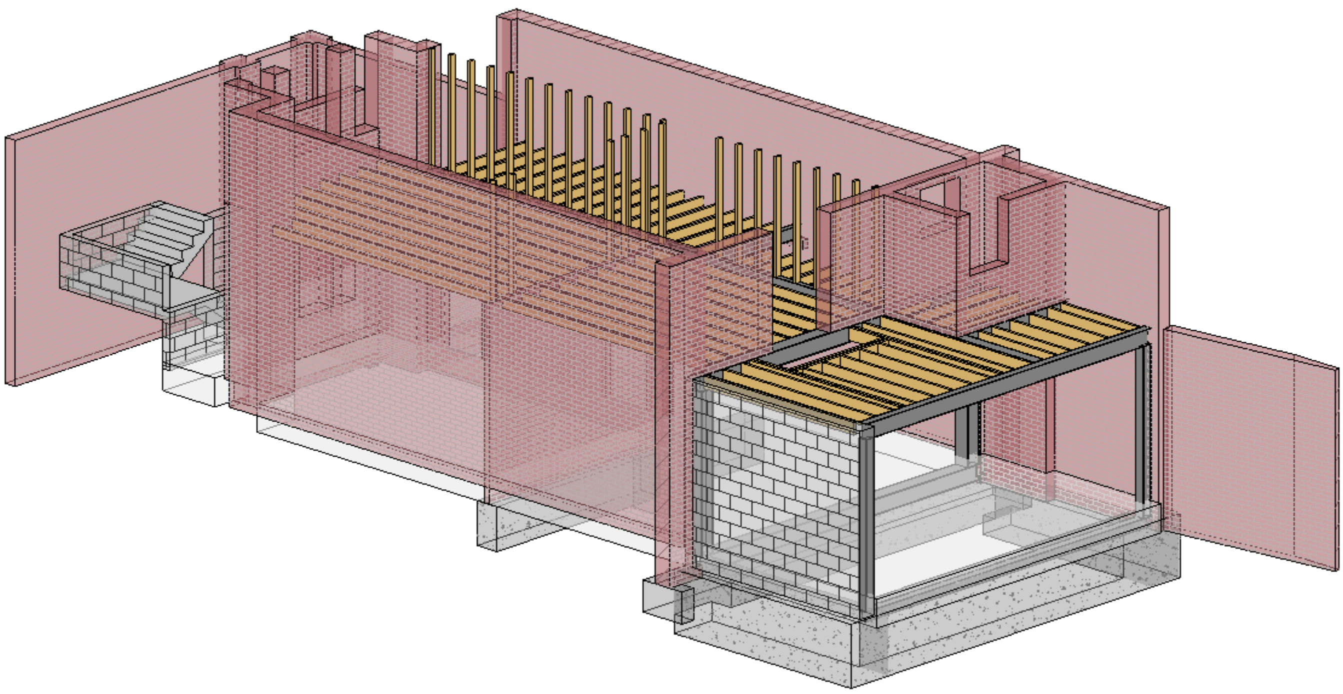
28 Burghley Road
Lower ground floor back extension
Internal alterations



Structural Design Calculation Package

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Project Number: SDS 187
Date: June 2017
Version: 1

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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.

Loads

GENERAL LOADS CONSIDERED ON THE STRUCTURAL DESIGN

Roof Loading - pitched roof

Roof slope; θ = **30.0** °

Dead load

Tiles; $Roof_{D1}$ = **0.45** kN/m^2

Battens; $Roof_{D2}$ = **0.05** kN/m^2

Felt; $Roof_{D3}$ = **0.05** kN/m^2

Rafters; $Roof_{D4}$ = **0.10** kN/m^2

Dead load on slope; $Roof_{DL\_sroof}$ = sum($Roof_{D1}$,$Roof_{D2}$,$Roof_{D3}$,$Roof_{D4}$) = **0.65** kN/m^2

Ceiling joists; $Roof_{D5}$ = **0.05** kN/m^2

Insulation; $Roof_{D6}$ = **0.05** kN/m^2

Plasterboard and skim; $Roof_{D7}$ = **0.14** kN/m^2

Services; $Roof_{D8}$ = **0.05** kN/m^2

Dead load on plan; $Roof_{DL\_proof}$ = sum($Roof_{D5}$,$Roof_{D6}$,$Roof_{D7}$,$Roof_{D8}$) = **0.29** kN/m^2

Total dead load on plan; $Roof_{DL}$ = $Roof_{DL\_sroof}$ / cos(θ) + $Roof_{DL\_proof}$= **1.04** kN/m^2

Imposed load

Sloped roof imposed load; $Roof_{IL}$ = **0.60** kN/m^2 ; on plan

Flat roof imposed load; $Roof_{IL}$ = **0.75** kN/m^2 ; on plan

Timber floor Loading

Dead load

Boards; $Floor_{grnd\_D1}$ = **0.15** kN/m^2

Joists; $Floor_{grnd\_D2}$ = **0.15** kN/m^2

Ceiling; $Floor_{grnd\_D3}$ = **0.20** kN/m^2

Total dead load; $Floor_{grnd\_DL}$ = sum($Floor_{load}$) = **0.50** kN/m^2

Imposed load

Imposed load; $Floor_{grnd\_I1}$ = **1.50** kN/m^2

Partitions; $Floor_{grnd\_I2}$ = **0.00** kN/m^2

Total imposed load; $Floor_{grnd\_IL}$ = sum($Floor_{grnd\_I1}$,$Floor_{grnd\_I2}$) = **1.50** kN/m^2

Wall Loading

Timber wall - Dead load

Timber; IW_{D1} = **0.20** kN/m^2

Plaster (2 sides, lath and plaster); IW_{D2} = **0.30** kN/m^2

Total dead load; IW_{DL} = sum(IW_{D1},IW_{D2}) = **0.50** 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_{D2} = **0.50** kN/m^2

Total dead load; IW_{DL} = sum(IW_{D4}, IW_{D5},IW_{D2}) = **IW_{D4} + 0.5** kN/m^2

Dense Blockwork wall - Dead load

t_b – Thickness of Solid Block; IW_{D4} = 20kN/m^3 x t_b = 2 kN/m^2

Plaster (2 sides, lath and plaster); IW_{D2} = **0.50** kN/m^2

Total dead load; IW_{DL} = sum(IW_{D4}, IW_{D5},IW_{D2}) = **IW_{D4} + 0.5** kN/m^2

Steelwork

BOX-FRAME UNDER ORIGINAL BACK WALL

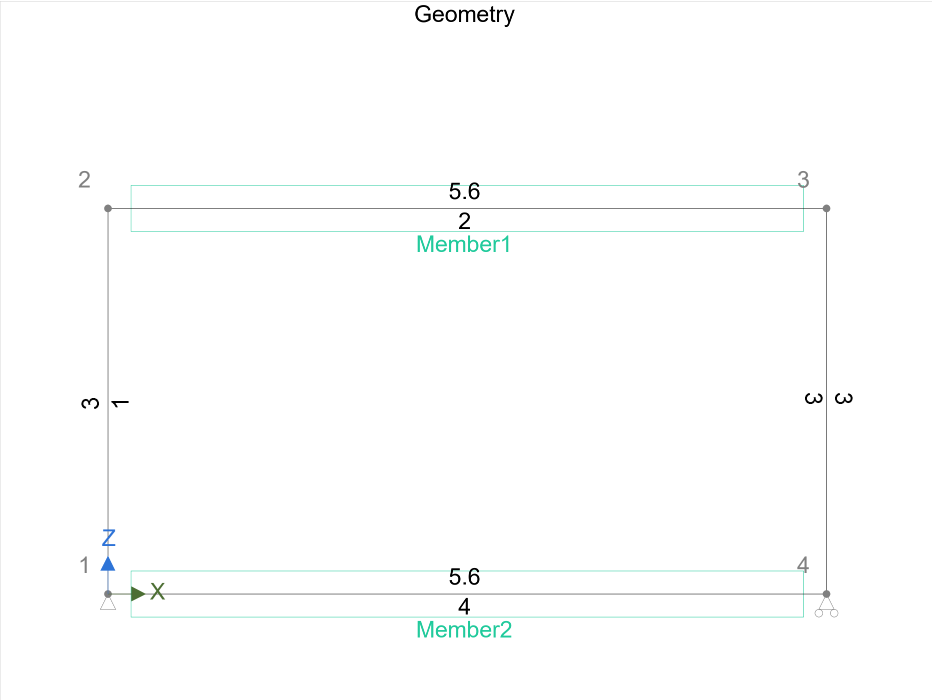
In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

Tedds calculation version 4.0.02

Analysis

Tedds calculation version 1.0.13

Geometry



Nodes

Node	Co-ordinates		Freedom			Coordinate system		Spring		
	X (m)	Z (m)	X	Z	Rot.	Name	Angle (°)	X (kN/m)	Z (kN/m)	Rot. 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 (kg/m^3)	Youngs Modulus kN/mm^2	Shear Modulus kN/mm^2	Thermal Coefficient °C^{-1}
Steel (EC3)	7850	210	80.8	0.000012

Sections

Name	Area	Moment of inertia		Shear area	
	(cm^2)	Major (cm^4)	Minor (cm^4)	A_y (cm^2)	A_z (cm^2)
UC 254x254x73	93	11407	3908	65	22
UC 254x254x89	113	14268	4857	80	27

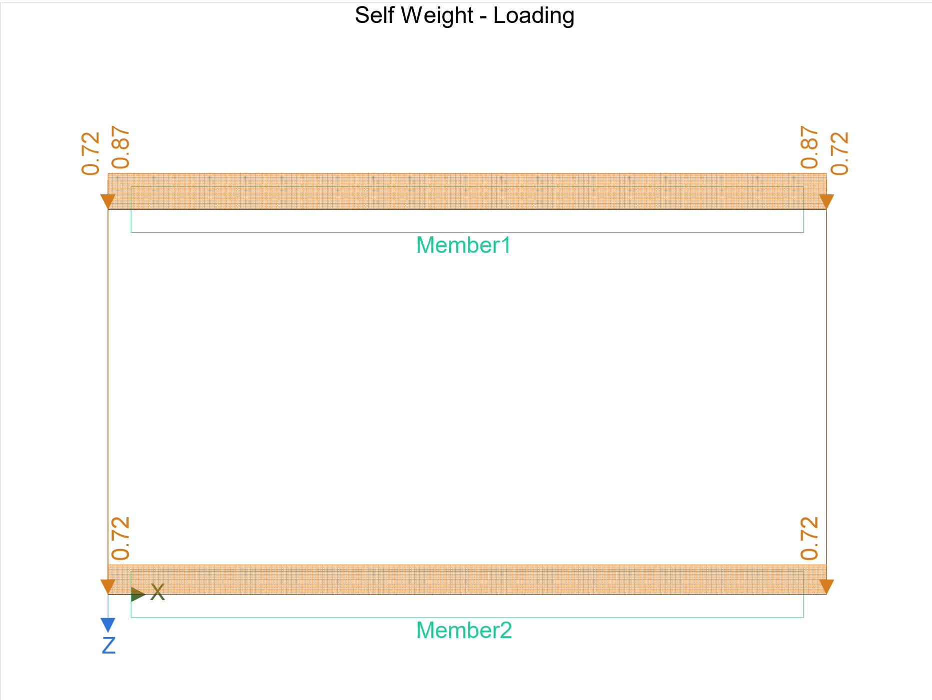
Elements

Element	Length (m)	Nodes		Section	Material	Releases			Rotated
		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

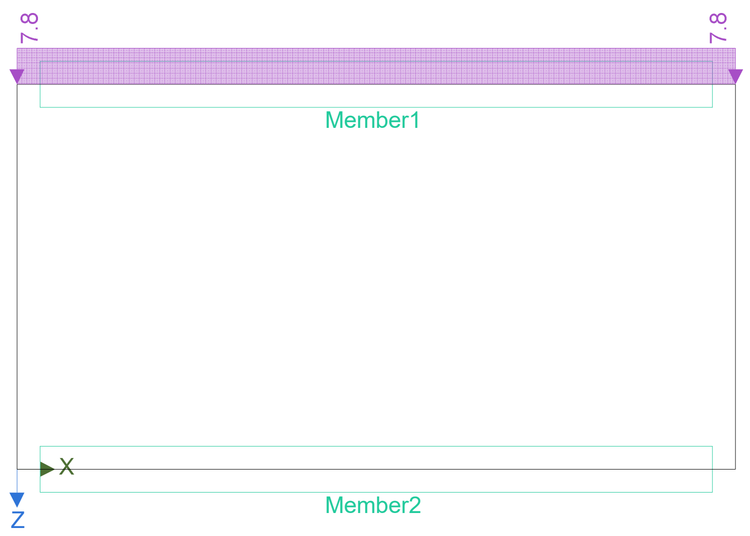
Loading



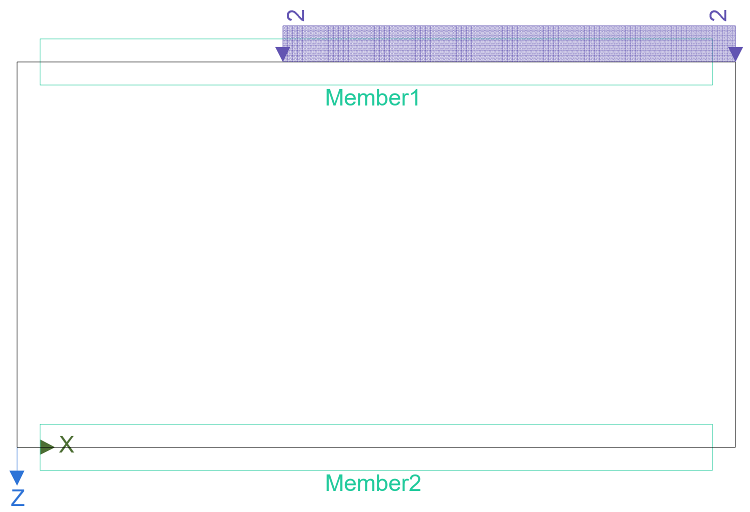
Solid 330mm brick wall above - 18kN/m3 x 0.33m x 9m - Loading

Floor dead load - 3 x 0.8kN/m2 x 2.15m - Loading

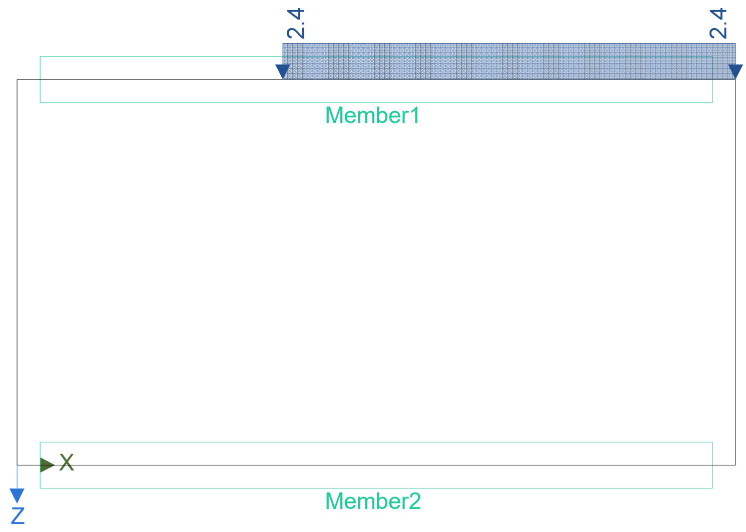
Floor live load - 3 x 1.5kN/m2 x 2.15m - Loading



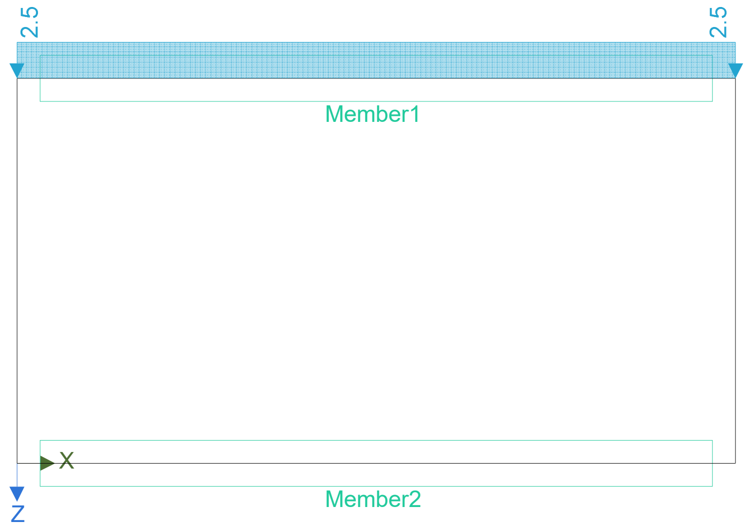
Terrace roof dead load - 1kN/m2 x 2m - Loading



Terrace roof live load - 1.5kN/m2 x 2m - Loading

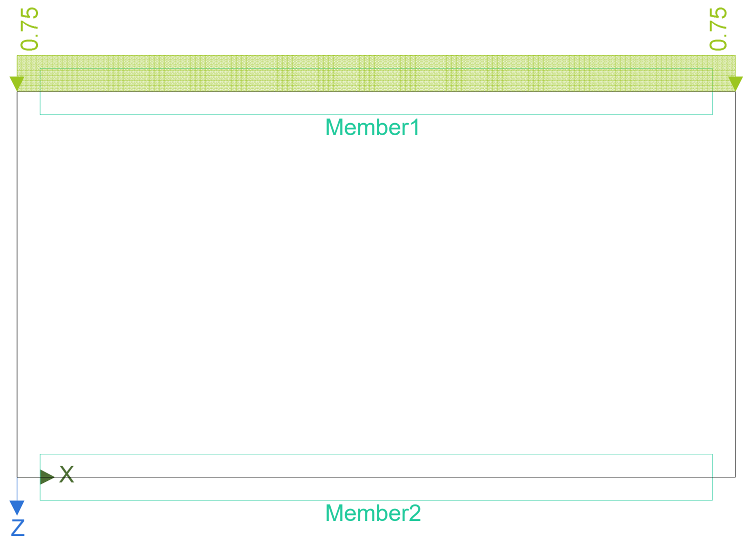


Roof dead load - 1kN/m2 x 2.5m - Loading



Roof live load - 0.6kN/m2 x 2.5m - Loading

Roof vertical wind load - 0.6kPa x sen(30) x 2.5m - Loading



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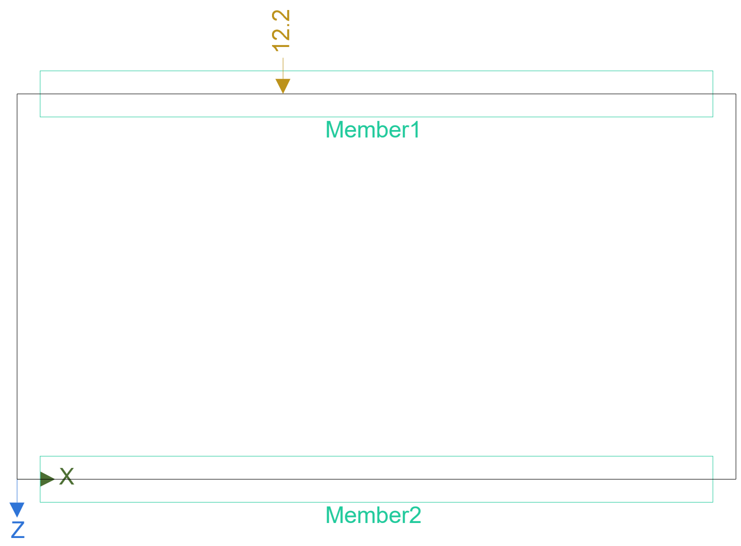
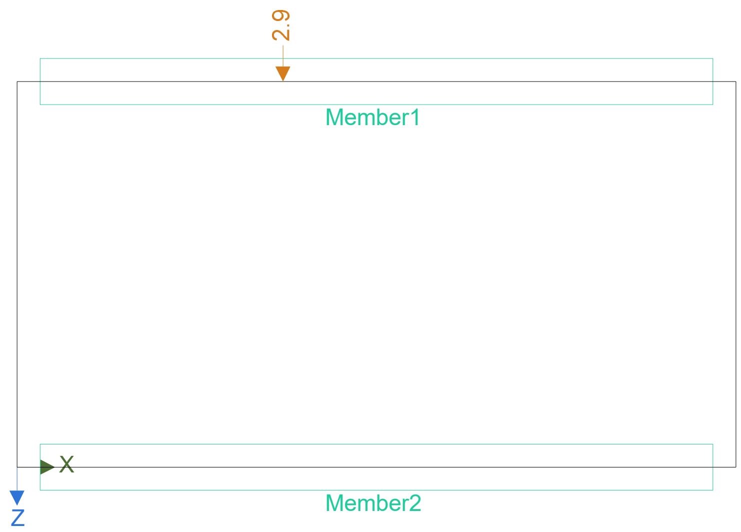
SB1 Dead Reaction - Loading



