sup>.</sup>	Class 1		
Outstand flamman, Table 7.6	ahaat 0 = ( 0)	$\mathbf{U} = \mathbf{U}\mathbf{Z}$	$I \land C \land = I \angle \land C,$	01033 1		
Outstand flanges - Table 5.2 (	sneet 2 of 3)	c /L 1		mm		
width of section;		$c = (b - I_w - C_w)$	· 2 × r) / 2 = <b>56.1</b>			
		$C / l_{\rm f} = 7.8$	$\times \varepsilon \leq \Im \times \varepsilon$	Class I	Cont	ion is class 1
<b>.</b>					Sect	IUII IS CIASS I
Check shear - Section 6.2.6						
Height of web;		h <sub>w</sub> = h - 2 >	× t <sub>f</sub> = <b>187.6</b> mm			
Shear area factor;		η = <b>1.000</b>				
		h <sub>w</sub> / t <sub>w</sub> < 72	2×ε/η	<b>.</b>		
Design about former		\/		Shear buckling	resistance ca	an be ignored
Design snear force;		$V_{Ed} = max($	(abs(V <sub>max</sub> ), abs()	$V_{min}$ ) = <b>12.5</b> kN	h	<b>0</b> man <sup>2</sup>
Silver area - $CI 0.2.0(3)$ ;	C(0).	$A_v = \max(A_v)$	$\neg - 2 \times D \times t_f + (t_w)$	$v + 2 \times r \times t_f, \eta \times t_f$	11 <sub>w</sub> × l <sub>w</sub> ) = 128	∠ !!!!!!*
Design shear resistance - cl 6.2	.o(∠);	$V_{c,Rd} = V_{pl,l}$	Rd = Av × (ĭy / √[3 SS - Design she	οj) / γмο = 203.5 k ear resistance e	xceeds desig	n shear force

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	()					
Check bending moment major	(y-y) axis - Se	ction 6.2.5			<b>F</b> 1.1.1	
Design bending moment;	nt og 6 12.	$M_{Ed} = max$	(aDS(IVIs1_max), a	$DS(IVIs1_min)) = 43.$	5 KINM	
Design bending resistance mome	nii - eq 0.13,	$M_{c,Rd} = M_{pl}$	Rd =			
		VVpi.y × Ty /	mo = 70.9  kinifi			
Slenderness ratio for lateral tor	sional buckli	ng				
Correction factor - Table 6.6;		k <sub>c</sub> = <b>0.94</b>	1 100			
		$C_1 = 1 / k_c^2$	= 1.132			
Curvature factor;		$g = \sqrt{1 - (1_2)}$	$[z / I_y)] = 0.932$			
Poissons ratio;		v = <b>0.3</b>	<i></i>			
Shear modulus;		G = E / [2 >	< (1 + v)] = 8076	<b>59</b> N/mm <sup>2</sup>		
Unrestrained length;		$L = 1.2 \times L$	<sub>s1</sub> + 2 × h = <b>328</b>	<b>6</b> mm		
Elastic critical buckling moment;		$M_{cr} = C_1 \times C_1$	$\pi^2 \times E \times I_z / (L^2)$	$\times$ g) $\times \sqrt{[I_w / I_z + L^2]}$	$^{2}$ × G × I <sub>t</sub> / ( $\pi^{2}$	$\times E \times I_z)] =$
		<b>95.4</b> kNm				
Slenderness ratio for lateral torsic	onal buckling;	$\lambda_{LT} = \sqrt{W}$	$pl.y \times fy / M_{cr}) = 0$	0.862		
Limiting slenderness ratio;		λ <sub>LT,0</sub> = <b>0.4</b>				
			λιτ > λιτ,ο - La	teral torsional b	uckling cann	ot be ignored
Design resistance for buckling	- Section 6.3.	2.1				
Buckling curve - Table 6.5;		b				
Imperfection factor - Table 6.3;		$\alpha_{\text{LT}} = 0.34$				
Correction factor for rolled section	ıs;	$\beta = 0.75$				
LTB reduction determination factor	or;	$\phi_{LT} = 0.5  imes$	$[1 + \alpha_{LT} \times (\overline{\lambda}_{LT})]$	- $\overline{\lambda}_{LT,0}$ ) + $\beta \times \overline{\lambda}_{LT,0}$	<sup>2</sup> ] = <b>0.857</b>	
LTB reduction factor - eq 6.57;		$\chi_{LT} = min(1)$	/ $[\phi_{LT} + \sqrt{(\phi_{LT}^2 - $	$\beta \times \overline{\lambda}_{LT}^{2}$ )], 1, 1 /	$\overline{\lambda}_{LT}{}^2) = \boldsymbol{0.782}$	
Modification factor;		f = min(1 -	$0.5 \times (1 - k_c) \times [$	1 - 2 × ( $\overline{\lambda}_{LT}$ - 0.8)	<sup>2</sup> ], 1) = <b>0.970</b>	
Modified LTB reduction factor - ed	q 6.58;	$\chi_{LT,mod} = m$	$in(\chi_{LT} / f, 1) = 0$	.806		
Design buckling resistance mome	ent - eq 6.55;	$M_{b,Rd} = \chi_{LT}$	$_{mod} \times W_{pl.y} \times f_y$ /	γ <sub>M1</sub> = <b>57.1</b> kNm		
	PASS	- Design buckl	ing resistance	moment exceed	ls design ben	ding moment
Check vertical deflection - Sect	ion 7.2.1					
Consider deflection due to perma	nent and varia	ble loads				
Limiting deflection;		$\delta_{\text{lim}} = \min(1)$	0 mm, L <sub>s1</sub> / 500	0) = <b>4.8</b> mm		
Maximum deflection span 1;		$\delta = \max(ab)$	$s(\delta_{max}), abs(\delta_{mi})$	n)) = <b>3.878</b> mm		
•		PAS	S - Maximum	deflection does	not exceed d	eflection limit

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# NEW STEEL BEAM THAT SUPPORT THE SIDE WALL OF THE OUTRIGGER

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

TEDDS calculation version 3.0.13





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kN 62.402 - 62.4		Shear Force En	velope				
		41.9					
0.0-							
-32.052		-30.5	4000		-32.1		
mm [ A			1		] B		
Support conditions							
Support A		Vertically r	estrained				
		Rotational	ly free				
Support B		Vertically r	estrained				
		Rotationali	y free				
Applied loading							
Beam loads		Steel self v	weight - Perma	nent self weight o	f beam $\times$ 1		
		SB2 Dead	load - Perman	ent point load 46.	2 kN at 1500	) mm	
		SB2 Live k	oad - Variable p	point load 6.7 kN a	at 1500 mm		
		Solid 215n	nm brick wall -	18kN/m3 x 0.215r	n x 2.5m - P	ermanent	
		partial UDI	_ 9.675 KIN/M Tr	rom 0 mm to 1500	mm		
Load combinations							
Load combination 1		Support A		Permar	nent × 1.35		
				Variable	e×1.50		
		Span 1		Permar	nent × 1.35		
				Variable	e×1.50		
		Support B		Permar	nent $ imes$ 1.35		
				Variable	e × 1.50		
Analysis results			0.1.1.1				
Maximum moment;		$M_{max} = 78.2$	2 kNm;	M <sub>min</sub> = 0	U KNM		
waximum shear;		$V_{max} = 62.4$	+ KIN;	V min = -	32.1 KN		
Dellection;		$o_{max} = \delta m$	111; D 4 1/NI+	o <sub>min</sub> = 0	$\delta_{\min} = 0 \text{ mm}$		
Infactored permanent load rea	, ation at support A	$RA_{max} = 0$	2.4 KIN,	ha_min =	= <b>02.4</b> KIN		
Unfactored variable load reactiv	n at support A:		t = 41.0 KN				
Maximum reaction at support R	a support A,	R <sub>P</sub> may - <b>2</b>		Ro	- <b>32 1</b> kN		
Unfactored permanent load rea	, iction at support F	B: RB Permanent	t = <b>21</b> kN	• 'D_IIII -	•=== 1 1/1 <b>1</b>		
Unfactored variable load reaction	on at support B;	RB_Variable =	= <b>2.5</b> kN				
Section details							
Section type;		UC 203x2	03x46 (BS4-1)				
Steel grade;		S275					
	products of stru	ctural steels					
EN 10025-2:2004 - Hot rolled							
EN 10025-2:2004 - Hot rolled Nominal thickness of element;		$t = max(t_f, $	tw) = <b>11.0</b> mm				
EN 10025-2:2004 - Hot rolled Nominal thickness of element; Nominal yield strength;		$t = max(t_f, f_y = 275 N/t_f)$	t <sub>w</sub> ) = <b>11.0</b> mm				
EN 10025-2:2004 - Hot rolled Nominal thickness of element; Nominal yield strength; Nominal ultimate tensile streng	th;	$t = max(t_f, f_y = 275 N/f_u = 410 N/f_u = 21000$	t <sub>w</sub> ) = <b>11.0</b> mm /mm <sup>2</sup> /mm <sup>2</sup>				

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	-	±					
	:	È Î					
		∙—		203.6	►		
Partial factors - Section 6.1							
Resistance of cross-sections;	- I= : 1:4		γ <sub>M0</sub> = <b>1.00</b>				
Resistance of members to Insta Resistance of tancile members	to froot	Iro.	$\gamma_{M1} = 1.00$				
		ле,	γm2 = 1.10				
Lateral restraint			Snan 1 ha	e latoral rostrair	at at supports only		
			Spannia	S Idleral restrail	it at supports only	y	
Effective length factors	avie:		k _ 1 000				
Effective length factor in minor	axis;		$K_{z} = 1.000$				
Effective length factor for torsic	on;		K <sub>LT.A</sub> = <b>1.2</b>	<b>00</b> ; + 2 × h			
-			К <sub>LТ.В</sub> = <b>1.2</b>	<b>00</b> ; + 2 × h			
Classification of cross section	ons - Se	ction 5.5					
			ε = √[235 ľ	V/mm² / f <sub>y</sub> ] = <b>0.9</b>	92		
Internal compression parts s	ubiect t	o bendir	ng - Table 5.2 (	sheet 1 of 3)			
Width of section;			c = d = 16	<b>0.8</b> mm			
			c / t <sub>w</sub> = 24.	$2 \times \varepsilon \le 72 \times \varepsilon;$	Class 1		
Outstand flanges - Table 5.2	(sheet 2	2 of 3)					
Width of section;			c = (b - t <sub>w</sub> -	- 2 × r) / 2 = <b>88</b>	mm		
			$c \ / \ t_{f} = 8.7$	$\times \varepsilon \le 9 \times \varepsilon;$	Class 1		
						Se	ction is class 1
Check shear - Section 6.2.6							
Height of web;			h <sub>w</sub> = h - 2 :	× t <sub>f</sub> = <b>181.2</b> mm			
Shear area factor;			η = <b>1.000</b>				
			h <sub>w</sub> / t <sub>w</sub> < 72	2×ε/η			
Desire shoey fares					Shear buckling	resistance	can be ignored
Design snear force;			$V_{Ed} = max$	(abs(V <sub>max</sub> ), abs(	(Vmin)) = <b>62.4</b> KIN	(h, v, t) = 16	<b>208</b> mm <sup>2</sup>
Design shear resistance $= 016$	2 6/21.		$M_V = MaX(A$	$\neg = \angle \times \cup \times \text{ If } + (l$	.w + こ べ i ) × lf, i   × 3]) / ハ៱៱ – <b>260 F</b> レ	$1 \le 1 \le$	JEO 111111-
Boolgh andar realaitue - Cl 0.4	U( <i>Ľ</i> ),		v с, на = v pl, РА	SS - Desian sh	ear resistance e	xceeds des	ian shear force
Check bending moment mair	or (v_v) /	avie - Co	ction 6 2 5				
Design bending moment	//(y−y)č	113 - 30	MEd = may	(abs(Mai may) a	$bs(M_{s1 min})) = 78$	<b>2</b> kNm	
			Lu – max		······// - / •		

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Docian bonding registance mon	ont og 6 12:	M	D						
Design bending resistance mon	ienii - eq 0.13,	$W_{pl,v} \times f_v / \gamma$	na – 1 <b>36.8</b> kNm						
Slandarness ratio for lateral t	orsional buckli	na							
Correction factor - Table 6.6		ka = 0.94							
		$C_1 = 1 / k_c^2$	= 1.132						
Curvature factor:		q = √[1 - (]₂	/ l <sub>v</sub> )] = <b>0.813</b>						
Poissons ratio:		v = 0.3	y = 0.3						
Shear modulus;		G = E / [2 >	G = E / [2 × (1 + v)] = <b>80769</b> N/mm <sup>2</sup>						
Unrestrained length;		L = 1.2 × L	$s_1 + 2 \times h = 5206$	<b>6</b> mm					
Elastic critical buckling moment	,	$M_{cr} = C_1 \times c$	$\pi^2 \times E \times I_z / (L^2 \times$	< g) × √[l <sub>w</sub> / l <sub>z</sub> + L <sup>2</sup>	$^{2}$ × G × I <sub>t</sub> / ( $\pi^{2}$ >	$\times E \times I_z)] =$			
		<b>257.1</b> kNm	<b>257.1</b> kNm						
Slenderness ratio for lateral tors	ional buckling;	$\overline{\lambda}_{LT} = \sqrt{(W_{\text{pl.y}} \times f_y \ / \ M_{cr})} = 0.729$							
Limiting slenderness ratio;		$\overline{\lambda}_{LT,0} = 0.4$	$\overline{\lambda}_{LT,0} = 0.4$						
			λιτ > λιτ,ο - Lat	eral torsional b	uckling canno	ot be ignored			
Design resistance for bucklin	g - Section 6.3.	2.1							
Buckling curve - Table 6.5;		b							
Imperfection factor - Table 6.3;		$\alpha_{LT} = 0.34$							
Correction factor for rolled secti	ons;	$\beta = 0.75$							
LTB reduction determination fac	ctor;	$\phi_{LT} = 0.5  imes$	$[1 + \alpha_{LT} \times (\overline{\lambda}_{LT})]$	- $\overline{\lambda}_{LT,0}$ ) + $\beta \times \overline{\lambda}_{LT}$	<sup>.2</sup> ] = <b>0.756</b>				
LTB reduction factor - eq 6.57;		χ∟⊤ = min(1	/ [ $\phi_{LT} + \sqrt{(\phi_{LT}^2 - $	$\beta  imes \overline{\lambda}_{LT^2}$ ], 1, 1 /	$\overline{\lambda}_{LT}^2$ ) = <b>0.855</b>				
Modification factor;		f = min(1 -	0.5 × (1 - k <sub>c</sub> )× [1	- 2 $\times$ ( $\overline{\lambda}_{LT}$ - 0.8)	<sup>2</sup> ], 1) = <b>0.970</b>				
Modified LTB reduction factor -	eq 6.58;	$\chi_{\text{LT,mod}} = m$	in(χ <sub>LT</sub> / f, 1) = <b>0.</b>	881					
Design buckling resistance mon	nent - eq 6.55;	$M_{b,Rd} = \chi_{LT}$	$_{mod}  imes W_{pl.y}  imes f_y$ /	γ <sub>M1</sub> = <b>120.5</b> kNm					
	PASS	- Design buckli	ing resistance i	moment exceed	s design ben	ding moment			
Check vertical deflection - Se	ction 7.2.1								

#### 

Maximum deflection span 1;

 $\delta = \max(abs(\delta_{max}), abs(\delta_{min})) = 7.962 \text{ mm}$ 

PASS - Maximum deflection does not exceed deflection limit

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# STEEL POST THAT SUPPORTS NEW STAIRWAY BEAAM

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





# Column and loading details

Column section;	SHS 90x90x4.0
System length for buckling about y axis;	L <sub>y</sub> = ; <b>3000</b> ; mm
System length for buckling about z axis;	L <sub>z</sub> = ; <b>3000</b> ; mm;

#### Sway

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

#### **Column loading**

Axial load;	N <sub>Ed</sub> = <b>21</b> kN; (Compression)
Moment about y axis at end 1;	M <sub>y,Ed1</sub> = <b>0.0</b> kNm
Moment about y axis at end 2;	M <sub>y,Ed2</sub> = <b>0.0</b> kNm
Moment about z axis at end 1;	M <sub>z,Ed1</sub> = <b>0.0</b> kNm
Moment about z axis at end 2;	$M_{z,Ed2} = 0.0 \text{ kNm}$
Shear force parallel to z axis;	$V_{z,Ed} = 0 \text{ kN}$
Shear force parallel to y axis;	$V_{y,Ed} = 0 \ kN$
Material details	
Steel grade;	S275
Yield strength;	f <sub>y</sub> = <b>275</b> N/mm <sup>2</sup>
Ultimate strength;	f <sub>u</sub> = <b>410</b> N/mm <sup>2</sup>
Modulus of elasticity;	E = <b>210</b> kN/mm <sup>2</sup>
Poisson's ratio;	v = <b>0.3</b>
Shear modulus;	G = E / $[2 \times (1 + v)]$ = 80.8 kN/mm <sup>2</sup>
Buckling length for flexural buckling al	bout y axis
Buckling length;	L <sub>cr_y</sub> = <b>6000</b> mm
Buckling length for flexural buckling al	bout z axis
Buckling length;	L <sub>cr_z</sub> = <b>6000</b> mm
Section classification	
Web section classification (Table 5.2)	
Coefficient depending on fy;	$\epsilon = \sqrt{(235 \text{ N/mm}^2 / f_y)} = 0.924$

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Depth between fillets;		$C_w = h - 3 \times$	<t <b="" =="">78.0 mm</t>			
Rallo of c/l;	a du	$ratio_w = c_w$	/ (1 = 19.50)	) 0.5 mm		
	au,	$I_W = IIIIII(INE)$	$d / (2 \times I_y \times l), C$	w) =9.5 mm		
For class 1 & 2 proportion in con	npression,	$\alpha = (C_W/2 + 1)$	$I_{W}/2$ ) / $C_{W} = 0.50$			
Limit for class 1 web,		$\Box IIIIIIII = (3$	590 × 8) / (13 × 0	(x - 1) = ,56.15	Тро и	vah ie claee 1
	(T-1-1- 5 0)				ine w	60 13 01033 1
Flange section classification (	(Table 5.2)	a h 0	. 70.0			
Depth between lillets;		$C_f = D - 3 \times$	t = 78.0 mm			
Ratio of C/L,		$fallO_f = C_f /$	l = 19.50			
Limit for class 1 flange:		l imit₁₁ – 33	×ε – 30 51			
Limit for class 2 flange:		$Limit_{2} = 38$	×ε = 35.13			
Limit for class 3 flange:		$Limit_{2f} = 42$	$2 \times c = 38.83$			
					The flar	nae is class 1
Overall section elessification						.ge ie eidee i
Overall Section Classification					The sect	ion is class 1
Resistance of cross section (	<u>cl. 6.2)</u>					
Compression (cl. 6.2.4)						
Design force;		N <sub>Ed</sub> = <b>21</b> kl	N			
Design resistance;		$N_{c,Rd} = N_{pl,l}$	$Rd = A \times f_y / \gamma_{MO}$	= <b>374</b> kN		
		PASS - The c	compression d	esign resistance	exceeds the	design force
Buckling resistance (cl. 6.3)						
Yield strength for buckling resist	tance;	f <sub>y</sub> = <b>275</b> N/	mm²			
Flexural buckling about y axis	3					
Elastic critical buckling force;		$N_{cr,y}=\pi^2\times$	$E \times I_y / L_{cr_y^2} = 2$	<b>96</b> kN		
Non-dimensional slenderness;		$\overline{\lambda}_y = (A \times$	(f <sub>y</sub> / N <sub>cr,y</sub> ) = <b>1.97</b>	76		
Buckling curve (Table 6.2);		а				
Imperfection factor (Table 6.1);		$\alpha_y = 0.21$				
Parameter $\Phi$ ;		$\Phi_{y}$ = 0.5 $ imes$	$[1 + \alpha_y \times (\overline{\lambda}_y - 0)]$	$(0.2) + \overline{\lambda}y^2] = 2.63$	8	
Reduction factor;		$\chi_y = \min(1.$	0, 1 / $[\Phi_y + \sqrt{\Phi_y}]$	$y^{2} - \overline{\lambda}y^{2})]) = 0.228$		
Design buckling resistance;		$N_{b,y,Rd} = \chi_y$	$ imes$ A $ imes$ f <sub>y</sub> / $\gamma_{M1}$ =	<b>85.2</b> kN		
I	PASS - The flex	ural buckling r	esistance abo	ut the y axis exc	eeds the desi	gn axial load
Flexural buckling about z axis	6					
Elastic critical buckling force;		$N_{cr,z} = \pi^2 \times$	$E \times I_z / L_{cr_z^2} = 2$	96 kN		
Non-dimensional slenderness;		$\overline{\lambda}_z = \sqrt{(A \times A)^2}$	$(f_y / N_{cr,z}) = 1.97$	76		
Buckling curve (Table 6.2);		а				
Imperfection factor (Table 6.1);		$\alpha_{z} = 0.21$		_		
Parameter $\Phi$ ;		$\Phi_z$ = 0.5 $ imes$	$[1 + \alpha_z \times (\overline{\lambda}_z - 0)]$	$(0.2) + \overline{\lambda}z^2] = 2.63$	8	
Reduction factor;		$\chi_z = min(1.$	0, 1 / [ $\Phi_z + \sqrt{\Phi_z}$	$(z^2 - \overline{\lambda} z^2)]) = 0.228$		
Design buckling resistance;		$N_{b,z,Rd} = \chi_z$	$\times$ A $\times$ f <sub>y</sub> / $\gamma_{M1}$ =	<b>85.2</b> kN		
I	PASS - The flex	ural buckling r	esistance abo	ut the z axis exc	eeds the desi	gn axial load
Minimum buckling resistance						

Minimum buckling resistance;

 $N_{b,Rd} = min(N_{b,y,Rd}, N_{b,z,Rd}) = \textbf{85.2 kN}$  **PASS - The axial load buckling resistance exceeds the design axial load** 

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# **Steel connections**

Note: See SD Structures plans drawings for beams annotations

#### SB1 AND SB3 CONNECTION TO MAIN BOX-FRAME

#### Section Details

Supporting Beam UKC 254x254x89;; Gradesupporting = "S275"Supported Beam 1 - UB 203x102x23;;Gradesupportedb1 = "S275"Supported beam 1 endplate - 130 x 190 x 10;; Gradeendplateb1 = "S275"Supported Beam 2 - UKC 203x203x46;;Gradesupportedb2 = "S275"Supported beam 2 endplate - 130 x 190 x 10;; Gradeendplateb2 = "S275"Supported beam 2 endplate - 130 x 190 x 10;; Gradeendplateb2 = "S275"

Bolts M16 (Grade 8.8)





SECTION A - A

TEDDS calculation version 2.0.14

# **Connection Details**

#### Both beams ; Bolt pitch;; $p_{bolts} = 50 \text{ mm}$ $g_{bolts} = 90 \text{ mm}$ Bolt gauge; : Beam 1 number of bolt rows; $n_{boltsb1} = 2$ End plate end distance (top & bottom); e1endplateb1 = 40 mm End plate edge distance; e<sub>2endplateb1</sub> = (d<sub>endplateb1</sub> - g<sub>bolts</sub>) / 2 = 50 mm $I_{endplateb1} = p_{bolts} \times (n_{boltsb1}-1) + 2 \times e_{1endplateb1} = 130 \text{ mm}$ End plate length; Weld leg length; Sweldb1 = 6 mmQ<sub>b1</sub> = **22.0** kN Supported beam end reaction; Beam 2 number of bolt rows; $n_{boltsb2} = 2$ ; e1endplateb2 = 40 mm End plate end distance (top & bottom); End plate edge distance; e2endplateb2 = (dendplateb2 - gbolts) / 2 = 50 mm End plate length; lendplateb2 = pbolts×(nboltsb2-1)+2×e1endplateb2 = **130** mm Weld leg length; $S_{weldb2} = 6 \text{ mm}$ Supported beam end reaction; Qb2 = 63.0 kN ; Notch details

#### Beam 1

; Top notch length; C<sub>topnotchb1</sub> = **133** mm

Top notch depth; d<sub>ctopnotchb1</sub> = **30** mm

Beam 2

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	_					
; Top notch length;	Ctopnotchb2 = 133	mm				
; I op notch depth; c	$d_{ctopnotchb2} = 30 \text{ m}$	1m				
Check 1 - Essential detail	ing requiremen	Its				
Both beams		<b>C</b> hatha	_ 00 mm· DAS	e		
, Don gauge, Ream 1		goots	= 30 mm, FAS	5		
: End plate thicknes	s:	tendola	nteb1 = <b>10</b> mm; <b>I</b>	PASS		
; End plate Length;	,	lendpla	<sub>teb1</sub> = <b>130</b> mm			
;		End	plate length fo	or torsional requ	irements : PA	ASS
Beam 2				•		
; End plate thicknes	s;	tendpla	uteb2 = <b>10</b> mm; <b>F</b>	PASS		
; End plate Length;		lendpla	<sub>teb2</sub> = <b>130</b> mm			
• •		End	plate length fo	or torsional requ	irements : P/	ASS
Check 2 - Shear capacity of be	olt group conne	ecting end plate	e to supportin	g beam		
Beam 1						
Shear capacity of top pa	air of bolts;;;;					
Pbsendplateb1 = <b>460</b> N/mm <sup>2</sup>	2					
$P_{sbolts1b1} = min(P_{sbolts}, 0)$	$5 \times e_{1 \text{ endplateb1}} \times c$	$t_{endplateb1}  imes p_{bsendp}$	olateb1) = <b>58.9</b> kl	N		
Shear capacity of other	bolts	· · · · · · · · · · · · · · · · · · ·	,			
$P_{\text{shelts}} = 58.9 \text{ kN}$						
Shear capacity of bolt group - si	um of bolt capac	vitios				
$P_{sboltssumb1} = 2 \times P_{sbolts1b1} + 2 \times ($	$(\Pi_{\text{boltsb1}} - 1) \times P_{\text{sb}}$	olts = 235.5  KIN				
;Shear on bolt group; $Q_{b1} = 22.0$	) kN					
Utilisation factor; $U_{check2b1} = Q_{b1}$	$/ P_{sboltssumb1} = 0.$	093				
		S	hear capacity	of bolt group to	Supporting I	Beam : PASS
Beam 2						
Shear capacity of top pa	air of bolts;;;;					
Pbsendplateb2 = 460 N/mm <sup>2</sup>	2					
<b>_</b>	$5 \times e_{1endplateb2} \times \frac{1}{2}$		) 50.011			
$P_{sbolts1b2} = min(P_{sbolts}, 0.)$		[endplateb2 $ imes$ $p$ bsendp	olateb2) = <b>58.9</b> KI	N		
$P_{sbolts1b2} = min(P_{sbolts}, 0.)$ Shear capacity of other	bolts	Iendplateb2 $ imes$ $p$ bsend	blateb2) = <b>58.9</b> KI	N		
$P_{sbolts1b2} = min(P_{sbolts}, 0.)$ Shear capacity of other $P_{sbolts} = 58.9 \text{ kN}$	bolts	Iendplateb2 $ imes$ $p$ bsend;	blateb2) = <b>58.9</b> Kl	N		
$P_{sbolts1b2} = min(P_{sbolts}, 0.$ Shear capacity of other $P_{sbolts} = 58.9 \text{ kN}$ Shear capacity of bolt group - su	bolts um of bolt capac	ities	olateb2) = <b>58.9</b> KI	N		
$P_{sbolts1b2} = min(P_{sbolts}, 0).$ Shear capacity of other $P_{sbolts} = 58.9 \text{ kN}$ Shear capacity of bolt group - su 'P_{sboltsump2} = 2 \times P_{sbolts1b2} + 2 \times (2)	bolts um of bolt capac	ities	olateb2) = <b>58.9</b> Kl	N		
$P_{sbolts1b2} = min(P_{sbolts}, 0)$ Shear capacity of other $P_{sbolts} = 58.9 \text{ kN}$ Shear capacity of bolt group - si ;P_{sboltssumb2} = 2 × P_{sbolts1b2} + 2 × ( :Shear on bolt group: Or a = 63.0	bolts um of bolt capac $(n_{boltsb2} - 1) \times P_{sb}$	lendplateb2 × Pbsendp tities olts = <b>235.5</b> kN	olateb2) = <b>58.9</b> KI	N		
$P_{sbolts1b2} = min(P_{sbolts}, 0).$ Shear capacity of other $P_{sbolts} = 58.9 \text{ kN}$ Shear capacity of bolt group - su ;P_{sboltssumb2} = 2 × P_{sbolts1b2} + 2 × ( ;Shear on bolt group; Q <sub>b2</sub> = 63.0	bolts um of bolt capac $(n_{boltsb2} - 1) \times P_{sb}$ 0  kN	ities olts = <b>235.5</b> kN	olateb2) = <b>58.9</b> Kl	N		
$P_{sbolts1b2} = min(P_{sbolts}, 0)$ $Shear capacity of other$ $P_{sbolts} = 58.9 \text{ kN}$ Shear capacity of bolt group - su ;P_{sboltssumb2} = 2 × P_{sbolts1b2} + 2 × (0) ;Shear on bolt group; $Q_{b2} = 63.00$ Utilisation factor; $U_{check2b2} = Q_{b2}$	bolts um of bolt capac $(n_{boltsb2} - 1) \times P_{sb}$ 0  kN / $P_{sboltssumb2} = 0.$	ities olts = 235.5 kN 268	lateb2) = <b>58.9</b> Kl	N		
$P_{sbolts1b2} = min(P_{sbolts}, 0).$ Shear capacity of other $P_{sbolts} = 58.9 \text{ kN}$ Shear capacity of bolt group - su ;P_{sboltssumb2} = 2 × P_{sbolts1b2} + 2 × () ;Shear on bolt group; Q <sub>b2</sub> = 63.0 Utilisation factor; U <sub>check2b2</sub> = Q <sub>b2</sub>	bolts um of bolt capac $(n_{boltsb2} - 1) \times P_{sb}$ 0  kN / $P_{sboltssumb2} = 0.$	lendplateb2 × Pbsendp bities bolts = 235.5 kN 268 S	hear capacity	of bolt group to	Supporting I	Beam : PASS
$P_{sbolts1b2} = min(P_{sbolts}, 0).$ Shear capacity of other $P_{sbolts} = 58.9 \text{ kN}$ Shear capacity of bolt group - su ;P_{sboltssumb2} = 2 × P_{sbolts1b2} + 2 × ( ;Shear on bolt group; Q_{b2} = 63.0 Utilisation factor; U_{check2b2} = Q_{b2} Check 3 - Shear and bearing of	bolts um of bolt capac $(n_{boltsb2} - 1) \times P_{sb}$ 0  kN / P <sub>sboltssumb2</sub> = 0.	ities olts = 235.5 kN 268 plate	hear capacity	N of bolt group to	Supporting I	Beam : PASS
$P_{sbolts1b2} = min(P_{sbolts}, 0).$ Shear capacity of other $P_{sbolts} = 58.9 \text{ kN}$ Shear capacity of bolt group - su ;P_{sboltssumb2} = 2 × P_{sbolts1b2} + 2 × () ;Shear on bolt group; Q <sub>b2</sub> = 63.0 Utilisation factor; U <sub>check2b2</sub> = Q <sub>b2</sub> Check 3 - Shear and bearing of Beam 1	bolts um of bolt capac $(n_{boltsb2} - 1) \times P_{sb}$ 0  kN $/ P_{sboltssumb2} = 0.$ capacity of end	lendplateb2 × Pbsendp oities olts = 235.5 kN 268 S plate	hear capacity	N of bolt group to	Supporting I	Beam : PASS
$P_{sbolts1b2} = min(P_{sbolts}, 0).$ Shear capacity of other $P_{sbolts} = 58.9 \text{ kN}$ Shear capacity of bolt group - su ;P_{sboltssumb2} = 2 × P_{sbolts1b2} + 2 × (); Shear on bolt group; Q_{b2} = 63.0) Utilisation factor; U_{check2b2} = Q_{b2} Check 3 - Shear and bearing of Beam 1 for shear	bolts um of bolt capac $(n_{boltsb2} - 1) \times P_{sb}$ 0  kN / $P_{sboltssumb2} = 0.$ capacity of end	lendplateb2 × Pbsendp bities olts = 235.5 kN 268 S plate	hear capacity	N of bolt group to	Supporting I	Beam : PASS
$P_{sbolts1b2} = min(P_{sbolts}, 0).$ Shear capacity of other $P_{sbolts} = 58.9 \text{ kN}$ Shear capacity of bolt group - su ;P_{sboltssumb2} = 2 × P_{sbolts1b2} + 2 × (); Shear on bolt group; Q_{b2} = 63.0 Utilisation factor; U_{check2b2} = Q_{b2} Check 3 - Shear and bearing of Beam 1 for shear ;;	bolts um of bolt capac (n <sub>boltsb2</sub> - 1) × P <sub>sb</sub> ) kN / P <sub>sboltssumb2</sub> = 0.	lendplateb2 × Pbsendp olts = 235.5 kN 268 plate	hear capacity	of bolt group to	Supporting I	Beam : PASS
$P_{sbolts1b2} = min(P_{sbolts}, 0).$ Shear capacity of other $P_{sbolts} = 58.9 \text{ kN}$ Shear capacity of bolt group - su ;P_{sboltssumb2} = 2 × P_{sbolts1b2} + 2 × (0); Shear on bolt group; $Q_{b2} = 63.00$ Utilisation factor; $U_{check2b2} = Q_{b2}$ Check 3 - Shear and bearing of Beam 1 for shear ;; $p_{yendplateb1} = 275$	bolts um of bolt capac $(n_{boltsb2} - 1) \times P_{sb}$ 0  kN $/ P_{sboltssumb2} = 0.$ capacity of end $5 \text{ N/mm}^2$	lendplateb2 × Pbsendp bities olts = 235.5 kN 268 plate	hear capacity	of bolt group to	Supporting I	Beam : PASS

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www.sd-structures.com	110	20/00/2011	WD	20/00/2017			
;; A <sub>vendplateb1</sub> = 0.9	imes (2 $ imes$ e <sub>1aendplate</sub>	$_{b1}$ + ( $n_{boltsb1}$ - 1)	imes p <sub>bolts</sub> ) $ imes$ t <sub>endplate</sub>	<sub>b1</sub> = <b>1170</b> mm <sup>2</sup>			
; $A_{vnetendplateb1} = A$	vendplateb1 - Nboltsb	$_{\rm h1}  imes {\sf D}_{\rm hbolts}  imes t_{\rm endpl}$	lateb1 = <b>810</b> mm <sup>2</sup>				
Effective net area coeffi	cient						
Keendplateb1 = 1.2	0						
Plain shear capacity of e	end plate						
$P_{vPendplateb1} = min(0.6 \times p)$	$D_{yendplateb1}  imes A_{vendplateb1}$	dplateb1, $0.7  imes K_{ee}$	ndplateb1 $ imes$ $p_{ ext{yendplat}}$	$_{eb1}  imes A_{vnetendplateb1}$	) = <b>187.1</b> kN		
$A_{v1endplateb1} = (e_1)$	laendplateb1 + (Nbolt	sb1 - 1) $ imes$ p <sub>bolts</sub> ) $ imes$	tendplateb1 = <b>900</b>	mm²			
; Ateffendplateb1 = (e	2endplateb1 - $0.5 imes$	$D_{hbolts}) \times t_{endplate}$	b1 = <b>410</b> mm <sup>2</sup>				
Block shear capacity of	end plate						
$P_{vBendplateb1} = 0.6 \times p_{vendplateb1}$	blateb1 × $A_{v1endplate}$	$_{eb1} + 0.6 \times K_{eendo}$	lateb1 $ imes$ <b>D</b> vendplateb1	$\times A_{\text{teffendplateb1}} =$	229.7 kN		
Shear capacity of the end plate:	Pvendplateb1 = mil	n (PvPendplateb1, P	vBendplateb1) = $187$	<b>′.1</b> kN			
:Shear force on each end plate	shear plane: Qb	1/2 = 11.0  kN					
Utilisation factor: Ucheck3shearb1 =	$Q_{b1} / (2 \times P_{vendpl})$	(1) = 0.059					
				Shear ca	nacity of end	nlate · PASS	
for bearing					<i>py</i> e. e	<i>p</i>	
$e_{endn ateh1} = e_{1endn at$	<b>40</b> mm						
bearing strength of the $\epsilon$	and plate						
$D_{brondplatch1} = 460 \text{ N/mm}^2$							
For top bolt							
bearing capacity of the	end plate per bo	olt					
$P_{bsendplate1b1} = min(d_{bolts})$	× t <sub>endplateb1</sub> × $p_{bse}$	endplateb1, $0.5  imes e_e$	ndplateb1 $ imes$ tendplateb	$_{\text{o1}} \times \mathbf{p}_{\text{bsendplateb1}} =$	<b>73.6</b> kN		
For other bolts,							
bearing capacity of the	end plate per bo	olt					
$P_{bsendplateb1} = d_{bolts} \times t_{endplateb1}$	lateb1 $ imes$ pbsendplatel	<sub>b1</sub> = <b>73.6</b> kN					
Capacity of bolt group;							
$P_{bsendplatesumb1} = 2 \times P_{bsendplate1b1}$	+ 2 $\times$ (n <sub>boltsb1</sub> - 1	$) \times P_{bsendplateb1} =$	<b>294.4</b> kN				
Bearing force on bolt group; Qb1	= <b>22.0</b> kN						
Utilisation factor; Ucheck3bearingb1 =	= Qb1 / Pbsendplates	sumb1 = <b>0.075</b>					
				Bearing ca	apacity of end	dplate : PASS	
Beam 2							
for shear							
· · · · · · · · · · · · · · · · · · ·							
Pyendplateb2 = 275	N/mm <sup>2</sup>						
;; e1aendplateb2 = e1e	endplateb2 = <b>40</b> mm	n					
;; Avendplateb2 = 0.9	imes (2 $ imes$ e1aendplate	b2 + (N <sub>boltsb2</sub> - 1)	imes <b>p</b> bolts) $ imes$ <b>t</b> endplate	<sub>b2</sub> = <b>1170</b> mm <sup>2</sup>			
; Avnetendplateb2 = A	vendplateb2 - Nboltsb	$_{2} \times D_{hbolts} \times t_{endpl}$	$t_{ateb2} = 810 \text{ mm}^2$				
Effective net area coeffi	cient						
Keendplateb2 = <b>1.2</b>	0						
Plain shear capacity of	end plate						
$P_{vPendplateb2} = min(0.6 \times r)$	$D_{vendplateh2} \times A_{vendplateh2}$	dplateb2, 0.7 × Kaa	ndplateb2 $ imes$ Dvendplat	eb2 $\times$ Avnetendolateb	2) = <b>187.1</b> kN		
	- , 5				,		

Strend Stren		Project				Job Ref.	
SD Structures       Section       Building Control Approval       129         Vior Place Streak London       Biol Control Approval       129         Activity Streak Streak Structures Control       Discontrol Approval       129         Millistation factor; Uprovations Control Approval       Discontrol Approval       129         Presentative Control Approval       Discontrol Approval       Discontrol Approval       129         Bearing capacity of the ond plate per bolt       Prevenduate Control Approval       Discontrol Approval       73.6 kN         Capacity of the ond plate per bolt       Prevenduate Control Control Approval       Discontrol Approval       73.6 kN	<b>CD</b> structures		28 Burg	hley Road		5	SDS187
Sol Structures       Building Control Approval       129         Inter Field Statu       Case, by       Date       Chikit by       Date       Applit by       Date         International Control Approval       RC       20/06/2017       MD       20/06/2017       Applit by       Date         Anteroplance 1 (Discottance - 0.5 × Directing) × Longinates = 400 mm <sup>2</sup> Anteroplance - 0.5 × Directing) × Longinates = 400 mm <sup>2</sup> Block shear capacity of end plate       Proceedpance = 0.5 × Directing) × Longinates = 410 mm <sup>2</sup> Structures       0.6 × Directing × Longinates = 0.6 × (2 × Participates = 0.5 × Directing) × Longinates = 0.7 kN       Shear capacity of the end plate, Provide and anteroplanet = 0.7 kN         Shear capacity of the end plate, Provide and = 0.0 × (2 × Participates = 0.168       Shear capacity of end plate : Professional = 0.0 × (2 × Participates = 0.0 × (2 × Participates = 0.0 × 0.0 × Directing)       Shear capacity of end plate : Professional = 0.0 × (2 × Participates > Participates = 0.0 × Directing = 0.0	SD Structures Chartered Engineers	Section				Sheet no./rev	Ι.
$ \begin{array}{c c c c c c c c c c c c c c c c c c c $	SD Structures		Building Co	ontrol Approval	l		129
www.ad structures.com         RC         26/06/2017         MD         26/06/2017           Auto-optimize = (0:anorphanes = (Counce - 1) × Pacing) × Longitudes = 900 mm <sup>2</sup> Auto-optimize = (0:anorphanes - 0.5 × Droom) × Longitudes = 900 mm <sup>2</sup> Block shear capacity of end plate         P-derodyname = 0.6 × Pyrophanes × Auto-optimes + 0.6 × Kenrophanes = 209.7 kN           Shear capacity of the end plate         P-derodyname = 0.6 × Pyrophanes × Auto-optimes + 0.6 × Kenrophanes = 187.1 kN           Shear capacity of the end plate         P-derodyname = 0.6 × (2 × Porophanes = 0.166 <i>for bearing Bearing</i> @engloads = e1 indpanet = 40 mm <i>Bearing</i> bearing capacity of the end plate <i>Porophanes</i> × porophanes × porophanes × porophanes + 0.5 × Europhanet × toophanes + toophanes + porophanes + 0.5 × Europhanet × toophanes + 0.5 × Europhanet × porophanes + 0.5 × Europhanet × toophanes + 0.5 × Europhanet × porophanes + 0.5 × Europhanet × toophanet × porophanet + P/           for bearing         Bearing capacity of the end plate per bolt         Pserophanet = 2 × Poworphanet × poworphanet = 73.6 kN           Capacity of bol group;         Bearing capacity of the end plate per bolt         Pserophanet = 2 × (Poworphanet = 1) × Pserophanet = 294.4 kN           Bearing capacity of the end plate per bolt         Pserophanet = 2 × (Poworphanet = 0.214         Bearing capacity of endplate : P/           Check 4 - shear capacity of the supported beam web at the endplate         Bearing capacity of	107 Fleet Street, London EC4A 2AB	Calc. by	Date	Chk'd by	Date	App'd by	Date
Arrequisted = (framplated + (floated - 1) × Pools) × Englande = 900 mm <sup>2</sup> ; Autorquance = (02000plate: 2.0.5 × Director) × Englande = 410 mm <sup>2</sup> Block shear capacity of end plate Petendiates = 0.6 × pereplates × Averequance = 0.6 × Konsequance × pereplates × Astronguance = 229.7 kN Shear capacity of the end plate shear plane; Qar / 2 = 31.5 kN Utilisation factor; Unexclationare = Oae / (2 × Pereplates = 0.168 Shear capacity of end plate : Pereplates = 40 mm bearing strength of the end plate pereplates = 460 N/mm <sup>2</sup> For top bolt, bearing capacity of the end plate per bolt Promotypates = 640 N/mm <sup>2</sup> For otop bolt, bearing capacity of the end plate per bolt Promotypates = 0 the subject = 73.6 kN Capacity of bolt group; Pereplates = 0 the subject = 27.6 kN Capacity of bolt group; Pereplates = 0 the subject = 27.6 kN Capacity of bolt group; Pereplates = 0 the subject = 27.8 kN Earling capacity of the end plate per bolt Promotypates = 0 the subject = 27.8 kN Earling capacity of end plate : PP Check 4 - shear capacity of the supported beam web at the endplate Beam 1 ; properties = 0.9 × min((endplate), despondent) × toppondent) = :632; mm <sup>2</sup> ; Shear capacity of supported beam web Promotypates = 0.9 × min((endplate), despondent) = :04.2 kN ; thear force on supported beam; On = = 22.0 kN Utilisation factor; Unexclosures = 0.9 × min((endplate) = 0.214 Shear capacity of supported beam web Progenome = 0.9 × min((endplate) = 0.22.0 kN Utilisation factor; Unexclosures = 0.9 × min((endplate) = 0.211 Shear capacity of supported beam web : PP Beam 2 ; psupported beam web : PZ Beam 2 ; psupported beam web : PZ	www.sd-structures.com	RC	26/06/2017	MD	26/06/2017		
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<pre>comparative = (continues = 0.00 monoparative / continues = 0.00 min) Block shear capacity of end plate Pdeequates = 0.6 × preprint = 0.6 × preprint = 0.6 × Keeroparates × Anteroparates = 229.7 kN Shear capacity of the end plate; Pveroparates = min (Pveroparates, Pveroparates) = 187.1 kN Shear capacity of end plate shear plane; Que / 2 = 31.5 kN Utilisation factor; U-exclusive: = 40 mm bearing ecodemics = 0 rendometer = 0 kr / (2 × Pveroparates) = 0.168 Shear capacity of end plate : P/ for bearing ecodemics = 0 rendometer = 40 mm bearing strength of the end plate per bolt Pseroparates = min(Conte × Integrates × poweroparates) = 0.5 × 0eroparates × Introductes × poweroparates) = 73.6 kN For other bolts, bearing capacity of the end plate per bolt Pseroparates = 0 × min(Conte × 1 × Pseroparates = 73.6 kN Capacity of bolt group; Poweroparates = 0 × 0 × 0 × 0 × 0 × 0 × 0 × 0 × 0 × 0</pre>	· Atoffandplatch2 -	(Crachopiatobe - 0 F	5 × Dhhalla) × tandriat	$-410 \text{ mm}^2$			
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<pre>pbsendplatetz = 460 N/mm<sup>2</sup> For top bolt, bearing capacity of the end plate per bolt Pbsendplatetiz = min(dbate × tendplateb2 × pbsendplateti2, 0.5 × 6endplateti2 × tendplateti2 × pbsendplateti2) = 73.6 kN For other bolts, bearing capacity of the end plate per bolt Pbsendplatesum2 = 2 × Pbsendplateti2 × 2 pbsendplateti2 = 73.6 kN Capacity of bolt group; Pbsendplatesum2 = 2 × Pbsendplateti2 + 2 × (Notest2 - 1) × Pbsendplateti2 = 294.4 kN Bearing capacity of endplate ti2 + 2 × (Notest2 - 1) × Pbsendplateti2 = 294.4 kN Bearing force on bolt group; Qbz = 63.0 kN Utilisation factor; Ucheckteeningb2 = Qbz / Pbsendplatesum2 = 0.214  Check 4 - shear capacity of the supported beam web at the endplate Beam 1 ; Pysupportedb1 = 0.9 × min(lendplate1, dsupportedb1) × tappontedb1 = ;632; mm<sup>2</sup>; Shear capacity of supported beam web Psupported beam web Psupported beam; Qb1 = 22.0 kN Utilisation factor; Ucheckteetat1 = Qb1 / Psupportedb1 = 104.2 kN ;Shear force on supported beam; Qb1 = 22.0 kN Utilisation factor; Ucheckteetat1 = Qb1 / Psupportedb1 = 0.211  Fean 2 ; pysupportedb2 = 275 N/mm<sup>2</sup> ; </pre>	bearing strength of th	e end plate					
For top bolt, bearing capacity of the end plate per bolt Posendplatet2 = min(dbolts × tendplatet2 × posendplatet2, 0.5 × 0endplatet2 × tendplatet2 × posendplatet2) = <b>73.6</b> kN For other bolts, bearing capacity of the end plate per bolt Posendplatet2 = dbolts × tendplatet2 × posendplatet2 = <b>73.6</b> kN Capacity of bolt group; Posendplatet2 = dbolts × tendplatet2 × posendplatet2 = <b>73.6</b> kN Capacity of bolt group; Posendplatet2 = 2 × Posendplatet12 + 2 × (Noutsb2 - 1) × Posendplatet2 = <b>294.4</b> kN Bearing capacity of endplate = <b>0.2</b> / Posendplatesum02 = <b>0.214</b> Check 4 - shear capacity of the supported beam web at the endplate Beam 1 ; pysupportedb = 0.9 × min(lendplatet), dsupportedb = ;632; mm <sup>2</sup> ; Shear capacity of supported beam; web Pvsupportedb = 0.9 × min(lendplatet), dsupportedb = = 104.2 kN ;Shear force on supported beam; Qbit = <b>22.0</b> kN Utilisation factor; Uchecketerant = Qbit / Pvsupportedb = 0.211 Shear capacity of supported beam web : PA Beam 2 ; pysupportedb = <b>275</b> N/mm <sup>2</sup> ; pysupportedb = <b>275</b> N/mm <sup>2</sup>	pbsendplateb2 = 460 N/m	1m²					
bearing capacity of the end plate per bolt Posendplatetiz = min(dbolts × tendplatetiz × posendplatetiz, 0.5 × @endplatetiz × tendplatetiz × posendplatetiz) = <b>73.6</b> kN For other bolts, bearing capacity of the end plate per bolt Posendplatetiz = douts × tendplatetiz × Dosendplatetiz = <b>73.6</b> kN Capacity of bolt group; Posendplatesumbz = 2 × Posendplatetiz + 2 × (nboltstiz - 1) × Posendplatetiz = <b>294.4</b> kN Bearing force on bolt group; Qoz = <b>63.0</b> kN Utilisation factor; Ucheckdetearingtiz = Qoz / Posendplatetiz = <b>0.214</b> <b>Bearing capacity of endplate</b> : <b>P</b> / <b>Check 4 - shear capacity of the supported beam web at the endplate</b> <b>Beam 1</b> ; pysupportedb = <b>275</b> N/mm <sup>2</sup> ;; , Avsupportedb = 0.9 × min([endplatetin, dsupportedb ] × tsupportedb ] = ; <b>632</b> ; mm <sup>2</sup> ; Shear capacity of supported beam; Qo <sub>1</sub> = <b>22.0</b> kN Utilisation factor; Uchecketstearb = Qo <sub>1</sub> / Pysupportedb ] = <b>104.2</b> kN ;Shear force on supported beam; Qo <sub>1</sub> = <b>22.0</b> kN Utilisation factor; Uchecketstearb = Qo <sub>1</sub> / Pysupportedb ] = <b>0.211</b> <b>Beam 2</b> ; pysupportedb = <b>275</b> N/mm <sup>2</sup> ; pysupportedb = <b>275</b> N/mm <sup>2</sup> ; pysupportedb = <b>275</b> N/mm <sup>2</sup>	For top bolt,						
Pisendplatetiz = min(dbots × tendplateb2 × pisendplateb2, 0.5 × 6endplateb2 × pisendplateb2) = 73.6 kN For other holts, bearing capacity of the end plate per bolt Pisendplateb2 = dbots × tendplateb2 × pisendplateb2 = 73.6 kN Capacity of bolt group; Pisendplatesumb2 = 2 × Pisendplate1tb2 + 2 × (nbottst2 - 1) × Pisendplateb2 = 294.4 kN Bearing force on bolt group; Obz = 63.0 kN Utilisation factor; Uchecksbearingb2 = Obz / Pisendplatesumb2 = 0.214 <i>Bearing capacity of endplate</i> : <i>Pi</i> Check 4 - shear capacity of the supported beam web at the endplate Beam 1 ; pysupportedb1 = 0.9 × min((lendplateb1, dsupportedb1) × tsupportedb1 = ;632; mm <sup>2</sup> ; Shear capacity of supported beam web Pvsupportedb1 = 0.6 × pysupportedb1 = 104.2 kN ;Shear force on supported beam; Qb1 = 22.0 kN Utilisation factor; Ucheckshearb1 = Cb1 / Pvsupportedb1 = 0.211 <i>Shear capacity of supported beam web</i> : <i>Pi</i> Beam 2 ; pysupportedb2 = 275 N/mm <sup>2</sup> ; pysupportedb2 = 275 N/mm <sup>2</sup>	bearing capacity of th	ie end plate per	bolt				
For other holds, bearing capacity of the end plate per bolt Posendplateb2 = dbolts × tendplateb2 × Posendplateb2 = <b>73.6</b> kN Capacity of bolt group; Posendplatesumb2 = 2 × Posendplate102 + 2 × (fboltsb2 - 1) × Posendplateb2 = <b>294.4</b> kN Bearing force on bolt group; Qu2 = <b>63.0</b> kN Utilisation factor; Ucheck3bearingb2 = Qu2 / Posendplatesumb2 = <b>0.214</b> <b>Bearing capacity of endplate</b> : PA <b>Check 4 - shear capacity of the supported beam web at the endplate</b> <b>Beam 1</b> ; pysupportedb1 = <b>275</b> N/mm <sup>2</sup> ;; Avsupportedb1 = 0.9 × min(lendplateb1, dsupportedb1 = ; <b>632</b> ; mm <sup>2</sup> ; Shear capacity of supported beam web Pvsupportedb1 = 0.6 × Pysupportedb1 = <b>104.2</b> kN ;Shear force on supported beam; Qu5 = <b>22.0</b> kN Utilisation factor; Ucheck4shearb1 = Qu5 / Pvsupportedb1 = <b>0.211</b> <b>Shear capacity of supported beam web</b> : PA <b>Beam 2</b> ; pysupportedb2 = <b>275</b> N/mm <sup>2</sup> ;; pysupportedb2 = <b>275</b> N/mm <sup>2</sup>	$P_{bsendplate1b2} = min(d_{bc})$	olts $ imes$ tendplateb2 $ imes$ f	Obsendplateb2, $0.5  imes e$	endplateb2 $ imes$ tendpl	lateb2 $ imes$ <b>P</b> bsendplateb2) :	= <b>73.6</b> kN	
Posendplateb2 = dbotts × tendplateb2 × posendplateb2 = 73.6 kN Capacity of bolt group; Posendplatesumb2 = 2 × Posendplate1b2 + 2 × (flootb2 - 1) × Posendplateb2 = 294.4 kN Bearing force on bolt group; Ob2 = 63.0 kN Utilisation factor; Ucheck3bearingb2 = Ob2 / Posendplatesumb2 = 0.214 <i>Bearing capacity of endplate : PA</i> Check 4 - shear capacity of the supported beam web at the endplate Beam 1 ; pysupportedb1 = 275 N/mm <sup>2</sup> ;; Avsupportedb1 = 0.9 × min(lendplate1, dsupportedb1) × tsupportedb1 = ;632; mm <sup>2</sup> ; Shear capacity of supported beam web Pvsupportedb1 = 0.6 × pysupportedb1 = 104.2 kN ;Shear force on supported beam; Ob1 = 22.0 kN Utilisation factor; Ucheck4tehearb1 = Ob1 / Pvsupportedb1 = 0.211 <i>Shear capacity of supported beam web</i> : <i>PA</i> Beam 2 ; pysupportedb2 = 275 N/mm <sup>2</sup>	For other bolts, bearing capacity of th	ne end plate per	bolt				
Capacity of bolt group; Posendplatesumts2 = 2 × Posendplate tb2 + 2 × (nbottsb2 - 1) × Posendplateb2 = 294.4 kN Bearing force on bolt group; Qb2 = 63.0 kN Utilisation factor; Ucheck3bearingb2 = Qb2 / Posendplatesumts2 = 0.214 <b>Bearing capacity of endplate :</b> P/ <b>Check 4 - shear capacity of the supported beam web at the endplate</b> <b>Beam 1</b> ; pysupportedb1 = 275 N/mm <sup>2</sup> ;; Avsupportedb1 = 0.9 × min(lenoptateb1, dsupportedb1) × tsupportedb1 = ;632; mm <sup>2</sup> ; Shear capacity of supported beam web Pvsupportedb1 = 0.6 × pysupportedb1 × Avsupportedb1 = 104.2 kN ;Shear force on supported beam; Qb1 = 22.0 kN Utilisation factor; Ucheck3hearb1 = Qb1 / Pvsupportedb1 = 0.211 <b>Shear capacity of supported beam web</b> : P/ <b>Beam 2</b> ; pysupportedb2 = 275 N/mm <sup>2</sup> ;; pysupportedb2 = 275 N/mm <sup>2</sup>	$P_{bsendplateb2} = d_{bolts} \times t_{d}$	endplateb2 $ imes$ $p$ bsendp	olateb2 = <b>73.6</b> kN				
Posendplatesumb2 = 2 × Posendplate1b2 + 2 × (Notisb2 - 1) × Posendplateb2 = 294.4 kN Bearing force on bolt group; Qb2 = 63.0 kN Utilisation factor; Ucheck3bearingb2 = Qb2 / Posendplatesumb2 = 0.214 <b>Bearing capacity of endplate</b> : P/ <b>Check 4 - shear capacity of the supported beam web at the endplate</b> <b>Beam 1</b> ;; pysupportedb1 = 275 N/mm <sup>2</sup> ;; ; Avsupportedb1 = 0.9 × min(lendplate1, dsupportedb1) × tsupportedb1 = ;632; mm <sup>2</sup> ; Shear capacity of supported beam web Pvsupportedb1 = 0.6 × pysupportedb1 × Avsupportedb1 = 104.2 kN ;Shear force on supported beam; Qb1 = 22.0 kN Utilisation factor; Ucheck4shearb1 = Qb1 / Pvsupportedb1 = 0.211 <b>Beam 2</b> ;; pysupportedb2 = 275 N/mm <sup>2</sup> ;; pysupportedb2 = 275 N/mm <sup>2</sup>	Capacity of bolt group;						
Bearing force on bolt group; Q <sub>62</sub> = 63.0 kN Utilisation factor; U <sub>checkdbearingb2</sub> = Q <sub>62</sub> / P <sub>bsendplatesumb2</sub> = 0.214 <i>Bearing capacity of endplate : PJ</i> Check 4 - shear capacity of the supported beam web at the endplate Beam 1 ;; pysupportedb1 = 275 N/mm <sup>2</sup> ;; Avsupportedb1 = 0.9 × min(lendplateb1, dsupportedb1) × tsupportedb1 = ;632; mm <sup>2</sup> ; Shear capacity of supported beam web Pvsupportedb1 = 0.6 × pysupportedb1 × Avsupportedb1 = 104.2 kN ;Shear force on supported beam; Q <sub>b1</sub> = 22.0 kN Utilisation factor; U <sub>checkdshearb1</sub> = Q <sub>b1</sub> / Pvsupportedb1 = 0.211 <i>Shear capacity of supported beam web : PJ</i> Beam 2 ; pysupportedb2 = 275 N/mm <sup>2</sup> ;; pysupportedb2 = 275 N/mm <sup>2</sup>	$P_{bsendplatesumb2} = 2 \times P_{bsendplate1}$	$_{1b2}$ + 2 × ( $n_{boltsb2}$	- 1) × P <sub>bsendplateb2</sub> =	= <b>294.4</b> kN			
Utilisation factor; U <sub>check3bearingb2</sub> = Q <sub>b2</sub> / P <sub>bsendplatesumb2</sub> = 0.214 <b>Bearing capacity of endplate :</b> PJ <b>Check 4 - shear capacity of the supported beam web at the endplate</b> <b>Beam 1</b> ;; $p_{ysupportedb1} = 275 \text{ N/mm}^2$ ;; Avsupportedb1 = 0.9 × min(lendplateb1, dsupportedb1) × tsupportedb1 = ;632; mm <sup>2</sup> ; Shear capacity of supported beam web $P_{vsupportedb1} = 0.6 \times p_{ysupportedb1} \times Avsupportedb1 = 104.2 \text{ kN}$ ;Shear force on supported beam; Q <sub>b1</sub> = 22.0 kN Utilisation factor; U <sub>check4shearb1</sub> = Q <sub>b1</sub> / P <sub>vsupportedb1</sub> = 0.211 <b>Shear capacity of supported beam web :</b> PA <b>Beam 2</b> ;; $p_{ysupportedb2} = 275 \text{ N/mm}^2$ ;;	Bearing force on bolt group;	Q <sub>b2</sub> = <b>63.0</b> kN					
Bearing capacity of endplate : PA Check 4 - shear capacity of the supported beam web at the endplate Beam 1 ; pysupportedb1 = 275 N/mm2 ;; Avsupportedb1 = 0.9 × min(lendplateb1, dsupportedb1) × tsupportedb1 = ;632; mm2; Shear capacity of supported beam web Pvsupportedb1 = 0.6 × pysupportedb1 × Avsupportedb1 = 104.2 kN ;Shear force on supported beam; Qb1 = 22.0 kN Utilisation factor; Ucheck4shearb1 = Qb1 / Pvsupportedb1 = 0.211 Shear capacity of supported beam web : PA Beam 2 ; pysupportedb2 = 275 N/mm2 ;; puspportedb2 = 275 N/mm2	Utilisation factor; Ucheck3bearing	$p_{2} = Q_{b2} / P_{bsendp}$	latesumb2 = 0.214				
Check 4 - shear capacity of the supported beam web at the endplate Beam 1 ;; pysupportedb1 = 275 N/mm <sup>2</sup> ;; Avsupportedb1 = 0.9 × min(lendplateb1, dsupportedb1) × tsupportedb1 = ;632; mm <sup>2</sup> ; Shear capacity of supported beam web Pvsupportedb1 = 0.6 × pysupportedb1 × Avsupportedb1 = 104.2 kN ;Shear force on supported beam; Qb1 = 22.0 kN Utilisation factor; Ucheck4shearb1 = Qb1 / Pvsupportedb1 = 0.211 Shear capacity of supported beam web : PA Beam 2 ; pysupportedb2 = 275 N/mm <sup>2</sup> ;; pysupportedb2 = 275 N/mm <sup>2</sup>					Bearing c	apacity of e	endplate : PA
Beam 1 ;; pysupportedb1 = 275 N/mm <sup>2</sup> ;; Avsupportedb1 = 0.9 × min(lendplateb1, dsupportedb1) × tsupportedb1 = ;632; mm <sup>2</sup> ; Shear capacity of supported beam web Pvsupportedb1 = 0.6 × pysupportedb1 × Avsupportedb1 = 104.2 kN ;Shear force on supported beam; Q <sub>b1</sub> = 22.0 kN Utilisation factor; Ucheck4shearb1 = Q <sub>b1</sub> / Pvsupportedb1 = 0.211 Shear capacity of supported beam web : PA Beam 2 ;; pysupportedb2 = 275 N/mm <sup>2</sup> ;;	Check 4 - shear capaci	ty of the suppo	orted beam web a	t the endplate	e		
;; $p_{ysupportedb1} = 275 \text{ N/mm}^2$ ;; Avsupportedb1 = 0.9 × min(lendplateb1, dsupportedb1) × tsupportedb1 = ;632; mm^2; Shear capacity of supported beam web $P_{vsupportedb1} = 0.6 × p_{ysupportedb1} × Avsupportedb1 = 104.2 \text{ kN}$ ;Shear force on supported beam; $Q_{b1} = 22.0 \text{ kN}$ Utilisation factor; Ucheck4shearb1 = $Q_{b1} / P_{vsupportedb1} = 0.211$ Shear capacity of supported beam web : P/ Beam 2 ;; $p_{ysupportedb2} = 275 \text{ N/mm}^2$ ;;	Beam 1						
$p_{ysupportedb1} = 275 \text{ N/mm}^{2}$ ;; Avsupportedb1 = 0.9 × min(lendplateb1, dsupportedb1) × tsupportedb1 = ;632; mm <sup>2</sup> ; Shear capacity of supported beam web Pvsupportedb1 = 0.6 × pysupportedb1 × Avsupportedb1 = 104.2 kN ;Shear force on supported beam; Qb1 = 22.0 kN Utilisation factor; U <sub>check4shearb1</sub> = Qb1 / Pvsupportedb1 = 0.211 Shear capacity of supported beam web : PA Beam 2 ;; pysupportedb2 = 275 N/mm <sup>2</sup> ;;	"						
<pre>;;; ; Avsupportedb1 = 0.9 × min(lendplateb1, dsupportedb1) × tsupportedb1 = ;632; mm<sup>2</sup>; Shear capacity of supported beam web Pvsupportedb1 = 0.6 × pysupportedb1 × Avsupportedb1 = 104.2 kN ;Shear force on supported beam; Qb1 = 22.0 kN Utilisation factor; Ucheck4shearb1 = Qb1 / Pvsupportedb1 = 0.211 Shear capacity of supported beam web : PA Beam 2 ;; pysupportedb2 = 275 N/mm<sup>2</sup> ;; </pre>	Pysuppor	<sub>tedb1</sub> = <b>275</b> N/mr	m²				
<pre>, Avsupportedb1 = 0.9 × min(lendplateb1, dsupportedb1) × tsupportedb1 = ;632; mm<sup>2</sup>; Shear capacity of supported beam web Pvsupportedb1 = 0.6 × pysupportedb1 × Avsupportedb1 = 104.2 kN ;Shear force on supported beam; Qb1 = 22.0 kN Utilisation factor; Ucheck4shearb1 = Qb1 / Pvsupportedb1 = 0.211 Shear capacity of supported beam web : PA Beam 2 ;; pysupportedb2 = 275 N/mm<sup>2</sup> ;; ; </pre>							
Shear capacity of supported beam web $P_{vsupportedb1} = 0.6 \times p_{ysupportedb1} \times A_{vsupportedb1} = 104.2 \text{ kN}$ ;Shear force on supported beam; $Q_{b1} = 22.0 \text{ kN}$ Utilisation factor; $U_{check4shearb1} = Q_{b1} / P_{vsupportedb1} = 0.211$ <b>Beam 2</b> ;; $p_{ysupportedb2} = 275 \text{ N/mm}^2$ ;;	, Δ	0 × min/l	d) × t		mm <sup>2</sup> ·		
Product of predict of our for the section in the section of the section in the section of the section is the section in the section is the section is the section is the section in the section is the s	Avsupported bit = 0.3	rted beam web	, Usupportedb1) × Isupp	ortedb1 = ,032, 1			
<pre>;Shear force on supported beam; Q<sub>b1</sub> = 22.0 kN Utilisation factor; U<sub>check4shearb1</sub> = Q<sub>b1</sub> / P<sub>vsupportedb1</sub> = 0.211 Beam 2 ;; pysupportedb2 = 275 N/mm<sup>2</sup> ;; ;</pre>	$P_{\text{vsupportedb1}} = 0.0$	$6 \times D_{\text{VSUpportedb1}} \times$	Avsupportedb1 = $104$	. <b>2</b> kN			
Utilisation factor; U <sub>check4shearb1</sub> = Q <sub>b1</sub> / P <sub>vsupportedb1</sub> = <b>0.211</b> Shear capacity of supported beam web : P <sub>4</sub> Beam 2 ;; p <sub>ysupportedb2</sub> = <b>275</b> N/mm <sup>2</sup> ;; ;	;Shear force on supporte	ed beam; $Q_{b1} = 2$	22.0 kN				
Shear capacity of supported beam web : PA Beam 2 ;; pysupportedb2 = 275 N/mm <sup>2</sup> ;; ;	Utilisation factor; Ucheck4s	hearb1 = $Q_{b1} / P_{vs}$	upportedb1 = <b>0.211</b>				
Beam 2 ;; pysupportedb2 = 275 N/mm <sup>2</sup> ;; ;				Sh	ear capacity of su	ipported be	am web : PA
;; pysupportedb2 = <b>275</b> N/mm <sup>2</sup> ;;	Beam 2						
pysupportedb2 = <b>275</b> N/mm <sup>2</sup> ;;	;;						
;; ;	Pysuppor	tedb2 = <b>275</b> N/mr	n²				
	,,,,						
	;	0	-l ``.	~ ~ ~	2.		