SB1 Live Reaction - Loading



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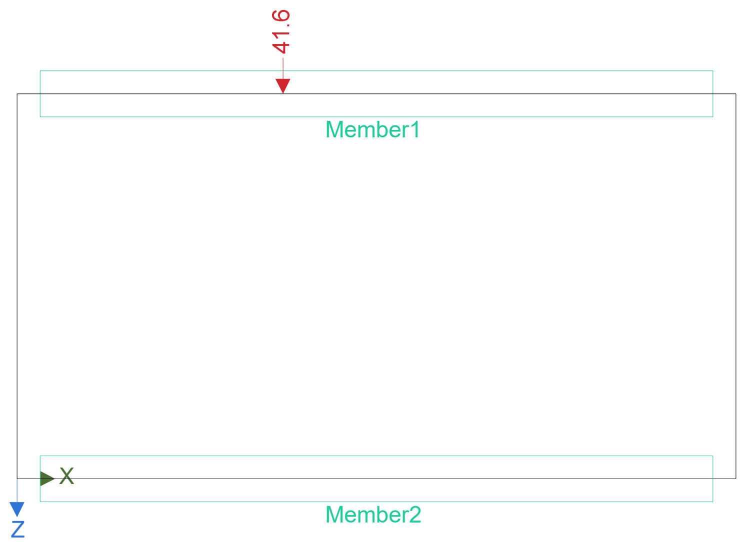
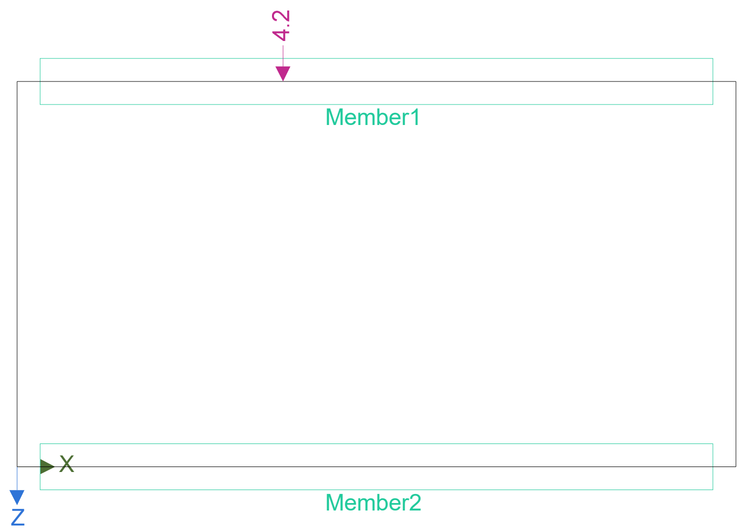
	Section Building Control Approval				Sheet no./rev. 14	
	Calc. by RC	Date 26/06/2017	Chk'd by MD	Date 26/06/2017	App'd by	Date

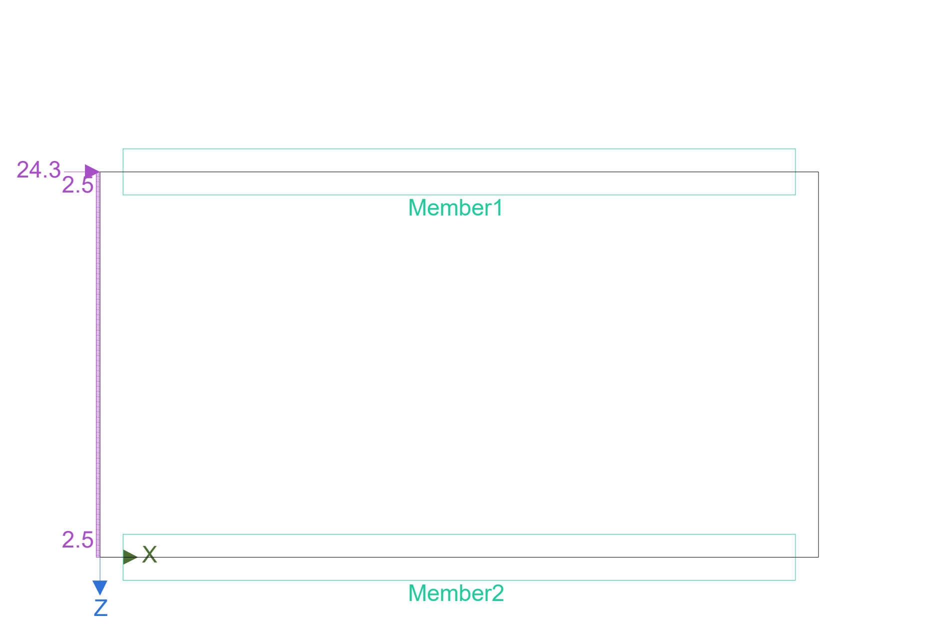
SB3 Dead Reaction - Loading



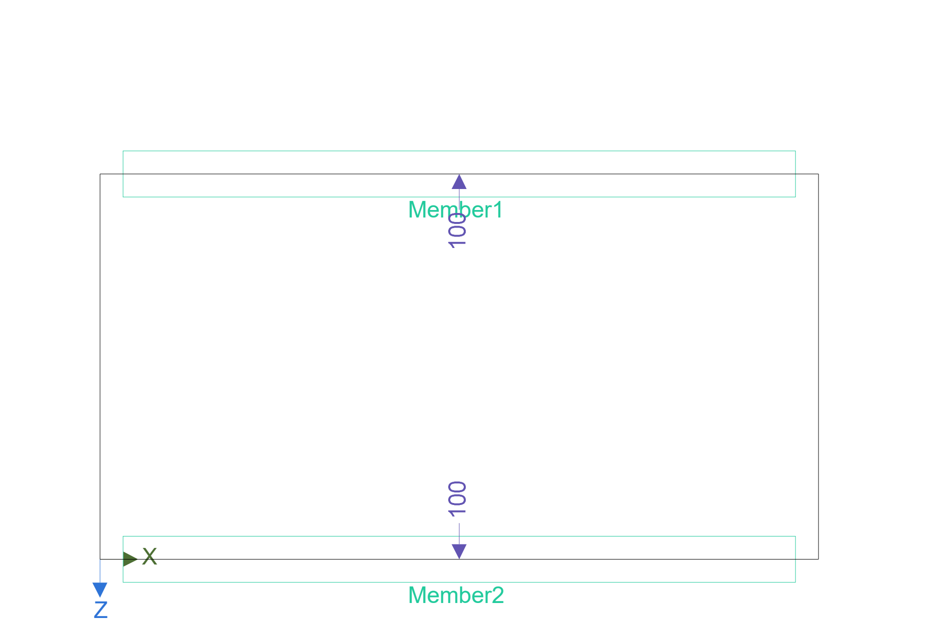
SB3 Live Reaction - Loading



Wind - Loading



Pre-Deflection - Loading



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	Type	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)

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 + 1.0ψ_2Q (Quasi)

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 + 1.5ψ_0S (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)

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: $1.35G + 1.5\psi_0 Q + 1.5S$ (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: $1.35G + 1.5Q + 1.5\psi_0 S + 1.5\psi_0 W$ (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
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 + 1.5ψ_0Q + 1.5S + 1.5ψ_0W (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
Wind	0.75

Load combination: 1.0G + 1.0ψ_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 + 1.5ψ_0Q + 1.5ψ_0S + 1.5W (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
Wind	1.5

Load combination: 1.0G + 1.0ψ_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)

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 + 1.5ψ_0Q + 1.5ψ_0RQ (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: 1.35ξG + 1.5Q + 1.5RQ (Strength)

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

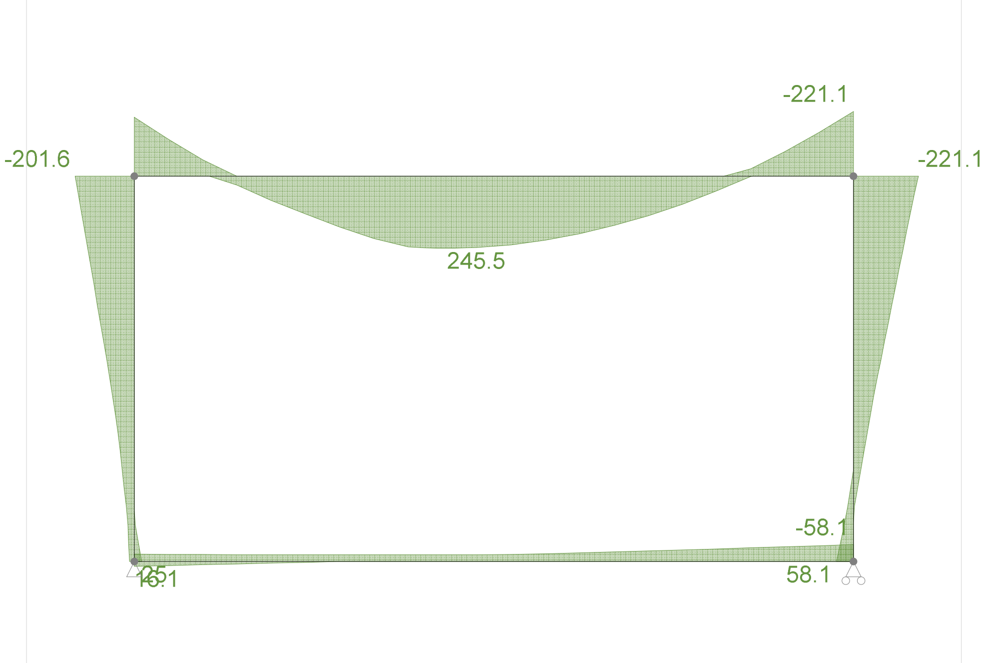
Element	Load case	Position		Load	Orientation
		Type	Start	(kN)	
2	SB1 Dead Reaction	Ratio	0.37	12.2	GlobalZ
2	SB1 Live Reaction	Ratio	0.37	2.9	GlobalZ
2	SB3 Dead Reaction	Ratio	0.37	41.6	GlobalZ
2	SB3 Live Reaction	Ratio	0.37	4.2	GlobalZ
1	Wind	Ratio	1	24.3	GlobalX
2	Pre-Deflection	Ratio	0.5	-100	GlobalZ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

Element UDL loads

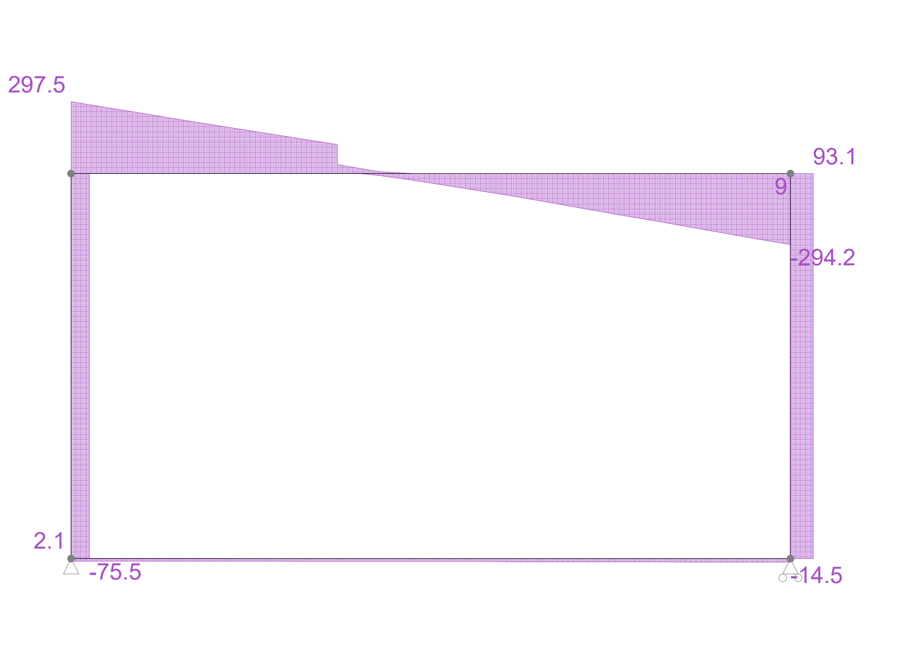
Element	Load case	Position			Load	Orientation
		Type	Start	End	(kN/m)	
2	Solid 330mm brick wall above - 18kN/m3 x 0.33m x 9m	Ratio	0	1	45	GlobalZ
2	Floor dead load - 3 x 0.8kN/m2 x 2.15m	Ratio	0	1	5.2	GlobalZ
2	Floor live load - 3 x 1.5kN/m2 x 2.15m	Ratio	0	1	7.8	GlobalZ
2	Terrace roof dead load - 1kN/m2 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

Results

Forces

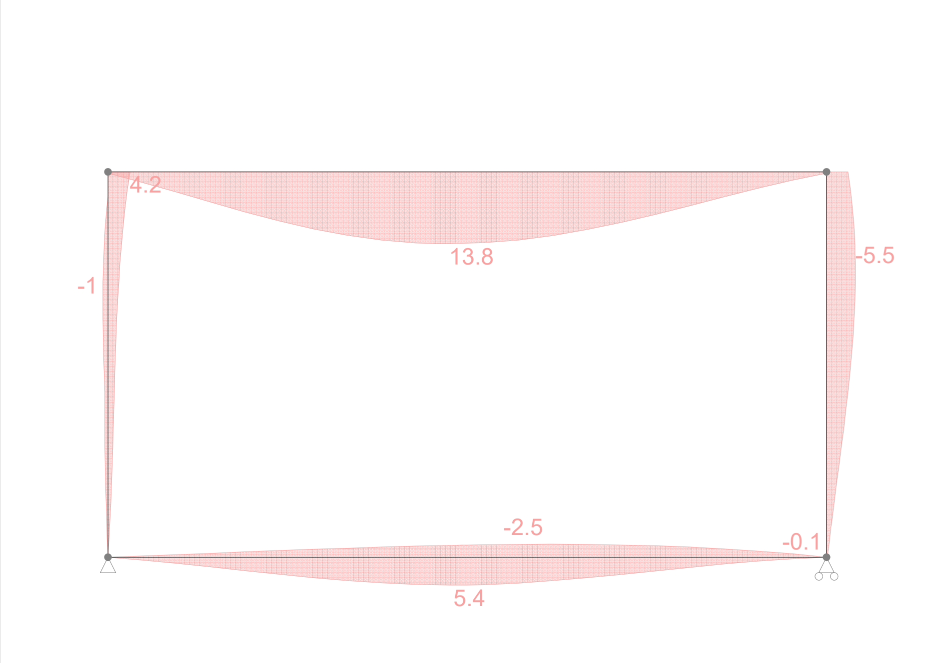


Strength combinations - Moment envelope (kNm)



Strength combinations - Shear envelope (kN)

Service combinations - Deflection envelope (mm)



;

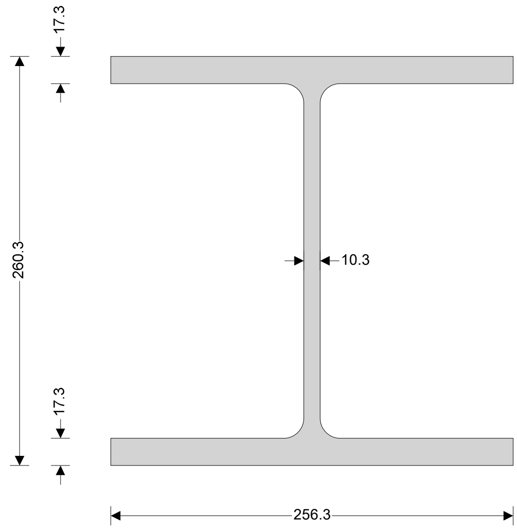
Partial factors - Section 6.1

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

<u>Member1 - Span 1</u>

Section details

Section type;	UC 254x254x89 (BS4-1)
Steel grade - EN 10025-2:2004;	S275
Nominal thickness of element;	t_{nom} = max(t_f, t_w) = **17.3** mm
Nominal yield strength;	f_y = **265** N/mm^2
Nominal ultimate tensile strength;	f_u = **410** N/mm^2
Modulus of elasticity;	E = **210000** N/mm^2



UC 254x254x89 (BS4-1)
Section depth, h, 260.3 mm
Section breadth, b, 256.3 mm
Mass of section, Mass, 88.9 kg/m
Flange thickness, t_f, 17.3 mm
Web thickness, t_w, 10.3 mm
Root radius, r, 12.7 mm
Area of section, A, 11331 mm^2
Radius of gyration about y-axis, i_y, 112 mm
Radius of gyration about z-axis, i_z, 65 mm
Elastic section modulus about y-axis, $W_{el.y}$, 1096254 mm^3
Elastic section modulus about z-axis, $W_{el.z}$, 379046 mm^3
Plastic section modulus about y-axis, $W_{pl.y}$, 1223863 mm^3
Plastic section modulus about z-axis, $W_{pl.z}$, 575307 mm^3
Second moment of area about y-axis, I_y, 142677465 mm^4
Second moment of area about z-axis, I_z, 48574741 mm^4

Lateral restraint

Both flanges have lateral restraint at supports only

Consider Combination 1 - 1.35G + 1.5Q + 1.5RQ (Strength)

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]} = \mathbf{0.94}$

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

Width of section; $c = d = \mathbf{200.3}$ mm

$\alpha = \min([h / 2 + N_{Ed} / (2 \times t_w \times f_y) - (t_f + r)] / c, 1) = \mathbf{0.580}$

$c / t_w = 19.4 = 20.7 \times \varepsilon <= 396 \times \varepsilon / (13 \times \alpha - 1)$; Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section; $c = (b - t_w - 2 \times r) / 2 = \mathbf{110.3}$ mm

$c / t_f = 6.4 = 6.8 \times \varepsilon <= 9 \times \varepsilon$; Class 1

Section is class 1

Check compression - Section 6.2.4

Design compression force; $N_{Ed} = \mathbf{88}$ kN

Design resistance of section - eq 6.10; $N_{c,Rd} = N_{pl,Rd} = (A - \rho_{y,v} \times h \times t_w) \times f_y / \gamma_{M0} = \mathbf{2953.8}$ kN

$N_{Ed} / N_{c,Rd} = \mathbf{0.03}$

PASS - Design compression resistance exceeds design compression

Slenderness ratio for y-y axis flexural buckling - Section 6.3.1.3

Critical buckling length; $L_{cr,y} = L_{m1\_s1} = \mathbf{5600}$ mm

Critical buckling force; $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2 = \mathbf{9429.7}$ kN

Slenderness ratio for buckling - eq 6.50; $\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})} = \mathbf{0.564}$

Check y-y axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2; b

Imperfection factor - Table 6.1; $\alpha_y = \mathbf{0.34}$

Buckling reduction determination factor; $\phi_y = 0.5 \times (1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2) = \mathbf{0.721}$

Buckling reduction factor - eq 6.49; $\chi_y = \min(1 / (\phi_y + \sqrt{(\phi_y^2 - \bar{\lambda}_y^2)}), 1) = \mathbf{0.855}$

Design buckling resistance - eq 6.47; $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1} = \mathbf{2566.1}$ kN

$N_{Ed} / N_{b,y,Rd} = \mathbf{0.034}$

PASS - Design buckling resistance exceeds design compression

Slenderness ratio for z-z axis flexural buckling - Section 6.3.1.3

Critical buckling length; $L_{cr,z} = L_{m1\_s1\_seg1} = \mathbf{5600}$ mm

Critical buckling force; $N_{cr,z} = \pi^2 \times E \times I_z / L_{cr,z}^2 = \mathbf{3210.4}$ kN

Slenderness ratio for buckling - eq 6.50; $\bar{\lambda}_z = \sqrt{(A \times f_y / N_{cr,z})} = \mathbf{0.967}$

Check z-z axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2; c

Imperfection factor - Table 6.1; $\alpha_z = \mathbf{0.49}$

Buckling reduction determination factor; $\phi_z = 0.5 \times (1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2) = \mathbf{1.156}$

Buckling reduction factor - eq 6.49; $\chi_z = \min(1 / (\phi_z + \sqrt{(\phi_z^2 - \bar{\lambda}_z^2)}), 1) = \mathbf{0.559}$

Design buckling resistance - eq 6.47; $N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1} = \mathbf{1679.2}$ kN

$N_{Ed} / N_{b,z,Rd} = \mathbf{0.052}$

PASS - Design buckling resistance exceeds design compression

Check torsional and torsional-flexural buckling - Section 6.3.1.4

Torsional buckling length; $L_{cr,T} = L_{m1\_s1\_seg1\_R} = \mathbf{5600}$ mm

Distance from shear centre to centroid in y axis; $y_0 = \mathbf{0.0}$ mm

Distance from shear centre to centroid in z axis; z_0 = **0.0** mm

Radius of gyration; $i_0 = \sqrt{(i_y^2 + i_z^2)}$ = **129.9** mm

Elastic critical torsional buckling force; $N_{cr,T} = 1 / i_0^2 \times (G \times I_t + \pi^2 \times E \times I_w / L_{cr,T}^2)$ = **7705.2** kN

Torsion factor; $\beta_T = 1 - (y_0 / i_0)^2$ = **1**

Elastic critical torsional-flexural buckling force

$N_{cr,TF} = N_{cr,y} / (2 \times \beta_T) \times [1 + N_{cr,T} / N_{cr,y} - \sqrt{[(1 - N_{cr,T} / N_{cr,y})^2 + 4 \times (y_0 / i_0)^2 \times N_{cr,T} / N_{cr,y}]}]$ = **7705.2** kN

Elastic critical buckling force; $N_{cr} = \min(N_{cr,T}, N_{cr,TF})$ = **7705.2** kN

Slenderness ratio for torsional buckling - eq 6.52; $\bar{\lambda}_T = \sqrt{[A \times f_y / N_{cr}]}$ = **0.624**

Design resistance for torsional and torsional-flexural buckling - Section 6.3.1.1

Buckling curve - Table 6.2; c

Imperfection factor - Table 6.1; α_T = **0.49**

Buckling reduction determination factor; $\phi_T = 0.5 \times (1 + \alpha_T \times (\bar{\lambda}_T - 0.2) + \bar{\lambda}_T^2)$ = **0.799**

Buckling reduction factor - eq 6.49; $\chi_T = \min(1 / (\phi_T + \sqrt{(\phi_T^2 - \bar{\lambda}_T^2)}), 1)$ = **0.771**

Design buckling resistance - eq 6.47; $N_{b,T,Rd} = \chi_T \times A \times f_y / \gamma_{M1}$ = **2314.9** kN