Shear capacity of supported beam web

		Project				Job Ref.	
	ctructuroc		28 Burg	SE	S187		
	Chartered Engineers	Section				Sheet no./rev.	
	SD Structures		Building Co	ntrol Approval			130
10	07 Fleet Street, London	Calc. by	Date	Chk'd by	Date	App'd by	Date
v	EC4A 2AB www.sd-structures.com	RC	26/06/2017	MD	26/06/2017		
					1		
	$P_{vsupportedb2} = 0.62$	$\times p_{ysupportedb2} \times d$	Avsupportedb2 = <b>139.</b>	<b>0</b> kN			
	;Shear force on supported	beam; $Q_{b2} = 6$	<b>3.0</b> kN				
	Utilisation factor; Ucheck4she	$_{arb2} = Q_{b2} / P_{vsu}$	pportedb2 = <b>0.453</b>				
				Shea	ar capacity of su	pported bea	m web : PASS
(	Check 5 - capacity of the	fillet welds c	onnecting the en	d plate to the	supported beam	web	
ļ	Beam 1						
	; Effective throat si	ze of weld; a <sub>we</sub>	$_{\text{ldb1}} = S_{\text{weldb1}} \times 0.7$	= <b>4.2</b> mm			
	,,,						
	Effective length o	f weld; I <sub>weldb1</sub> =	$2 \times (min(l_{endplateb1}))$	, $2 \times r_{supportedb1}$ -	+ d <sub>supportedb1</sub> ) - 2 ×	Sweldb1) = ;236	<b>5.0</b> ; mm
	; Design strength c	of weld; p <sub>weldb1</sub> :	= <b>220</b> N/mm <sup>2</sup>				
(	Capacity of fillet welds; Pw	eldb1 = $p_{weldb1} \times$	$I_{weldb1} \times a_{weldb1} = 2$	1 <b>8.1</b> kN			
	;Utilisation factor; Ucheck5we	Idb1 = Qb1 / Pwel	db1 = <b>0.101</b>				
					Ca	pacity of fille	t weld : PASS
	Beam 2						
	: Effective throat si	ze of weld: awe	$Idh2 = Sweldh2 \times 0.7$	= <b>4.2</b> mm			
	Fffective length o	f weld: Iweldb2 =	$2 \times (min(l_{endplateb}))$	$2 \times r_{supportedb2}$ -	+ dsupportedb2) - 2 ×	Sweldh2) = .236	<b>.0</b> . mm
	· Design strength c	of weld: nweldb2 =	= 220 N/mm <sup>2</sup>			Gweldb2) - , <b>20</b> 4	
	Capacity of fillet welds: Pw	$aldb2 = \mathbf{D}_{weldb2} \times \mathbf{C}_{weldb2}$	$l_{weldb2} \times a_{weldb2} = 2$	18.1 kN			
	:Utilisation factor: Ucheck5we	Idh2 = Qh2 / Pwell	db2 = 0.289				
					Ca	pacity of fille	t weld : PASS
	Chook 6a - bonding cona	aity of roduce	d supported bog	m soction at t	ha natah - 1 flan	ao notohod	
	Roam 1	city of reduce	a supported bed	in section at t		genotcheu	
	Properties of notched sect	ion ianorina ro	ot radii				
	Denth of web: d		1 - dataanatahki - Tau		mm		
	:: Area of notched a		$d = d_{uubbd} \times t_{uubbd}$		······	<b>832</b> mm <sup>2</sup>	
	Distance from top of notch	to centroid of	notched section			0.02	
I		$d_{\rm max}^2 \times t_{\rm max}^2$		ья — Т м. н. м.	(during + Trunger		le e elle d
	y barb1 = (						nedbi
	Inertia of notched section	27					
	Inotchedb1 = tsupported	$_{\rm Ib1}  imes {\sf d}_{\rm webb1}^3 / 12$ B <sub>SUPDOTEdb1</sub> $ imes$ T	$2 + t_{supportedb1} \times d_{we}$	$(y_{barb1} \times (y_{barb1} - d_{v})$	$(2)^2 + (d_{webb1} \times (d_{webb1} + d_{webb1}))^2$	T <sub>supportedb1</sub> / 2	- V <sub>barb1</sub> ) <sup>2</sup>
	Inotchedb1 = 542 cm	4		,,opi	,		- /
	Modulus of notched sectio	n					
	Znotchedb1 = Inotchedl	$y_{barb1} = 42.8$	<b>3</b> cm <sup>3</sup>				
	Moment capacity of notch	ed section; Mca	apnotchedb1 = $\mathbf{p}_{\text{Vsupport}}$	tedb1 $ imes$ Znotchedb1	= <b>11.8</b> kNm		
	; Top notch length:	Ctopnotchb1 = 13	<b>3</b> mm				
	;;Moment applied to notch	ed section: Man	pnotchedb1 = $Q_{b1} \times ($	Ctopnotchb1+ tendola	ateb1) = <b>3.1</b> kNm		
	Utilisation factor; Ucheck6amo	omentb1 = Mappnot	chedb1 / Mcapnotchedb	i = 0.268	,		
	,					Моте	nt capacity of
						notched se	ection : PASS
I	Beam 2						
	Properties of notched sect	ion ignoring roo	ot radii				

;;; Depth of web;  $d_{webb2} = D_{supportedb2} - d_{ctopnotchb2} - T_{supportedb2} = 162.2 \text{ mm}$ 

;; Area of notched section;  $A_{notchedb2} = d_{webb2} \times t_{supportedb2} + B_{supportedb2} \times T_{supportedb2} = 3407 \text{ mm}^2$ 

		Project				Job Ref.	
C	nstructures		28 Burg		SDS187		
3	Chartered Engineers	Section				Sheet no./rev.	
10	SD Structures		Building Co	ntrol Approval			131
10	EC4A 2AB	Calc. by	Date	Chk'd by	Date	App'd by	Date
w	ww.sd-structures.com	RC	26/06/2017	MD	26/06/2017		
[	Distance from top of notch	to centroid of n	otched section				
	$y_{barb2} = (0)$	$d_{webb2^2}  imes t_{supporte}$	db2 / 2 + B <sub>supported</sub>	$_{b2}  imes T_{supportedb2}  imes$	(dwebb2 + Tsupporte	db2 / 2)) / A <sub>notch</sub>	iedb2
	<b>y</b> <sub>barb2</sub> = <b>1</b>	<b>38</b> mm					
l	nertia of notched section						
	Inotchedb2 = tsupported	$b_2 \times d_{webb2^3} / 12$ Bsupportedb2 $\times T_{st}$	+ $t_{supportedb2} \times d_{we}$ upportedb2 <sup>3</sup> / 12 + B	$_{ m abb2}  imes (y_{ m barb2} - d_{ m we}$ supportedb2 $ imes T_{ m support}$	$_{bb2} / 2)^2 +$ ortedb2 × (dwebb2 +	T <sub>supportedb2</sub> / 2 -	Ybarb2) <sup>2</sup>
	$I_{notchedb2} = 834 \text{ cm}^3$	4					
Ν	Modulus of notched section	1 ,	2				
	Znotchedb2 = Inotchedb	2 / Ybarb2 = <b>60.4</b>	cm <sup>3</sup>	-			
;	Moment capacity of notche	ed section; M <sub>cap</sub>	pnotchedb2 = $p_{ysupport}$	tedb2 $\times$ Znotchedb2 =	₌ <b>16.6</b> kNm		
;	I op notch length;	Ctopnotchb2 = 133	s mm				
;	invoment applied to notche	eu section; Mapp	notchedb2 = $Qb2 \times (0)$	Ctopnotchb2+ lendplate	eb2) = <b>9.0</b> KNM		
Ĺ	JUIISAUOTI TACIOF, Ucheck6amo	mentb2 = IVIappnotch	nedb2 / IVIcapnotchedb2	2 = 0.342		Momon	t conceity of
						notched se	ction : PASS
Chec	k 6b - local stability of n	otched suppor	rted beams rest	rained against I	ateral torsional	buckling - 1	flange
notch	ned						
Bean	n 1						
;;	Depth of Notch; dctopnotc	hb1 = <b>30</b> mm					
	Length of notch; ctopnotch	hb1 = <b>133</b> mm					
	-			:Depth	of top notch le	ss than limit	of D/2 : PASS
•	Gradesupportedb1 = "S275	5''		· •			
;;	dot <sub>b1</sub> = D <sub>supportedb1</sub> /t <sub>suppor</sub>	tedb1 = <b>37.6</b>					
	Climitb1 = if(or(Gradesupport	rtedb1 == "S275"	, Gradesupportedb1 =	== "300"), if(dot <sub>b</sub>	1<=54.3,D <sub>supporte</sub>	db1,160000 $ imes$	
Dsuppo	$brtedb1/dotb1^3),$						
	if(dot <sub>b1</sub> <=48.0,	Dsupportedb1,1100	$000 \times D_{supportedb1/c}$	dot <sub>b1</sub> 3))			
Climitb1	= <b>203.2</b> mm						
				L	Length of top n	otch less thai	n limit : PASS
Bean	n 2						
;;	Depth of Notch; d <sub>ctopnotc</sub>	hb2 = <b>30</b> mm					
	Length of notch; ctopnotch	<sub>hb2</sub> = <b>133</b> mm					
	-			:Depth	of top notch le	ss than limit	of D/2 : PASS
:	Gradesupportedb? = "\$275	5''		, - <b>p</b>	,		
	$dot_{h2} = D_{supportsdh2}/t_{support}$	tedh2 = <b>28</b> .2					
,,	$Climith_2 = if(Or(Grade_{action}))$	rtedb2 == "\$275"	Grade	== "300") if(dot⊾	2<=54.3 Dournette		
Dsuppo	$\frac{1}{1000} = \frac{1}{1000} \frac{1}{10$		, and supported 2 -			1029 1 0 0 0 0 0 A	
- Juhhr	if(dot <sub>b2</sub> <=48.0.	D <sub>supportedb2</sub> ,1100	$000  imes D_{supportedb2}/c$	dot <sub>b2</sub> 3))			
	= <b>203.2</b> mm	,					
				I	Lenath of ton n	otch less that	n limit : PASS
Chec	k 7 - Local shear and be	aring canacity	of Sunnorting F	- Beam web			
for of		anny capacity	si sapporting t				
	o <b>00</b> mm						
,,		- (	\ <b>.</b>	、 <b>.</b>	1057 0		
,,,,	$A_{vsupporting} = (p_{bolts} + (m))$	n(nboltsb1, nboltsb2	$(-1) \times p_{bolts} + e_{ts}$	upporting) $ imes$ $\mathfrak{l}_{ ext{supporting}}$	<sub>ig</sub> = 1957 mm²		

	Project				Job Ref.	
structures		28 Burg		SDS187		
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SD Structures		Building Co	ntrol Approval	Γ		132
EC4A 2AB	Calc. by	Date	Chk'd by	Date	App'd by	Date
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; Avnetsupporting = Avsupporting	<sub>g</sub> - min(n <sub>boltsb1</sub> ,n <sub>bo</sub>	$_{ m itsb2})  imes D_{ m hbolts}  imes t_{ m s}$	supporting = <b>1586</b> m	1m²		
,						
pysupporting = 265	<b>5</b> N/mm²					
Effective net area coeff	ficient					
Kesupporting = <b>1.2</b>	20					
$P_{vsupporting} = min(0.6 \times p)$	$\mathbf{D}_{ysupporting} \times \mathbf{A}_{vsupport}$	orting, $0.7  imes K_{esupp}$	porting $\times \mathbf{p}_{\text{ysupporting}}$	$\times A_{\text{vnetsupporting}} =$	<b>311.2</b> kN	
$\therefore \qquad Q_v = Q_{h1} \times if(n_{holtsh2} < n_{holtsh2})$	oltsb1. Nboltsb2/Nboltst	$_{1.1}$ ) + $Q_{b2} \times if(n)$	holtsh1 <nboltsh2. nbc<="" td=""><td><math>h_{\rm holtsb2}(1) = 8</math></td><td>85 kN</td><td></td></nboltsh2.>	$h_{\rm holtsb2}(1) = 8$	85 kN	
Shear load on supporting beam	$n \cdot \Omega_v / 2 = 42.5 \text{ k}$	N			• • • •	
Utilisation factor: Usestation = (	$\sum / (2 \times P)$	-) <b>- 0 137</b>				
Othisation lactor, Ocheck/shear – C		g) – <b>0.13</b> 7	l ocal choar	consoity of Su	nnorting Bo	am wab. DASS
for boaring			LUCAI SIICAI	capacity of Su	рропппу ве	ann web. FA33
	.2					
Pbssupporting = 460 IN/IIII						
$P_{\text{bssupporting}} = d_{\text{bolts}} \times t_{\text{supporting}} \times p$	Obssupporting = <b>75.8</b>	KN				
$E_1 = O_1 \cdot I (2 \times p_1 + 1) \cdot O_1 \cdot I (1)$	$(2 \times n_{\text{boltsb2}}) = 21.3$	<b>3</b> kN				
$F_{bs} = Q_{b1} / (2 \times \Pi_{b0} I_{sb1}) + Q_{b2} / ($		- 0 280				
Utilisation factor; Ucheck7bearing =	+ Hbs / Pbssupporting =	- 0.200				
Utilisation factor; Ucheck7bearing =	E Hos / Possupporting =	- 0.200	Local bearing	capacity of Sup	porting Bea	am web : PASS
Utilisation factor; U <sub>check7bearing</sub> =	E Hos / Pbssupporting =	- 0.200	Local bearing	capacity of Sup	oporting Bea	am web : PASS
Build (2 × Hooltsbil) + Qb2 / (         Utilisation factor; Ucheck7bearing =         SUMMARY OF RESULTS         Check 2 - Shear capacity of b	• Fbs / Pbssupporting = polt group connection	ecting end plat	Local bearing e to supporting	capacity of Sup	oporting Bea	am web : PASS
Bos = Qb1 / (2 × Tholtsb1) + Qb2 / (         Utilisation factor; Ucheck7bearing =         SUMMARY OF RESULTS         Check 2 - Shear capacity of b         Beam 1	Fbs / Pbssupporting = polt group connection	ecting end plat	Local bearing e to supporting	capacity of Sup	oporting Bea	am web : PASS
Fbs = Qb1 / (2 × Hoottsb1) + Qb2 / (         Utilisation factor; Ucheck7bearing =         SUMMARY OF RESULTS         Check 2 - Shear capacity of b         Beam 1         Shear utilisation factor;	; U <sub>check2b1</sub> = <b>0.093</b>	ecting end plat	<i>Local bearing</i> e to supporting	capacity of Sup Beam PASS	oporting Bea	am web : PASS
S       Gb1 / (2 × Hoottsb1) + Gb2 / (         Utilisation factor; Ucheck7bearing =         SUMMARY OF RESULTS         Check 2 - Shear capacity of b         Beam 1         Shear utilisation factor;         Beam 2	; U <sub>check2b1</sub> = <b>0.093</b>	ecting end plat 3;	<i>Local bearing</i> e to supporting	capacity of Sup Beam PASS	oporting Bea	am web : PASS
Seam 1       Shear utilisation factor;         Shear utilisation factor;	F bs / Pbssupporting = polt group connection; Ucheck2b1 = 0.093 ; Ucheck2b2 = 0.268	ecting end plat 3; 3;	Local bearing	capacity of Sup Beam PASS PASS	oporting Bea	am web : PASS
Fbs = Qb1 / (2 × Hoottsb1) + Qb2 / (         Utilisation factor; Ucheck7bearing =         SUMMARY OF RESULTS         Check 2 - Shear capacity of b         Beam 1         Shear utilisation factor;         Beam 2         Shear utilisation factor;         Check 3 - Capacity of end pla	; U <sub>check2b2</sub> = <b>0.268</b>	ecting end plat 3; 3;	<i>Local bearing</i> e to supporting	capacity of Sup Beam PASS PASS	oporting Bea	am web : PASS
Seam 1         Shear utilisation factor;         Shear utilisation factor;         Beam 1         Shear utilisation factor;         Beam 2         Shear utilisation factor;         Check 3 - Capacity of end pla         Beam 1	<pre>&gt; Fbs / Pbssupporting = &gt; Dolt group conne ; Ucheck2b1 = 0.093 ; Ucheck2b2 = 0.268 ate</pre>	ecting end plat 3; 3;	<i>Local bearing</i> e to supporting	capacity of Sup Beam PASS PASS	oporting Bea	am web : PASS
Pbs = Qb1 / (2 × Hoottsb1) + Qb2 / (         Utilisation factor; Ucheck7bearing =         SUMMARY OF RESULTS         Check 2 - Shear capacity of b         Beam 1         Shear utilisation factor;         Beam 2         Shear utilisation factor;         Check 3 - Capacity of end pla         Beam 1         Shear utilisation factor;         Check 3 - Capacity of end pla         Beam 1         Shear utilisation factor;	F bs / Pbssupporting = colt group connection ; $U_{check2b1} = 0.093$ ; $U_{check2b2} = 0.268$ ate	ecting end plat 3; 3;	<i>Local bearing</i> e to supporting	capacity of Sup Beam PASS PASS	porting Bea	am web : PASS
Bes = Qb1 / (2 × Hoottsb1) + Qb2 / (         Utilisation factor; Ucheck7bearing =         Summary of RESULTS         Check 2 - Shear capacity of b         Beam 1         Shear utilisation factor;         Beam 2         Shear utilisation factor;         Check 3 - Capacity of end pla         Beam 1         Shear utilisation factor;         Check 3 - Capacity of end pla         Beam 1         Shear utilisation factor;         Beam 1         Shear utilisation factor;         Beam 1         Shear utilisation factor;	<pre>&gt; Fbs / Pbssupporting = poolt group connel ; Ucheck2b1 = 0.093 ; Ucheck2b2 = 0.268 ate ; Ucheck3shearb1 = 0 pril Ucheck3shearb1 = 0</pre>	ecting end plat 3; 3; .059; - 0 075:	<i>Local bearing</i> e to supporting	Capacity of Sup Beam PASS PASS PASS PASS	porting Bea	am web : PASS
Pbs = Qb1 / (2 × Hoottsb1) + Qb2 / (         Utilisation factor; Ucheck7bearing =         SUMMARY OF RESULTS         Check 2 - Shear capacity of b         Beam 1         Shear utilisation factor;         Beam 2         Shear utilisation factor;         Check 3 - Capacity of end pla         Beam 1         Shear utilisation factor;         Check 3 - Capacity of end pla         Beam 1         Shear utilisation factor;         Bearing utilisation factor;	<pre>&gt; Fbs / Pbssupporting = &gt; Dolt group conne ; Ucheck2b1 = 0.093 ; Ucheck2b2 = 0.268 ate ; Ucheck3shearb1 = 0 Dr; Ucheck3bearingb1 =</pre>	ecting end plat 3; 3; .059; = 0.075;	Local bearing	capacity of Sup Beam PASS PASS PASS PASS	porting Bea	am web : PASS
Summary of results         Summary of results         Check 2 - Shear capacity of b         Beam 1         Shear utilisation factor;         Beam 2         Shear utilisation factor;         Check 3 - Capacity of end pla         Beam 1         Shear utilisation factor;         Beam 2         Shear utilisation factor;         Beam 1         Shear utilisation factor;         Beam 1         Shear utilisation factor;         Bear 1         Shear utilisation factor;         Bear 2         Shear utilisation factor;         Bearing utilisation factor;         Bearing utilisation factor;	<pre>&gt; Fbs / Pbssupporting = &gt; Dolt group connel ; Ucheck2b1 = 0.093 ; Ucheck2b2 = 0.268 ate ; Ucheck3shearb1 = 0 Dr; Ucheck3bearingb1 = </pre>	ecting end plat 3; 3; .059; = 0.075;	Local bearing	Capacity of Sup Beam PASS PASS PASS PASS PASS	porting Bea	am web : PASS
Summary of results         Summary of results         Check 2 - Shear capacity of the seam 1         Shear utilisation factor;         Beam 2         Shear utilisation factor;         Check 3 - Capacity of end plate         Beam 1         Shear utilisation factor;         Beam 2         Shear utilisation factor;         Beam 1         Shear utilisation factor;         Beam 1         Shear utilisation factor;         Beam 1         Shear utilisation factor;         Beam 2         Shear utilisation factor;         Bearing utilisation factor;	<pre>&gt; Fbs / Pbssupporting = &gt; Dolt group conne ; Ucheck2b1 = 0.093 ; Ucheck2b2 = 0.268 ate ; Ucheck3shearb1 = 0 Dr; Ucheck3shearb1 = 0 pr; Ucheck3shearb2 = 0</pre>	ecting end plat 3; .059; = 0.075; .168;	Local bearing	capacity of Sup Beam PASS PASS PASS PASS PASS	porting Bea	am web : PASS
Summary of Results         Summary of Results         Check 2 - Shear capacity of b         Beam 1         Shear utilisation factor;         Beam 2         Shear utilisation factor;         Check 3 - Capacity of end pla         Beam 1         Shear utilisation factor;         Beam 2         Shear utilisation factor;         Beam 1         Shear utilisation factor;         Bearing utilisation factor;	<pre>&gt; Fbs / Pbssupporting = &gt; Dolt group connel ; Ucheck2b1 = 0.093 ; Ucheck2b2 = 0.268 ate ; Ucheck3shearb1 = 0 Dr; Ucheck3bearingb1 = ; Ucheck3bearingb1 = ; Ucheck3bearingb1 = </pre>	ecting end plat 3; .059; = 0.075; .168; = 0.214;	Local bearing	capacity of Sup Beam PASS PASS PASS PASS PASS PASS PASS	porting Bea	am web : PASS
Pbs = Qb1 / (2 × Hoottsb1) + Qb2 / (         Utilisation factor; Ucheck7bearing =         Summary of RESULTS         Check 2 - Shear capacity of b         Beam 1         Shear utilisation factor;         Beam 2         Shear utilisation factor;         Check 3 - Capacity of end pla         Beam 1         Shear utilisation factor;         Beam 1         Shear utilisation factor;         Bearing utilisation factor;	<ul> <li>Fbs / Pbssupporting =</li> <li>Dolt group connect</li> <li>; Ucheck2b1 = 0.093</li> <li>; Ucheck2b2 = 0.268</li> <li>ate</li> <li>; Ucheck3shearb1 = 0</li> <li>Dor; Ucheck3shearb1 = 0</li> <li>Dor; Ucheck3shearb2 = 0</li> <li>Dor; Ucheck3shearb2 = 0</li> </ul>	ecting end plat 3; .059; = 0.075; .168; = 0.214; eam web at the	Local bearing	capacity of Sup Beam PASS PASS PASS PASS PASS PASS	porting Bea	am web : PASS
Pbs = Qb1 / (2 × Hoottsb1) + Qb2 / (         Utilisation factor; Ucheck7bearing =         SUMMARY OF RESULTS         Check 2 - Shear capacity of b         Beam 1         Shear utilisation factor;         Beam 2         Shear utilisation factor;         Check 3 - Capacity of end pla         Beam 1         Shear utilisation factor;         Beam 1         Shear utilisation factor;         Bearing utilisation factor;      <	<pre>&gt; Fbs / Pbssupporting = &gt; Dolt group connel ; Ucheck2b1 = 0.093 ; Ucheck2b2 = 0.268 ate ; Ucheck3shearb1 = 0 Dr; Ucheck3bearingb1 = ; Ucheck3bearingb1 = ; Ucheck3bearingb2 = ; he supported bearingb2</pre>	ecting end plat 3; .059; = 0.075; .168; = 0.214; eam web at the	Local bearing	capacity of Sup Beam PASS PASS PASS PASS PASS PASS PASS	porting Bea	am web : PASS
Pbs = Qb1 / (2 × Hootsb1) + Qb2 / (         Utilisation factor; Ucheck7bearing =         Summary of RESULTS         Check 2 - Shear capacity of b         Beam 1         Shear utilisation factor;         Beam 2         Shear utilisation factor;         Check 3 - Capacity of end pla         Beam 1         Shear utilisation factor;         Beam 1         Shear utilisation factor;         Bearing utilisation factor;         Bear utilisation factor;	F bs / Pbssupporting = Dolt group connects ; Ucheck2b1 = 0.093 ; Ucheck2b2 = 0.268 ate ; Ucheck3shearb1 = 0 Dr; Ucheck3shearb1 = 0 Dr; Ucheck3shearb2 = 0 Dr; Ucheck3bearingb2 = the supported beam ; Ucheck4shearb1 = 0	ecting end plat 3; .059; = 0.075; .168; = 0.214; eam web at the .211;	Local bearing	capacity of Sup Beam PASS PASS PASS PASS PASS PASS PASS	porting Bea	am web : PASS
Pbs = Qb1 / (2 × Hootsb1) + Qb2 / (         Utilisation factor; Ucheck7bearing =         SUMMARY OF RESULTS         Check 2 - Shear capacity of b         Beam 1         Shear utilisation factor;         Beam 2         Shear utilisation factor;         Check 3 - Capacity of end pla         Beam 1         Shear utilisation factor;         Beam 1         Shear utilisation factor;         Bearing utilisation factor;         Bear utilisation factor;         Bear utilisation factor;         Bear 1         Shear utilisation factor;	F bs / Pbssupporting = Dolt group connects ; Ucheck2b1 = 0.093 ; Ucheck2b2 = 0.268 ate ; Ucheck3shearb1 = 0 Dr; Ucheck3shearb1 = 0 Dr; Ucheck3shearb2 = 0 Dr; Ucheck3shearb2 = 0 or; Ucheck3bearingb1 = the supported beaution ; Ucheck4shearb1 = 0	ecting end plat 3; .059; = 0.075; .168; = 0.214; eam web at the .211;	Local bearing	capacity of Sup Beam PASS PASS PASS PASS PASS PASS PASS	porting Bea	am web : PASS
Pbs = Qb1 / (2 × Hootsb1) + Qb2 / (         Utilisation factor; Ucheck7bearing =         SUMMARY OF RESULTS         Check 2 - Shear capacity of b         Beam 1         Shear utilisation factor;         Beam 2         Shear utilisation factor;         Check 3 - Capacity of end pla         Beam 1         Shear utilisation factor;         Beam 2         Shear utilisation factor;         Bearing utilisation factor;         Bear utilisation factor;         Bear utilisation factor;         Bear 1         Shear utilisation factor;         Bear 2         Shear utilisation factor;         Bear 1         Shear utilisation factor;         Bear 2         Shear utilisation factor;	Fbs / Pbssupporting =         polt group conner;         ; Ucheck2b1 = 0.093;         ; Ucheck2b2 = 0.268;         ate         ; Ucheck3shearb1 = 0         por; Ucheck3shearb1 = 0         por; Ucheck3shearb2 = 0         por; Ucheck3bearingb1 =         ; Ucheck4shearb2 = 0         por; Ucheck4shearb1 = 0         ; Ucheck4shearb1 = 0	ecting end plat 3; .059; = 0.075; .168; = 0.214; eam web at the .211; .453;	Local bearing	capacity of Sup Beam PASS PASS PASS PASS PASS PASS PASS PAS	porting Bea	am web : PASS
Pbs = Qb1 / (2 × Hootsb1) + Qb2 / (         Utilisation factor; Ucheck7bearing =         SUMMARY OF RESULTS         Check 2 - Shear capacity of b         Beam 1         Shear utilisation factor;         Beam 2         Shear utilisation factor;         Check 3 - Capacity of end pla         Beam 1         Shear utilisation factor;         Beam 1         Shear utilisation factor;         Bearing utilisation factor;         Bear utilisation factor; </td <td><pre>boolt group conner coolt group conner ; Ucheck2b1 = 0.093 ; Ucheck2b2 = 0.268 ate ; Ucheck3shearb1 = 0 Or; Ucheck3shearb1 = 0 Or; Ucheck3shearb2 = 0 Or; Ucheck3shearb2 = 0 ; Ucheck4shearb1 = 0 ; Ucheck4shearb1 = 0 ; Ucheck4shearb2 = 0 et welds connec</pre></td> <td>ecting end plat 3; .059; = 0.075; .168; = 0.214; eam web at the .211; .453; :ting the end pla</td> <td>Local bearing e to supporting e endplate ate to the supp</td> <td>capacity of Sup Beam PASS PASS PASS PASS PASS PASS PASS PAS</td> <td>porting Bea</td> <td>am web : PASS</td>	<pre>boolt group conner coolt group conner ; Ucheck2b1 = 0.093 ; Ucheck2b2 = 0.268 ate ; Ucheck3shearb1 = 0 Or; Ucheck3shearb1 = 0 Or; Ucheck3shearb2 = 0 Or; Ucheck3shearb2 = 0 ; Ucheck4shearb1 = 0 ; Ucheck4shearb1 = 0 ; Ucheck4shearb2 = 0 et welds connec</pre>	ecting end plat 3; .059; = 0.075; .168; = 0.214; eam web at the .211; .453; :ting the end pla	Local bearing e to supporting e endplate ate to the supp	capacity of Sup Beam PASS PASS PASS PASS PASS PASS PASS PAS	porting Bea	am web : PASS
Pbs = Qb1 / (2 × Hootsb1) + Qb2 / (         Utilisation factor; Ucheck7bearing =         SUMMARY OF RESULTS         Check 2 - Shear capacity of b         Beam 1         Shear utilisation factor;         Beam 2         Shear utilisation factor;         Check 3 - Capacity of end pla         Beam 1         Shear utilisation factor;         Beam 2         Shear utilisation factor;         Bearing utilisation factor;         Bearing utilisation factor;         Bearing utilisation factor;         Bear 1         Shear utilisation factor;         Beam 2         Shear utilisation factor;         Beam 1         Shear utilisation factor;         Beam 1         Shear utilisation factor;         Beam 2         Shear utilisation factor;         Beam 1         Shear utilisation factor;         Check 5 - Capacity of the fille         Beam 1	F bs / Pbssupporting = Dolt group connects ; Ucheck2b1 = 0.093 ; Ucheck2b2 = 0.268 ate ; Ucheck3shearb1 = 0 Dr; Ucheck3shearb1 = 0 Dr; Ucheck3shearb2 = 0 Dr; Ucheck4shearb2 = 0 ; Ucheck4shearb1 = 0	ecting end plat 3; .059; = 0.075; .168; = 0.214; eam web at the .211; .453; :ting the end pla	Local bearing	capacity of Sup Beam PASS PASS PASS PASS PASS PASS PASS PAS	porting Bea	am web : PASS
Pbs = Qb17 (2 × Hootsb1) + Qb27 (         Utilisation factor; Ucheck7bearing =         SUMMARY OF RESULTS         Check 2 - Shear capacity of b         Beam 1         Shear utilisation factor;         Beam 2         Shear utilisation factor;         Check 3 - Capacity of end pla         Beam 1         Shear utilisation factor;         Beam 1         Shear utilisation factor;         Bearing utilisation factor;         Bear utilisation factor;         Beam 1         Shear utilisation factor;         Beam 2         Shear utilisation factor;         Beam 1         Shear utilisation factor;         Check 5 - Capacity of the fille         Beam 1         Shear utilisation factor;	Fbs / Pbssupporting =         coolt group conner;         ; Ucheck2b1 = 0.093;         ; Ucheck2b2 = 0.268;         ate         ; Ucheck3shearb1 = 0         Or; Ucheck3shearb1 = 0         or; Ucheck3shearb2 = 0         or; Ucheck3shearb2 = 0         ; Ucheck4shearb2 = 0         ; Ucheck4shearb1 = 0         ; Ucheck4shearb1 = 0         ; Ucheck4shearb1 = 0         ; Ucheck4shearb2 = 0         ; Ucheck4shearb2 = 0         ; Ucheck4shearb2 = 0	ecting end plat 3; .059; = 0.075; .168; = 0.214; eam web at the .211; .453; ting the end plat 101;	Local bearing	capacity of Sup Beam PASS PASS PASS PASS PASS PASS PASS orted beam wet PASS	porting Bea	am web : PASS
Pibs = Qb17 (2 × Thoritsol) + Qb27 (         Utilisation factor; Ucheck7bearing =         SUMMARY OF RESULTS         Check 2 - Shear capacity of b         Beam 1         Shear utilisation factor;         Beam 2         Shear utilisation factor;         Beam 1         Shear utilisation factor;         Beam 1         Shear utilisation factor;         Bearing utilisation factor;         Bear utilisation factor;         Bear 1         Shear utilisation factor;         Beam 2         Shear utilisation factor;         Beam 1         Shear utilisation factor;         Check 5 - Capacity of the fille         Beam 1         Shear utilisation factor;         Beam 1         Shear utilisation factor;         Beam 1         Shear utilisation factor;         Beam 2         Shear utilisation factor;         Beam 1         Shear utilisation factor;         Beam 2	Fbs / Pbssupporting =         coolt group conner;         ; Ucheck2b1 = 0.093;         ; Ucheck2b2 = 0.268;         ate         ; Ucheck3shearb1 = 0         por; Ucheck3shearb1 = 0         por; Ucheck3shearb2 = 0         por; Ucheck3bearingb2 =         the supported box         ; Ucheck4shearb1 = 0         ; Ucheck4shearb1 = 0         ; Ucheck4shearb1 = 0         ; Ucheck4shearb1 = 0         ; Ucheck4shearb2 = 0         ; Ucheck4shearb1 = 0         ; Ucheck5weldb1 = 0.	ecting end plat 3; 3; .059; = 0.075; .168; = 0.214; eam web at the .211; .453; ting the end plat 101;	Local bearing	capacity of Sup Beam PASS PASS PASS PASS PASS PASS PASS orted beam web	porting Bea	am web : PASS
Shear utilisation factor; Beam 1 Shear utilisation factor; Check 2 - Shear capacity of the Beam 1 Shear utilisation factor; Beam 2 Shear utilisation factor; Bear 1 Shear utilisation factor; Bearing utilisation factor; Bearing utilisation factor; Bearing utilisation factor; Bearing utilisation factor; Bear 1 Shear utilisation factor; Bear 2 Shear utilisation factor;	Fbs / Pbssupporting =         coolt group connect         ; Ucheck2b1 = 0.093         ; Ucheck2b2 = 0.268         ate         ; Ucheck3shearb1 = 0         or; Ucheck3shearb1 = 0         or; Ucheck3shearb2 = 0         or; Ucheck3shearb2 = 0         or; Ucheck3shearb2 = 0         ; Ucheck4shearb1 = 0         ; Ucheck4shearb1 = 0         ; Ucheck4shearb2 = 0         ; Ucheck5weldb1 = 0.         ; Ucheck5weldb1 = 0.	ecting end plat 3; .059; = 0.075; .168; = 0.214; eam web at the .211; .453; tting the end plat 101; 289;	Local bearing	capacity of Sup Beam PASS PASS PASS PASS PASS PASS PASS orted beam web PASS PASS	porting Bea	am web : PASS

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www.sd-structures.com	RC	26/06/2017	MD	26/06/2017				
Beam 1								
Utilisation factor; L	Jcheck6amomentb1	= <b>0.268</b> ;		I	PASS			
Beam 2								
Utilisation factor; L	Jcheck6amomentb2	= <b>0.542</b> ;		I	PASS			
Check 6b - local stability notched	of notched su	pported beams	restrained ag	gainst lateral torsi	onal buckling	g - 1 flange		
Beam 1								
Depth of top not	ch less than li	mit of D/2 : PAS	S					
Length of top not	tch less than	limit : PASS						
Beam 2								
Depth of top note	ch less than li	mit of D/2 : PAS	S					
Length of top not	tch less than	limit : PASS						
Check 7 - Local shear and	d bearing capa	acity of Support	ing Beam we	b				
Shear utilisation fa	actor; Ucheck7she	ar = <b>0.137</b> ;		I	PASS			
Bearing utilisation	factor; U <sub>check7b</sub>	earing = <b>0.280</b> ;			PASS	3		

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#### SB3 TO SB4 CONNECTION

#### Section Details

Supporting Beam UKB 254x146x37;; Grade<sub>supporting</sub> = "S275" Supported Beam UKC 203x203x46;; Grade<sub>supported</sub> = "S275"

Endplate - 150 x 190 x 10;; Grade<sub>endplate</sub> = "S275"

Bolts M16 (Grade 8.8)



SECTION THROUGH SUPPORTING BEAM

# **Connection Details**



SECTION A - A

TEDDS calculation version 2.0.14



					1	
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www.su-structures.com				20,00,2011		
Utilisation factor; Ucheck2 = Q /	Psboltssum = 0.28	5				
		5	Shear capacit	y of bolt group to	Supporting	Beam : PASS
Check 3 - Shear and bearing	g capacity of e	ndplate				
for shear						
;;						
pyendplate = 27	<b>5</b> N/mm <sup>2</sup>					
;; e1aendplate = e	1endplate = 40 mm					
;; Avendplate = 0.9	9 imes (2 $ imes$ e1aendplat	$_{e}$ + (n <sub>bolts</sub> - 1) × p <sub>bo</sub>	$t_{ts}) \times t_{endplate} = $	<b>1350</b> mm²		
; Avnetendplate =	Avendplate - $n_{bolts} \times$	$\times$ Dhbolts $\times$ tendplate =	<b>990</b> mm²			
Effective net area coe	efficient					
Keendplate = 1.2	20					
Plain shear capacity	of endplate					
$P_{vPendplate} = min(0.6 \times$	$p_{yendplate}  imes A_{vendplate}$	plate, $0.7 imes K_{eendplate}$	$\times p_{\text{yendplate}} \times A$	vnetendplate) = 222.8	kN	
$A_{v1endplate} = (e)$	e1aendplate + (Nbolts	- 1) × p <sub>bolts</sub> ) × t <sub>endp</sub>	olate = <b>1100</b> mm	1 <sup>2</sup>		
; Ateffendplate = (	$e_{2endplate} - 0.5  imes I$	$D_{hbolts}) \times t_{endplate} = 4$	<b>410</b> mm <sup>2</sup>			
Block shear capacity	of endplate	, .				
$P_{vBendplate} = 0.6 \times p_{vendplate}$	dplate $\times A_{v1endplate}$	+ $0.6 \times K_{eendplate} \times$	$p_{vendplate}  imes A_{teff}$	fendplate = <b>262.7</b> kN		
Shear capacity of the endplat	e: Pvendplate = m	in (PyPendolate, PyBer	(1) = 222.8	kN		
:Shear force on each endplat	e shear plane: C	2/2 = 33.5  kN				
Utilisation factor: Ucheck3shear =	$\Omega / (2 \times P_{vendola})$	$(t_{e}) = 0.150$				
				Shear c	apacity of en	dplate : PASS
for bearing					,,,,,,,,,,,,,,	
eendplate = e1endplate = 4	<b>0</b> mm					
bearing strength of th	e endplate					
p <sub>bsendplate</sub> = <b>460</b> N/mm	1 <sup>2</sup>					
For top bolt,						
bearing capacity of th	ne endplate per l	bolt				
$P_{bsendplate1} = min(d_{bolts})$	$\times$ tendplate $\times$ pbsen	adplate, $0.5 imes e_{ ext{endplate}}$	$h_{e}  imes t_{endplate}  imes p_{bs}$	sendplate) = <b>73.6</b> kN		
For other bolts, bearing capacity of th	ie endplate per l	bolt				
$P_{bsendplate} = d_{bolts} \times t_{end}$	dplate $ imes$ <b>p</b> bsendplate :	= <b>73.6</b> kN				
Capacity of bolt group;						
$P_{bsendplatesum} = 2 \times P_{bsendplate1} +$	$-2 \times (n_{bolts} - 1) \times$	Pbsendplate = <b>294.4</b>	kN			
Bearing force on bolt group; (	Q = <b>67.0</b> kN					
Utilisation factor; Ucheck3bearing	= Q / P <sub>bsendplates</sub>	um = <b>0.228</b>				
,				Bearing c	apacity of en	dplate : PASS
Check 4 - shear capacit	ty of the suppo	rted beam web a	t the endplate	9		~
;;						
Pysuppor	ted = <b>275</b> N/mm <sup>2</sup>	2				

;;; ;

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 $A_{vsupported} = 0.9 \times min(l_{endplate}, d_{supported}) \times t_{supported} = ;972; mm^2;$ Shear capacity of supported beam web

 $P_{vsupported} = 0.6 \times p_{ysupported} \times A_{vsupported} = 160.4 \text{ kN}$ 

;Shear force on supported beam; Q = 67.0 kN

Utilisation factor; Ucheck4shear = Q / Pvsupported = 0.418

Shear capacity of supported beam web : PASS

# Check 5 - capacity of the fillet welds connecting the endplate to the supported beam web

Effective throat size of weld;  $a_{weld} = s_{weld} \times 0.7 = 4.2 \text{ mm}$ 

; ;;;

Effective length of weld; Iweld = 2 × (min(Iendplate, 2 × rsupported + dsupported) - 2 × sweld) = ;276.0; mm

Design strength of weld; pweld = 220 N/mm<sup>2</sup> :

Capacity of fillet welds;  $P_{weld} = p_{weld} \times I_{weld} \times a_{weld} = 255.0 \text{ kN}$ 

;Utilisation factor; Ucheck5weld = Q / Pweld = 0.263

Capacity of fillet weld : PASS

### Check 6a - bending capacity of reduced supported beam section at the notch - 1 flange notched

Properties of notched section ignoring root radii

;;; Depth of web; dweb = Dsupported - dctopnotch - Tsupported

Area of notched section;  $A_{notched} = d_{web} \times t_{supported} + B_{supported} \times T_{supported} = 3407 \text{ mm}^2$ ;;

Distance from top of notch to centroid of notched section

ybar = (dweb<sup>2</sup> × tsupported / 2 + Bsupported × Tsupported × (dweb + Tsupported / 2)) / Anotched = 138 mm

Inertia of notched section

Inotched = t<sub>supported</sub> × d<sub>web</sub><sup>3</sup> / 12 + t<sub>supported</sub> × d<sub>web</sub> × (y<sub>bar</sub> - d<sub>web</sub> / 2)<sup>2</sup> +

 $B_{supported} \times T_{supported}{}^3 \ / \ 12 \ + \ B_{supported} \times T_{supported} \times (d_{web} \ + \ T_{supported} \ / \ 2 \ - \ y_{bar})^2$ 

 $I_{notched} = 834 \text{ cm}^4$ 

Modulus of notched section

Znotched = Inotched / ybar = 60.4 cm<sup>3</sup>

;Moment capacity of notched section;  $M_{capnotched} = p_{ysupported} \times Z_{notched} = 16.6 \text{ kNm}$ 

;;;Moment applied to notched section; Mappnotched = Q × (Ctopnotch+ tendplate) = 6.0 kNm