$N_{Ed} / N_{b,T,Rd}$ = **0.038**

PASS - Design buckling resistance exceeds design compression

<u>Check design at start of span</u>

Check shear - Section 6.2.6

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

$h_w / t_w = 21.9 = 23.3 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force; $V_{y,Ed}$ = **297.5** kN

Shear area - cl 6.2.6(3); $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **3081** mm^2

Design shear resistance - cl 6.2.6(2); $V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **471.4** kN

$V_{y,Ed} / V_{c,y,Rd}$ = **0.631**

PASS - Design shear resistance exceeds design shear force

Shear reduction factor - cl.6.2.8(3); $\rho_{y,v} = (2 \times V_{y,Ed} / V_{pl,y,Rd} - 1)^2$ = **0.069**

Check bending moment - Section 6.2.5 and 6.2.8

Design bending moment; $M_{y,Ed}$ = **191** kNm

Design bending resistance moment - eq 6.13; $M_{c,y,Rd} = M_{pl,y,Rd} = (W_{pl.y} - \rho_{y,v} \times t_w \times h^2 / 4) \times f_y / \gamma_{M0}$ = **321.1** kNm

$M_{y,Ed} / M_{c,y,Rd}$ = **0.595**

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6; k_c = **0.897**

$C_1 = 1 / k_c^2$ = **1.244**

Poissons ratio; ν = **0.3**

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

Unrestrained effective length; $L = 1.0 \times L_{m1\_s1\_seg1\_B}$ = **5600** mm

Elastic critical buckling moment; $M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **803.6** kNm

Slenderness ratio for lateral torsional buckling; $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **0.635**

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

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

Check buckling resistance - Section 6.3.2.1

Buckling curve - Table 6.5; b

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

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

LTB reduction determination factor; $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **0.691**

LTB reduction factor - eq 6.57; $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.901**

Modification factor; $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.951**

Modified LTB reduction factor - eq 6.58; $\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.947**

Design buckling resistance moment - eq 6.55; $M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1}$ = **307.2** kNm

$M_{y,Ed} / M_{b,y,Rd}$ = **0.622**

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; $N_{y,lim} = \min(0.25 \times N_{pl,Rd}, 0.5 \times h_w \times t_w \times (1 - \rho_{y,v}) \times f_y / \gamma_{M0})$ = **286.8** kN

$N_{Ed} / N_{y,lim}$ = **0.307**

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_y = -191 kNm / -219.3 kNm = **0.871**

α_y = -219.3 kNm / 239.5 kNm = **-0.916**

$C_{my} = 0.95 + 0.05 \times \alpha_y$ = **0.904**

; ψ_{LT} = -191 kNm / -219.3 kNm = **0.871**

α_{LT} = -219.3 kNm / 239.5 kNm = **-0.916**

$C_{mLT} = 0.95 + 0.05 \times \alpha_{LT}$ = **0.904**

Interaction factors k_{ij} for members susceptible to torsional deformations - Table B.2

Characteristic moment resistance; $M_{y,Rk} = W_{pl.y} \times f_y$ = **324.3** kNm

Characteristic moment resistance; $M_{z,Rk} = W_{pl.z} \times f_y$ = **152.5** kNm

Characteristic resistance to normal force; $N_{Rk} = A \times f_y$ = **3002.8** kN

Interaction factors; $k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}))$ = **0.916**

$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1})$ = **0.992**

Interaction formulae - eq 6.61 & eq 6.62; $N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.573**

$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.637**

PASS - Combined bending and compression checks are satisfied

Check design 2441 mm along span

Check bending moment - Section 6.2.5 and 6.2.8

Design bending moment; $M_{y,Ed}$ = **245.5** kNm

Design bending resistance moment - eq 6.13; $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **324.3** kNm

$M_{y,Ed} / M_{c,y,Rd}$ = **0.757**

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6; k_c = **0.897**

$C_1 = 1 / k_c^2$ = **1.244**

Poissons ratio; ν = **0.3**

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

Unrestrained effective length; $L = 1.0 \times L_{m1\_s1\_seg1\_T}$ = **5600** mm

Elastic critical buckling moment; $M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **803.6** kNm

Slenderness ratio for lateral torsional buckling; $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **0.635**

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

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

Check buckling resistance - Section 6.3.2.1

Buckling curve - Table 6.5; b

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

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

LTB reduction determination factor; $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **0.691**

LTB reduction factor - eq 6.57; $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{}(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)], 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.901**

Modification factor; $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.951**

Modified LTB reduction factor - eq 6.58; $\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.947**

Design buckling resistance moment - eq 6.55; $M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1}$ = **307.2** kNm

$M_{y,Ed} / M_{b,y,Rd}$ = **0.799**

PASS - Design buckling resistance moment exceeds design bending moment

Check bending and axial force - Section 6.2.9

Check combined bending and compression - Section 6.3.3

Equivalent uniform moment factors - Table B.3; ψ_y = -191 kNm / -219.3 kNm = **0.871**

α_y = -219.3 kNm / 239.5 kNm = **-0.916**

$C_{my} = 0.95 + 0.05 \times \alpha_y$ = **0.904**

; ψ_{LT} = -191 kNm / -219.3 kNm = **0.871**

α_{LT} = -219.3 kNm / 239.5 kNm = **-0.916**

$C_{mLT} = 0.95 + 0.05 \times \alpha_{LT}$ = **0.904**

Interaction factors k_{ij} for members susceptible to torsional deformations - Table B.2

Characteristic moment resistance; $M_{y,Rk} = W_{pl.y} \times f_y$ = **324.3** kNm

Characteristic moment resistance; $M_{z,Rk} = W_{pl.z} \times f_y$ = **152.5** kNm

Characteristic resistance to normal force; $N_{Rk} = A \times f_y$ = **3002.8** kN

Interaction factors; $k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}))$ = **0.916**

$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1})$ = **0.992**

Interaction formulae - eq 6.61 & eq 6.62; $N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.727**

$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.803**

PASS - Combined bending and compression checks are satisfied

<u>Check design at end of span</u>

Check shear - Section 6.2.6

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

$h_w / t_w = 21.9 = 23.3 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force; $V_{y,Ed}$ = **294.2** kN

Shear area - cl 6.2.6(3); $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **3081** mm^2

Design shear resistance - cl 6.2.6(2); $V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{}(3)) / \gamma_{M0}$ = **471.4** kN

$V_{y,Ed} / V_{c,y,Rd}$ = **0.624**

PASS - Design shear resistance exceeds design shear force

Shear reduction factor - cl.6.2.8(3); $\rho_{y,v} = (2 \times V_{y,Ed} / V_{pl,y,Rd} - 1)^2$ = **0.062**

Check bending moment - Section 6.2.5 and 6.2.8

Design bending moment; $M_{y,Ed}$ = **219.3** kNm

Design bending resistance moment - eq 6.13; $M_{c,y,Rd} = M_{pl,y,Rd} = (W_{pl.y} - \rho_{y,v} \times t_w \times h^2 / 4) \times f_y / \gamma_{M0}$ = **321.5** kNm

$M_{y,Ed} / M_{c,y,Rd}$ = **0.682**

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6; k_c = **0.897**

$C_1 = 1 / k_c^2$ = **1.244**

Poissons ratio; ν = **0.3**

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

Unrestrained effective length; $L = 1.0 \times L_{m1\_s1\_seg1\_B}$ = **5600** mm

Elastic critical buckling moment; $M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **803.6** kNm

Slenderness ratio for lateral torsional buckling; $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **0.635**

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

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

Check buckling resistance - Section 6.3.2.1

Buckling curve - Table 6.5; b

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

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

LTB reduction determination factor; $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **0.691**

LTB reduction factor - eq 6.57; $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.901**

Modification factor; $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.951**

Modified LTB reduction factor - eq 6.58; $\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.947**

Design buckling resistance moment - eq 6.55; $M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1}$ = **307.2** kNm

$M_{y,Ed} / M_{b,y,Rd}$ = **0.714**

PASS - Design buckling resistance moment exceeds design bending moment

Check bending and axial force - Section 6.2.9

Check combined bending and compression - Section 6.3.3

Equivalent uniform moment factors - Table B.3; ψ_y = -191 kNm / -219.3 kNm = **0.871**

α_y = -219.3 kNm / 239.5 kNm = **-0.916**

$C_{my} = 0.95 + 0.05 \times \alpha_y$ = **0.904**

; ψ_{LT} = -191 kNm / -219.3 kNm = **0.871**

α_{LT} = -219.3 kNm / 239.5 kNm = **-0.916**

$C_{mLT} = 0.95 + 0.05 \times \alpha_{LT}$ = **0.904**

Interaction factors k_{ij} for members susceptible to torsional deformations - Table B.2

Characteristic moment resistance; $M_{y,Rk} = W_{pl.y} \times f_y$ = **324.3** kNm

Characteristic moment resistance; $M_{z,Rk} = W_{pl.z} \times f_y$ = **152.5** kNm

Characteristic resistance to normal force; $N_{Rk} = A \times f_y$ = **3002.8** kN

Interaction factors; $k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}))$ = **0.916**

$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1})$ = **0.992**

Interaction formulae - eq 6.61 & eq 6.62; $N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.653**

$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.723**

PASS - Combined bending and compression checks are satisfied

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; δ_y = **9.4** mm

Allowable deflection; $\delta_{y,Allowable}$ = Min(L_{m1_s1} / 500, 10 mm) = **10** mm

δ_y / $\delta_{y,Allowable}$ = **0.936**

PASS - Allowable deflection exceeds design deflection

BOX-FRAME ON THE ALLIGMENT OF THE NEW REAR

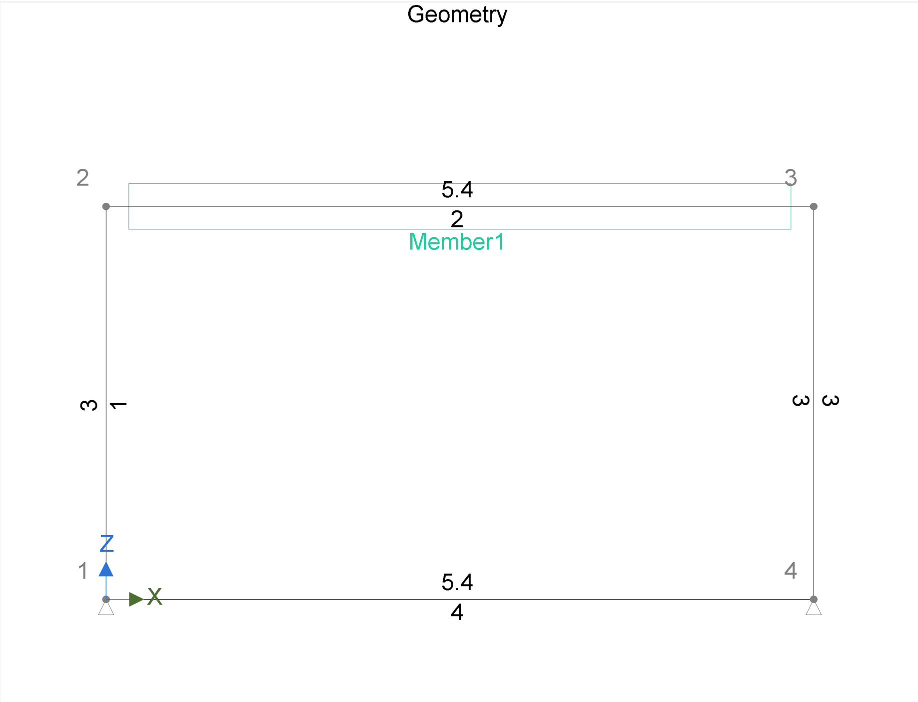
In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

Tedds calculation version 4.0.02

Analysis

Tedds calculation version 1.0.13

Geometry



Nodes

Node	Co-ordinates X (m)	Co-ordinates Z (m)	Freedom X	Freedom Z	Freedom Rot.	Coordinate system Name	Coordinate system Angle (°)	Spring X (kN/m)	Spring Z (kN/m)	Spring Rot. 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 (kg/m^3)	Youngs Modulus kN/mm^2	Shear Modulus kN/mm^2	Thermal Coefficient °C^{-1}
Steel (EC3)	7850	210	80.8	0.000012

Sections

Name	Area (cm^2)	Moment of inertia Major (cm^4)	Moment of inertia Minor (cm^4)	Shear area A_y (cm^2)	Shear area A_z (cm^2)
UC 152x152x30	38	1748	560	26	10
UB 254x146x37	47	5537	571	29	16

Elements

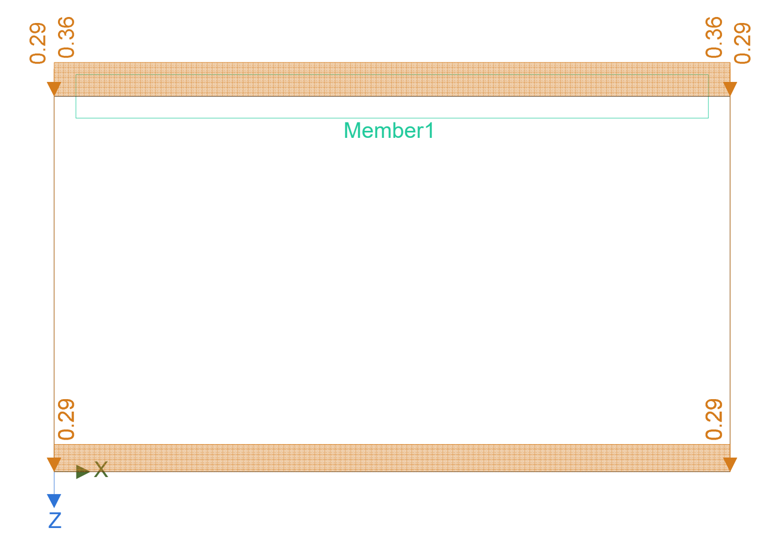
Element	Length (m)	Nodes		Section	Material	Releases			Rotated
		Start	End			Start moment	End moment	Axial	
1	3	1	2	UC 152x152x30	Steel (EC3)	Fixed	Fixed	Fixed	
2	5.4	2	3	UB 254x146x37	Steel (EC3)	Fixed	Fixed	Fixed	
3	3	3	4	UC 152x152x30	Steel (EC3)	Fixed	Fixed	Fixed	
4	5.4	1	4	UC 152x152x30	Steel (EC3)	Fixed	Fixed	Fixed	

Members

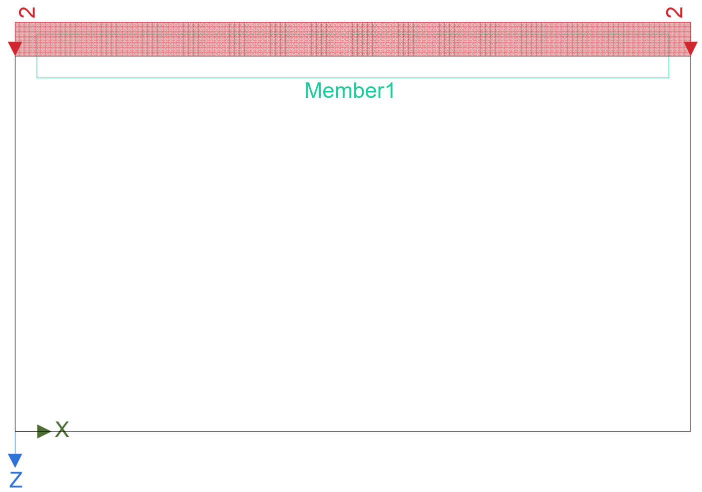
Name	Elements	
	Start	End
Member1	2	2

Loading

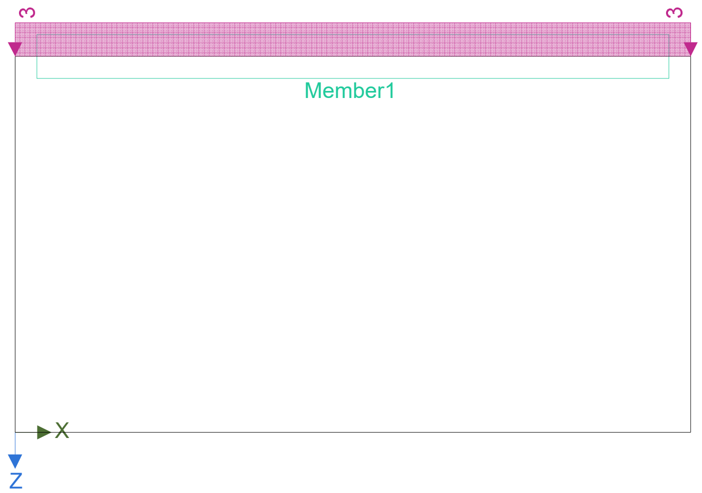
Self Weight - Loading



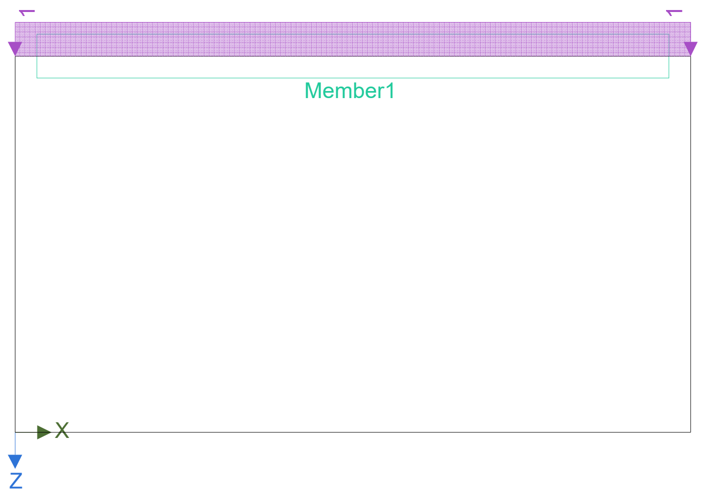
Terrace roof dead load - 1kN/m2 x 2m - Loading



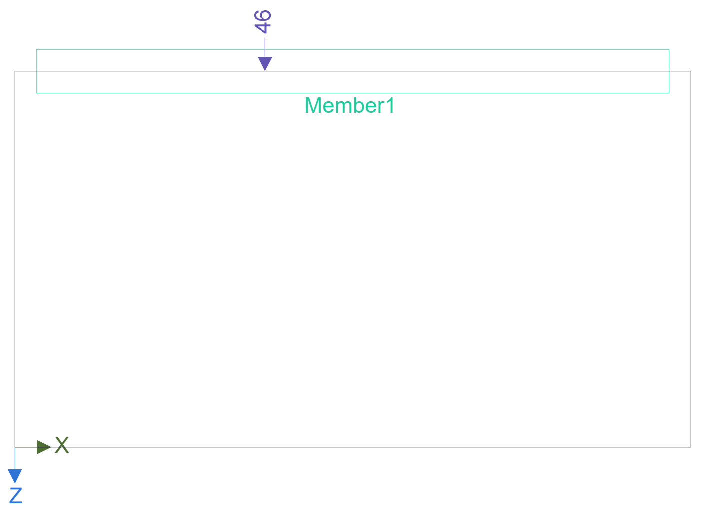
Terrace roof live load - 1.5kN/m2 x 2m - Loading



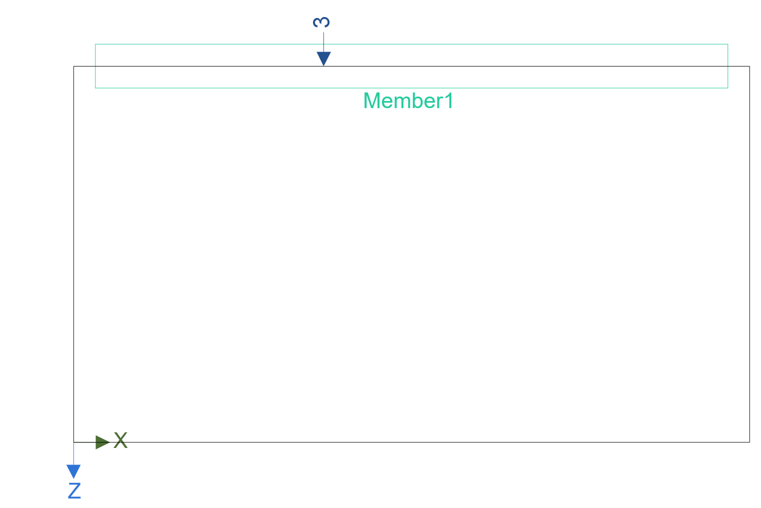
Glass balustrade - 25kN/m2 x 0.022m x 1.5m - Loading



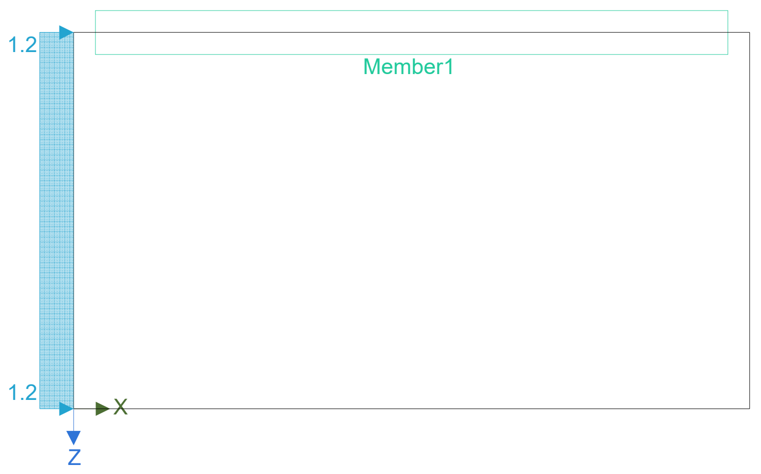
SB3 Dead Reaction - Loading



SB3 Live Reaction - Loading



Wind - Loading



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	Type	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)