Utilisation factor; Ucheck6amoment = Mappnotched / Mcapnotched = 0.363

Moment capacity of notched section : PASS

# Check 6b - local stability of notched supported beams restrained against lateral torsional buckling - 1 flange notched

;	Depth of Notch; d <sub>ctopnotch</sub> = <b>30</b> mm
;	Length of notch; c <sub>topnotch</sub> = <b>80</b> mm
	;Depth of top notch less than limit of D/2 : PASS
;	Grade <sub>supported</sub> = "S275"
;	dot = D <sub>supported</sub> /t <sub>supported</sub> = <b>28.2</b>
	$\begin{aligned} \text{Climit} = \text{if}(\text{or}(\text{Grade}_{\text{supported}} == \text{``S275''}, \text{Grade}_{\text{supported}} == \text{``300''}), \text{ if}(\text{dot} <= 54.3, \text{D}_{\text{supported}}, 160000 \times \text{D}_{\text{supported}}/\text{dot}^3), \\ \text{if}(\text{dot} <= 48.0, \text{D}_{\text{supported}}, 110000 \times \text{D}_{\text{supported}}/\text{dot}^3)) \end{aligned}$
	Climit = <b>203.2</b> mm
	Length of top notch less than limit : PASS
Check	7 - Local shear and bearing capacity of Supporting Beam web

for shear

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www.sd-structures.com	RC	26/06/2017	MD	26/06/2017			
;; etsupporting = <b>70</b> mm							
;;; Avsupporting = (pbolts + (n	bolts - 1) $ imes$ pbolts + e	tsupporting) $ imes$ tsupport	rting = <b>1323</b> mm <sup>2</sup>	2			
; Avnetsupporting = Avsupporti	$_{ m ng}$ - $n_{ m bolts}  imes D_{ m hbolts}  imes$	t <sub>supporting</sub> = <b>1096</b>	6 mm²				
;							
$p_{ysupporting} = 27$	<b>75</b> N/mm²						
Effective net area coe	efficient						
Kesupporting = 1	.20						
$P_{vsupporting} = min(0.6 \times p_{ysupportin})$	$hg  imes A_{vsupporting}, 0.7$	$ imes K_{esupporting}  imes p_y$	supporting $ imes A_{vnetsu}$	upporting) = <b>218.3</b> k	N		
;Shear load on Supporting Be	am; Q / 2 = <b>33.5</b> k	N					
Utilisation factor; U <sub>check7shear</sub> =	Q / (2 $\times$ P <sub>vsupporting</sub>	) = <b>0.153</b>					
			Local shea	r capacity of Su	pporting Bear	n web: PASS	
for bearing							
pbssupporting = <b>460</b> N/mr	m²						
$P_{bssupporting} = d_{bolts} \times t_{supporting} \times$	Pbssupporting = <b>46.4</b>	kN					
Q / (2 × n <sub>bolts</sub> ) = <b>16.8</b> kN							
Utilisation factor; Ucheck7bearing =	= Q / $(2 \times n_{bolts} \times F)$	bssupporting) = 0.3	61				
Utilisation factor; Ucheck7bearing :	= Q / (2 × $n_{bolts}$ × F	bssupporting) = <b>0.3</b>	61 <i>Local bearing</i>	capacity of Sup	oporting Beam	n web : PASS	
Utilisation factor; Ucheck7bearing :	$= Q / (2 \times n_{bolts} \times F)$	bssupporting) = 0.3	61 <i>Local bearing</i>	capacity of Sup	oporting Bean	n web : PASS	
Utilisation factor; U <sub>check7bearing</sub> : <u>SUMMARY OF RESULTS</u> Check 2 - Shear capacity of	= Q / (2 × n <sub>bolts</sub> × F	Possupporting) = 0.30	61 Local bearing	capacity of Sup	oporting Bean	n web : PASS	
Utilisation factor; U <sub>check7bearing</sub> : <u>SUMMARY OF RESULTS</u> Check 2 - Shear capacity of Shear utilisation facto	= Q / (2 × n <sub>bolts</sub> × F bolt group conner:	Possupporting) = 0.30	61 <i>Local bearing</i> e to supporting	capacity of Sup Beam	pporting Bean	n web : PASS	
Utilisation factor; U <sub>check7bearing</sub> : <u>SUMMARY OF RESULTS</u> Check 2 - Shear capacity of Shear utilisation facto Check 3 - Capacity of endnli	= Q / (2 × n <sub>bolts</sub> × F bolt group conne r; U <sub>check2</sub> = 0.285;	Possupporting) = 0.30	61 <i>Local bearing</i> e to supporting	capacity of Sup Beam PASS	oporting Bean	n web : PASS	
Utilisation factor; U <sub>check7bearing</sub> : <u>SUMMARY OF RESULTS</u> Check 2 - Shear capacity of Shear utilisation facto Check 3 - Capacity of endpla Shear utilisation facto	= Q / (2 × n <sub>bolts</sub> × F bolt group conne r; U <sub>check2</sub> = 0.285; ate	Possupporting) = 0.30	61 <i>Local bearing</i> e to supporting	capacity of Sup Beam PASS	pporting Beam	ו web : PASS	
Utilisation factor; U <sub>check7bearing</sub> : <u>SUMMARY OF RESULTS</u> Check 2 - Shear capacity of Shear utilisation facto Check 3 - Capacity of endplation Shear utilisation facto Bearing utilisation facto	= Q / (2 × n <sub>bolts</sub> × F <b>bolt group conn</b> er; U <sub>check2</sub> = <b>0.285</b> ; <b>ate</b> r; U <sub>check3shear</sub> = <b>0.1</b> tor: U <sub>check3shear</sub> = <b>0.1</b>	Possupporting) = 0.30 ecting endplate 50;	61 <i>Local bearing</i> e to supporting	Beam PASS	oporting Bean	n web : PASS	
Utilisation factor; U <sub>check7bearing</sub> : <u>SUMMARY OF RESULTS</u> Check 2 - Shear capacity of Shear utilisation facto Check 3 - Capacity of endplation Shear utilisation factor Bearing utilisation factor Check 4 - Shear capacity of	= Q / (2 × n <sub>bolts</sub> × F <b>bolt group conn</b> r; U <sub>check2</sub> = <b>0.285</b> ; <b>ate</b> r; U <sub>check3shear</sub> = <b>0.1</b> tor; U <sub>check3bearing</sub> = <b>the supported b</b>	Possupporting) = <b>0.3</b> Possupporting endplate 50; 0.228; 223 web at the	61 <i>Local bearing</i> to supporting	capacity of Sup Beam PASS PASS PASS	pporting Beam	n web : PASS	
Utilisation factor; U <sub>check7bearing</sub> : <u>SUMMARY OF RESULTS</u> Check 2 - Shear capacity of Shear utilisation facto Check 3 - Capacity of endpla Shear utilisation facto Bearing utilisation facto Shear utilisation facto	= Q / (2 × n <sub>bolts</sub> × F <b>bolt group conn</b> r; U <sub>check2</sub> = <b>0.285</b> ; <b>ate</b> r; U <sub>check3shear</sub> = <b>0.1</b> tor; U <sub>check3bearing</sub> = <b>the supported bo</b> r: U <sub>check4bear</sub> = <b>0.4</b>	Possupporting) = <b>0.3</b> Possupporting endplate 50; 0.228; Poam web at the 18:	61 <i>Local bearing</i> e to supporting endplate	Beam PASS PASS PASS	pporting Bean	n web : PASS	
Utilisation factor; U <sub>check7bearing</sub> : <u>SUMMARY OF RESULTS</u> Check 2 - Shear capacity of Shear utilisation facto Check 3 - Capacity of endpla Shear utilisation facto Bearing utilisation facto Check 4 - Shear capacity of Shear utilisation facto	= Q / (2 × n <sub>bolts</sub> × F <b>bolt group conne</b> r; Ucheck2 = <b>0.285</b> ; <b>ate</b> r; Ucheck3shear = <b>0.1</b> tor; Ucheck3bearing = <b>the supported be</b> r; Ucheck4shear = <b>0.4</b> let welds connect	bssupporting) = 0.30 ecting endplate 50; 0.228; eam web at the 18; ting the endpla	61 <i>Local bearing</i> e to supporting endplate	capacity of Sup Beam PASS PASS PASS PASS	pporting Beam	ו web : PASS	
Utilisation factor; U <sub>check7bearing</sub> : <u>SUMMARY OF RESULTS</u> Check 2 - Shear capacity of Shear utilisation facto Check 3 - Capacity of endpla Shear utilisation facto Bearing utilisation facto Check 4 - Shear capacity of Shear utilisation facto Check 5 - Capacity of the fill Shear utilisation facto	= Q / (2 × n <sub>bolts</sub> × F <b>bolt group conn</b> r; U <sub>check2</sub> = <b>0.285</b> ; <b>ate</b> r; U <sub>check3shear</sub> = <b>0.1</b> tor; U <sub>check3bearing</sub> = <b>the supported be</b> r; U <sub>check4shear</sub> = <b>0.4</b> <b>let welds connec</b> r: U <sub>check5weld</sub> = <b>0.2</b>	Possupporting) = 0.30 ecting endplate 50; 0.228; eam web at the 18; ting the endpla	61 <i>Local bearing</i> e to supporting endplate ite to the supp	capacity of Sup Beam PASS PASS PASS PASS orted beam web	pporting Beam	n web : PASS	
Utilisation factor; Ucheck7bearing : SUMMARY OF RESULTS Check 2 - Shear capacity of Shear utilisation facto Check 3 - Capacity of endpla Shear utilisation facto Bearing utilisation facto Check 4 - Shear capacity of Shear utilisation facto Check 5 - Capacity of the fill Shear utilisation facto	= Q / (2 × nbolts × F bolt group conner r; Ucheck2 = 0.285; ate r; Ucheck3shear = 0.1 tor; Ucheck3bearing = the supported bord r; Ucheck4shear = 0.4 let welds connec r; Ucheck5weld = 0.26 posity of reduced	Possupporting) = 0.30 ecting endplate 50; 0.228; eam web at the 18; ting the endpla 53; supported box	61 <i>Local bearing</i> e to supporting endplate Ite to the supp	capacity of Sup Beam PASS PASS PASS orted beam web PASS	pporting Bean	n web : PASS	
Utilisation factor; U <sub>check7bearing</sub> : <u>SUMMARY OF RESULTS</u> Check 2 - Shear capacity of Shear utilisation facto Check 3 - Capacity of endpla Shear utilisation facto Bearing utilisation facto Check 4 - Shear capacity of Shear utilisation facto Check 5 - Capacity of the fill Shear utilisation facto Check 6a - bending cap Utilisation factor; U <sub>check6an</sub>	= Q / (2 × nbolts × F bolt group conner r; Ucheck2 = 0.285; ate r; Ucheck3shear = 0.1 tor; Ucheck3bearing = the supported bo r; Ucheck4shear = 0.4 let welds connec r; Ucheck5weld = 0.26 acity of reduced noment = 0.363;	Possupporting) = 0.30 Possupporting endplate 50; 0.228; eam web at the 18; ting the endpla 53; supported bea	61 <i>Local bearing</i> to supporting endplate te to the supp m section at th	e capacity of Sup Beam PASS PASS PASS orted beam web PASS ne notch - 1 flan	pporting Beam ge notched PASS	n web : PASS	
Utilisation factor; Ucheck7bearing : SUMMARY OF RESULTS Check 2 - Shear capacity of Shear utilisation facto Check 3 - Capacity of endpla Shear utilisation facto Bearing utilisation facto Check 4 - Shear capacity of Shear utilisation facto Check 5 - Capacity of the fill Shear utilisation facto Check 6a - bending cap Utilisation factor; Ucheck6an	= Q / (2 × nbolts × F bolt group conner r; Ucheck2 = 0.285; ate r; Ucheck3shear = 0.1 tor; Ucheck3bearing = the supported bo r; Ucheck4shear = 0.4 let welds connec r; Ucheck5weld = 0.26 acity of reduced noment = 0.363; v of notched corn	Possupporting) = 0.36 ecting endplate 50; 0.228; eam web at the 18; ting the endpla 53; supported bea	61 <i>Local bearing</i> e to supporting endplate te to the supp m section at th	e capacity of Sup Beam PASS PASS PASS orted beam web PASS ne notch - 1 flan	ge notched PASS	n web : PASS	
Utilisation factor; Ucheck7bearing : SUMMARY OF RESULTS Check 2 - Shear capacity of Shear utilisation facto Check 3 - Capacity of endpla Shear utilisation facto Bearing utilisation facto Check 4 - Shear capacity of Shear utilisation facto Check 5 - Capacity of the fill Shear utilisation facto Check 6a - bending cap Utilisation factor; Ucheck6ar Check 6b - local stabilit notched	= Q / (2 × nbolts × F bolt group conner r; Ucheck2 = 0.285; ate r; Ucheck3shear = 0.1 tor; Ucheck3bearing = the supported bor r; Ucheck4shear = 0.4 let welds connec r; Ucheck5weld = 0.26 acity of reduced noment = 0.363; y of notched sup	Possupporting) = 0.30 ecting endplate 50; 0.228; eam web at the 18; ting the endpla 53; supported beams	61 <i>Local bearing</i> e to supporting endplate ate to the supp m section at th restrained aga	capacity of Sup Beam PASS PASS PASS orted beam web PASS ne notch - 1 flan inst lateral torsi	pporting Beam ge notched PASS onal buckling	n web : PASS	
Utilisation factor; U <sub>check7bearing</sub> : <u>SUMMARY OF RESULTS</u> Check 2 - Shear capacity of Shear utilisation facto Check 3 - Capacity of endpla Shear utilisation facto Bearing utilisation facto Check 4 - Shear capacity of Shear utilisation facto Check 5 - Capacity of the fill Shear utilisation facto Check 6a - bending cap Utilisation factor; U <sub>check6ar</sub> Check 6b - local stabilit notched Depth of top notch less	= Q / (2 × nbolts × F bolt group conner r; Ucheck2 = 0.285; ate r; Ucheck3shear = 0.1 tor; Ucheck3shear = 0.1 tor; Ucheck3bearing = the supported bo r; Ucheck4shear = 0.4 let welds connec r; Ucheck5weld = 0.26 acity of reduced noment = 0.363; y of notched sup than limit of D/2	Possupporting) = 0.30 ecting endplate 50; 0.228; eam web at the 18; ting the endpla 53; supported beams ported beams : PASS	61 <i>Local bearing</i> e to supporting endplate tte to the supp m section at th restrained aga	e capacity of Sup Beam PASS PASS PASS orted beam web PASS ne notch - 1 flan inst lateral torsi	porting Beam ge notched PASS onal buckling	n web : PASS	
Utilisation factor; U <sub>check7bearing</sub> : <u>SUMMARY OF RESULTS</u> Check 2 - Shear capacity of Shear utilisation facto Check 3 - Capacity of endpla Shear utilisation facto Bearing utilisation facto Check 4 - Shear capacity of Shear utilisation facto Check 5 - Capacity of the fill Shear utilisation facto Check 6a - bending cap Utilisation factor; U <sub>check6ar</sub> Check 6b - local stabilit notched Depth of top notch less Length of top notch less	= Q / (2 × nbolts × F bolt group conner r; Ucheck2 = 0.285; ate r; Ucheck3shear = 0.1 tor; Ucheck3bearing = the supported be r; Ucheck4shear = 0.4 let welds connec r; Ucheck4shear = 0.4 let welds connec r; Ucheck5weld = 0.26 acity of reduced noment = 0.363; y of notched sup than limit of D/2 s than limit : PAS	Possupporting) = 0.30 ecting endplate 50; 0.228; eam web at the 18; ting the endpla 53; supported beams ported beams : PASS SS	61 <i>Local bearing</i> e to supporting endplate ate to the supp m section at th restrained aga	e capacity of Sup Beam PASS PASS PASS orted beam web PASS ne notch - 1 flan inst lateral torsi	pporting Beam ge notched PASS onal buckling	n web : PASS	
Utilisation factor; U <sub>check7bearing</sub> : <u>SUMMARY OF RESULTS</u> Check 2 - Shear capacity of Shear utilisation facto Check 3 - Capacity of endpla Shear utilisation facto Bearing utilisation facto Check 4 - Shear capacity of Shear utilisation facto Check 5 - Capacity of the fill Shear utilisation facto Check 6a - bending cap Utilisation factor; U <sub>check6ar</sub> Check 6b - local stabilit notched Depth of top notch less Length of top notch less Check 7 - Local shear a	= Q / (2 × nbolts × F bolt group conner r; Ucheck2 = 0.285; ate r; Ucheck3shear = 0.1 tor; Ucheck3shear = 0.1 tor; Ucheck3bearing = the supported bo r; Ucheck4shear = 0.4 let welds connec r; Ucheck5weld = 0.26 acity of reduced noment = 0.363; y of notched sup than limit of D/2 s than limit : PAS nd bearing capad	Possupporting) = 0.34 ecting endplate 50; 0.228; eam web at the 18; ting the endpla 53; supported beams ported beams : PASS SS city of Support	61 <i>Local bearing</i> e to supporting endplate tte to the supp m section at th restrained aga	e capacity of Sup Beam PASS PASS PASS orted beam web PASS ne notch - 1 flan inst lateral torsi	ge notched PASS onal buckling	n web : PASS	
Utilisation factor; Ucheck7bearing : SUMMARY OF RESULTS Check 2 - Shear capacity of Shear utilisation facto Check 3 - Capacity of endpla Shear utilisation facto Bearing utilisation facto Check 4 - Shear capacity of Shear utilisation facto Check 5 - Capacity of the fill Shear utilisation facto Check 6a - bending cap Utilisation factor; Ucheck6ar Check 6b - local stabilit notched Depth of top notch less Length of top notch less Check 7 - Local shear a Shear utilisation	= Q / (2 × nbolts × F bolt group conner r; Ucheck2 = 0.285; ate r; Ucheck3shear = 0.1 tor; Ucheck3shear = 0.1 tor; Ucheck3bearing = the supported bord r; Ucheck4shear = 0.4 let welds connec r; Ucheck4shear = 0.4 let welds connec r; Ucheck4shear = 0.4 let welds connec r; Ucheck5weld = 0.26 actity of reduced noment = 0.363; y of notched sup than limit of D/2 s than limit : PAS nd bearing capace factor; Ucheck7shear	Possupporting) = 0.34 ecting endplate 50; 0.228; eam web at the 18; ting the endpla 53; supported beams : PASS SS city of Supporti = 0.153;	61 <i>Local bearing</i> e to supporting endplate net to the supp m section at th restrained aga	e capacity of Sup Beam PASS PASS PASS orted beam web PASS ne notch - 1 flan inst lateral torsi	ge notched PASS	n web : PASS	

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#### SB3 TO SB3 CONNECTION

#### Section Details

Supporting Beam UKC 203x203x46;; Grade<sub>supporting</sub> = "S355" Supported Beam UKC 203x203x46;; Grade<sub>supported</sub> = "S355" Endplate - 140 x 190 x 10;; Grade<sub>endplate</sub> = "S275"

Bolts M16 (Grade 8.8)



### SECTION THROUGH SUPPORTING BEAM





TEDDS calculation version 2.0.14



#### number of bolt rows; $n_{bolts} = 2$ Bolt pitch;; $p_{bolts} = 70 \text{ mm}$ $g_{bolts} = 90 \text{ mm}$ Bolt gauge; Endplate end distance (top & bottom); e1endplate = 35 mm Endplate edge distance; e<sub>2endplate</sub> = (d<sub>endplate</sub> - g<sub>bolts</sub>) / 2 = 50 mm Endplate length; lendplate = pbolts×(nbolts-1)+2×e1endplate = 140 mm Weld leg length; s<sub>weld</sub> = 6 mm supported beam end reaction; Q = **73.0** kN ; Notch details Top notch length; ctopnotch = 105 mm Top notch depth; d<sub>ctopnotch</sub> = **25** mm Bottom notch length; cbottomnotch = **105** mm Bottom notch depth; dcbottomnotch = 25 mm **Check 1 - Essential detailing requirements** Endplate thickness; tendplate = 10 mm; PASS ; Bolt gauge; gbolts = 90 mm; PASS Endplate Length; lendplate = 140 mm Endplate length for torsional requirements : PASS Check 2 - Shear capacity of bolt group connecting endplate to supporting beam Shear capacity of top pair of bolts;;;; pbsendplate = 460 N/mm<sup>2</sup> $P_{sbolts1} = min(P_{sbolts}, 0.5 \times e_{1endplate} \times t_{endplate} \times p_{bsendplate}) = 58.9 \text{ kN}$ Shear capacity of other bolts Psbolts = 58.9 kN Shear capacity of bolt group - sum of bolt capacities ; $P_{sboltssum} = 2 \times P_{sbolts1} + 2 \times (n_{bolts} - 1) \times P_{sbolts} = 235.5 \text{ kN}$

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;Shear on bolt group; Q = 73.	) kN							
Utilisation factor; $U_{check2} = Q /$	Psboltssum = 0.31	0						
		S	Shear capacity	of bolt group to	Supporting	Beam : PASS		
Check 3 - Shear and bearing	g capacity of e	ndplate						
for shear								
·· ;;								
pyendplate = 27	<b>5</b> N/mm <sup>2</sup>							
;; e1aendplate = e1	endplate = <b>35</b> mm							
;; A <sub>vendplate</sub> = 0.9	$0  imes (2  imes e_{1aendplate})$	$_{e}$ + (n <sub>bolts</sub> - 1) × p <sub>bo</sub>	$_{\rm its}) \times t_{\rm endplate} = 1$	<b>260</b> mm <sup>2</sup>				
, $A_{vnetendplate} = h$	$A_{vendplate} - n_{bolts} \times$	$D_{hbolts} \times t_{endplate} =$	<b>900</b> mm <sup>2</sup>					
Effective net area coe	fficient	- <b>F</b>						
Keendplate = 1.2	20							
Plain shear capacity o	of endplate							
$P_{vPondploto} = \min(0.6 \times 10^{-1})$		Nata 07 X Kaandalata	$\times$ Dyondolato $\times$ A	(notondolato) = 207 9	) kN			
$\Delta_{\rm ut} = mm(0.0)$		$-1) \times n_{halle}) \times t_{and m}$	usto – 1050 mm	2				
		$(-1) \times (-1) \times (-1)$	110 mm <sup>2</sup>					
; Ateffendplate = $(e_{2endplate} - U.5 \times D_{hbolts}) \times I_{endplate} = 410 \text{ mm}^2$								
		0.0						
$PvBendplate = 0.6 \times Pyend$	lplate × Av1endplate	+ U.6 × Keendplate ×	Pyendplate × Ateffe	endplate = 234.4 KIN				
Shear capacity of the endplate	e; Pvendplate = m	IN (PvPendplate, PvBer	ndplate) = 207.9 k	<n .<="" td=""><td></td><td></td></n>				
;Shear force on each endplate	e shear plane; C	2 / 2 = <b>36.5</b> kN						
Utilisation factor; Ucheck3shear =	Q / (2 × Pvendplat	te) = <b>0.176</b>						
				Shear c	apacity of en	dplate : PASS		
for bearing								
$e_{endplate} = e_{1endplate} = 3$	<b>5</b> mm							
bearing strength of th	e endplate							
p <sub>bsendplate</sub> = <b>460</b> N/mm	2							
For top bolt,								
bearing capacity of th	e endplate per t	oolt						
$P_{bsendplate1} = min(d_{bolts})$	$\times$ tendplate $\times$ <b>p</b> bsen	dplate, $0.5  imes e$ endplate	$1  imes t_{endplate}  imes p_{bs}$	endplate) = <b>73.6</b> kN				
For other bolts,								
bearing capacity of th	e endplate per t							
$P_{bsendplate} = d_{bolts} \times t_{end}$	plate $ imes$ <b>p</b> bsendplate =	= <b>73.6</b> kN						
Capacity of bolt group;								
$P_{bsendplatesum} = 2 \times P_{bsendplate1} +$	$2 \times (n_{bolts} - 1) \times$	Pbsendplate = 294.4	kN					
Bearing force on bolt group; C	Q = <b>73.0</b> kN							
Utilisation factor; Ucheck3bearing	= Q / Pbsendplatesu	ım = <b>0.248</b>						
				Bearing o	apacity of en	dplate : PASS		
Check 4 - shear capacit	y of the suppo	rted beam web a	t the endplate					
;;	<b>.</b>							
Pysupport 	<sub>ed</sub> = <b>355</b> N/mm <sup>2</sup>							
,,,								

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;

 $A_{vsupported} = 0.9 \times min(l_{endplate}, d_{supported}) \times t_{supported} = ;907; mm^{2};$ 

Shear capacity of supported beam web

 $P_{vsupported} = 0.6 \times p_{ysupported} \times A_{vsupported} = 193.2 \text{ kN}$ 

;Shear force on supported beam; Q = 73.0 kN

Utilisation factor; Ucheck4shear = Q / Pvsupported = 0.378

#### Shear capacity of supported beam web : PASS

### Check 5 - capacity of the fillet welds connecting the endplate to the supported beam web

; Effective throat size of weld;  $a_{weld} = s_{weld} \times 0.7 = 4.2 \text{ mm}$ ;;;

Effective length of weld;  $I_{weld} = 2 \times (min(I_{endplate}, 2 \times r_{supported} + d_{supported}) - 2 \times s_{weld}) = ;256.0; mm$ 

Design strength of weld; pweld = 220 N/mm<sup>2</sup>

Capacity of fillet welds;  $P_{weld} = p_{weld} \times I_{weld} \times a_{weld} = 236.5 \text{ kN}$ 

;Utilisation factor; Ucheck5weld = Q / Pweld = 0.309

#### Capacity of fillet weld : PASS

# Check 6a - bending capacity of reduced supported beam section at the notch - 2 flanges notched

;;;; Znotched = tsupported × (Dsupported - dctopnotch - dctopnotch)<sup>2</sup> / 6 = 28164 mm<sup>3</sup>
;Moment capacity of notched section; Mcapnotched = pysupported × Znotched = 10.0 kNm
;;;;Moment applied to notched section; Mappnotched = Q × (max(Ctopnotch, Cbottomnotch) + tendplate)= 8.4 kNm
Utilisation factor; Ucheck6amoment = Mappnotched / Mcapnotched = 0.840

Moment capacity of

notched section : PASS

Check 6b - local stability of notched supported beams restrained against lateral torsional buckling - 2 flanges notched

,	Depth of top notch; d <sub>ctopnotch</sub> = <b>25</b> mm
;	Length of top notch; ctopnotch = 105 mm
;	Depth of bottom notch; d <sub>cbottomnotch</sub> = <b>25</b> mm
;	Length of bottom notch; c <sub>bottomnotch</sub> = <b>105</b> mm
	Check <sub>depth</sub> = max(d <sub>ctopnotch</sub> , d <sub>cbottomnotch</sub> ) = <b>25</b> mm
	Check <sub>length</sub> = max(C <sub>topnotch</sub> , C <sub>bottomnotch</sub> ) = <b>105</b> mm
	;Depth of both notches less than limit of D/5 : PASS
;	Grade <sub>supported</sub> = "S355"
;	dot = D <sub>supported</sub> /t <sub>supported</sub> = <b>28.2</b>
	$\begin{aligned} c_{\text{limit}} = if(or(Grade_{\text{supported}} == \text{``S275''}, Grade_{\text{supported}} == \text{``300''}), & if(dot <= 54.3, D_{\text{supported}}, 160000 \times D_{\text{supported}}/dot^3), \\ & if(dot <= 48.0, D_{\text{supported}}, 110000 \times D_{\text{supported}}/dot^3)) \end{aligned}$
	Climit = <b>203.2</b> mm
	Length of both notches less than limit : PASS
Check	7 - Local shear and bearing capacity of Supporting Beam web
for she	ar
;;	etsupporting = 70 mm
;;;	$A_{vsupporting} = (p_{bolts} + (n_{bolts} - 1) \times p_{bolts} + e_{tsupporting}) \times t_{supporting} = 1512 \text{ mm}^2$

;  $A_{vnetsupporting} = A_{vsupporting} - n_{bolts} \times D_{hbolts} \times t_{supporting} = 1253 \text{ mm}^2$ 

;

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Pysupporting = <b>355</b>	N/mm <sup>2</sup>						
Effective net area coeff	icient						
Kesupporting = 1.1	0						
$P_{vsupporting} = min(0.6 \times p_{ysupporting}$	$\times A_{vsupporting}, 0.7$	$7  imes K_{esupporting}  imes p$	ysupporting $ imes A_{vne}$	tsupporting) = <b>322.1</b> k	N		
;Shear load on Supporting Bear	m; Q / 2 = <b>36.5</b>	kN					
Utilisation factor; Ucheck7shear = C	P / (2 × P <sub>vsupportin</sub>	ng) = <b>0.113</b>					
			Local she	ear capacity of Su	pporting Be	eam web: PASS	
for bearing							
pbssupporting = 550 N/mm <sup>2</sup>	2						
$P_{\text{bssupporting}} = d_{\text{bolts}} \times t_{\text{supporting}} \times p_{\text{bssupporting}}$	ossupporting = 63.4	k N					
Q / (2 × $n_{bolts}$ ) = <b>18.3</b> kN							
Utilisation factor; Ucheck7bearing =	Q / (2 $\times$ n <sub>bolts</sub> $\times$	P <sub>bssupporting</sub> ) = <b>0.2</b>	88				
			Local bearin	ng capacity of Sup	porting Be	am web : PASS	
SUMMARY OF RESULTS							
Check 2 - Shear capacity of h	olt aroun conr	necting endplate	e to supportir	ng Beam			
Shear utilisation factor:	$U_{check2} = 0.310$	:		PASS	5		
Check 3 - Capacity of endplat	e	,			-		
Shear utilisation factor:	$U_{\text{check3shear}} = 0.$	176:		PASS	5		
$Bearing utilisation factor:   _{check3hearing} = 0.248$				<b>j</b>			
Check 4 - Shear capacity of th	ne supported b	beam web at the	endplate				
Shear utilisation factor;	$U_{check4shear} = 0.1$	378;	·	PASS	5		
Check 5 - Capacity of the fille	t welds conne	cting the endpla	ate to the sup	ported beam web	)		
Shear utilisation factor;	Ucheck5weld = 0.3	309;		PASS	6		
Check 6a - bending capa	city of reduced	d supported bea	m section at	the notch - 2 flan	ges notche	d	
Utilisation factor; Ucheck6amo	ment = <b>0.840</b> ;				PA	SS	
Check 6b - local stability of n	otched suppor	rted beams rest	rained agains	at lateral torsiona	l buckling -	2 flanges	
notched							
Depth of both notches less th	an limit of D/5	: PASS					
Length of both notches less t	han limit : PAS	SS					
Check 7 - Local shear and	d bearing capa	acity of Support	ing Beam we	b	<b>B</b> 4 6 6		
Shear utilisation factor; $U_{check7shear} = 0.113$ ;					PASS		
	actor, Ucheck7b	earing = <b>0.200</b> ,			PA	55	
i							