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_2Q (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_0S (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 + 1.5ψ_0Q + 1.5S (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

Load combination: 1.35G + 1.5Q + 1.5ψ_0S + 1.5ψ_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	1.5
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 + 1.5ψ_0Q + 1.5S + 1.5ψ_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 + 1.0ψ_0Q + 1.0S + 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	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 + 1.5ψ₀Q + 1.5ψ₀S + 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 + 1.0ψ₀Q + 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)

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

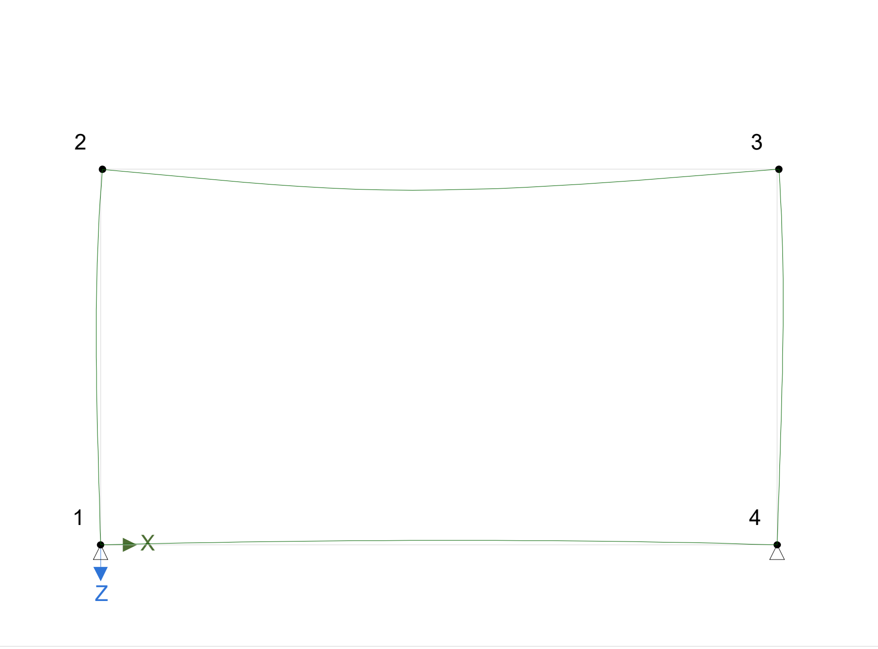
Element UDL loads

Element	Load case	Position Type	Position Start	Position End	Load (kN/m)	Orientation
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

<u>Results</u>

Total deflection

1.35G + 1.5Q + 1.5RQ (Strength) - Total deflection @ 10x

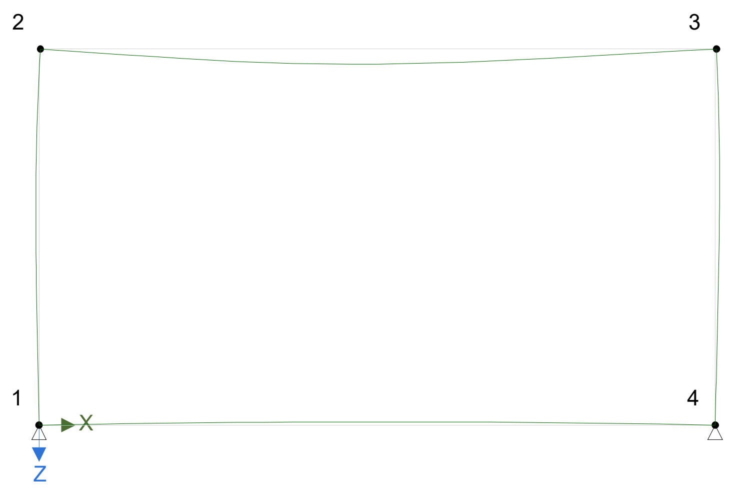


Member results

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

Member	Deflection				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

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

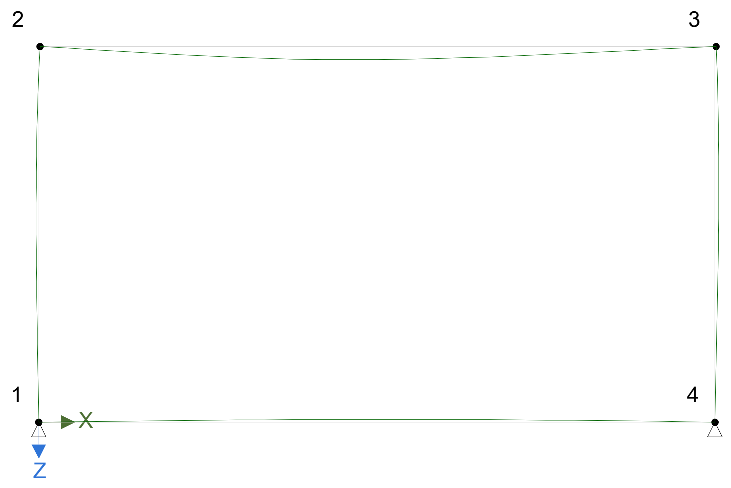


Member results

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

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

1.0G + 1.0ψ_2Q (Quasi) - Total deflection @ 10x

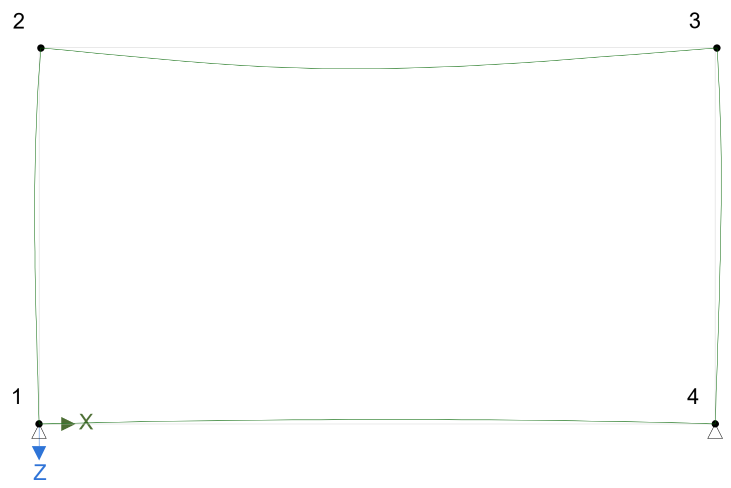


Member results

Load combination: 1.0G + 1.0ψ_2Q (Quasi)

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

1.35G + 1.5Q + 1.5ψ_0S (Strength) - Total deflection @ 10x

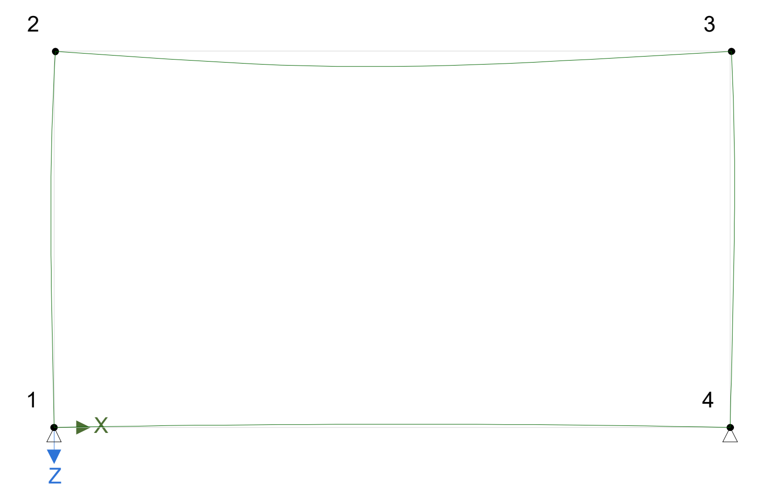


Member results

Load combination: 1.35G + 1.5Q + 1.5ψ_0S (Strength)

Member	Deflection				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

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

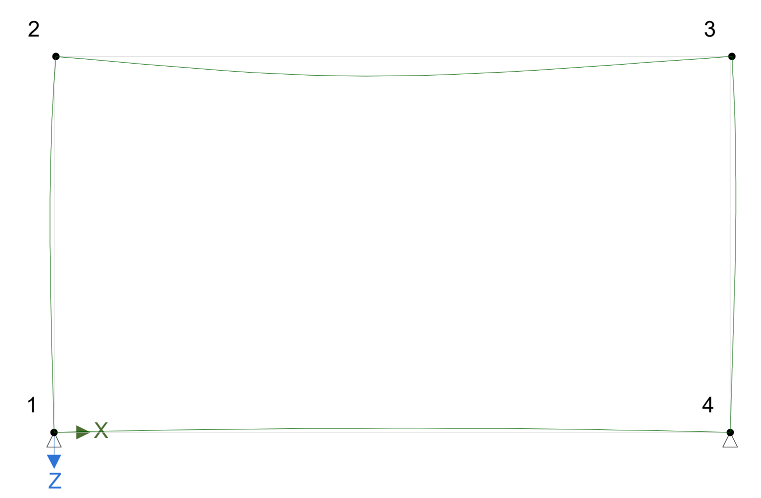


Member results

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

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

1.35G + 1.5ψ_0Q + 1.5S (Strength) - Total deflection @ 10x

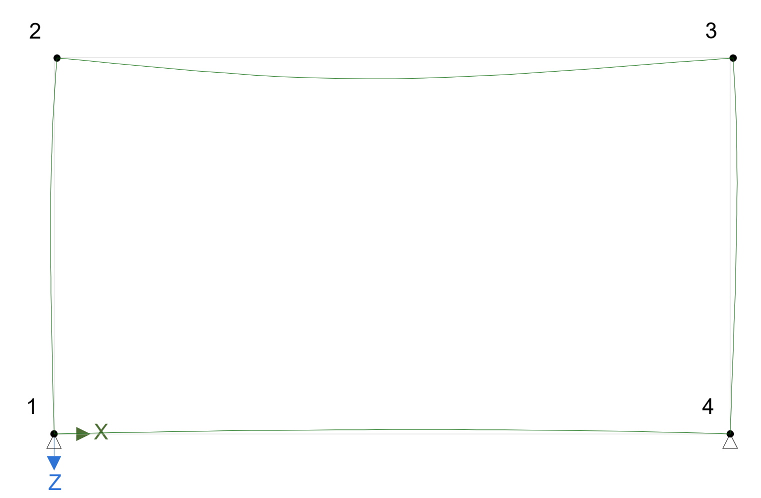


Member results

Load combination: 1.35G + 1.5ψ_0Q + 1.5S (Strength)

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

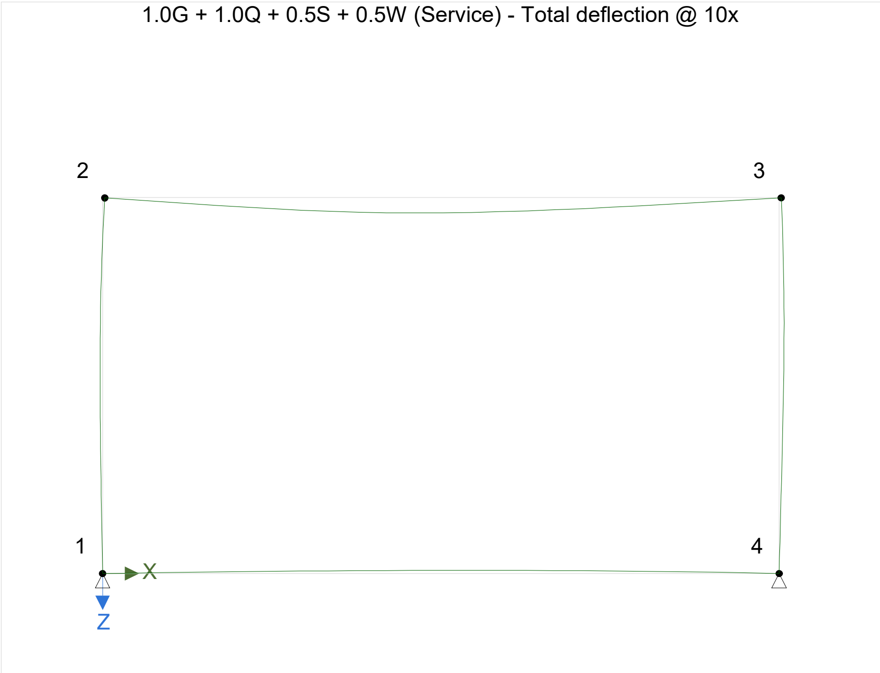
1.35G + 1.5Q + 1.5ψ_0S + 1.5ψ_0W (Strength) - Total deflection @ 10x



Member results

Load combination: 1.35G + 1.5Q + 1.5ψ_0S + 1.5ψ_0W (Strength)

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

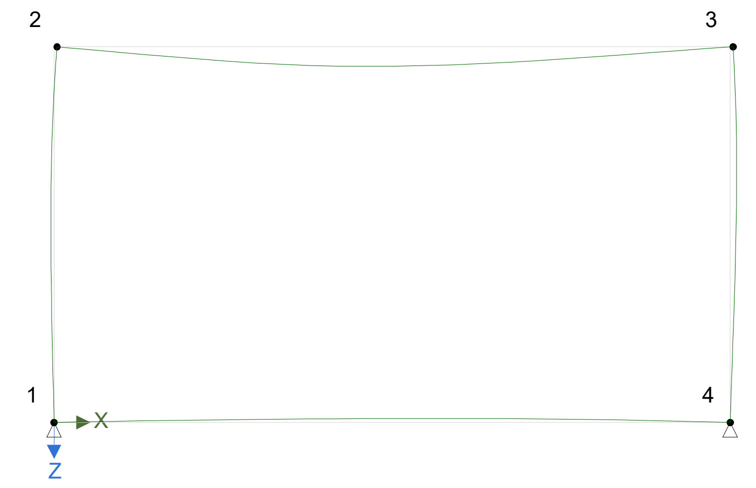


Member results

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

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

$1.35G + 1.5\psi_0Q + 1.5S + 1.5\psi_0W$ (Strength) - Total deflection @ 10x

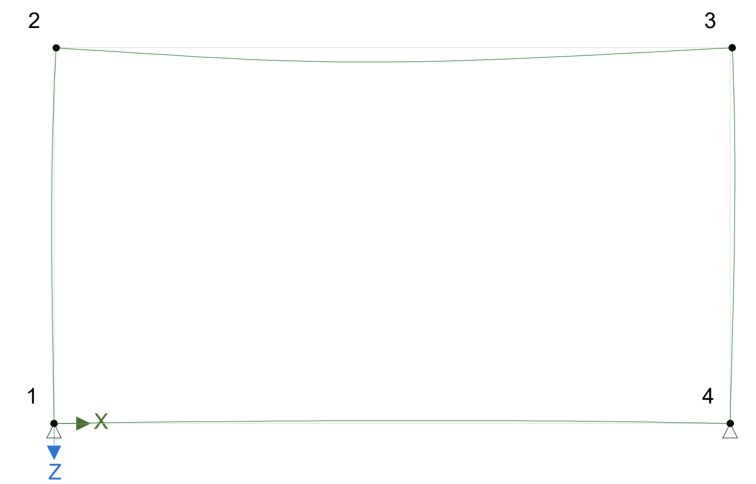


Member results

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

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

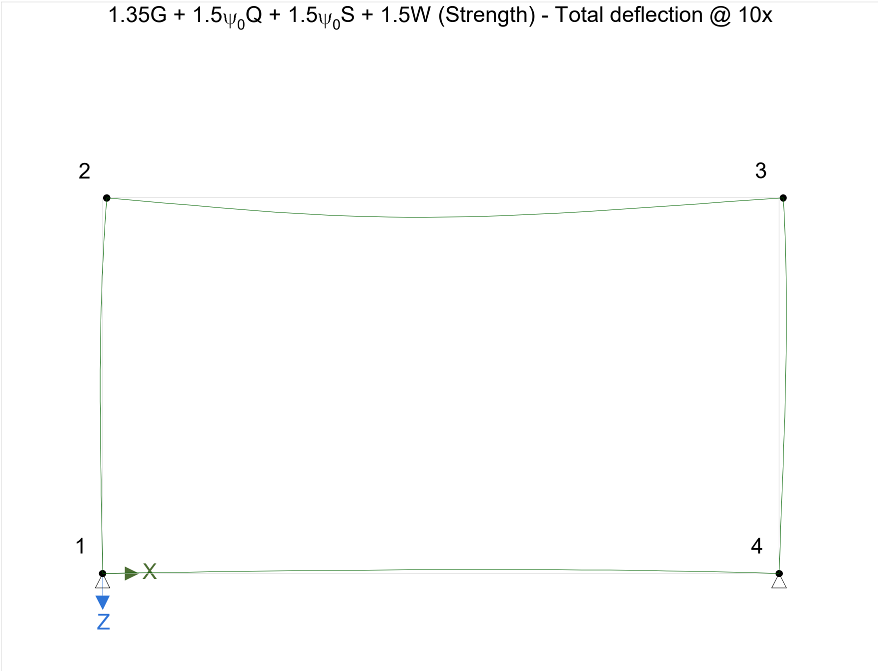
$1.0G + 1.0\psi_0Q + 1.0S + 0.5W$ (Service) - Total deflection @ 10x



Member results

Load combination: 1.0G + 1.0ψ0Q + 1.0S + 0.5W (Service)

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

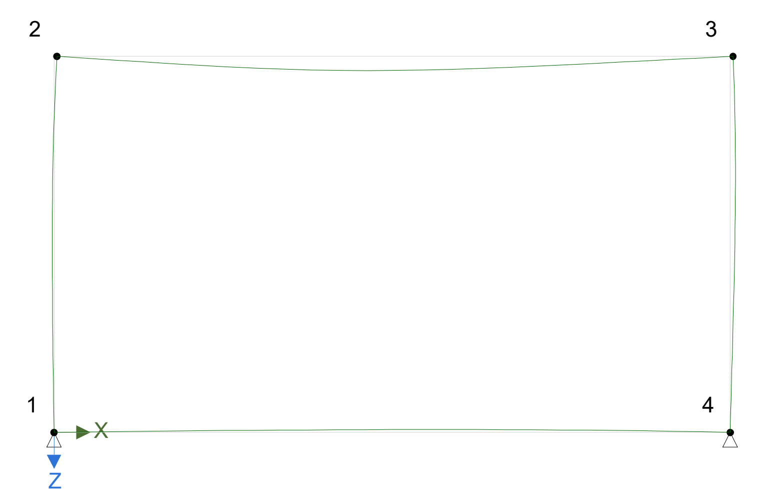


Member results

Load combination: 1.35G + 1.5ψ0Q + 1.5ψ0S + 1.5W (Strength)

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

1.0G + 1.0ψ_0Q + 0.5S + 1.0W (Service) - Total deflection @ 10x

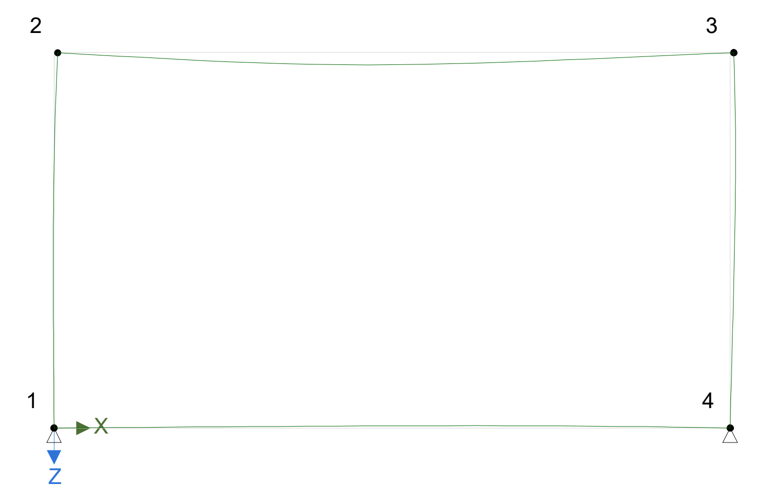


Member results

Load combination: 1.0G + 1.0ψ₀Q + 0.5S + 1.0W (Service)

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

1.0G + 1.5W (Strength) - Total deflection @ 10x

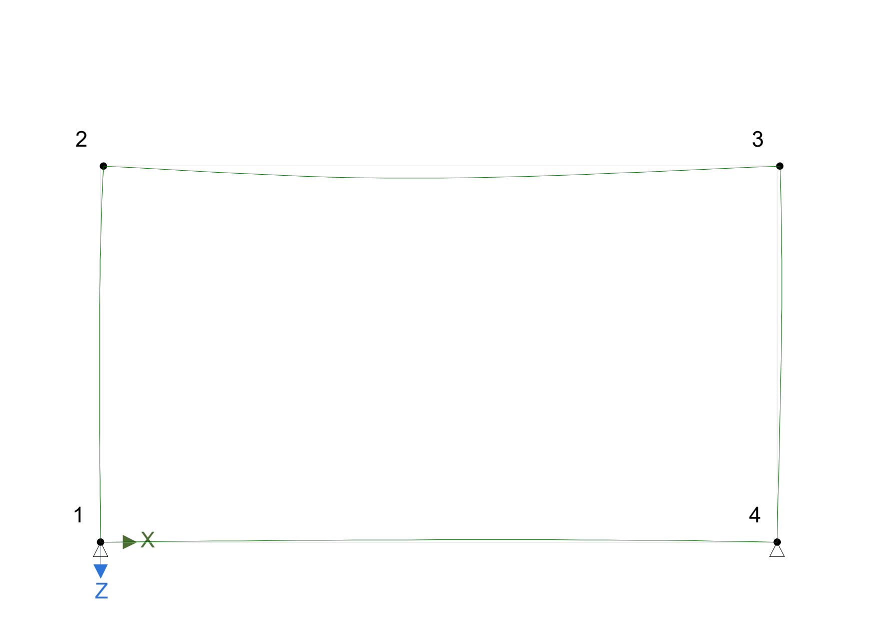


Member results

Load combination: 1.0G + 1.5W (Strength)

Member	Deflection				Axial deflection			
	Pos (m)	Max (mm)	Pos (m)	Min (mm)	Pos (m)	Max (mm)	Pos (m)	Min (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 (m)	Max (mm)	Pos (m)	Min (mm)	Pos (m)	Max (mm)	Pos (m)	Min (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
	X (mm)	Z (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
	X (mm)	Z (mm)	(°)	
1	0	0	-0.10836	

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

Load combination: 1.0G + 1.0ψ_2Q (Quasi)

Node	Deflection X (mm)	Deflection Z (mm)	Rotation (°)	Co-ordinate system
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_0S (Strength)

Node	Deflection X (mm)	Deflection Z (mm)	Rotation (°)	Co-ordinate system
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 X (mm)	Deflection Z (mm)	Rotation (°)	Co-ordinate system
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 + 1.5ψ_0Q + 1.5S (Strength)

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

Load combination: 1.35G + 1.5Q + 1.5ψ_0S + 1.5ψ_0W (Strength)

Node	Deflection		Rotation	Co-ordinate system
	X (mm)	Z (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
	X (mm)	Z (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 + 1.5ψ_0Q + 1.5S + 1.5ψ_0W (Strength)

Node	Deflection		Rotation	Co-ordinate system
	X (mm)	Z (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 + 1.0ψ_0Q + 1.0S + 0.5W (Service)

Node	Deflection		Rotation	Co-ordinate system
	X (mm)	Z (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 + 1.5ψ_0Q + 1.5ψ_0S + 1.5W (Strength)

Node	Deflection		Rotation	Co-ordinate system
	X (mm)	Z (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	

Load combination: 1.0G + 1.0ψ_0Q + 0.5S + 1.0W (Service)

Node	Deflection		Rotation	Co-ordinate system
	X (mm)	Z (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
	X (mm)	Z (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)

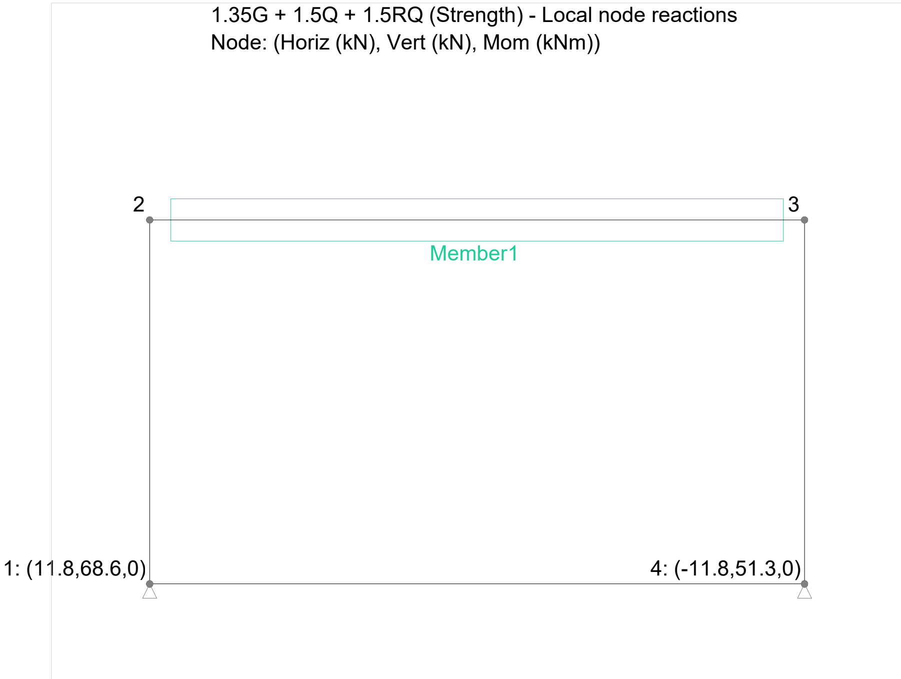
Node	Deflection		Rotation	Co-ordinate system
	X (mm)	Z (mm)	(°)	
1	0	0	-0.05764	
2	2.3	0.1	0.28919	
3	2.2	0.1	-0.21678	
4	0	0	0.11356	

Total base reactions

Load case/combination	Force	
	FX (kN)	FZ (kN)
1.35G + 1.5Q + 1.5RQ (Strength)	0	120
1.0G + 1.0Q + 1.0RQ (Service)	0	86.7
1.0G + 1.0ψ_2Q (Quasi)	0	73.3
1.35G + 1.5Q + 1.5ψ_0S (Strength)	0	120
1.0G + 1.0Q + 0.5S (Service)	0	86.7
1.35G + 1.5ψ_0Q + 1.5S (Strength)	0	111.3
1.35G + 1.5Q + 1.5ψ_0S + 1.5ψ_0W (Strength)	-2.7	120
1.0G + 1.0Q + 0.5S + 0.5W (Service)	-1.8	86.7
1.35G + 1.5ψ_0Q + 1.5S + 1.5ψ_0W (Strength)	-2.7	111.3
1.0G + 1.0ψ_0Q + 1.0S + 0.5W (Service)	-1.8	81
1.35G + 1.5ψ_0Q + 1.5ψ_0S + 1.5W (Strength)	-5.4	111.3
1.0G + 1.0ψ_0Q + 0.5S + 1.0W (Service)	-3.6	81
1.0G + 1.5W (Strength)	-5.4	67.5

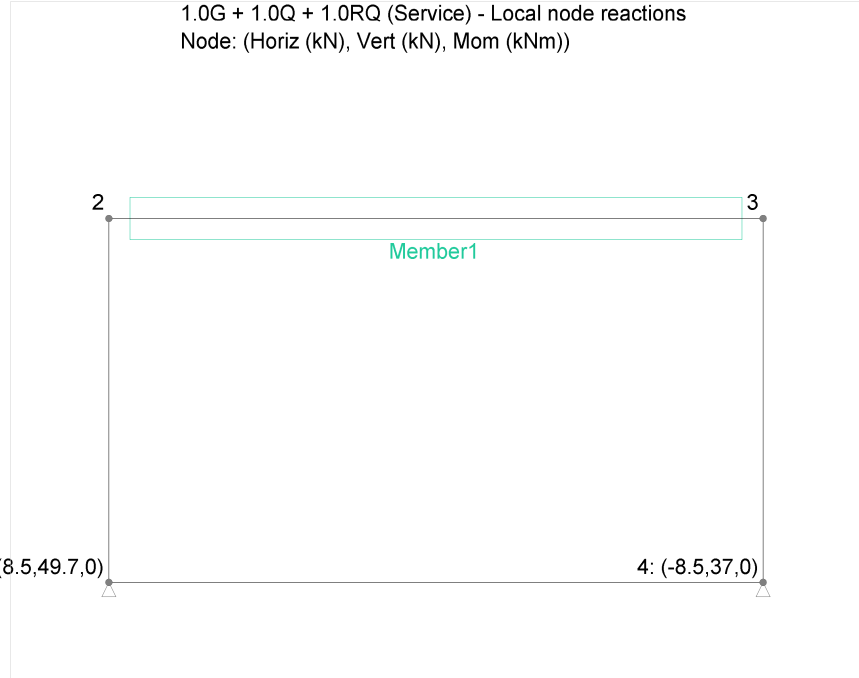
Load case/combination	Force	
	FX (kN)	FZ (kN)
1.0G + 1.0W (Service)	-3.6	67.5

Reactions



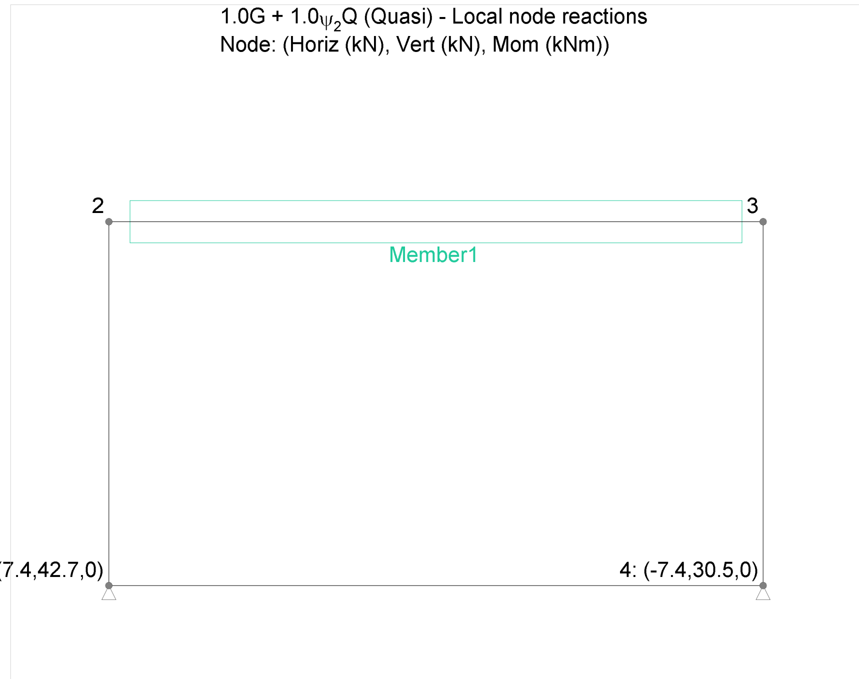
Load combination: 1.35G + 1.5Q + 1.5RQ (Strength)

Node	Force		Moment
	Fx (kN)	Fz (kN)	My (kNm)
1	11.8	68.6	0
4	-11.8	51.3	0



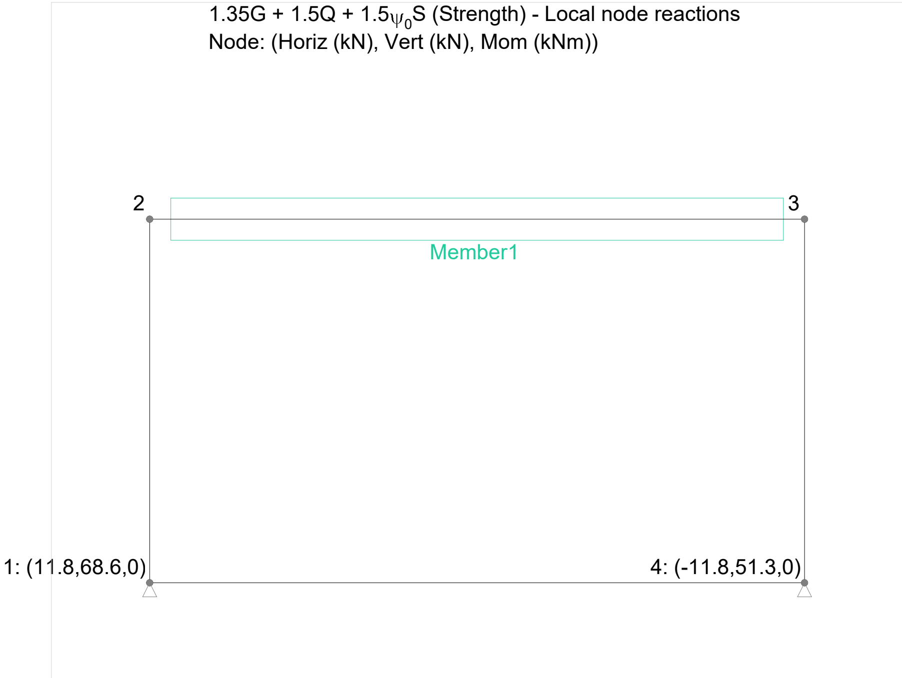
Load combination: 1.0G + 1.0Q + 1.0RQ (Service)

Node	Force		Moment
	Fx (kN)	Fz (kN)	My (kNm)
1	8.5	49.7	0
4	-8.5	37	0



Load combination: 1.0G + 1.0ψ_2Q (Quasi)

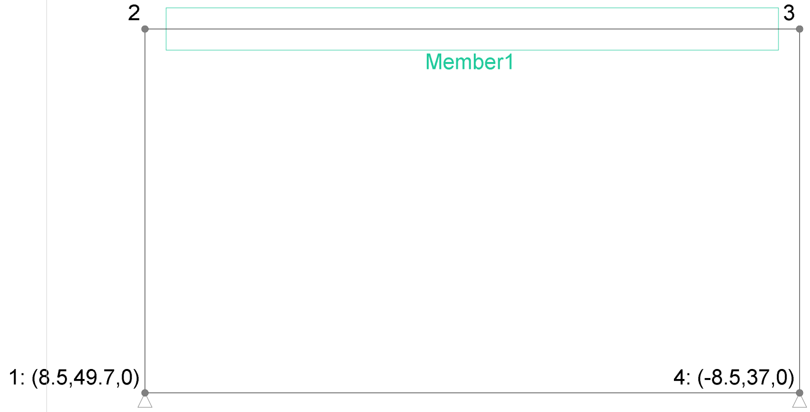
Node	Force		Moment
	Fx (kN)	Fz (kN)	My (kNm)
1	7.4	42.7	0
4	-7.4	30.5	0



Load combination: 1.35G + 1.5Q + 1.5ψ_0S (Strength)

Node	Force		Moment
	Fx (kN)	Fz (kN)	My (kNm)
1	11.8	68.6	0
4	-11.8	51.3	0

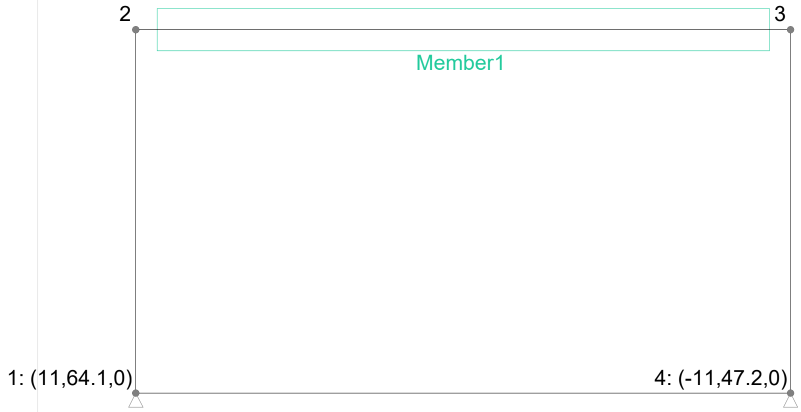
1.0G + 1.0Q + 0.5S (Service) - Local node reactions
Node: (Horiz (kN), Vert (kN), Mom (kNm))



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

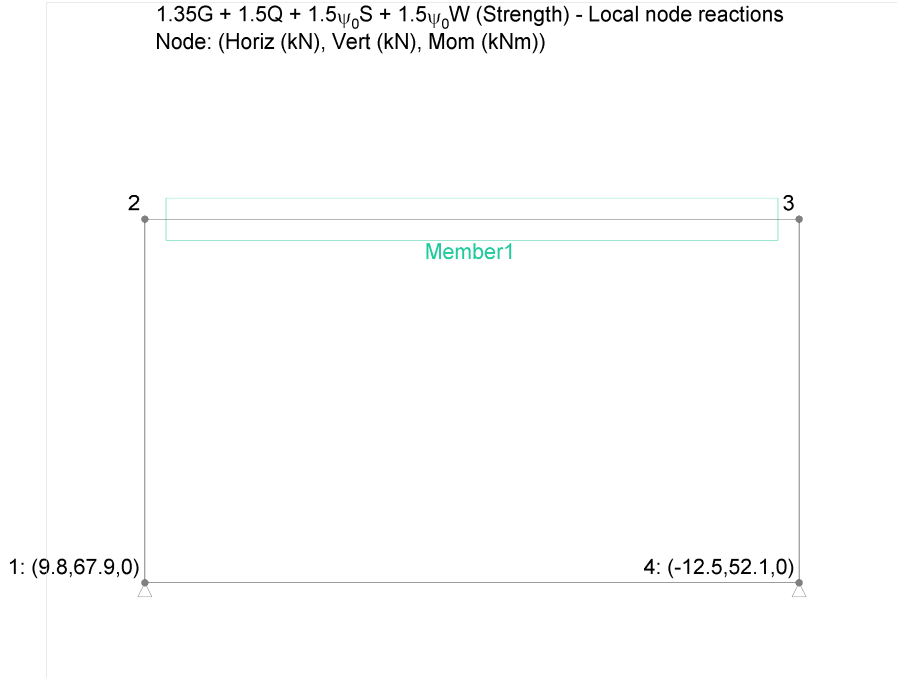
Node	Force		Moment
	Fx (kN)	Fz (kN)	My (kNm)
1	8.5	49.7	0
4	-8.5	37	0

1.35G + 1.5ψ_0Q + 1.5S (Strength) - Local node reactions
Node: (Horiz (kN), Vert (kN), Mom (kNm))



Load combination: 1.35G + 1.5ψ_0Q + 1.5S (Strength)

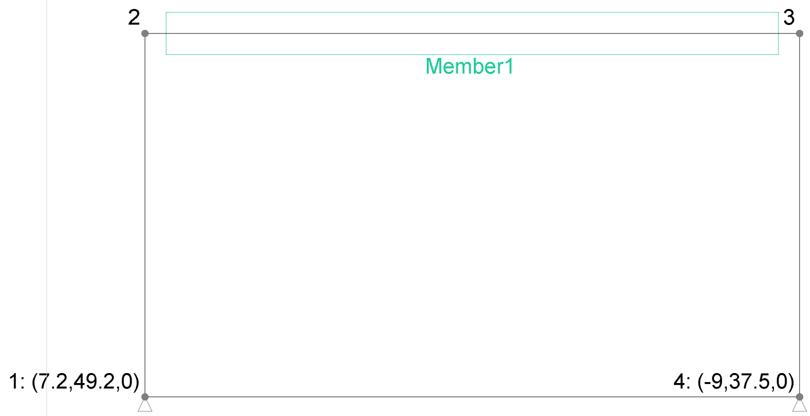
Node	Force		Moment
	Fx (kN)	Fz (kN)	My (kNm)
1	11	64.1	0
4	-11	47.2	0



Load combination: 1.35G + 1.5Q + 1.5ψ_0S + 1.5ψ_0W (Strength)

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

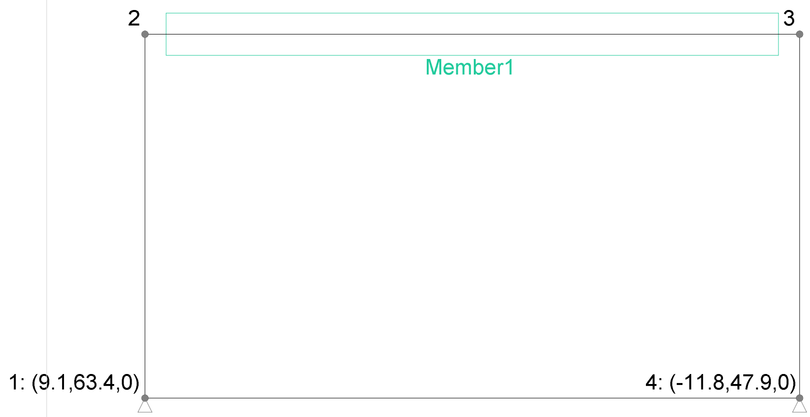
1.0G + 1.0Q + 0.5S + 0.5W (Service) - Local node reactions
Node: (Horiz (kN), Vert (kN), Mom (kNm))



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

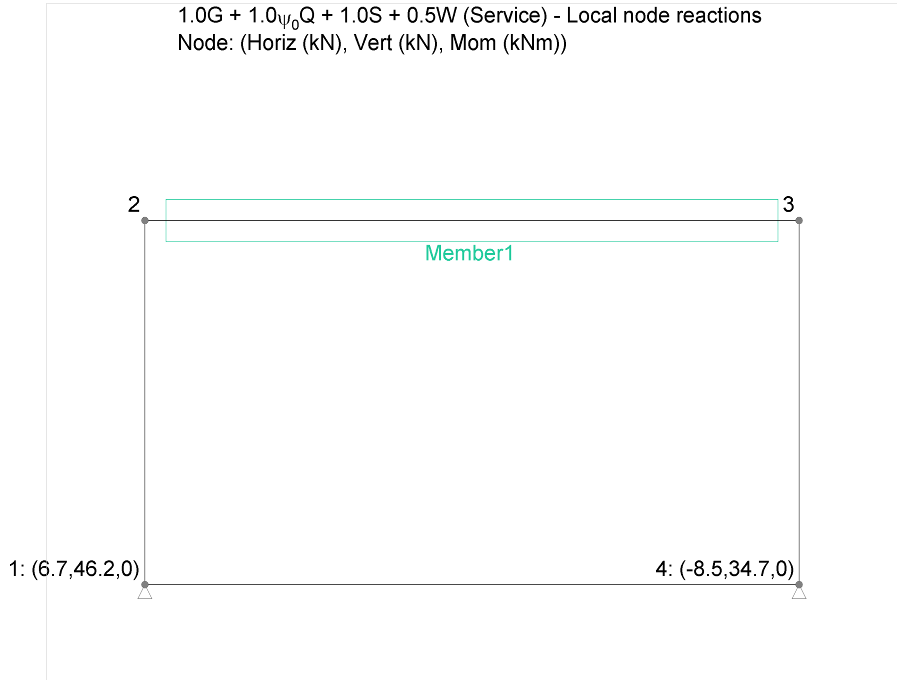
Node	Force		Moment
	Fx (kN)	Fz (kN)	My (kNm)
1	7.2	49.2	0
4	-9	37.5	0