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Diagonal stiffner requierement;

# $ds=if(M_{Ed_h}/D < V_{pl,Rd}$ , "Not required", "Required")= "Required"

# Hogging moment design:

;Number of bolts under shear; Shear force on bolts; Shear force per bolt;

Shear/Bearing capacity per bolt; Design check; Utilization;

#### Opening up moment design:

;Number of bolts under tension; Tension force on bolts; Tension force per bolt;

Tension/Punching capacity per bolt; Design check; Utilization; <u>Bending of plate subjected to tension from bolts:</u> Bending moment on plate; Plate thicknesss required; N/mm<sup>2</sup>))))=

#### Tension check with reduced flange:

Tension force on bolts; A<sub>net</sub>; Tension capacity of reduced flange; Design check; Utilization; 
$$\begin{split} n_h &= \textbf{8.000} \ ; \\ V_{Ed,b} &= M_{Ed\_h}/D = \textbf{845.179} \ kN \\ F_{b,Ed} &= V_{Ed,b} \ /n_h = \textbf{105.647} \ kN \\ k_1 &= min(2.8^*(e_2/D_h) - 1.7, 1.4^*(p/D_h) - 1.7, 2.5) = \textbf{2.177} \\ \alpha_v &= min(e_1/(3^*D_h), p/(3^*D_h) - 0.25, f_{ub}/f_{u}, 1) = \textbf{0.615} \\ F_{v,Rd} &= F_{b,Rd} = min(k_1^*\alpha_v^*f_u^*d^*t_p/\gamma_{M2}, 0.6^*f_{ub}^*A_s/\gamma_{M2}) = \textbf{135.552} \ kN \end{split}$$

hc=if(F<sub>b,Ed</sub><F<sub>b,Rd</sub>,"PASS","FAIL")=**"PASS"** F<sub>b,Ed</sub>/ F<sub>b,Rd</sub>=**0.779** 

$$\begin{split} &n_{ou} = \textbf{2.000} \ ; \\ &T_{Ed,b} = M_{Ed\_ou}/(D+p) = \textbf{0.000} \ kN \\ &T_{b,Ed} = T_{Ed,b} / n_{ou} = \textbf{0.000} \ kN \\ &k_2 = 0.63 \\ &F_{t,Rd} = B_{p,Rd} = min(k_2 * f_{ub} * A_s / \gamma_{M2}, 0.6 * \pi^* D_h * t_p * f_u / \gamma_{M2}) = \textbf{142.330} \ kN \\ &om = if(T_{b,Ed} < F_{t,Rd}, "PASS", "FAIL") = "PASS" \\ &T_{b,Ed} / \ F_{t,Rd} = \textbf{0.000} \end{split}$$

$$\begin{split} &M_{Ed\_plate}{=} T_{Ed,b}{}^{*}p{=}\textbf{0.000} \ kNm \\ &t_{p}{=}if(t_{p}{>}\sqrt{((\ M_{Ed\_plate}{*}4) \ / \ (B^{*} \ 275 \ N/mm^{2}))}, \ t_{p}, \ (\sqrt{((\ M_{Ed\_plate}{\times} \ 4) \ / \ (B^{*} \ 275 \ 17.300 \ mm)}) \\ &\textbf{17.300} \ mm \end{split}$$

$$\begin{split} & V_{Ed,b} = M_{Ed\_h}/D = \textbf{845.179 \ kN} \\ & A_{net} = (B\text{-}(n_{ou}\text{*}d))\text{*}T\text{=}\textbf{3603.590}\text{mm}^2 \\ & F_{ft,Rd} = (A_{net}\text{*}f_u)/\gamma_{M2}\text{=}\textbf{1181.978}\text{kN} \\ & trf\text{=} if(V_{Ed,b} < F_{ft,Rd},\text{"PASS"},\text{"FAIL"})\text{=}\textbf{"PASS"} \\ & V_{Ed,b} / F_{ft,Rd} = \textbf{0.715} \end{split}$$

# Summary information:

Diagonal Stiffner Required; Hogging Moment Check PASS; Opening-up Moment Check PASS; Reduced flange section tension check PASS;

Number of bolts on column: 8 bolts; Number of bolts on beams: 2 bolts; Required thickness of steel plates: 17.30 mm


Design shear resistance - cl 6.2.6(2)

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Diagonal stiffner requierement;

#### $ds=if(M_{Ed_h}/D < V_{pl,Rd}$ , "Not required", "Required")= "Required"

#### Hogging moment design:

;Number of bolts under shear; Shear force on bolts; Shear force per bolt;

Shear/Bearing capacity per bolt; Design check; Utilization;

#### Opening up moment design:

;Number of bolts under tension; Tension force on bolts; Tension force per bolt;

Tension/Punching capacity per bolt; Design check; Utilization; <u>Bending of plate subjected to tension from bolts:</u> Bending moment on plate; Plate thicknesss required; N/mm<sup>2</sup>))))=

#### Tension check with reduced flange:

Tension force on bolts; A<sub>net</sub>; Tension capacity of reduced flange; Design check; Utilization; 
$$\begin{split} n_h &= \textbf{4.000} \ ; \\ V_{Ed,b} &= M_{Ed_h}/D = \textbf{190.355} \ kN \\ F_{b,Ed} &= V_{Ed,b}/n_h = \textbf{47.589} \ kN \\ k_1 &= min(2.8^*(e_2/D_h) - 1.7, 1.4^*(p/D_h) - 1.7, 2.5) = \textbf{2.033} \\ \alpha_v &= min(e_1/(3^*D_h), p/(3^*D_h) - 0.25, f_{ub}/f_{u,1}) = \textbf{0.593} \\ F_{v,Rd} &= F_{b,Rd} = min(k_1^*\alpha_v^*f_u^*d^*t_p/\gamma_{M2}, 0.6^*f_{ub}A_s/\gamma_{M2}) = \textbf{59.441} \ kN \\ hc &= if(F_{b,Ed} < F_{b,Rd}, "PASS", "FAIL") =$$
**"PASS"** $\end{split}$ 

 $F_{b,Ed}\!\!/\;F_{b,Rd}\!\!=\!\!\boldsymbol{0.801}$ 

$$\begin{split} n_{ou} &= \textbf{2.000} \ ; \\ T_{Ed,b} &= M_{Ed\_ou}/(D+p) = \textbf{0.000} \ kN \\ T_{b,Ed} &= T_{Ed,b} \ / n_{ou} = \textbf{0.000} \ kN \\ k_2 &= 0.63 \\ F_{t,Rd} &= B_{p,Rd} &= min(k_2 * f_{ub} * A_s / \gamma_{M2}, 0.6 * \pi^* D_h * t_p * f_u / \gamma_{M2}) = \textbf{63.302} \ kN \\ om &= if(T_{b,Ed} < F_{t,Rd}, "PASS", "FAIL") = "PASS" \\ T_{b,Ed} \ / \ F_{t,Rd} = \textbf{0.000} \end{split}$$

$$\begin{split} &M_{Ed\_plate}{=} T_{Ed,b}{}^{*}p{=}\textbf{0.000} \ kNm \\ &t_{p}{=}if(t_{p}{>}\sqrt{((\ M_{Ed\_plate}{*}4) \ / \ (B^{*} \ 275 \ N/mm^{2}))}, \ t_{p}, \ (\sqrt{((\ M_{Ed\_plate}{\times} \ 4) \ / \ (B^{*} \ 275 \ \textbf{9.400} \ mm)} \end{split}$$

$$\begin{split} & V_{Ed,b} = M_{Ed\_h}/D = 190.355 \text{ kN} \\ & A_{net} = (B-(n_{ou}*d))*T = 1136.460 \text{mm}^2 \\ & F_{ft,Rd} = (A_{net}*f_u)/\gamma_{M2} = 372.759 \text{kN} \\ & trf = if(V_{Ed,b} < F_{ft,Rd},"PASS","FAIL") = "PASS" \\ & V_{Ed,b} / F_{ft,Rd} = 0.511 \end{split}$$

## Summary information:

Diagonal Stiffner Required; Hogging Moment Check PASS; Opening-up Moment Check PASS; Reduced flange section tension check PASS;

Number of bolts on column: 4 bolts; Number of bolts on beams: 2 bolts; Required thickness of steel plates: 9.40 mm

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## Masonry structural members

#### PADSTONE SUPPORT OF STEEL SUPPORT BACK WALL OF OUTRIGGER

In accordance with EN1996-1-1:2005 + A1:2012, incorporating Corrigenda February 2006 and July 2009 and the UK National Annex.

Tedds calculation version 1.0.05

Masonry panel details	
Panel length;	L = <b>2000</b> mm
Panel height;	h = <b>2500</b> mm
Thickness of load bearing leaf;	t = <b>215</b> mm
Effective height;	h <sub>ef</sub> = <b>2500</b> mm
Effective thickness;	t <sub>ef</sub> = <b>215</b> mm
Masonry material details	
Unit type;	Clay - Group 2
Compressive strength of masonry unit;	f <sub>c</sub> = <b>20</b> N/mm <sup>2</sup>
Height of unit;	h <sub>u</sub> = <b>65</b> mm
Width of unit;	w <sub>u</sub> = <b>215</b> mm
Conditioning factor;	k = <b>1.0</b>
- Conditioning to the air dry condition in accordance	e with cl.7.3.2
Shape factor - Table A.1;	d <sub>sf</sub> = <b>0.685</b>
Mean compressive strength of masonry unit;	$f_b = f_c \times k \times d_{sf} =$ <b>13.7</b> N/mm <sup>2</sup>
Specific weight of units;	γ = <b>18</b> kN/m <sup>3</sup>
Mortar type:	, M4 - General Purpose
Compressive strength of mortar;	f <sub>m</sub> = <b>4.0</b> N/mm <sup>2</sup>
Compressive strength factor - Tbl. NA 4;	K = <b>0.40</b>
Characteristic compressive strength of the masonry	- eq. 3.1
	$f_k = K \times f_b^{0.7} \times f_m^{0.3} = 3.79 \text{ N/mm}^2$
Short term secant modulus of elasticity factor:	K <sub>E</sub> = <b>1000</b>
Modulus of elasticity - cl.3.7.2:	E <sub>w</sub> = K <sub>F</sub> × f <sub>k</sub> = <b>3788</b> N/mm <sup>2</sup>
Design compressive strength of mesonry	
Cotogory of monufacturing controls	Cotogon/II
Class of execution control:	
Partial factor for material strength in direct or flowurs	
	$\gamma M = 3.00$
Cross-sectional area of wall;	$A = L \times t = 0.43 m^2;$
Design compressive strength of masonry;	$t_d = t_k / \gamma_M = 1.26 \text{ N/mm}^2$
Partial safety factors for design loads	
Partial safety factor for permanent load;	γ <sub>fG</sub> = <b>1.35</b>
Partial safety factor for variable load;	$\gamma_{fQ} = 1.50$
Superimposed vertical loading details	
Permanent UDL at top of wall:	a <sub>k</sub> = <b>2.00</b> kN/m
Variable UDL at top of wall:	g <sub>k</sub> = <b>1.30</b> kN/m
Eccentricity of permanent UDL load:	$e_{au} = 0 \text{ mm}$
Eccentricity of variable UDL load;	$e_{au} = 0 \text{ mm}$
Slanderness ratio of masonry well - Soction 5 5	1 Л
Sienderness ratio limit:	דיי אייי = 27
Sienderness ratio	$n_{\rm HM} = 21$
Sienuemess ralio,	$\lambda = \Pi_{\text{ef}} / \eta_{\text{ef}} = \Pi_{\text{III}} \mathbf{D}$

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#### PASS - Slenderness ratio is less than slenderness limit

#### **Concentrated Load 1 details - SB2 Reaction**



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Design of spreader beam



 $B_{\alpha P11} = 1/2 \times (\cosh(\alpha_1 \times P_{11}) \times \sin(180 \times \alpha_1 \times P_{11} / \pi) + \sinh(\alpha_1 \times P_{11}) \times$  $\cos(180 \times \alpha_1 \times P_{11} / \pi)) = 0.55$ 

Krilov's functions at the point load;

Type of spreader;

Height of spreader; Width of spreader;

Winkler's constant;

Using method of initial conditions

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						•		
Initial moment of LH edge;		$M_{01} = 0 \text{ kN}$	m					
Initial shear of LH edge;		$V_{01} = \mathbf{U} \text{ KIN}$						
which gives;		$(4 \times \alpha_1^2 \times 0)$	$\int \alpha  1 \times \partial 0  + 4 \times 0$	$(1 \times D_{\alpha   1} \times \Phi_{01}) \times$	Esp1 × Isp1 - Βα	$\alpha_1 \times \alpha_1 \times$		
and		$NEdc1 = \mathbf{U}.\mathbf{U}$		2		NI		
ano,		$(4 \times \alpha_1^\circ \times c$	$\sigma_{\alpha 1} \times \sigma_{01} + 4 \times \alpha$	$1^- \times \mathbf{U}_{\alpha 1} \times \Psi_{01} \times \mathbf{V}_{\alpha 1} \times \mathbf$	$\sqsubset_{sp1} \times I_{sp1} - A_0$	$P_{11} \times INEdc_1 =$		
Therefore		0.00 KIN						
Initial deflection of LH edge:		δ <sub>01</sub> = <b>0 619</b>	<b>39</b> mm					
Initial rotation of LH edge:		$\Phi_{01} = 0.010$	112					
milia rotationor Erredge,		<b>₩01 - 0.000</b>	7112					
Location of maximum deflection	;	Xdef1 = <b>350</b>	mm					
Krilov's functions at the spreade	r length;	$A_{\alpha x def1} = co$	$sh(\alpha_1 \times x_{def1}) \times 0$	$\cos(180 \times \alpha_1 \times x_c)$	lef1 / π) = <b>0.98</b>			
		$B_{\alpha x def1} = 1/2$	$2 \times (\cosh(\alpha_1 \times x))$	$_{def1}$ × sin(180 × $\alpha$	$\chi_1 \times \chi_{def1} / \pi) +$	$sinh(\alpha_1 \times$		
		$X_{def1}) \times COS$	$(180 \times \alpha_1 \times X_{def1})$	/ π)) = <b>0.55</b>	,	,		
Distance of point load right of lo	action;	p <sub>1def1</sub> = <b>0</b> m	im					
Krilov's functions at the spreade	r length;	$D_{\alpha p1 def1} = 1$	$/4 \times (\cosh(\alpha_1 \times$	p <sub>1def1</sub> ) × sin(180 >	$\propto \alpha_1 \times p_{1 def1} / \pi$	) - sinh( $\alpha_1 \times$		
		$p_{1def1}) \times cos$	$s(180  imes \alpha_1  imes p_{1d})$	ef1 / π)) = <b>0.00</b>				
Particular integral due to load;		$\delta'_1 = D_{\alpha p1def1} / \alpha_1^3 \times N_{Edc1} / (I_{sp1} \times E_{sp1}) = \textbf{0.000} mm$						
Maximum deflection;		$\delta_{max1} = A_{\alpha x \alpha}$	def1 × $\delta_{01}$ + Baxde	$f_{1} \times \Phi_{01} / \alpha_{1} + \delta'_{2}$	1 = <b>0.649</b> mm			
Location of maximum moment;		x <sub>M1</sub> = <b>350</b> r	nm					
Krilov's functions at the spreade	r length;	$C_{\text{ccxM1}} = 1/2 \times sinh(\alpha_1 \times x_{\text{M1}}) \times sin(180 \times \alpha_1 \times x_{\text{M1}} \ / \ \pi) = \textbf{0.15}$						
		$D_{\alpha xM1} = 1/4$	$\times (\cosh(\alpha_1 \times x_M))$	$(11) \times \sin(180 \times \alpha_1)$	$\times$ x <sub>M1</sub> / $\pi$ ) - sin	$h(\alpha_1 \times x_{M1}) \times$		
		$\cos(180 \times 6)$	$\alpha_1 \times \mathbf{x}_{M1} / \pi)) = 0$	.03				
Distance of point load right of lo	action;	$p_{1M1} = 0 m_1$	m					
Krilov's functions at the spreade	r length;	$B_{\alpha p1M1} = 1/2$	$2 \times (\cosh(\alpha_1 \times p))$	$_{1M1}$ × sin(180 × 0	αı×рıмı / π) +	$\sinh(\alpha_1 \times$		
<b>B</b>		$p_{1M1}) \times cos(180 \times \alpha_1 \times p_{1M1} / \pi)) = 0.00$						
Particular integral due to load;		$M^{1} = -B_{\alpha p 1 I}$	$\alpha_1 / \alpha_1 \times NEdc1 =$	0.00 KNM	··· <b>王</b> · ··· (1			
Maximum moment;		$M_{Edsp1} = (4 \times \alpha_1^2 \times C_{\alpha xM1} \times \delta_{01} + 4 \times \alpha_1 \times D_{\alpha xM1} \times \Phi_{01}) \times (I_{sp1} \times E_{sp1}) + M_{sp1} = C_{sp1} C_{sp1} C_{sp1} \times C_{sp1} $						
		IVI 1 = <b>5.60</b>	KINIII					
Location of maximum shear:		x <sub>v1</sub> = <b>350</b> n	nm					
Krilov's functions at the spreade	r length;	$B_{\alpha x V 1} = 1/2$	$\times (\cosh(\alpha_1 \times x_V))$	$(1) \times \sin(180 \times \alpha)$	× x <sub>V1</sub> / π) + sin	$h(\alpha_1 \times x_{V1}) \times$		
•	5 /	cos(180 × 0	$(\alpha_1 \times \mathbf{x}_{V1} / \pi)) = 0$	.55	,	( )		
		$C_{\alpha x V1} = 1/2$	$\times \sinh(\alpha_1 \times x_{V1})$	$\times \sin(180 \times \alpha_1 \times$	$x_{V1} / \pi = 0.15$	5		
Distance of point load right of lo	action;	p <sub>1V1</sub> = <b>0</b> mr	n	,	,			
Krilov's functions at the spreade	r length;	$A_{\alpha p1V1} = CO$	$sh(\alpha_1 \times p_{1V1}) \times \alpha_1$	$\cos(180 \times \alpha_1 \times p_1)$	v1 / π) = <b>1.00</b>			
Particular integral due to load;	-	$V'_1 = -A_{\alpha\beta1}V$	$1 \times N_{Edc1} = -67.5$	56 kN				
Shear at concentrated point load	d;	$V_1 = (4 \times \alpha$	$_{1}^{3} \times B_{\alpha x} v_{1} \times \delta_{01} +$	+ $4 \times \alpha_1^2 \times C_{\alpha x V 1}$	$ imes \Phi_{01})  imes (I_{sp1}  imes$	E <sub>sp1</sub> ) + V'1 =		
-		<b>-33.78</b> kN						
Maximum shear;		$V_{Edsp1} = Ma$	ax(Abs(V <sub>1</sub> ), N <sub>Edc</sub>	$_{1} - Abs(V_{1})) = 33$	.78 kN			
Maximum allowable stress unde	r spreader;	$\sigma_{\text{Rdsp1}} = \beta_1$	× f <sub>d</sub> = <b>1.26</b> N/mr	n²				
Maximum reaction;		$N_{Edsp1} = K_{c}$	$1 \times \delta_{max1} = 98.30$	kN/m				
Design stress;	<b>D</b> 4 6 6	$\sigma_{Edsp1} = N_{Edsp1}$	dsp1 / Wsp1 × (1 +	$6 \times e_{sp1} / W_{sp1} =$	1.20 N/mm <sup>2</sup>	a ulu a stara		
	PASS	- Design stress	under spreade	r is less than th	e allowable b	earing stress		

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## Walls subjected to mainly vertical loading - Section6.1.2

Eccentricity of permanent UDL at mid-height below concentrated load

 $e_{gmu1} = e_{gu} \times h_{c1} / (2 \times h) = 0.0 \text{ mm}$ 

Eccentricity of variable UDL at mid-height below concentrated load

	$e_{qmu1} = e_{qu} \times h_{c1} / (2 \times h) = 0.0 \text{ mm}$
Eccentricity of concentrated load at mid-height;	e <sub>mc1</sub> = e <sub>c1</sub> / 2 = <b>26.9</b> mm
Initial eccentricity - cl.5.5.1.1(4);	e <sub>init</sub> = h / 450 = <b>5.6</b> mm
Concentrated load at mid-height as UDL;	$N_{mc1} = N_{Edc1} / I_{efm1} = 42.86 \text{ kN/m}$
Vertical load at mid-height;	$N_{\text{Ed1}} = (g_k + \gamma \times t \times (h - h_{\text{c1}} / 2)) \times \gamma_{\text{fG}} + q_k \times \gamma_{\text{fQ}} + N_{\text{mc1}} = \textbf{54.04} \text{ kN/m}$
Design moment at mid-height;	$M_{Ed1} = g_k \times \gamma_{fG} \times e_{gmu1} + q_k \times \gamma_{fQ} \times e_{qmu1} + N_{mc1} \times e_{mc1} = \textbf{1.15} \text{ kNm/m}$
Eccentricities due to loads - eq. 6.7;	$e_{m1} = Abs(M_{Ed1}) / N_{Ed1} + e_{init} = 26.9 \text{ mm}$
Slenderness ratio limit for creep eccentricity;	$\lambda_{c} = 27$
Eccentricity due to creep;	e <sub>k1</sub> = <b>0.0</b> mm
Eccentricity at mid-height - eq. 6.6;	$e_{mk1} = Max(e_{m1} + e_{k1}, 0.05 \times t) = 26.9 \text{ mm}$
From eq. G2;	$A_{11} = 1 - 2 \times e_{mk1} / t = 0.75$
From eq. G3;	$u_1 = (h_{ef} / t_{ef} \times (1 / K_E)^{1/2} - 0.063) / (0.73 - 1.17 \times e_{mk1} / t) = 0.52$
Capacity reduction factor - eq. G1;	$\Phi_{m1} = A_{11} \times exp(-(u_1^2) / 2) = 0.65$
Design vertical resistance of panel - eq.6.2;	$N_{Rd1} = \Phi_{m1} \times t \times f_d = $ <b>177.68</b> kN/m
PA	SS - Design value of vertical resistance exceeds applied vertical load

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## **Timber structural members**

#### **NEW ROOF TERRACE JOISTS**

In accordance with EN1995-1-1:2004 + A1:2008 and Corrigendum No.1 and the UK National Annex incorporating National Amendment No.1

Tedds calculation version 1.5.11







Load Combination 1 (shown in proportion)







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Member details									
Load duration - cl.2.3.1.2;		Long-term	1						
Service class of timber - cl.2.3.1	.3;	1							
Length of bearing;		L <sub>b</sub> = <b>100</b> m	m						
Section properties									
Cross sectional area of member		$A = N \times b >$	< h = <b>10000</b> mm	2					
Section modulus;		$W_y = N \times b$	× h <sup>2</sup> / 6 = <b>3333</b>	<b>33</b> mm³					
		$W_z = h \times (N_z)$	$(1 \times b)^2 / 6 = 833$	<b>33</b> mm³					
Second moment of area;		$I_v = N \times b$	< h <sup>3</sup> / 12 = <b>3333</b>	<b>3333</b> mm⁴					
,		$I_z = h \times (N)$	× b) <sup>3</sup> / 12 = <b>208</b>	<b>3333</b> mm <sup>4</sup>					
Radius of ovration:		$r_v = \sqrt{I_v} / A$	) = <b>57.7</b> mm						
. ideide ei gjidtori,		$r_z = \sqrt{I_z} / \Delta$	) = 14.4  mm						
Denated for the state of the			/ -= • - <b>• • •</b>						
Partial factor for material prop	perties and resi	stances							
Partial factor for material proper	ties - Table 2.3;	γ <sub>M</sub> = <b>1.300</b>							
Modification factors									
Modification factor for load dura	tion and moistur	e content - Tabl	le 3.1						
		k <sub>mod</sub> = <b>0.70</b>	0						
Deformation factor for service cl	asses - Table 3	2; k <sub>def</sub> = <b>0.600</b>	k <sub>def</sub> = <b>0.600</b>						
Depth factor for bending - exp.3	.1;	k <sub>h.m</sub> = <b>1.000</b>							
Depth factor for tension - exp.3.	1;	k <sub>h.t</sub> = <b>1.000</b>	)						
Bending stress re-distribution fa	ctor - cl.6.1.6(2)	;  k <sub>m</sub> = <b>0.700</b>	k <sub>m</sub> = <b>0.700</b>						
Crack factor for shear resistance	ə - cl.6.1.7(2);	k <sub>cr</sub> = <b>0.670</b>							
Load configuration factor - exp.6	6.4;	k <sub>c.90</sub> = <b>1.50</b>	0						
System strength factor - cl.6.6;		k <sub>sys</sub> = <b>1.00</b>	D						
Effective length - Table 6.1;		$L_{ef} = 1.0 \times$	L <sub>s1</sub> = <b>3900</b> mm						
Critical bending stress - exp.6.3	2;	$\sigma_{\text{m.crit}} = 0.7$	$8 \times (N \times b)^2 \times E_0$	<sub>0.05</sub> / (h × L <sub>ef</sub> ) = <b>18</b>	<b>3.500</b> N/mm <sup>2</sup>				
Relative slenderness for bending	g - exp.6.30;	$\lambda_{\text{rel.m}} = \sqrt{[f_{\text{m}}]}$		9					
Lateral buckling factor - exp.6.34	4;	k <sub>crit</sub> = 1.56	$-0.75  imes \lambda_{rel.m} = 0$	0.706					
Compression perpendicular to	o the arain - cl.	6.1.5							
Design compressive stress:		σc 90 d = RΔ	$max / (N \times b \times (I))$	<sub>-b</sub> + min(L <sub>b</sub> , 30 m	m))) = <b>0.446</b> N	√mm²			
Design compressive strength		$f_{c,90,d} = k_{max}$	$\times \mathbf{k}_{svs} \times \mathbf{k}_{c}$ on $\times 1$	$f_{c,90,k} / \gamma_{M} = 2.019$	N/mm <sup>2</sup>				
		$\sigma_{c} = 1 / f_{c} = 0$	d = 0.221		,				
	PASS - Desig	n compressive	strenath exce	eds desian com	pressive stre	ss at bearing			
Ponding of 6.1.6	20019								
				~ <sup>2</sup>					
Design bending stress;		$\sigma_{m.d} = M / N$	/v <sub>y</sub> = <b>8.4/8</b> N/mi	m-					
Design bending strength;		$t_{m.d} = k_{h.m} \times$	$K_{mod} \times K_{sys} \times K_{cr}$	$_{\rm it} \times t_{\rm m.k} / \gamma_{\rm M} = 9.12$	21 N/mm <sup>2</sup>				
		$\sigma_{m.d} / f_{m.d} =$	0.930						
		PASS -	Design bending	g strength excee	eds design be	ending stress			
Shear - cl.6.1.7									
Applied shear stress;		$\tau_d = 3 \times F /$	$(2 \times k_{cr} \times A) = 0$	<b>.649</b> N/mm <sup>2</sup>					
Permissible shear stress;		$f_{v.d} = k_{mod} \times$	$k_{sys} \times f_{v.k} / \gamma_M =$	<b>2.154</b> N/mm <sup>2</sup>					
		$\tau_d \ / \ f_{v.d} = \bm{0}.$	301						
		PA	SS - Design sh	ear strength ex	ceeds design	shear stress			
Deflection - cl.7.2									
Deflection limit;		$\delta_{lim} = min(1)$	4 mm, 0.004 ×	L <sub>s1</sub> ) = <b>14.000</b> mm	ı				
· · · · · · · · · · · · · · · ·			,						

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Instantaneous deflection due to permanent load; Final deflection due to permanent load; Instantaneous deflection due to variable load; Factor for quasi-permanent variable action; Final deflection due to variable load;

Total final deflection;

#### $\delta_{\text{instG}} = \textbf{3.712} \text{ mm}$

 $\delta_{\text{finG}} = \delta_{\text{instG}} \times (1 + k_{\text{def}}) = \textbf{5.939} \text{ mm}$ 

 $\delta_{\text{instQ}} = \textbf{5.128} \text{ mm}$ 

ψ<sub>2</sub> = **0.3** 

 $\delta_{\text{finQ}} = \delta_{\text{instQ}} \times (1 + \psi_2 \times k_{\text{def}}) = \textbf{6.051} \text{ mm}$ 

 $\delta_{\text{fin}} = \delta_{\text{fin}G} + \delta_{\text{fin}Q} = \textbf{11.989} \ mm$ 

#### $\delta_{\text{fin}} \ / \ \delta_{\text{lim}} = \textbf{0.856}$

PASS - Total final deflection is less than the deflection limit

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## Foundations

#### FOUNDATION OF BOX-FRAME UNDER ORIGINAL BACK WALL

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

Analysis

Tedds calculation version 4.0.02

Tedds calculation version 1.0.13

#### Geometry Geometry Ceometry Ceomet

### Nodes

Node	Co-ord	linates		Freedom		Coordinate system		Spring		
	Х	Z	Х	Z	Rot.	Name	Angle	Х	Z	Rot.
	(m)	(m)					(°)	(kN/m)	(kN/m)	kNm/°
1	0	0	Fixed	Free	Free		0	0	2880	0
2	0.8	0	Free	Free	Free		0	0	2880	0
3	1.6	0	Free	Free	Free		0	0	2880	0
4	2.4	0	Free	Free	Free		0	0	2880	0
5	3.2	0	Free	Free	Free		0	0	2880	0
6	4	0	Free	Free	Free		0	0	2880	0
7	4.8	0	Free	Free	Free		0	0	2880	0
8	5.6	0	Free	Free	Free		0	0	2880	0

#### Materials

Name	Density (kg/m³)	Youngs Modulus kN/mm <sup>2</sup>	Shear Modulus kN/mm <sup>2</sup>	Thermal Coefficient °C <sup>-1</sup>
Steel (EC3)	7850	210	80.8	0.000012

#### Sections

Name	Area	Moment of inertia		Moment of inertia Shear ar	
		Major Minor		Ay	Az
	(cm²)	(cm4)	(cm4)	(cm²)	(cm²)
UC 254x254x89	113	14268	4857	80	27

#### Elements

Element	Length	No	des	Section	Material	Releases		Rotated	
	(m)	Start	End			Start moment	End moment	Axial	
1	0.8	1	2	UC 254x254x89	Steel (EC3)	Fixed	Fixed	Fixed	
2	0.8	2	3	UC 254x254x89	Steel (EC3)	Fixed	Fixed	Fixed	
3	0.8	3	4	UC 254x254x89	Steel (EC3)	Fixed	Fixed	Fixed	
4	0.8	4	5	UC 254x254x89	Steel (EC3)	Fixed	Fixed	Fixed	

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Element	Length	No	des	Section	Material	Releases		Rotated	
	(m)	Start	End			Start moment	End moment	Axial	
5	0.8	5	6	UC 254x254x89	Steel (EC3)	Fixed	Fixed	Fixed	
6	0.8	6	7	UC 254x254x89	Steel (EC3)	Fixed	Fixed	Fixed	
7	0.8	7	8	UC 254x254x89	Steel (EC3)	Fixed	Fixed	Fixed	

#### Members

Name	Elements				
	Start	End			
Member1	1	7			

#### Loading



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#### Load cases

Name	Enabled	Self weight factor	Patternable
Self Weight	yes	1	no
Left Column1	yes	0	no
Right Column1	yes	0	no
Pre-Deflection	yes	0	no
LoadCase5	yes	0	no

#### Load combinations

Load combination	Туре	Enabled	Patterned	
Foudations	Service	yes	no	

#### Load combination: Foudations (Service)

Load case	Factor
Self Weight	1
Left Column1	1
Right Column1	1
Pre-Deflection	1

#### Node loads

Node	Load case	Fo	Moment	
		X Z		
		(kN)	(kN)	(kNm)
1	Left Column1	0	223.4	0
8	Right Column1	0	212.5	0
1	Pre-Deflection	0	-50	0
8	Pre-Deflection	0	-50	0

#### Member VDL loads

Member	Load case	Position			Lo	Orientation	
		Туре	Start	End	Start	End	
					(kN/m)	(kN/m)	
Member1	Pre-Deflection	Ratio	0	0.5	0	35.7	GlobalZ
Member1	Pre-Deflection	Ratio	0.5	1	35.7	0	GlobalZ

#### Results

#### Forces



;

Partial factors - Section 6.1	
Resistance of cross-sections;	$\gamma_{MO} = 1$
Resistance of members to instability;	$\gamma_{M1} = 1$
Resistance of tensile members to fracture;	γ <sub>M2</sub> = <b>1.1</b>
Library item: Partial factors out	



#### Check design at start of span

#### Check y-y axis deflection - Section 7.2.1

Maximum deflection; Allowable deflection;  $\delta_y = 23.9 \text{ mm}$ 

 $\delta_{y,Allowable} = Min(L_{m1_{s1}} / 180, 25 \text{ mm}) = 25 \text{ mm}$ 

 $\delta_{y} \; / \; \delta_{y,\text{Allowable}} = \textbf{0.956}$ 

PASS - Design deflection does not exceed allowable deflection

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#### FOUNDATION UNDER REAR BOX-FRAME

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

#### Analysis

Tedds calculation version 4.0.02 Tedds calculation version 1.0.13

#### Geometry



#### Nodes

Node	Co-ord	dinates		Freedom		Coordina	te system		Spring	
	Х	Z	Х	Z	Rot.	Name	Angle	Х	Z	Rot.
	(m)	(m)					(°)	(kN/m)	(kN/m)	kNm/°
1	0	0	Fixed	Free	Free		0	0	1200	0
2	0.5	0	Free	Free	Free		0	0	1200	0
3	1	0	Free	Free	Free		0	0	1200	0
4	1.5	0	Free	Free	Free		0	0	1200	0
5	2	0	Free	Free	Free		0	0	1200	0
6	2.5	0	Free	Free	Free		0	0	1200	0
7	3	0	Free	Free	Free		0	0	1200	0
8	3.5	0	Free	Free	Free		0	0	1200	0
9	4	0	Free	Free	Free		0	0	1200	0
10	4.5	0	Free	Free	Free		0	0	1200	0
11	5	0	Free	Free	Free		0	0	1500	0

#### Materials

Name	Density	Youngs Modulus	Shear Modulus	Thermal Coefficient
	(kg/m³)	kN/mm <sup>2</sup>	kN/mm <sup>2</sup>	°C <sup>-1</sup>
Steel (EC3)	7850	210	80.8	0.000012

#### Sections

Name	Area	Moment of inertia		Shear area		
		Major Minor		Ay	Az	
	(cm²)	(cm <sup>4</sup> )	(cm4)	(cm²)	(cm²)	
UC 152x152x30	38	1748	560	26	10	

#### Elements

Element	Length	No	des	Section	Material	Releases		Rotated	
	(m)	Start	End			Start	End	Axial	
						moment	moment		
1	0.5	1	2	UC 152x152x30	Steel (EC3)	Fixed	Fixed	Fixed	
2	0.5	2	3	UC 152x152x30	Steel (EC3)	Fixed	Fixed	Fixed	
3	0.5	3	4	UC 152x152x30	Steel (EC3)	Fixed	Fixed	Fixed	

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Element	ement Length Nodes		Length Nodes Section Material			Rotated			
	(m)	Start	End			Start moment	End moment	Axial	
4	0.5	4	5	UC 152x152x30	Steel (EC3)	Fixed	Fixed	Fixed	
5	0.5	5	6	UC 152x152x30	Steel (EC3)	Fixed	Fixed	Fixed	
6	0.5	6	7	UC 152x152x30	Steel (EC3)	Fixed	Fixed	Fixed	
7	0.5	7	8	UC 152x152x30	Steel (EC3)	Fixed	Fixed	Fixed	
8	0.5	8	9	UC 152x152x30	Steel (EC3)	Fixed	Fixed	Fixed	
9	0.5	9	10	UC 152x152x30	Steel (EC3)	Fixed	Fixed	Fixed	
10	0.5	10	11	UC 152x152x30	Steel (EC3)	Fixed	Fixed	Fixed	

#### Members

Name	Elements					
	Start	End				
Member1	1	10				

#### Loading







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#### Load cases

Name	Enabled	Self weight factor	Patternable
Self Weight	yes	1	no
Left Column	yes	0	no
Right Column	yes	0	no
Pre-Deflection	yes	0	no

#### Load combinations

Load combination	Туре	Enabled	Patterned
Foudations	Service	yes	no

#### Load combination: Foudations (Service)

Load case	Factor
Self Weight	1
Left Column	1
Right Column	1
Pre-Deflection	1

#### Node loads

Node	Load case	Force		Moment
		х	Z	
		(kN)	(kN)	(kNm)
1	Left Column	0	49.1	0
11	Right Column	0	36	0
1	Pre-Deflection	0	0	0
11	Pre-Deflection	0	0	0

#### Member VDL loads

Member	Load case	Position		case Position Load			Orientation
		Туре	Start	End	Start	End	
					(kN/m)	(kN/m)	
Member1	Pre-Deflection	Ratio	0	0.5	0	0	GlobalZ
Member1	Pre-Deflection	Ratio	0.5	1	0	0	GlobalZ

Results



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#### **CANTILEVER RETAINING WALL DESIGN**

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

Tedds calculation v	version	2.9.00
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Retaining wall details	
Stem type;	Cantilever
Stem height;	h <sub>stem</sub> = <b>900</b> mm
Stem thickness;	t <sub>stem</sub> = <b>150</b> mm
Angle to rear face of stem;	$\alpha = 90 \text{ deg}$
Stem density;	$\gamma_{stem} = 25 \text{ kN/m}^3$
Toe length;	I <sub>toe</sub> = <b>1000</b> mm
Base thickness;	t <sub>base</sub> = <b>150</b> mm
Base density;	$\gamma_{\text{base}} = 25 \text{ kN/m}^3$
Height of retained soil;	h <sub>ret</sub> = <b>900</b> mm
Angle of soil surface;	$\beta = 0 \deg$
Depth of cover;	d <sub>cover</sub> = <b>0</b> mm
Height of water;	h <sub>water</sub> = <b>900</b> mm
Water density;	$\gamma_w = 9.8 \text{ kN/m}^3$
Retained soil properties	
Soil type;	Medium dense well graded sand
Moist density;	$\gamma_{mr} = 21 \text{ kN/m}^3$
Saturated density;	$\gamma_{sr} = 23 \text{ kN/m}^3$
Characteristic effective shear resistance angle;	φ'r.k = <b>30</b> deg
Characteristic wall friction angle;	$\delta_{r.k} = 0 \text{ deg}$
Base soil properties	
Soil type;	Medium dense well graded sand
Soil density;	$\gamma_{\rm b} = 18 \text{ kN/m}^3$
Characteristic effective shear resistance angle;	φ' <sub>b.k</sub> = <b>30</b> deg
Characteristic wall friction angle;	$\delta_{b,k} = 15 \text{ deg}$
Characteristic base friction angle;	$\delta_{bb.k} = 30 \text{ deg}$

Presumed bearing capacity;

#### Loading details

Variable surcharge load;

 $Surcharge_Q = 5 \text{ kN/m}^2$ 

 $P_{\text{bearing}} = 100 \text{ kN/m}^2$ 

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General arrangement

#### Calculate retaining wall geometry

Ibase = Itoe + tstem = 1150 mm Base length; Saturated soil height;  $h_{sat} = h_{water} + d_{cover} = 900 \text{ mm}$  $h_{moist} = h_{ret} - h_{water} = 0 mm$ Moist soil height; Length of surcharge load;  $I_{sur} = I_{heel} = 0 \text{ mm}$ - Distance to vertical component;  $x_{sur v} = I_{base} - I_{heel} / 2 = 1150 \text{ mm}$  $h_{eff} = h_{base} + d_{cover} + h_{ret} = 1050 \text{ mm}$ Effective height of wall; - Distance to horizontal component; x<sub>sur\_h</sub> = h<sub>eff</sub> / 2 = **525** mm Area of wall stem;  $A_{stem} = h_{stem} \times t_{stem} = 0.135 m^2$ - Distance to vertical component;  $x_{stem} = I_{toe} + t_{stem} / 2 = 1075 \text{ mm}$  $A_{\text{base}} = I_{\text{base}} \times t_{\text{base}} = 0.173 \text{ m}^2$ Area of wall base;  $x_{base} = I_{base} / 2 = 575 \text{ mm}$ - Distance to vertical component; **Using Rankine theory** At rest pressure coefficient;  $K_0 = 1 - sin(\phi'_{r.k}) = 0.500$  $K_{P} = (1 + \sin(\phi'_{b.k})) / (1 - \sin(\phi'_{b.k})) = 3.000$ Passive pressure coefficient; Bearing pressure check Vertical forces on wall  $F_{stem} = A_{stem} \times \gamma_{stem} = \textbf{3.4 kN}/m$ Wall stem; Wall base;  $F_{\text{base}} = A_{\text{base}} \times \gamma_{\text{base}} = \textbf{4.3} \text{ kN/m}$ Ftotal\_v = Fstem + Fbase + Fwater\_v = 7.7 kN/m Total: Horizontal forces on wall Surcharge load;  $F_{sur_h} = K_0 \times Surcharge_Q \times h_{eff} = 2.6 \text{ kN/m}$ 

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Saturated retained coil:		E., Ka	$\times (\alpha' \alpha') \times (b)$	$(1 + b)^{2}/2 - 3$	6 kN/m		
Water:		$\Gamma sat_n = R_0$	$\wedge (\gamma_{sr} - \gamma_{w}) \wedge (\Pi_{sr})$	at + Hbase) / 2 = 5	4 kN/m		
		$\Gamma_{water_h} = \gamma_{w}$	$v \times (\Pi water + U_{COVE})$	$h = \frac{12}{2} = 3$	.4 KIN/III		
Base soll,		$F_{pass_h} = -r$	ν γ <sub>b</sub> × (Ucover +	$(1base)^{-}/2 = -0.0$		NI/m	
Total,	F <sub>total_h</sub> = F <sub>sat_h</sub> + F <sub>moist_h</sub> + F <sub>pass_h</sub> + F <sub>water_h</sub> + F <sub>sur_h</sub> = <b>11.1</b> kN/m				IN/III		
Moments on wall							
Wall stem;	M <sub>stem</sub> = F <sub>stem</sub> × x <sub>stem</sub> = <b>3.6</b> kNm/m						
Wall base;		$M_{base} = F_{ba}$	$se \times Xbase = 2.5 k$	Nm/m			
Surcharge load;		$M_{sur} = -F_{sur}$	$h \times X_{sur_h} = -1.4$	kNm/m			
Saturated retained soil;		$M_{sat} = -F_{sat}$	_h × x <sub>sat_h</sub> = -1.3	kNm/m			
Water;		$M_{water} = -F_{w}$	water_h $\times$ Xwater_h =	<b>-1.9</b> kNm/m			
Total;		$M_{total} = M_{ster}$	em + Mbase + Msat	+ M <sub>moist</sub> + M <sub>water</sub>	+ M <sub>sur</sub> = <b>1.6</b> kM	Nm/m	
Check bearing pressure							
Propping force;		$F_{prop\_base} =$	$F_{total_h} = 11.1 \text{ kN}$	l/m			
Distance to reaction;		$\overline{x} = M_{total} /$	F <sub>total_v</sub> = <b>204</b> mr	n			
Eccentricity of reaction;		$e = \overline{x} - I_{bas}$	<sub>se</sub> / 2 = <b>-371</b> mm				
Loaded length of base;		$I_{load} = 3 \times$	x = <b>611</b> mm				
Bearing pressure at toe;		$q_{toe} = 2 \times F$	$t_{total_v} / I_{load} = 25.5$	<b>2</b> kN/m²			
Bearing pressure at heel;		$q_{heel} = 0 kN$	J/m²				
Factor of safety;		$FoS_{bp} = P_b$	<sub>earing</sub> / max(q <sub>toe</sub> , o	q <sub>heel</sub> ) = <b>3.971</b>			
	PASS - A	llowable bearii	ng pressure exc	ceeds maximum	n applied bear	ring pressure	

Retaining wall design

In accordance with EN1992-1-1:2004 incorporating Corrigendum dated January 2008 and the UK National Annex incorporating National Amendment No.1 and EN1996-1-1:2005 incorporating Corrigenda dated February 2006 and July 2009 and the UK National Annex

Tedds calculation version 2.9.00

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

Concrete strength class;	C30/37
Characteristic compressive cylinder strength;	f <sub>ck</sub> = <b>30</b> N/mm <sup>2</sup>
Characteristic compressive cube strength;	f <sub>ck,cube</sub> = <b>37</b> N/mm <sup>2</sup>
Mean value of compressive cylinder strength;	$f_{cm} = f_{ck} + 8 \text{ N/mm}^2 = 38 \text{ N/mm}^2$
Mean value of axial tensile strength;	$f_{ctm} = 0.3 \text{ N/mm}^2 \times (f_{ck} / 1 \text{ N/mm}^2)^{2/3} = 2.9 \text{ N/mm}^2$
5% fractile of axial tensile strength;	$f_{ctk,0.05} = 0.7 \times f_{ctm} = \textbf{2.0} \ N/mm^2$
Secant modulus of elasticity of concrete;	$E_{cm} = 22 \text{ kN/mm}^2 \times (f_{cm} / 10 \text{ N/mm}^2)^{0.3} = 32837 \text{ N/mm}^2$
Partial factor for concrete - Table 2.1N;	$\gamma_{\rm C} = 1.50$
Compressive strength coefficient - cl.3.1.6(1);	α <sub>cc</sub> = <b>0.85</b>
Design compressive concrete strength - exp.3.15;	$f_{cd} = \alpha_{cc} \times f_{ck} \ / \ \gamma_C = \textbf{17.0} \ N/mm^2$
Maximum aggregate size;	h <sub>agg</sub> = <b>20</b> mm
Maximum aggregate size; Reinforcement details	h <sub>agg</sub> = <b>20</b> mm
Maximum aggregate size; Reinforcement details Characteristic yield strength of reinforcement;	h <sub>agg</sub> = <b>20</b> mm f <sub>yk</sub> = <b>500</b> N/mm <sup>2</sup>
Maximum aggregate size; <b>Reinforcement details</b> Characteristic yield strength of reinforcement; Modulus of elasticity of reinforcement;	$h_{agg} = 20 \text{ mm}$ $f_{yk} = 500 \text{ N/mm}^2$ $E_s = 200000 \text{ N/mm}^2$
Maximum aggregate size; <b>Reinforcement details</b> Characteristic yield strength of reinforcement; Modulus of elasticity of reinforcement; Partial factor for reinforcing steel - Table 2.1N;	$h_{agg} = 20 \text{ mm}$ $f_{yk} = 500 \text{ N/mm}^2$ $E_s = 200000 \text{ N/mm}^2$ $\gamma_S = 1.15$
Maximum aggregate size; <b>Reinforcement details</b> Characteristic yield strength of reinforcement; Modulus of elasticity of reinforcement; Partial factor for reinforcing steel - Table 2.1N; Design yield strength of reinforcement;	$\begin{split} h_{agg} &= \textbf{20} \text{ mm} \\ f_{yk} &= \textbf{500} \text{ N/mm}^2 \\ E_s &= \textbf{200000} \text{ N/mm}^2 \\ \gamma_S &= \textbf{1.15} \\ f_{yd} &= f_{yk} \ / \ \gamma_S &= \textbf{435} \text{ N/mm}^2 \end{split}$
Maximum aggregate size; <b>Reinforcement details</b> Characteristic yield strength of reinforcement; Modulus of elasticity of reinforcement; Partial factor for reinforcing steel - Table 2.1N; Design yield strength of reinforcement; <b>Cover to reinforcement</b>	$\begin{split} h_{agg} &= \textbf{20} \text{ mm} \\ f_{yk} &= \textbf{500} \text{ N/mm}^2 \\ E_s &= \textbf{200000} \text{ N/mm}^2 \\ \gamma_S &= \textbf{1.15} \\ f_{yd} &= f_{yk} \ / \ \gamma_S &= \textbf{435} \text{ N/mm}^2 \end{split}$
Maximum aggregate size; <b>Reinforcement details</b> Characteristic yield strength of reinforcement; Modulus of elasticity of reinforcement; Partial factor for reinforcing steel - Table 2.1N; Design yield strength of reinforcement; <b>Cover to reinforcement</b> Top face of base;	$\label{eq:hagg} \begin{array}{l} h_{agg} = {\color{black}{20}} mm \\ f_{yk} = {\color{black}{500}} N/mm^2 \\ E_s = {\color{black}{200000}} N/mm^2 \\ \gamma_S = {\color{black}{1.15}} \\ f_{yd} = f_{yk} \ / \ \gamma_S = {\color{black}{435}} N/mm^2 \\ c_{bt} = {\color{black}{50}} mm \end{array}$
Maximum aggregate size; <b>Reinforcement details</b> Characteristic yield strength of reinforcement; Modulus of elasticity of reinforcement; Partial factor for reinforcing steel - Table 2.1N; Design yield strength of reinforcement; <b>Cover to reinforcement</b> Top face of base; Bottom face of base;	$\label{eq:hagg} \begin{array}{l} h_{agg} = {\color{black}{20}} mm \\ f_{yk} = {\color{black}{500}} N/mm^2 \\ E_s = {\color{black}{200000}} N/mm^2 \\ \gamma_S = {\color{black}{1.15}} \\ f_{yd} = f_{yk} \ / \ \gamma_S = {\color{black}{435}} N/mm^2 \\ \\ C_{bt} = {\color{black}{50}} mm \\ c_{bb} = {\color{black}{75}} mm \end{array}$

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Masonry details - Section 3.1	I	1			1	1		
Masonry type:		Autoclaved	d aerated con	crete - Group 1				
Normalised mean compressive	strength:	f <sub>b</sub> = <b>10.4</b> N	l/mm²					
Characteristic flexural strength -	cl.6.3.4(1);	f <sub>xk</sub> = <b>0</b> N/m	1m <sup>2</sup>					
Initial shear strength - Table NA	.5;	f <sub>vko</sub> = <b>0.2</b> N	l/mm²					
Mortar details - Section 3.2								
Mortar type;		General pu	urpose - M12,	prescribed mix				
Compressive strength of mortar	<b>.</b> ,	f <sub>m</sub> = <b>12</b> N/r	f <sub>m</sub> = <b>12</b> N/mm <sup>2</sup>					
Ultimate limit states - Table N	A.1							
Class of execution control;		1						
Category of manufacture contro	l;	2	2					
Partial factor for direct or flexura	l compression;	$\gamma_{Mc} = 2.3$						
Partial factor for flexural tension	,	$\gamma_{Mt} = 2.3$						
Partial factor for shear;		$\gamma_{Mv} = 2.0$	$\gamma_{Mv} = 2.0$					
Characteristic strengths of co	ncrete infill - T	able 3.2						
Concrete infill strength class;		C30/37						
Characteristic compressive stren	ngth;	$f_{ck,infill} = 25$	f <sub>ck,infill</sub> = <b>25</b> N/mm <sup>2</sup>					
Characteristic shear strength;		$f_{cvk,infill} = 0$	<b>45</b> N/mm²					
	$f_{cvd,infill} = f_{cvk,infill} / \gamma_{Mv}$			<sub>v</sub> = <b>0.225</b> N/mm <sup>2</sup>				



Check stem design at base of stem	
Depth of section;	t = <b>150</b> mm
Hollow wall details	
Face shell thickness;	t <sub>shell</sub> = <b>44</b> mm
Web shell thickness;	t <sub>web</sub> = <b>44</b> mm
Masonry characteristics	
Compressive strength constants - Table NA.4;	K = <b>0.55</b>
Characteristic compressive strength - cl.3.6.1.2(1);	$f_k = K \times f_b{}^{0.7} \times min(f_m,f_b,12~N/mm^2){}^{0.3} = \textbf{5.72}~N/mm^2$
Design compressive strength;	$f_d = min(f_k, f_{ck,infill}) / \gamma_{Mc} = 2.487 \text{ N/mm}^2$
Design flexural strength;	$f_{xd} = f_{xk} / \gamma_{Mt} = \boldsymbol{0} N/mm^2$
Height of masonry;	h <sub>wt</sub> = h <sub>stem</sub> = <b>900</b> mm
Compressive axial force combination 0;	$F_x = \gamma_{Gf} \times \gamma_{stem} \times h_{wt} \times t = \textbf{3.4 kN}/m$
Moment combination 0;	$M_x = \gamma_{Gf} \times \gamma_{stem} \times h_{wt} \times t^2 \ / \ 2 = \textbf{0.3} \ kNm/m$
Eccentricity of axial load:	$e = max(abs(t / 2 - M_x / F_x), 0.05 \times t) = 7.5 mm$