1.35G + 1.5ψ_0Q + 1.5S + 1.5ψ_0W (Strength) - Local node reactions
Node: (Horiz (kN), Vert (kN), Mom (kNm))



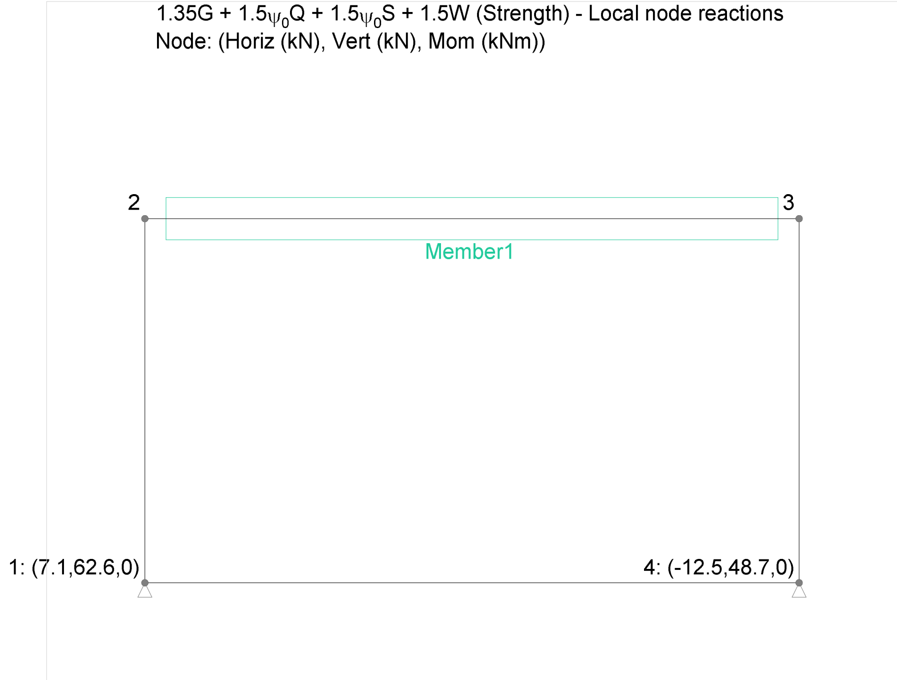
Load combination: 1.35G + 1.5ψ0Q + 1.5S + 1.5ψ0W (Strength)

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



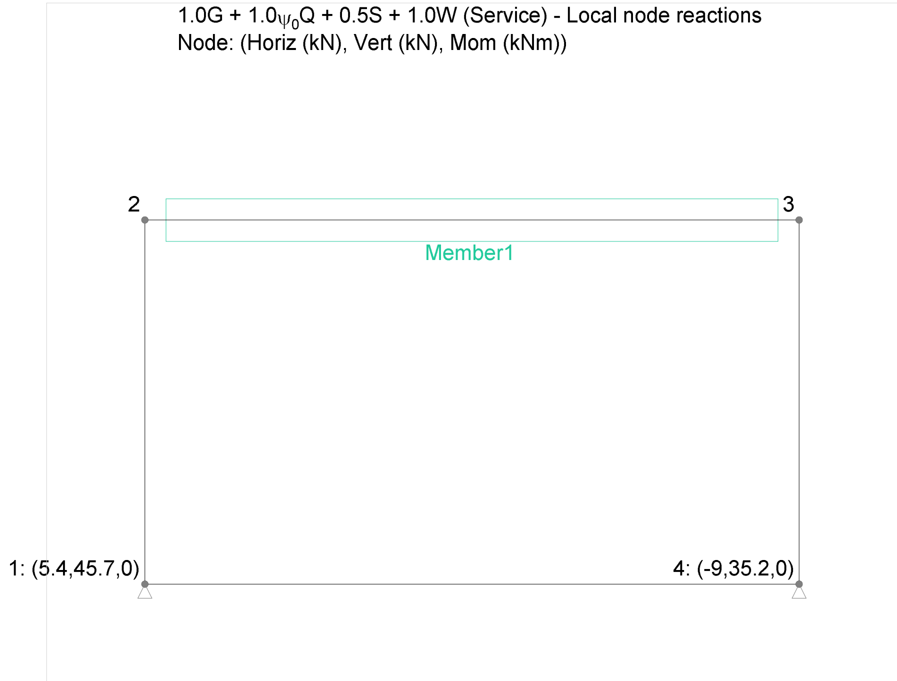
Load combination: 1.0G + 1.0ψ0Q + 1.0S + 0.5W (Service)

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



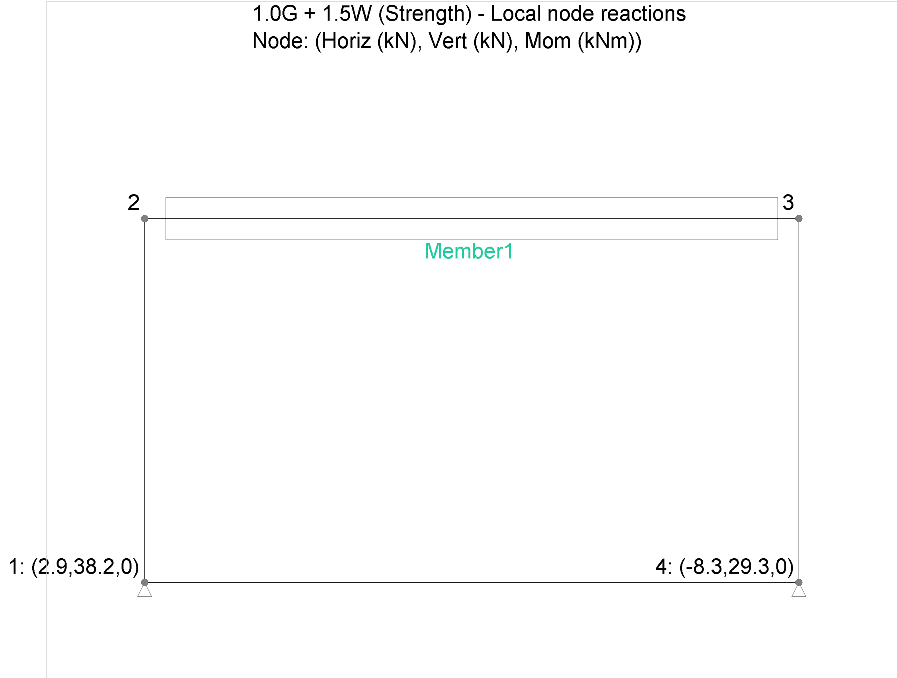
Load combination: 1.35G + 1.5ψ0Q + 1.5ψ0S + 1.5W (Strength)

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



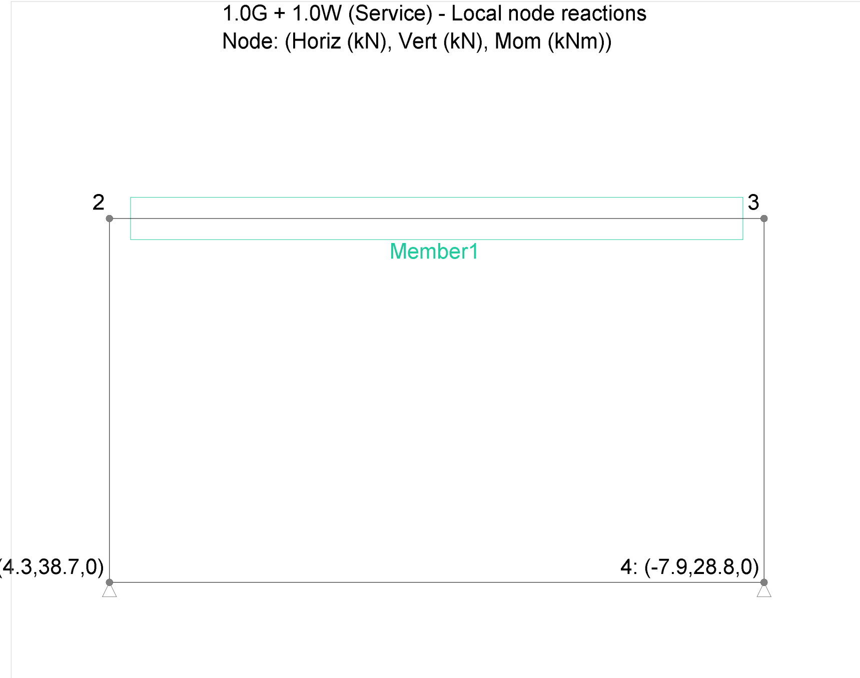
Load combination: 1.0G + 1.0ψ_0Q + 0.5S + 1.0W (Service)

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



Load combination: 1.0G + 1.5W (Strength)

Node	Force		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	Force		Moment
	Fx (kN)	Fz (kN)	My (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 (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (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

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

Load combination: 1.0G + 1.0ψ_2Q (Quasi)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (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 + 1.5ψ_0S (Strength)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (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 + 1.5ψ_0Q + 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

Load combination: 1.35G + 1.5Q + 1.5ψ_0S + 1.5ψ_0W (Strength)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (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 (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (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 + 1.5ψ_0Q + 1.5S + 1.5ψ_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 + 1.0ψ_0Q + 1.0S + 0.5W (Service)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (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

Load combination: 1.35G + 1.5ψ_0Q + 1.5ψ_0S + 1.5W (Strength)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (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 + 1.0ψ_0Q + 0.5S + 1.0W (Service)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (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)

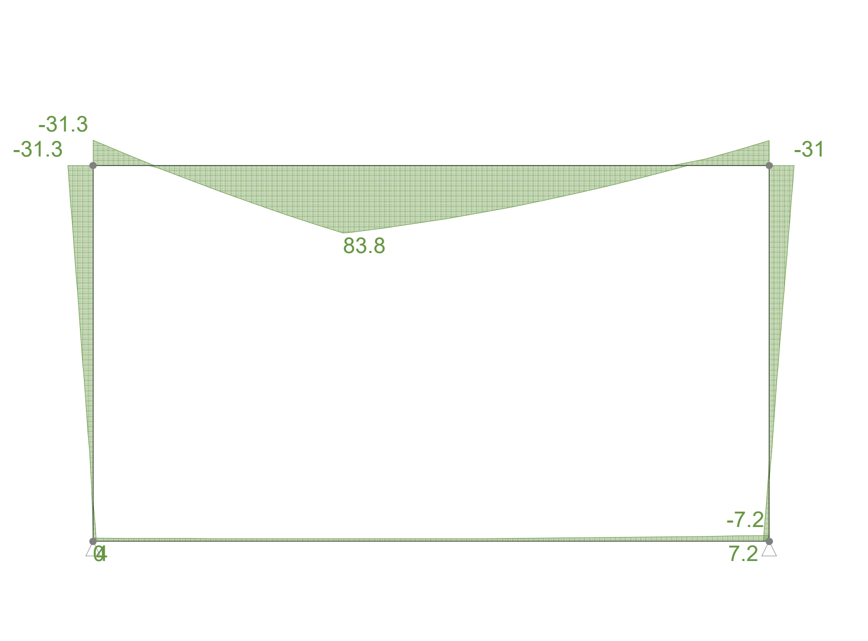
Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	3	1	-38.4	2.9	0
		2	37.6	-8.3	-16.9
2	5.4	2	-8.3	-37.6	16.9
		3	8.3	-26.6	-19.6
3	3	3	-26.6	-8.3	19.6
		4	27.5	8.3	5.4
4	5.4	1	0	0.2	0
		4	0	-1.8	-5.4

Load combination: 1.0G + 1.0W (Service)

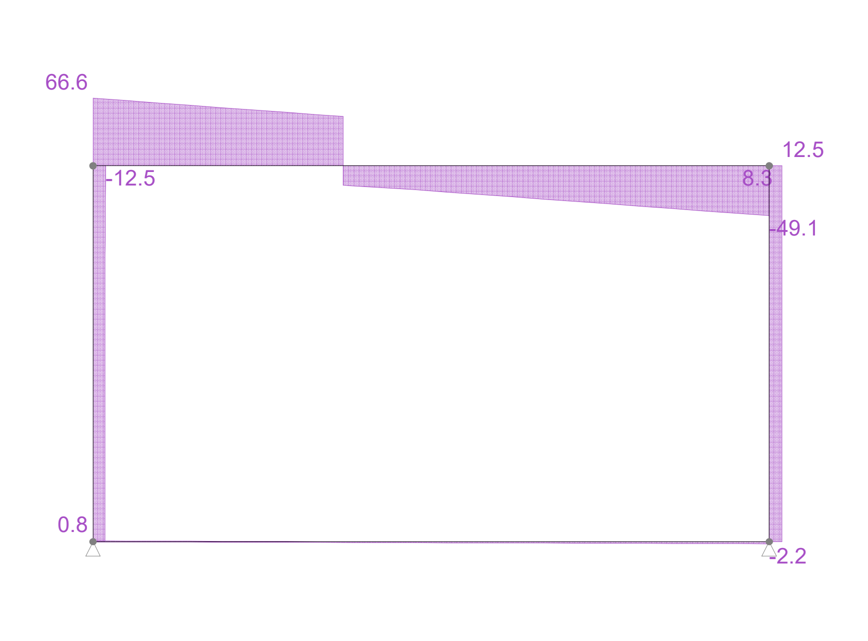
Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	3	1	-38.7	4.3	-0.8
		2	37.8	-7.9	-17.4
2	5.4	2	-7.9	-37.8	17.4
		3	7.9	-26.4	-18.8
3	3	3	-26.4	-7.9	18.8
		4	27.3	7.9	4.7
4	5.4	1	0	-0.1	0.8
		4	0	-1.5	-4.7

Forces

Strength combinations - Moment envelope (kNm)

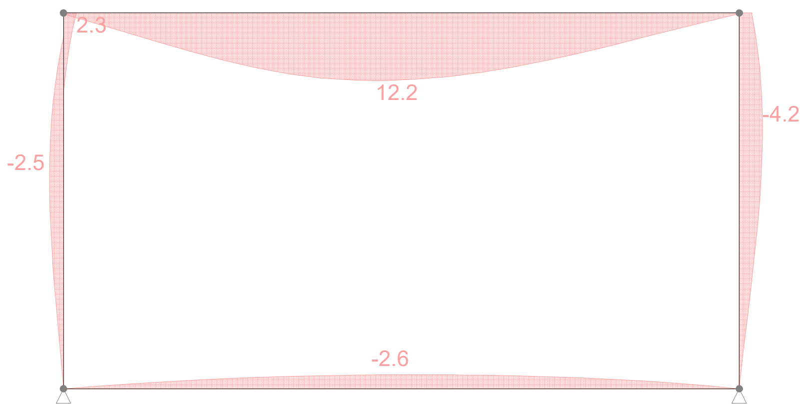


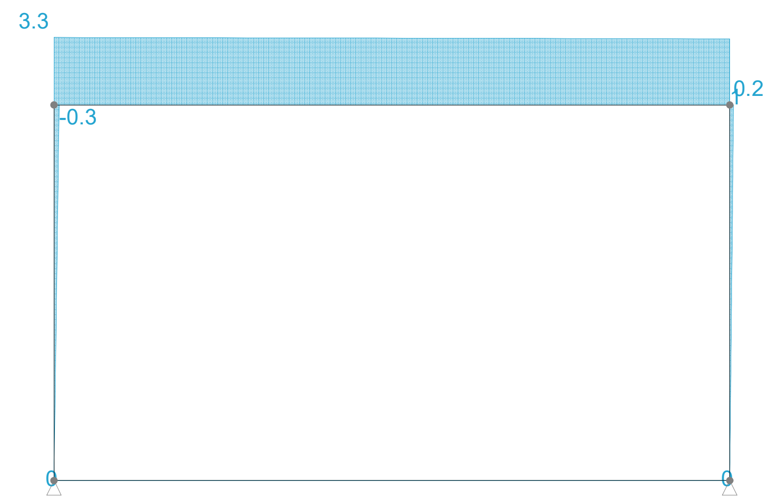
Strength combinations - Shear envelope (kN)



All combinations - Axial force envelope (kN)

Service combinations - Deflection envelope (mm)





Member results

Envelope - Strength combinations

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

Envelope - All combinations

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

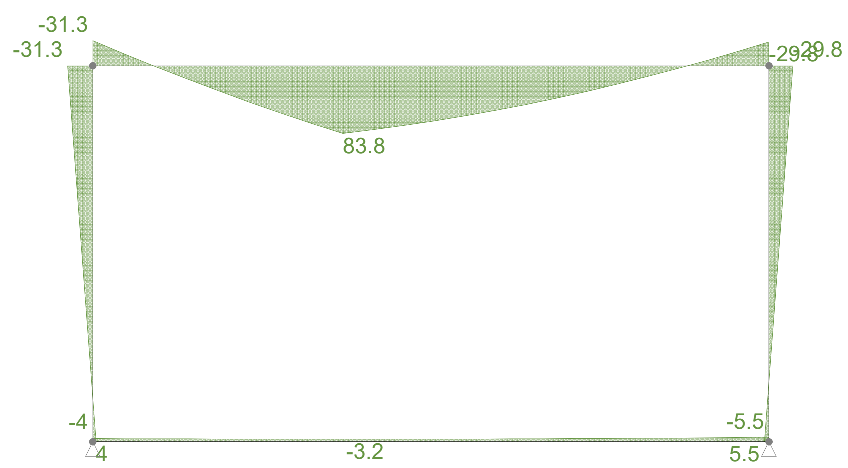
Envelope - Service combinations

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

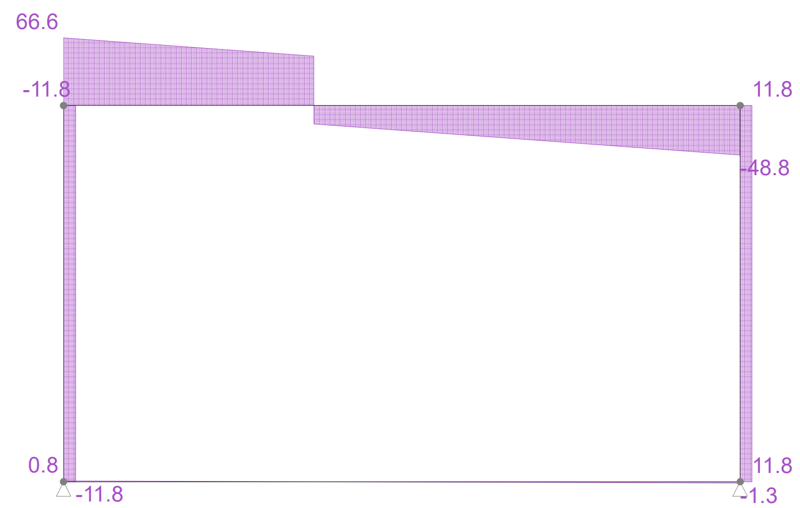
Envelope - All combinations

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

1.35G + 1.5Q + 1.5RQ (Strength) - Moment (kNm)



1.35G + 1.5Q + 1.5RQ (Strength) - Shear (kN)



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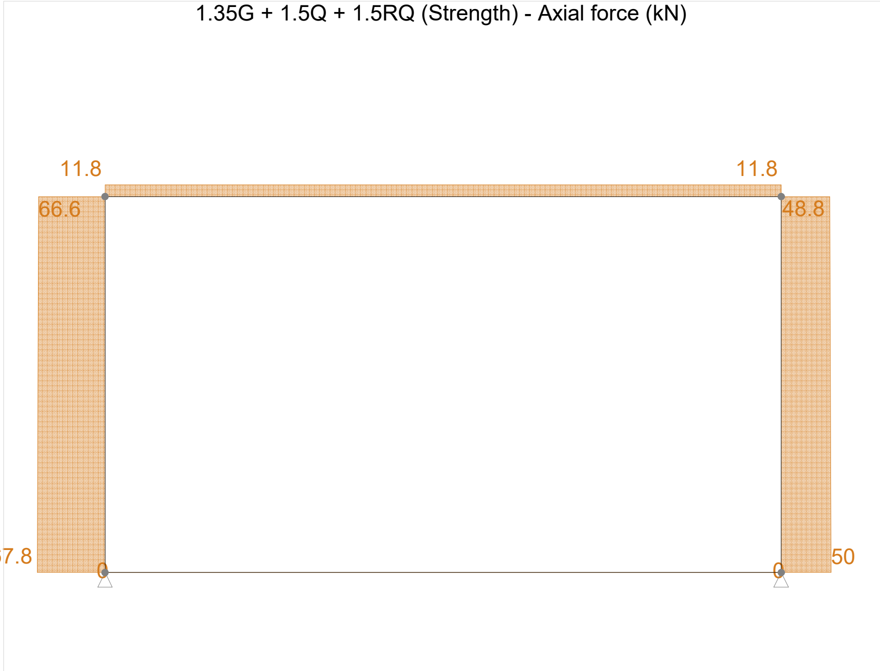
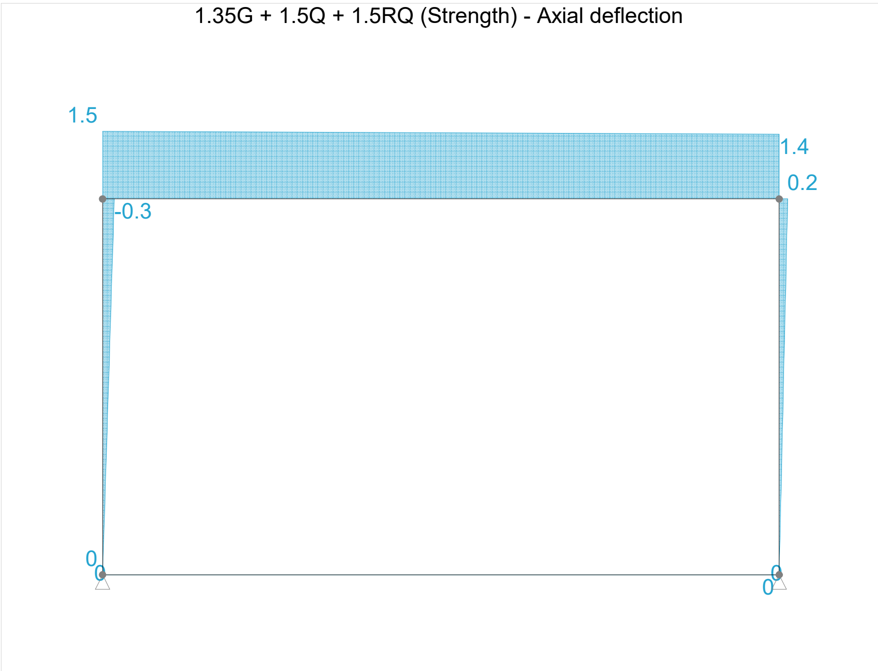
	Section: Building Control Approval				Sheet no./rev.: 73	
	Calc. by: RC	Date: 26/06/2017	Chk'd by: MD	Date: 26/06/2017	App'd by:	Date:





Member results

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

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

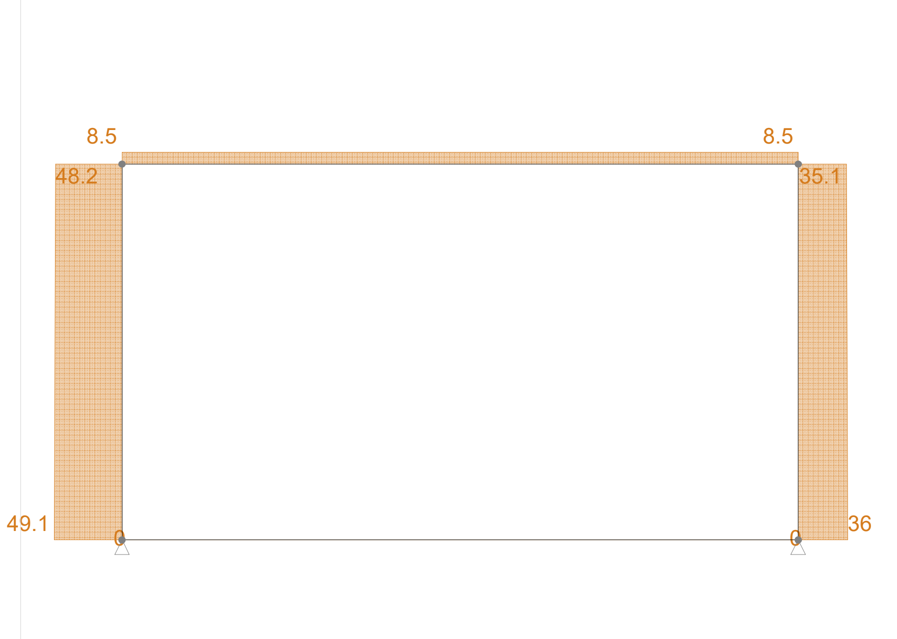
Load combination: 1.35G + 1.5Q + 1.5RQ (Strength)

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

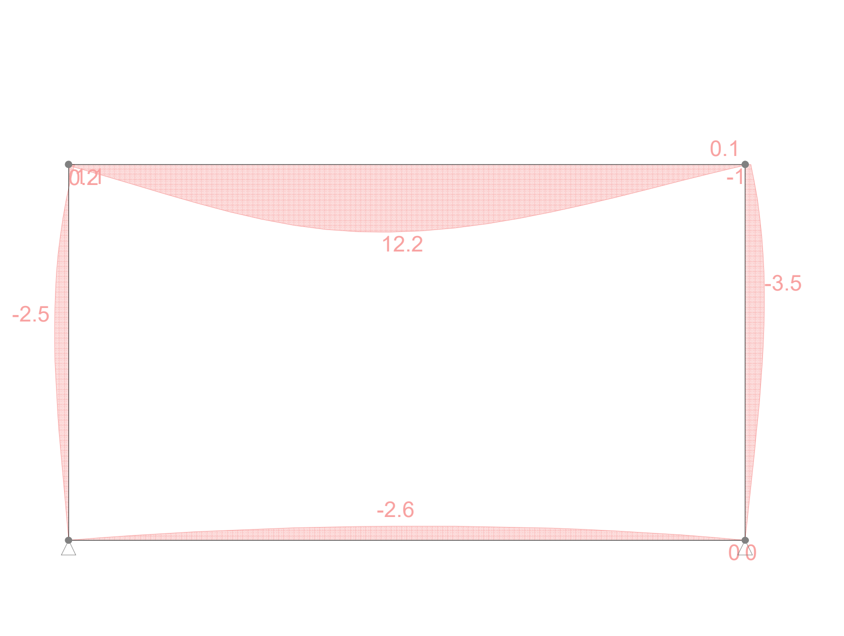
Load combination: 1.35G + 1.5Q + 1.5RQ (Strength)

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

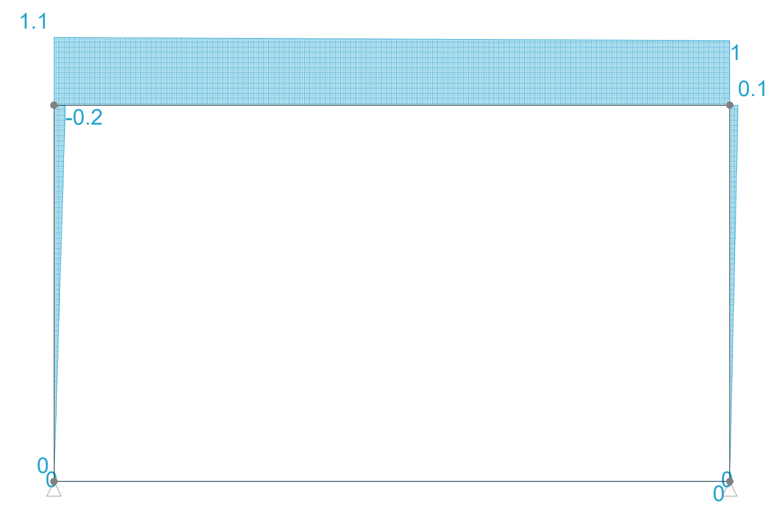
1.0G + 1.0Q + 1.0RQ (Service) - Axial force (kN)



1.0G + 1.0Q + 1.0RQ (Service) - Deflection (mm)



1.0G + 1.0Q + 1.0RQ (Service) - Axial deflection



Member results

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

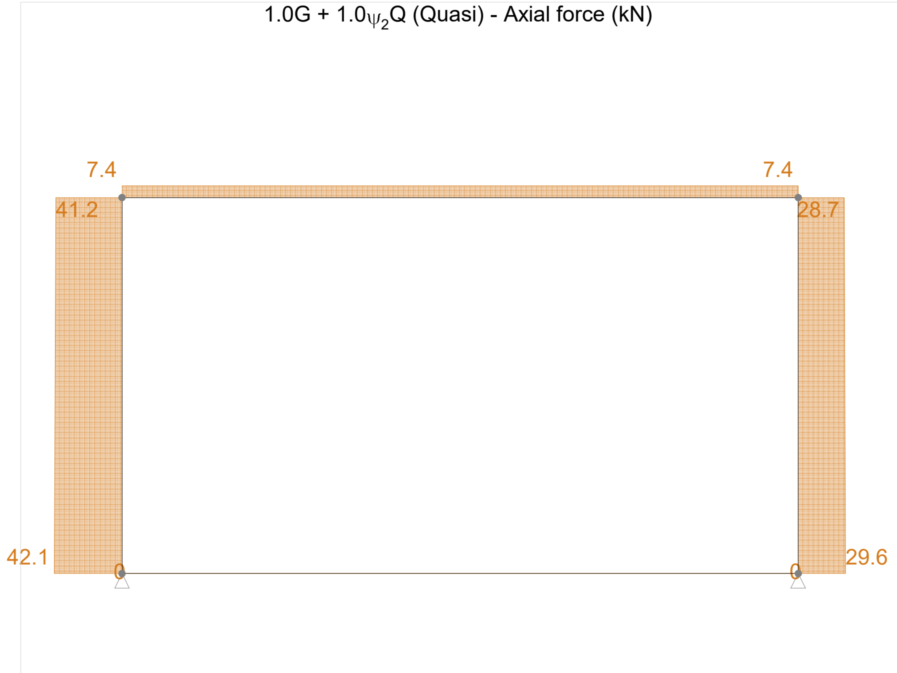
Member	Axial force			
	Pos (m)	Max (kN)	Pos (m)	Min (kN)
Member1	0	8.5	0	8.5

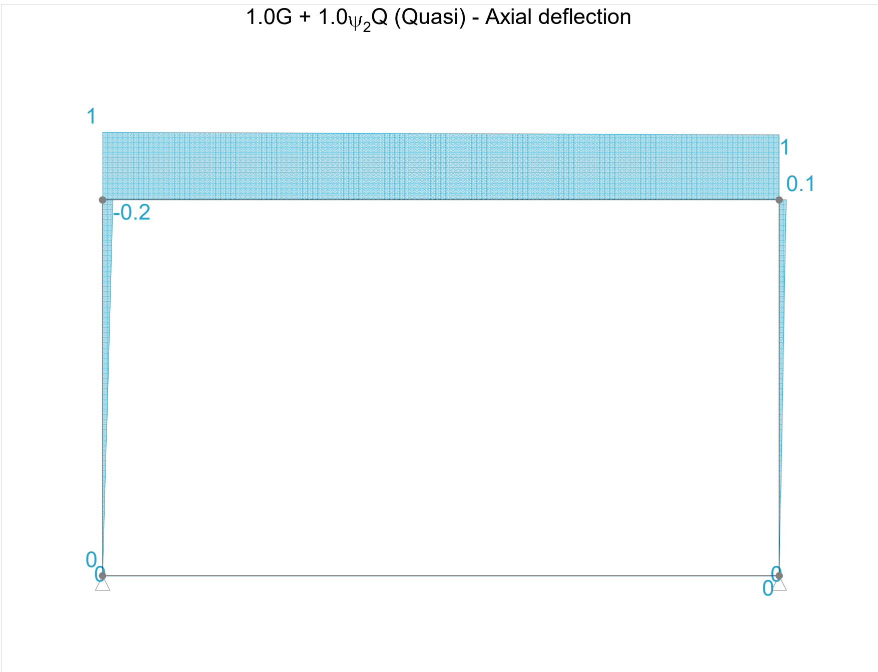
Load combination: 1.0G + 1.0Q + 1.0RQ (Service)

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

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

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





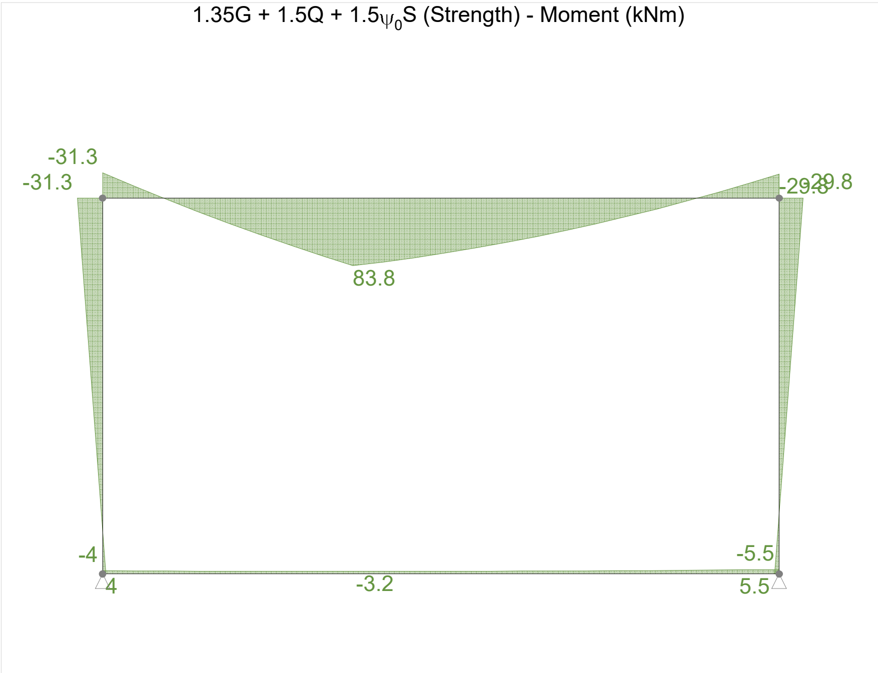
Member results

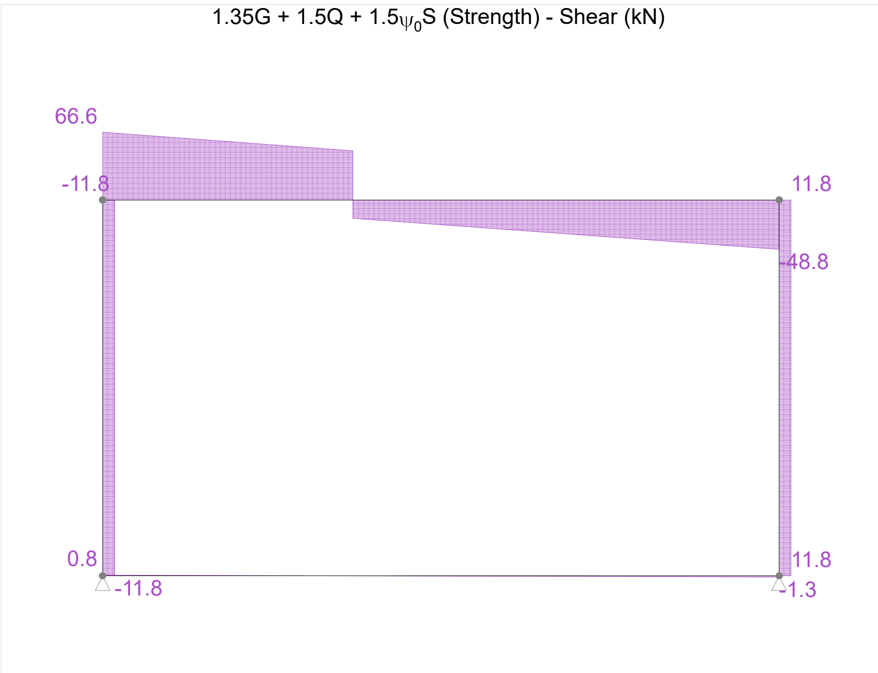
Load combination: 1.0G + 1.0ψ2Q (Quasi)

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

Load combination: 1.0G + 1.0ψ2Q (Quasi)

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





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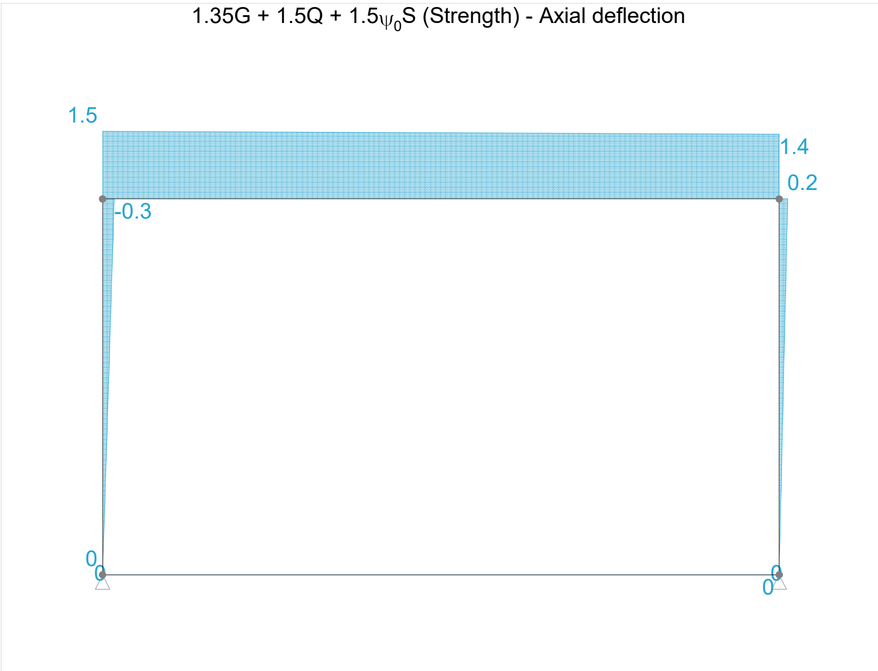
	Section Building Control Approval				Sheet no./rev. 78	
	Calc. by RC	Date 26/06/2017	Chk'd by MD	Date 26/06/2017	App'd by	Date



Member results

Load combination: 1.35G + 1.5Q + 1.5ψ0S (Strength)

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

Load combination: 1.35G + 1.5Q + 1.5ψ_0S (Strength)

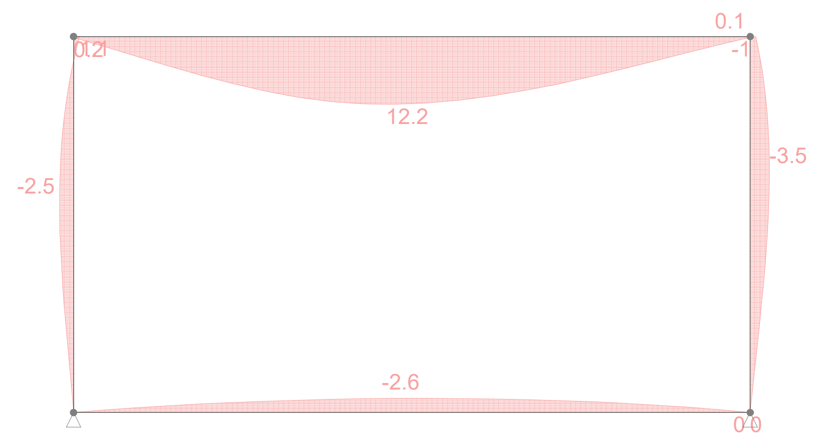
Member	Axial force			
	Pos (m)	Max (kN)	Pos (m)	Min (kN)
Member1	0	11.8	0	11.8

Load combination: 1.35G + 1.5Q + 1.5ψ_0S (Strength)

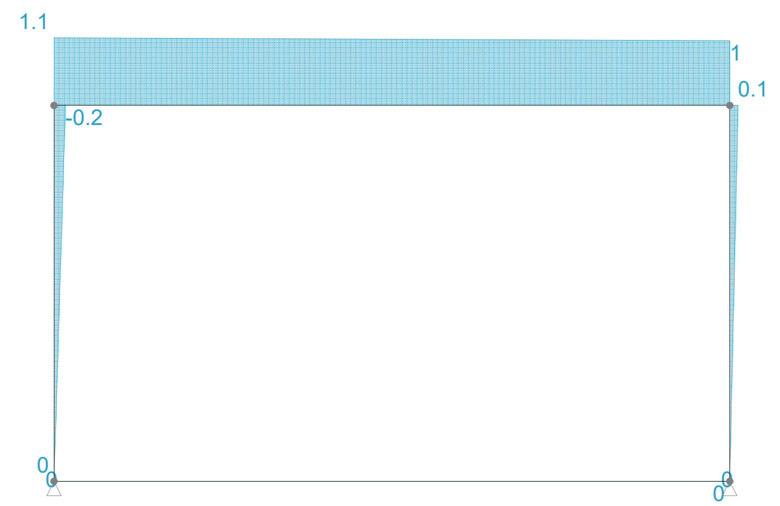
Member	Axial deflection			
	Pos (m)	Max (mm)	Pos (m)	Min (mm)
Member1	0	1.5	5.4	1.4

1.0G + 1.0Q + 0.5S (Service) - Axial force (kN)

1.0G + 1.0Q + 0.5S (Service) - Deflection (mm)



1.0G + 1.0Q + 0.5S (Service) - Axial deflection



Member results

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

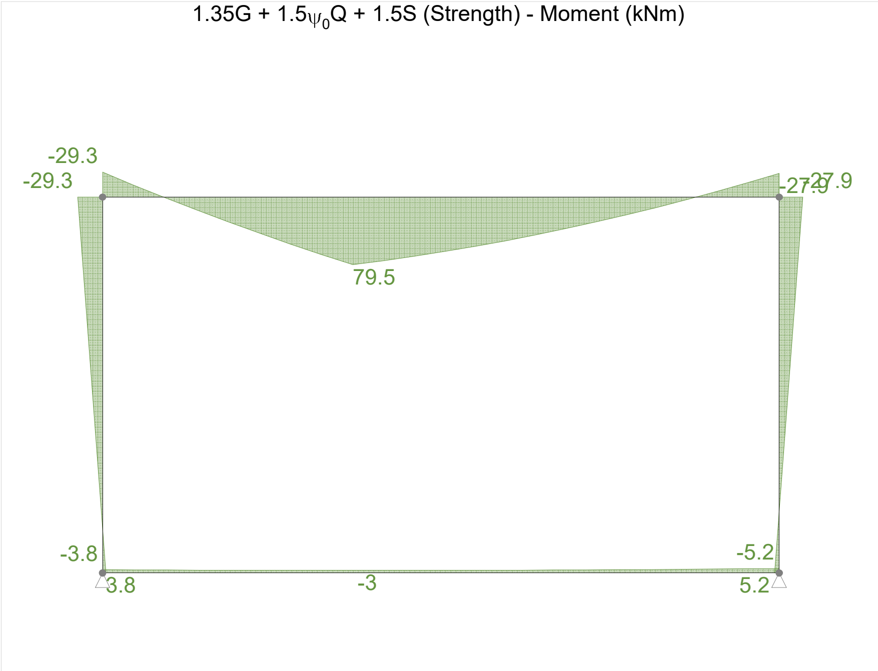
Member	Axial force			
	Pos (m)	Max (kN)	Pos (m)	Min (kN)
Member1	0	8.5	0	8.5