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Canadity reduction factor over 6	4.	<u> ተ 1 ዓ</u>	o / t 00				
Capacity reduction factor - exp.6	.4,	$\Psi = 1 - 2 \times$	e/l = 0.9				
Design vertical resistance - exp.	0.2,	$N_{Rd} = \Psi \times I$	$X I_d = 333.7 KIN/$	(11) +) 0.000 NI/mm	.2		
Design vertical compressive stre	ss;	$\sigma_d = \min(F)$	$$	l) = 0.023 N/mm	-		
Apparent design liexural strength	1 - exp.6.16;	$I_{xd,app} = I_{xd}$	$+ \sigma_d = 0.023 \text{ N/m}$	() () () () () () () () () () () () () (	N/mm²		
Characteristic shear strength - es	xp.3.5,	$I_{VK} = \Pi \Pi \Pi (I_{VK})$	$(0 + 0.4 \times 0_d, 0.0)$	$(00 \times 1_b) = 0.209$	N/11111-		
Design shear strength,		$I_{vd} = I_{vk} / \gamma_N$	v = 0.105  N/mm	-			
Reinforced masonry members	subjected to b	ending, bendi	ng and axial loa	ading, or axial lo	oading - Secti	on 6.6	
Design bending moment combin	ation 1;	M = <b>4.2</b> kN	m/m				
lension reinforcement provided;		12 dia.bars	@ 220 c/c				
Area of tension reinforcement pro	ovided;	$A_{sr.prov} = \pi$	$\times \phi_{sr^2} / (4 \times S_{sr}) =$	• <b>514</b> mm²/m			
Depth to tension reinforcement;	al 0.0.0/1\-	a = <b>80</b> mm		~2/~			
Minimum area of reinforcement -	cl.8.2.3(1);	$A_{sr.min} = 0.0$	$J005 \times d = 40$ mi	m²/m	05) <b>05</b>		
Lever arm - exp.6.23;		$z = a \times min$	$1(1 - 0.5 \times A_{sr.prov})$	$\times$ f <sub>yd</sub> / ( $\alpha \times$ f <sub>d</sub> ), 0.	95) = <b>35</b> mm		
Moment of resistance - exp.6.22	and exp.6.24a;	$M_{Rd} = min($	$A_{sr.prov} \times f_{yd} \times Z, 0$	$J.4 \times I_d \times d^2) = \mathbf{b}.$	<b>4</b> KNM/M		
		$IVI / IVI_{Rd} = U$	Moment of re	aiatanaa ayaaa	do applied do	oian momont	
		PA33		Sistance exceed	is applied des	sign moment	
Reinforced masonry members	subjected to s	hear loading -	Section 6.7				
Design shear force	40.	V = 12.344	KN/M	0.00 1.01/			
Design shear resistance - exp.6.40; $V_{Rd} = min(T_{vd}, T_{cvd,infill}) \times d = 8.36 \text{ KIV/m}$							
		V / VRd = I FAIL - Design	.411 shqar rasistan	co is loss than a	annliad dasiau	n shoar forco	
Herizentel veinferennent nevel		i m			ipplied deolgi		
Horizontal reinforcement para			005 d <b>10</b>				
Transverse reinforcement -	cl.o.2.3(4),	$A_{sx.req} = 0.0$	$0005 \times 0 = 40$ mi	11-/111			
Area of transverse reinforcement	eu, t provided:		× d= 200 C/C	<b>- 303</b> mm <sup>2</sup> /m			
Area of transverse reinforcement	PASS - Area of	R <sub>sx.prov</sub> = π	×ψsx / (4 × Ssx) : t nrovided is ai	eater than area	of reinforcen	nent required	
Ohaak haas dasim at tas			i protraca io gi			ient required	
Check base design at toe		h – <b>150</b> mr	'n				
		n = 150 m	11				
Rectangular section in flexure	- Section 6.1		,				
Design bending moment combine	ation 1;	M = <b>5.9</b> KN	m/m				
Depth to tension reinforcement;		$d = h - c_{bb} - \phi_{bb} / 2 = 69 \text{ mm}$					
		$K = M / (d^2 \times f_{ck}) = 0.042$					
		K' = <b>0.207</b>		la comprossion	rainfaraama	t in required	
Lover arm:		7 min(0 5			d 66 mm	it is required	
Level ann, Dooth of poutrol axis:		$Z = \Pi \Pi (0.3)$	(1 - 3.5)	5 × K) <sup>20</sup> , 0.95) ×	u = <b>00</b> mm		
Area of topology reinforcement re-	auirod:	$X = 2.5 \times (0)$	$(f_{1}, f_{2}, f_{3}, f_{3},$	$mm^2/m$			
Tansion reinforcement provided:	quileu,	12 dia bar	$(1_{yd} \times 2) = 200$	11111 /111			
Area of tension reinforcement provided: $A_{bb, prev} = \pi \times d_{bb}^2 / (4 \times s_{bb}) = 565 \text{ mm}^2/\text{m}$							
Minimum area of reinforcement -	area of reinforcement - exp.9.1N: Abb min = max(0.26 × fetm / fvk = 0.0013) × d = 104 mm <sup>2</sup> /m						
Maximum area of reinforcement	- c 9211/(3)	$12.1 \ 1/3). \qquad \Delta_{hh} \ max = 0.04 \ x \ h = 6000 \ mm^2/m$					
	$max(A_{bb reg}, A_{bb reg}) / A_{bb regv} = 0.369$						
	PASS - Area of	reinforcemen	t provided is ar	eater than area	of reinforcer	nent reauired	
Crack control - Section 7.2			,				
Limiting crack width:		Wmay - 0 2	mm				
Variable load factor - EN1000	Tahle A1 1.	wmax = 0.3					
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Serviceability bending moment;		Msls = 4.2 k	Nm/m						
Tensile stress in reinforcement;		$\sigma_s = M_{sls} / ($	$A_{bb.prov} \times z) = 11$	<b>4.6</b> N/mm <sup>2</sup>					
Load duration;		Long term							
Load duration factor;		$k_t = 0.4$							
Effective area of concrete in tens	sion;	$A_{c.eff} = min$	(2.5 × (h - d), (h	- x) / 3, h / 2) = 4	<b>47125</b> mm²/m				
Mean value of concrete tensile s	strength;	$f_{ct.eff} = f_{ctm} =$	<b>= 2.9</b> N/mm²						
Reinforcement ratio;		$\rho_{p.eff} = A_{bb.p}$	$A_{c.eff} = 0.012$	2					
Modular ratio;		$\alpha_e = E_s / E_s$	cm = <b>6.091</b>						
Bond property coefficient;		k1 = <b>0.8</b>							
Strain distribution coefficient;		k <sub>2</sub> = <b>0.5</b>							
		k <sub>3</sub> = <b>3.4</b>							
		$k_4 = 0.425$							
Maximum crack spacing - exp.7	.11;	$s_{r.max} = k_3 >$	$s_{r.max} = k_3 \times c_{bb} + k_1 \times k_2 \times k_4 \times \phi_{bb} \ / \ \rho_{p.eff} = \textbf{425} \ mm$						
Maximum crack width - exp.7.8;		$w_{k} = s_{r.max} \times max(\sigma_{s} - k_{t} \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_{e} \times \rho_{p.eff}), 0.6 \times \sigma_{s}) / E_{s}$							
		w <sub>k</sub> = <b>0.146</b> mm							
		$w_k / w_{max} =$	0.487						
		PASS	- Maximum cra	ack width is les	s than limiting	g crack width			
Rectangular section in shear -	Section 6.2								
Design shear force;		V = <b>7.8</b> kN/	′m						
		$C_{\text{Rd,c}} = 0.18$	B / γ <sub>C</sub> = <b>0.120</b>						
		k = min(1 +	√(200 mm / d),	2) = <b>2.000</b>					
Longitudinal reinforcement ratio	;	$\rho_{\rm I} = \min(A_{\rm bb, prov} / d, 0.02) = 0.008$							
		$v_{min} = 0.035 \ N^{1/2}/mm \times k^{3/2} \times f_{ck}^{0.5} = 0.542 \ N/mm^2$							
Design shear resistance - exp.6	.2a & 6.2b;	$V_{\text{Rd.c}} = \max(C_{\text{Rd.c}} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{\text{ck}})^{1/3}, v_{\text{min}}) \times d$							
				$V_{Bd,c} = 48.2 \text{ kN/m}$					
		$V / V_{Rd.c} = 0$	0.162						
		PAS	SS - Design she	ar resistance e	xceeds desig	n shear force			
Secondary transverse reinford	ement to base	Section 9.3							
Minimum area of reinforcement	– cl.9.3.1.1(2);	$A_{bx.req} = 0.2$	$2 \times A_{bb,prov} = 113$	mm²/m					
Maximum spacing of reinforcem	ent – cl.9.3.1.1(3	); Sbx_max = 45	5 <b>0</b> mm						
Transverse reinforcement provid	led;	10 dia.bars	@ 200 c/c						
Area of transverse reinforcemen	Area of transverse reinforcement provided;			$A_{bx,prov} = \pi \times \phi_{bx}^2 / (4 \times s_{bx}) = 393 \text{ mm}^2/\text{m}$					

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

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Reinforcement details

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#### FIXED-PINNED RETAINING WALL DESIGN

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

Tedds calculation version 2.9.00

Retaining wall details	
Stem type;	Cantilever
Stem height;	h <sub>stem</sub> = <b>1650</b> mm
Stem thickness;	t <sub>stem</sub> = <b>210</b> mm
Angle to rear face of stem;	$\alpha = 90 \text{ deg}$
Stem density;	$\gamma_{stem} = 25 \text{ kN/m}^3$
Toe length;	l <sub>toe</sub> = <b>1500</b> mm
Base thickness;	t <sub>base</sub> = <b>150</b> mm
Base density;	$\gamma_{\text{base}} = 25 \text{ kN/m}^3$
Height of retained soil;	h <sub>ret</sub> = <b>1650</b> mm
Angle of soil surface;	$\beta = 0 \text{ deg}$
Depth of cover;	$d_{cover} = 0 mm$
Height of water;	h <sub>water</sub> = <b>1650</b> mm
Water density;	$\gamma_w = 9.8 \text{ kN/m}^3$
Retained soil properties	
Soil type;	Medium dense well graded sand
Moist density;	$\gamma_{mr} = 21 \text{ kN/m}^3$
Saturated density;	$\gamma_{sr} = 23 \text{ kN/m}^3$
Characteristic effective shear resistance angle;	φ'r.k = <b>30</b> deg
Characteristic wall friction angle;	$\delta_{r.k} = 0 \text{ deg}$
Base soil properties	
Soil type;	Medium dense well graded sand
Soil density;	$\gamma_{\rm b} = 18 \text{ kN/m}^3$
Characteristic effective shear resistance angle;	φ' <sub>b.k</sub> = <b>30</b> deg
Characteristic wall friction angle;	$\delta_{b.k} = 15 \text{ deg}$
Characteristic base friction angle;	$\delta_{bb.k} = 30 \text{ deg}$

Pbearing = **100** kN/m<sup>2</sup>

#### Loading details

Variable surcharge load;

Presumed bearing capacity;

 $Surcharge_Q = \textbf{1.5} \text{ kN/m}^2$ 

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#### Calculate retaining wall geometry

Base length; Saturated soil height; Moist soil height; Length of surcharge load; - Distance to vertical component; Effective height of wall; - Distance to horizontal component; Area of wall stem: - Distance to vertical component; Area of wall base; - Distance to vertical component; **Using Rankine theory** At rest pressure coefficient; Passive pressure coefficient; Bearing pressure check Vertical forces on wall Wall stem; Wall base; Total; Horizontal forces on wall Surcharge load;

$$\begin{split} & |_{base} = |_{toe} + t_{stem} = 1710 \text{ mm} \\ & h_{sat} = h_{water} + d_{cover} = 1650 \text{ mm} \\ & h_{moist} = h_{ret} - h_{water} = 0 \text{ mm} \\ & l_{sur} = l_{heel} = 0 \text{ mm} \\ & x_{sur\_v} = l_{base} - l_{heel} / 2 = 1710 \text{ mm} \\ & h_{eff} = h_{base} + d_{cover} + h_{ret} = 1800 \text{ mm} \\ & x_{sur\_h} = h_{eff} / 2 = 900 \text{ mm} \\ & A_{stem} = h_{stem} \times t_{stem} = 0.347 \text{ m}^2 \\ & x_{stem} = l_{toe} + t_{stem} / 2 = 1605 \text{ mm} \\ & A_{base} = l_{base} \times t_{base} = 0.257 \text{ m}^2 \\ & x_{base} = l_{base} / 2 = 855 \text{ mm} \end{split}$$

$$\begin{split} & K_0 = 1 \, - \, sin(\varphi'_{r,k}) = \textbf{0.500} \\ & K_P = \left(1 \, + \, sin(\varphi'_{b,k})\right) \, / \, \left(1 \, - \, sin(\varphi'_{b,k})\right) = \textbf{3.000} \end{split}$$

$$\begin{split} F_{stem} &= A_{stem} \times \gamma_{stem} = \textbf{8.7 kN/m} \\ F_{base} &= A_{base} \times \gamma_{base} = \textbf{6.4 kN/m} \\ F_{total\_v} &= F_{stem} + F_{base} + F_{water\_v} = \textbf{15.1 kN/m} \end{split}$$

 $F_{sur\_h} = K_0 \times Surcharge_Q \times h_{eff} = \textbf{1.4 kN/m}$ 

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Saturated retained soil;		$F_{sat_h} = K_0$	$\times$ ( $\gamma_{ m sr}$ ' - $\gamma_{ m w}$ ') $ imes$ (h <sub>sa</sub>	$h_{t} + h_{base})^2 / 2 = 10$	<b>0.7</b> kN/m					
Water;		$F_{water_h} = \gamma_w$	$J' \times (h_{water} + d_{cover})$	$(r + h_{base})^2 / 2 = 13$	<b>5.9</b> kN/m					
Moist retained soil;		$F_{moist_h} = K_b$	$0  imes \gamma_{mr}'  imes ((h_{eff} - h_{mr}))$	n <sub>sat</sub> - h <sub>base</sub> ) <sup>2</sup> / 2 + (	(h <sub>eff</sub> - h <sub>sat</sub> - h <sub>bas</sub>	$_{\rm se})  imes (h_{ m sat} +$				
		h <sub>base</sub> )) = <b>0</b> ł	kN/m							
Base soil;		$F_{pass_h} = -K$	$P  imes \gamma_b'  imes (d_{cover} +$	h <sub>base</sub> ) <sup>2</sup> / 2 = -0.6	kN/m					
Total;		$F_{total_h} = F_{sa}$	at_h + Fmoist_h + F	pass_h + F <sub>water_h</sub> +	F <sub>sur_h</sub> = <b>27.3</b> k	N/m				
Moments on wall										
Wall stem;		$M_{stem} = F_{ste}$	m × X <sub>stem</sub> = <b>13.9</b>	kNm/m						
Wall base;	M <sub>base</sub> = F <sub>base</sub> × x <sub>base</sub> = <b>5.5</b> kNm/m									
Surcharge load;		M <sub>sur</sub> = -F <sub>sur</sub>	_h × X <sub>sur_h</sub> = -1.2	kNm/m						
Saturated retained soil;		$M_{sat} = -F_{sat}$	$h \times X_{sat_h} = -6.4$	kNm/m						
Water;		$M_{water} = -F_{w}$	$_{\text{vater}_h} \times X_{\text{water}_h} =$	<b>-9.5</b> kNm/m						
Moist retained soil;		M <sub>moist</sub> = -F <sub>n</sub>	$noist_h \times Xmoist_h =$	<b>0</b> kNm/m						
Total;		$M_{total} = M_{ste}$	m + M <sub>base</sub> + M <sub>sat</sub>	+ M <sub>moist</sub> + M <sub>water</sub>	+ M <sub>sur</sub> = <b>2.2</b> ki	Nm/m				
Check bearing pressure										
Propping force:		Eprop base =	Ftotal h = <b>27.3</b> kN	l/m						
Distance to reaction:		$\overline{\mathbf{X}} = \mathbf{M}_{\text{total}}$	$F_{total v} = 148 \text{ mr}$	n						
Eccentricity of reaction:		$e = \overline{x} - bas$	e / 2 = <b>-707</b> mm							
Loaded length of base:		$l_{load} = 3 \times$	x = <b>443</b> mm							
Bearing pressure at toe:		$Q_{\text{top}} = 2 \times F$	total $v / load = 68$ .	1 kN/m²						
Bearing pressure at heel:		$q_{\text{heel}} = 0 \text{ kN}$	l/m <sup>2</sup>							
Factor of safety:		$FoS_{bp} = P_{bp}$	earing / max(dtoe. (	Dheel) = <b>1.469</b>						
····· <b>·</b> ,	PASS - A	llowable bearir	ng pressure exc	ceeds maximum	applied bear	ring pressure				

Library item: Bearing FoS output Retaining wall design

In accordance with EN1992-1-1:2004 incorporating Corrigendum dated January 2008 and the UK National Annex incorporating National Amendment No.1 and EN1996-1-1:2005 incorporating Corrigenda dated February 2006 and July 2009 and the UK National Annex

Tedds calculation version 2.9.00

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

Concrete strength class;	C30/37
Characteristic compressive cylinder strength;	f <sub>ck</sub> = <b>30</b> N/mm <sup>2</sup>
Characteristic compressive cube strength;	f <sub>ck,cube</sub> = <b>37</b> N/mm <sup>2</sup>
Mean value of compressive cylinder strength;	$f_{cm} = f_{ck} + 8 \text{ N/mm}^2 = 38 \text{ N/mm}^2$
Mean value of axial tensile strength;	$f_{ctm} = 0.3 \ N/mm^2 \times (f_{ck} \ / \ 1 \ N/mm^2)^{2/3} = \textbf{2.9} \ N/mm^2$
5% fractile of axial tensile strength;	$f_{ctk,0.05} = 0.7 \times f_{ctm} = 2.0 \text{ N/mm}^2$
Secant modulus of elasticity of concrete;	$E_{cm} = 22 \text{ kN/mm}^2 \times (f_{cm} / 10 \text{ N/mm}^2)^{0.3} = 32837 \text{ N/mm}^2$
Partial factor for concrete - Table 2.1N;	$\gamma_{\rm C} = 1.50$
Compressive strength coefficient - cl.3.1.6(1);	$\alpha_{cc} = 0.85$
Design compressive concrete strength - exp.3.15;	$f_{cd} = \alpha_{cc} \times f_{ck} \ / \ \gamma_C = \textbf{17.0} \ N/mm^2$
Maximum aggregate size;	h <sub>agg</sub> = <b>20</b> mm
Reinforcement details	
Characteristic yield strength of reinforcement;	f <sub>yk</sub> = <b>500</b> N/mm <sup>2</sup>
Modulus of elasticity of reinforcement;	E <sub>s</sub> = <b>200000</b> N/mm <sup>2</sup>
Partial factor for reinforcing steel - Table 2.1N;	$\gamma_{\rm S} = 1.15$
Design yield strength of reinforcement;	f <sub>yd</sub> = f <sub>yk</sub> / γ <sub>S</sub> = <b>435</b> N/mm²

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	I	I	I	1		-1			
Cover to reinforcement									
Top face of base;		C <sub>bt</sub> = <b>35</b> mn	n						
Bottom face of base;		c <sub>bb</sub> = <b>35</b> mr	m						
Masonry details - Section 3.1									
Masonry type;		Autoclaved	l aerated concre	ete - Group 1					
Normalised mean compressive	strength;	f <sub>b</sub> = <b>10.4</b> N/	f <sub>b</sub> = <b>10.4</b> N/mm <sup>2</sup>						
Characteristic flexural strength -	$f_{xk} = 0 \text{ N/mm}^2$								
Initial shear strength - Table NA	A.5; $f_{vko} = 0.2 \text{ N/mm}^2$								
Mortar details - Section 3.2									
Mortar type;		General pu	General purpose - M12, prescribed mix						
Compressive strength of mortar	•	f <sub>m</sub> = <b>12</b> N/n	$f_m = 12 N/mm^2$						
Ultimate limit states - Table N	A.1								
Class of execution control;		1	1						
Category of manufacture contro	l;	2	2						
Partial factor for direct or flexura	I compression;	$\gamma_{Mc} = 2.3$	$\gamma_{Mc} = 2.3$						
Partial factor for flexural tension	,	γ <sub>Mt</sub> = <b>2.3</b>							
Partial factor for shear;		$\gamma_{Mv} = 2.0$							
Characteristic strengths of co	ncrete infill - Ta	able 3.2							
Concrete infill strength class;		C30/37							
Characteristic compressive stree	ngth;	$f_{ck,infill} = 25$	N/mm²						
Characteristic shear strength;		f <sub>cvk,infill</sub> = <b>0.45</b> N/mm <sup>2</sup>							
Design shear strength;		$f_{cvd,infill} = f_{cvl}$	$_{\rm k,infill}$ / $\gamma_{\rm Mv}$ = <b>0.22</b>	<b>5</b> N/mm²					



t = <b>210</b> mm
t <sub>shell</sub> = <b>44</b> mm
t <sub>web</sub> = <b>44</b> mm
JA 4 <sup>·</sup> K = 0.55

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Design flexural strength;		$f_{xd} = f_{xk} / \gamma_N$	$_{t} = 0 \text{ N/mm}^2$						
Height of masonry;		h <sub>wt</sub> = h <sub>stem</sub> :	= <b>1650</b> mm						
Compressive axial force combine	ation 0;	$F_x = \gamma_{Gf} \times \gamma_{f}$	$_{\text{stem}} \times h_{\text{wt}} \times t = 8.$	<b>7</b> kN/m					
Moment combination 0;		$M_x = \gamma_{Gf} \times \gamma_{ff}$	$h_{\rm stem} \times h_{\rm wt} \times t^2 / 2$	= <b>0.9</b> kNm/m					
Eccentricity of axial load;		e = max(at	os(t / 2 - M <sub>x</sub> / F <sub>x</sub> ),	, 0.05 × t) = <b>10.5</b>	mm				
Capacity reduction factor - exp.6	.4;	$\Phi = 1 - 2 \times$	e / t = <b>0.9</b>						
Design vertical resistance - exp.	6.2;	$N_{Rd} = \Phi \times 1$	:×f <sub>d</sub> = <b>470</b> kN/m	1					
Design vertical compressive stre	SS;	$\sigma_d = \min(F_2)$	$ < / t, 0.15 \times N_{Rd} / $	t) = <b>0.041</b> N/mm	2				
Apparent design flexural strengt	n - exp.6.16;	$f_{xd,app} = f_{xd}$	+ σ <sub>d</sub> = <b>0.041</b> N/n	nm²	_				
Characteristic shear strength - e	xp.3.5;	$f_{vk} = min(f_{vk})$	$c_{o}$ + 0.4 × $\sigma_{d}$ , 0.0	$65 \times f_b) = 0.217$ l	N/mm <sup>2</sup>				
Design shear strength;		$f_{vd} = f_{vk} / \gamma_{N}$	v = <b>0.108</b> N/mm	2					
Reinforced masonry members	subjected to b	ending, bendi	ng and axial loa	ading, or axial lo	oading - Sect	ion 6.6			
Design bending moment combin	ation 1;	M = <b>18.1</b> k	Nm/m						
Tension reinforcement provided;		12 dia.bars	; @ 220 c/c						
Area of tension reinforcement pr	ovided;	$A_{sr.prov} = \pi$	$\times \phi_{sr}^2 / (4 \times s_{sr}) =$	= <b>514</b> mm²/m					
Depth to tension reinforcement;		d = <b>140</b> mr	n	_					
Minimum area of reinforcement	· cl.8.2.3(1);	$A_{sr.min} = 0.0$	$A_{sr.min} = 0.0005 \times d = 70 \text{ mm}^2/\text{m}$						
Lever arm - exp.6.23; $z = d \times min(1 - 0.5 \times A_{sr,prov} \times f_{yd} / (d \times f_d), 0.95) = 95$				95) = <b>95</b> mm	5) = <b>95</b> mm				
Moment of resistance - exp.6.22	and exp.6.24a;	4a; $M_{Rd} = min(A_{sr.prov} \times f_{yd} \times z, 0.4 \times f_d \times d^2) = 19.5 \text{ kNm/m}$							
		$M / M_{Rd} = 0$	).929 Mamant of vo		de enviled de				
		PA55	- woment of re	Sistance exceed	is applied de	sign moment			
Reinforced masonry members	subjected to s	hear loading -	Section 6.7						
Design shear force	40.	V = 32.004	KN/m						
Design snear resistance - exp.6.	40;	$V_{Rd} = min(1)$	vd, Icvd,infill) $\times$ 0 =	13.133 KIN/III					
		FAIL - Design	shear resistan	ce is less than a	applied desig	n shear force			
Harizantal rainforcomant para	llal to face of a	om			ippilou uooig				
Minimum area of reinforcement		A _ 0.0	0005 × d – <b>70</b> m	m <sup>2</sup> /m					
Transverse reinforcement provid	ed:	Asx.req = 0.0	: @ 200 c/c	11 /111					
Area of transverse reinforcemen	t provided:		$A_{\text{exprov}} = \pi \times \phi_{\text{ex}}^2 / (4 \times s_{\text{ex}}) = 393 \text{ mm}^2/\text{m}$						
	PASS - Area of	reinforcemen	$r_{\text{ss,prov}} = \kappa + \varphi_{\text{ss}} + (+ + S_{\text{ss}}) = 000 \text{ mm /m}$						
Chack have design at too			. p. e		•••••••				
Depth of section:		h – <b>150</b> mr	n						
	•	n = <b>150</b> nn	11						
Rectangular section in flexure	- Section 6.1		,						
Design bending moment combin	ation I;	W = 22  kini	n/m						
Depth to tension reinforcement;		$\mathbf{d} = \mathbf{n} - \mathbf{C}_{bb}$	$d = h - c_{bb} - \phi_{bb} / 2 = 109 \text{ mm}$						
		$K = M / (d^2)$	× t <sub>ck</sub> ) = <b>0.062</b>						
		$\mathbf{K} = 0.207$	K' \ K - N	lo compression	reinforceme	nt is required			
l ever arm:		z = min(0 F)	K > K - K	3 ~ K) <sup>0.5</sup> 0 95) ~	d – <b>103</b> mm	ni is required			
Denth of neutral axis:		$x = 25 \vee 6$	(1 - z) = 16  mm	o∧n, ,0.30,×	a – 100 mm				
Area of tension reinforcement re	auired:	$A = 2.0 \times (0$	/ (fue × 7) – <b>493</b> I	mm²/m					
Tension reinforcement provided	44n 04,	12 dia haro	: (iyu ^ 2) = <b></b>						
Area of tension reinforcement or	ovided <sup>.</sup>	Abb prov = $\pi$	$\times \phi_{hh}^2 / (4 \times e_{hh})$	= <b>565</b> mm <sup>2</sup> /m					
Minimum area of reinforcement	exp.9 1N <sup>.</sup>	$A_{bb,min} = max(0.26 \times f_{ctm} / f_{vk} \ 0.0013) \times d = 164 \text{ mm}^2/\text{m}$							
Maximum area of reinforcement	- cl.9.2.1.1(3)	$A_{bb.max} = 0.04 \times h = 6000 \text{ mm}^2/\text{m}$							

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 $max(A_{bb.req}, A_{bb.min}) / A_{bb.prov} = 0.872$ 

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

Crack control - Section 7.3				
Limiting crack width;	w <sub>max</sub> = <b>0.3</b> mm			
Variable load factor - EN1990 – Table A1.1;	$\psi_2 = 0.6$			
Serviceability bending moment;	M <sub>sis</sub> = <b>16.2</b> kNm/m			
Tensile stress in reinforcement;	$\sigma_{s} = M_{sls} / (A_{bb,prov} \times z) = \textbf{278.4} \text{ N/mm}^{2}$			
Load duration;	Long term			
Load duration factor;	$k_{t} = 0.4$			
Effective area of concrete in tension;	$A_{c.eff} = min(2.5 \times (h - d), (h - x) / 3, h / 2) = 44746 mm^2/m$			
Mean value of concrete tensile strength;	$f_{ct.eff} = f_{ctm} = 2.9 \text{ N/mm}^2$			
Reinforcement ratio;	$\rho_{\text{p.eff}} = A_{\text{bb.prov}} \ / \ A_{\text{c.eff}} = \textbf{0.013}$			
Modular ratio;	$\alpha_{e} = E_{s} / E_{cm} = 6.091$			
Bond property coefficient;	k <sub>1</sub> = <b>0.8</b>			
Strain distribution coefficient;	k <sub>2</sub> = <b>0.5</b>			
	k <sub>3</sub> = <b>3.4</b>			
	k <sub>4</sub> = <b>0.425</b>			
Maximum crack spacing - exp.7.11;	$s_{r.max} = k_3 \times c_{bb} + k_1 \times k_2 \times k_4 \times \phi_{bb} \ / \ \rho_{p.eff} = \textbf{280} \ mm$			
Maximum crack width - exp.7.8;	$w_k = s_{r.max} \times max(\sigma_s - k_t \times (f_{ct.eff} \ / \ \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), \ 0.6 \times \sigma_s) \ / \ E_s$			
	w <sub>k</sub> = <b>0.252</b> mm			
	w <sub>k</sub> / w <sub>max</sub> = <b>0.84</b>			
	PASS - Maximum crack width is less than limiting crack width			
Rectangular section in shear - Section 6.2				
Design shear force;	V = <b>18.3</b> kN/m			
	$C_{Rd,c} = 0.18 / \gamma_{C} = 0.120$			
	k = min(1 + √(200 mm / d), 2) = <b>2.000</b>			
Longitudinal reinforcement ratio;	$\rho_{I} = min(A_{bb,prov} / d, 0.02) = 0.005$			
	$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5} = 0.542 \text{ N}/\text{mm}^2$			
Design shear resistance - exp.6.2a & 6.2b;	$V_{\text{Rd.c}} = max(C_{\text{Rd.c}} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{\text{min}}) \times d$			
	V <sub>Rd.c</sub> = <b>65.3</b> kN/m			
	V / V <sub>Rd.c</sub> = 0.280			
	PASS - Design shear resistance exceeds design shear force			
Secondary transverse reinforcement to base - S	ection 9.3			
Minimum area of reinforcement - cl.9.3.1.1(2);	$A_{\text{bx.req}} = 0.2 \times A_{\text{bb.prov}} = \textbf{113} \text{ mm}^2/\text{m}$			
Maximum spacing of reinforcement - cl.9.3.1.1(3);	s <sub>bx_max</sub> = <b>450</b> mm			
Transverse reinforcement provided;	10 dia.bars @ 200 c/c			
Area of transverse reinforcement provided;	$A_{bx,prov} = \pi \times \phi_{bx}^2 / (4 \times s_{bx}) = 393 \text{ mm}^2/\text{m}$			

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

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10 dia.bars @ 200 c/c transverse reinforcement in base

Reinforcement details