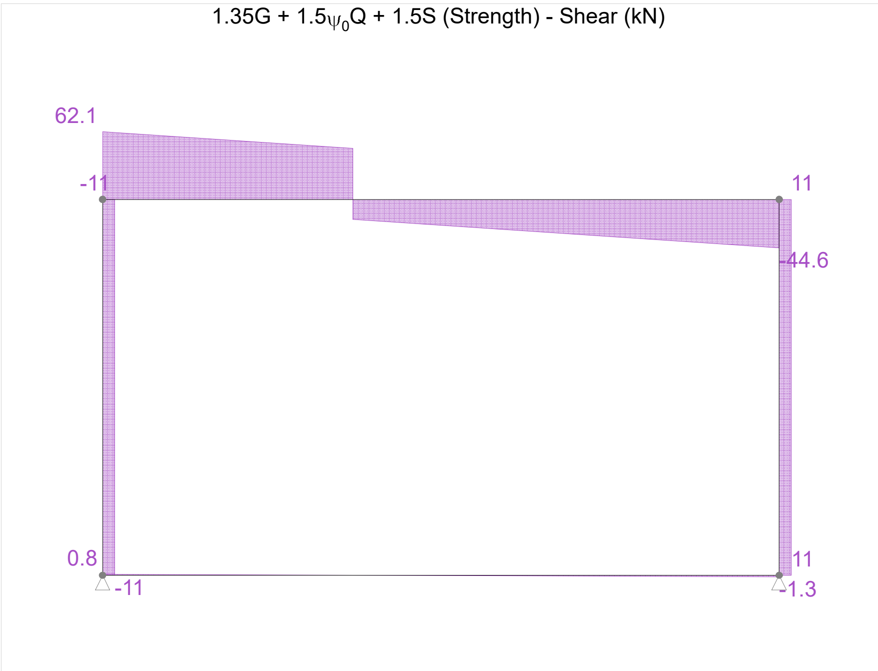
Load combination: 1.0G + 1.0Q + 0.5S (Service)

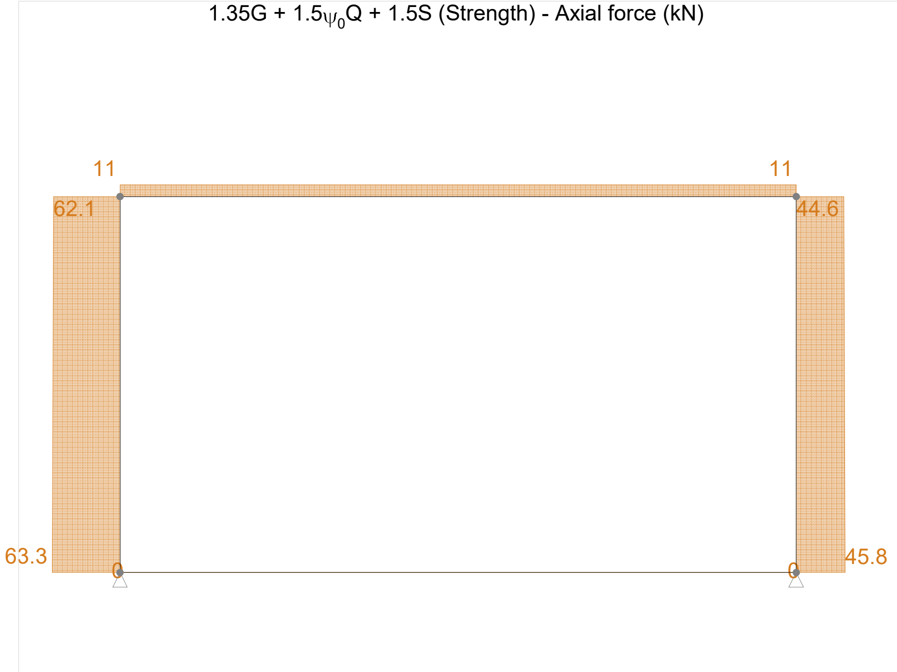
Member	Deflection			
	Pos (m)	Max (mm)	Pos (m)	Min (mm)
Member1	2.494	12.2	5.4	0.1

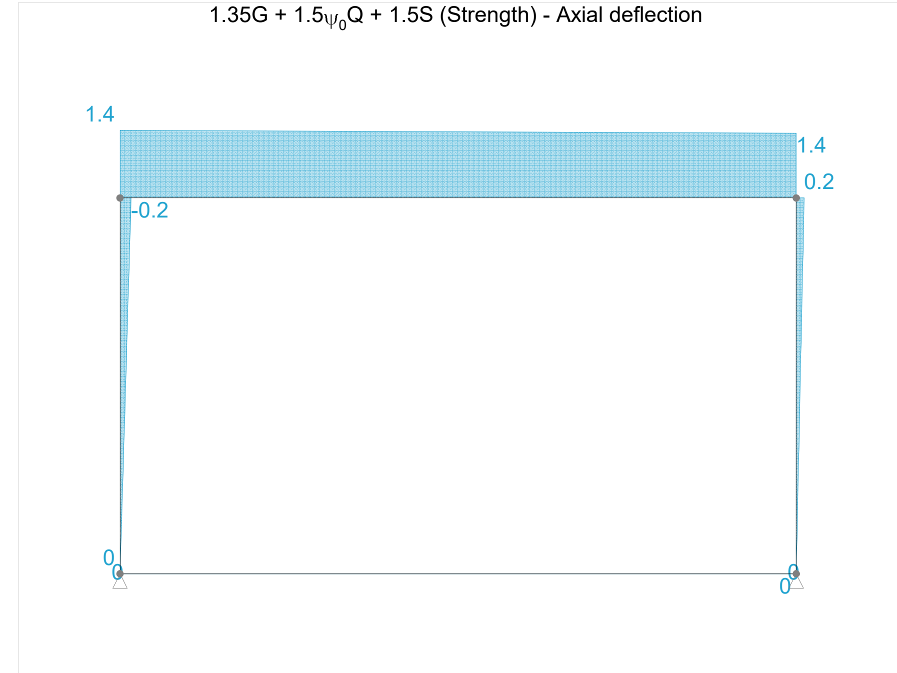
Load combination: 1.0G + 1.0Q + 0.5S (Service)

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









Member results

Load combination: 1.35G + 1.5ψ0Q + 1.5S (Strength)

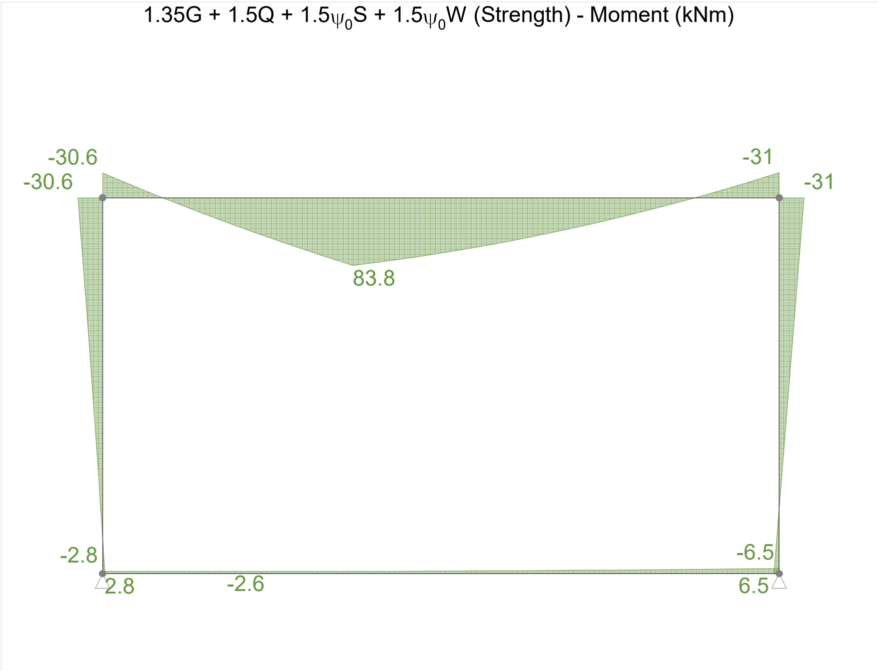
Member	Shear force		Moment			
	Pos (m)	Max abs (kN)	Pos (m)	Max (kNm)	Pos (m)	Min (kNm)
Member1	0	62.1	1.998	79.5	0	-29.3

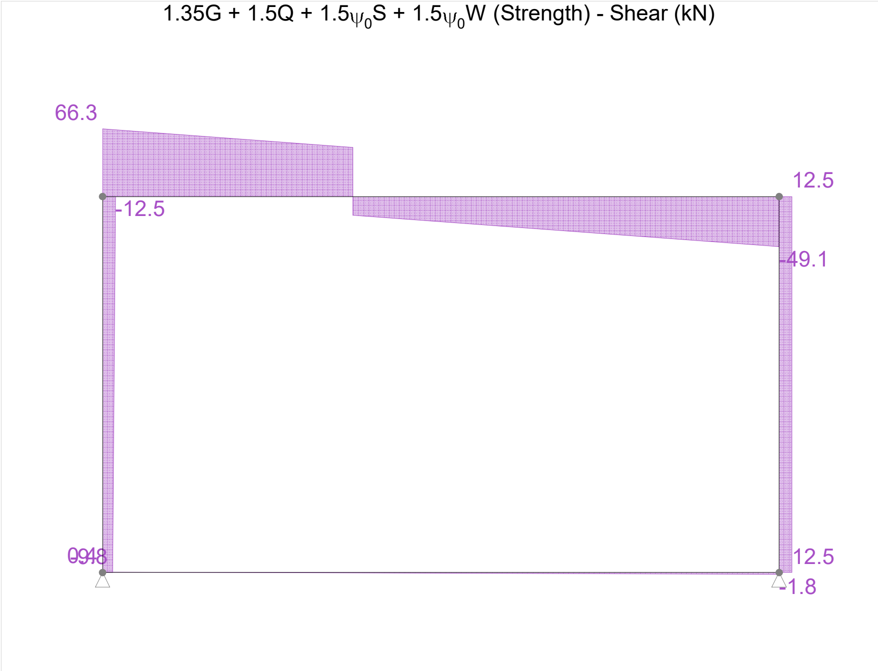
Load combination: 1.35G + 1.5ψ0Q + 1.5S (Strength)

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

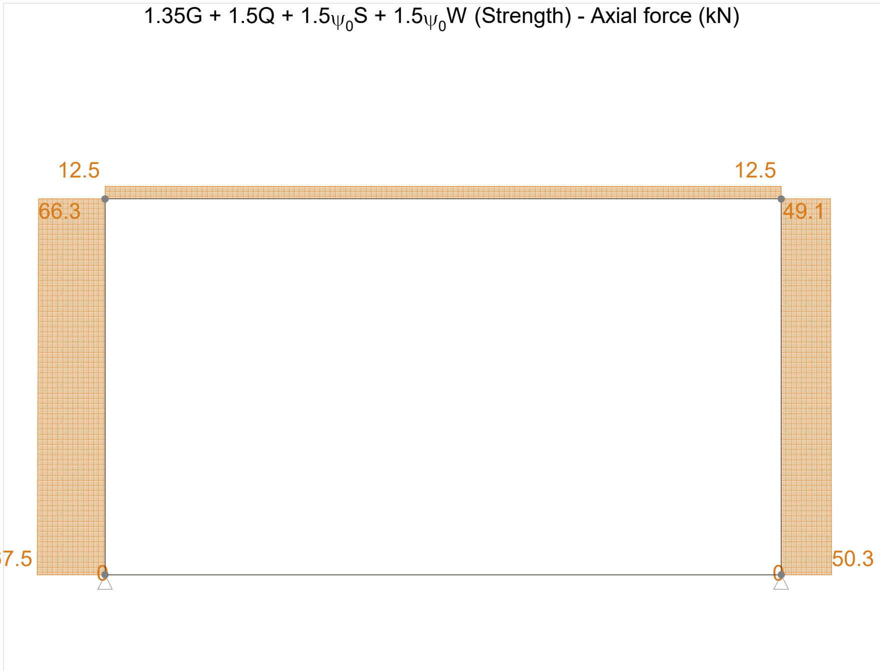
Load combination: 1.35G + 1.5ψ0Q + 1.5S (Strength)

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

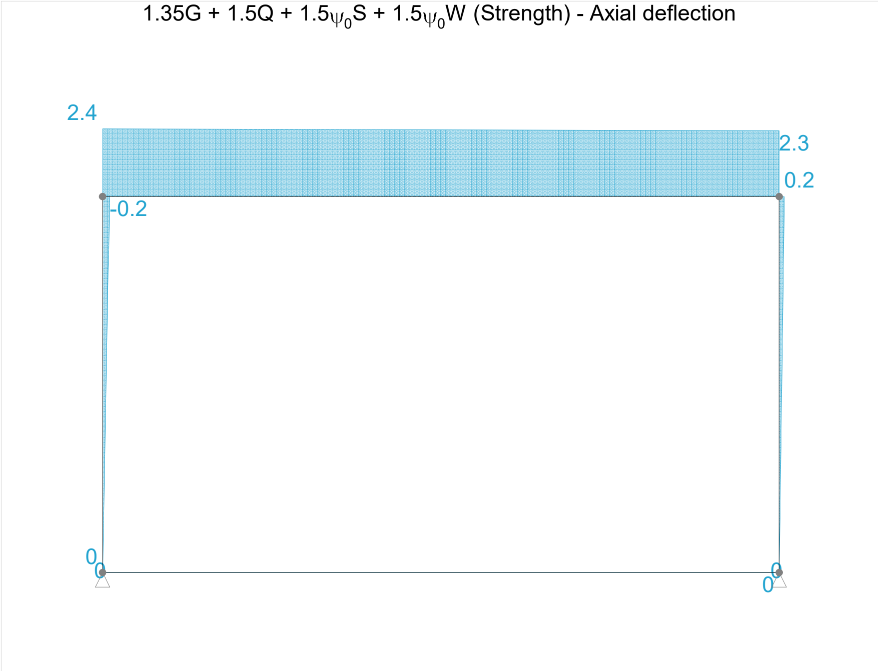




1.35G + 1.5Q + 1.5ψ0S + 1.5ψ0W (Strength) - Shear (kN)
66.3
-12.5
12.5
-49.1
0.48
12.5
-1.8



1.35G + 1.5Q + 1.5ψ0S + 1.5ψ0W (Strength) - Axial force (kN)
12.5
12.5
66.3
49.1
67.5
0
0
50.3



Member results

Load combination: 1.35G + 1.5Q + 1.5ψ_0S + 1.5ψ_0W (Strength)

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

Load combination: 1.35G + 1.5Q + 1.5ψ_0S + 1.5ψ_0W (Strength)

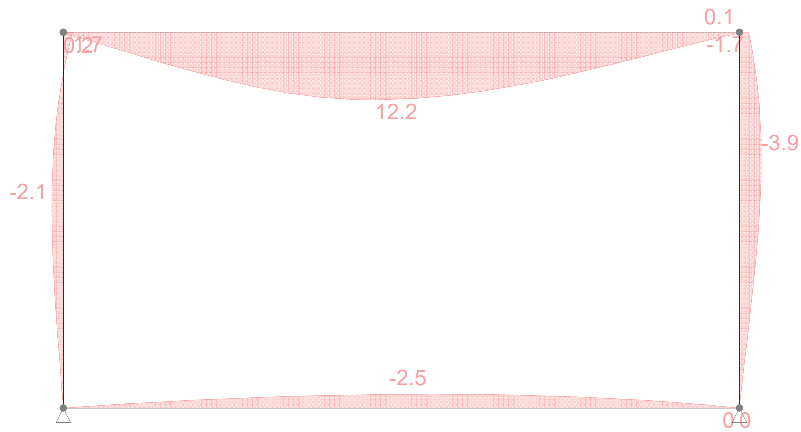
Member	Axial force			
	Pos (m)	Max (kN)	Pos (m)	Min (kN)
Member1	0	12.5	0	12.5

Load combination: 1.35G + 1.5Q + 1.5ψ_0S + 1.5ψ_0W (Strength)

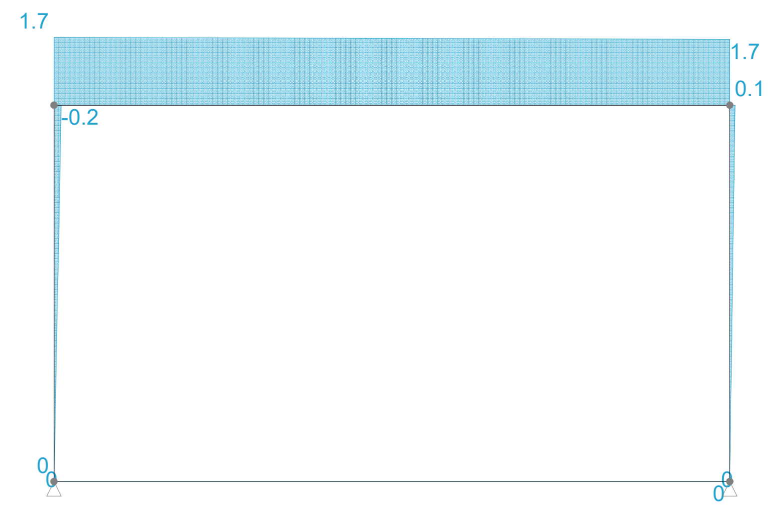
Member	Axial deflection			
	Pos (m)	Max (mm)	Pos (m)	Min (mm)
Member1	0	2.4	5.4	2.3

1.0G + 1.0Q + 0.5S + 0.5W (Service) - Axial force (kN)

1.0G + 1.0Q + 0.5S + 0.5W (Service) - Deflection (mm)



1.0G + 1.0Q + 0.5S + 0.5W (Service) - Axial deflection



Member results

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

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

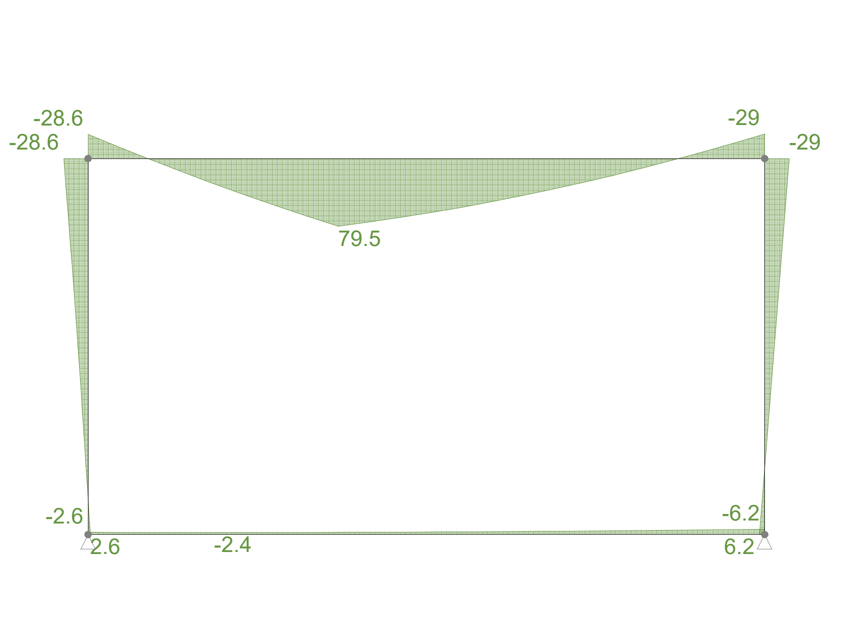
Load combination: 1.0G + 1.0Q + 0.5S + 0.5W (Service)

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

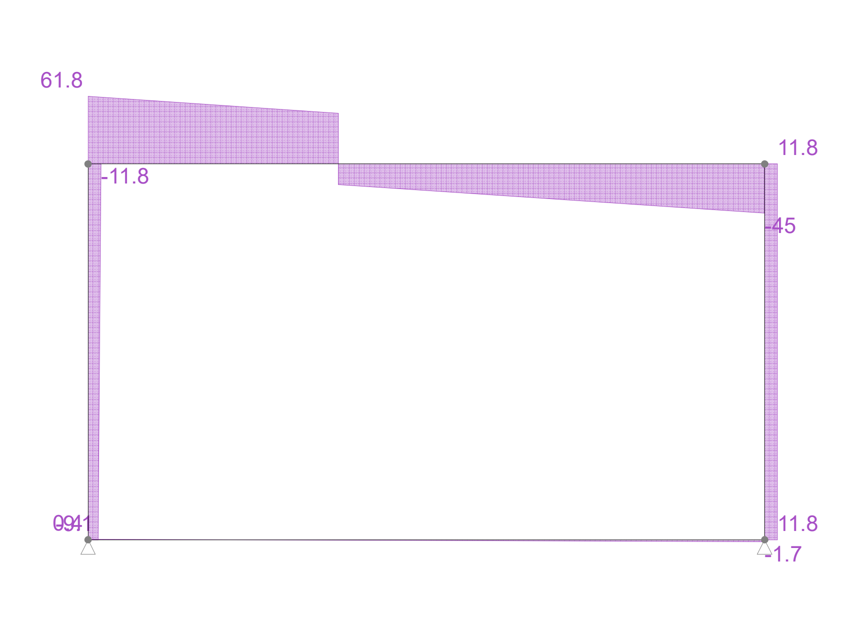
Load combination: 1.0G + 1.0Q + 0.5S + 0.5W (Service)

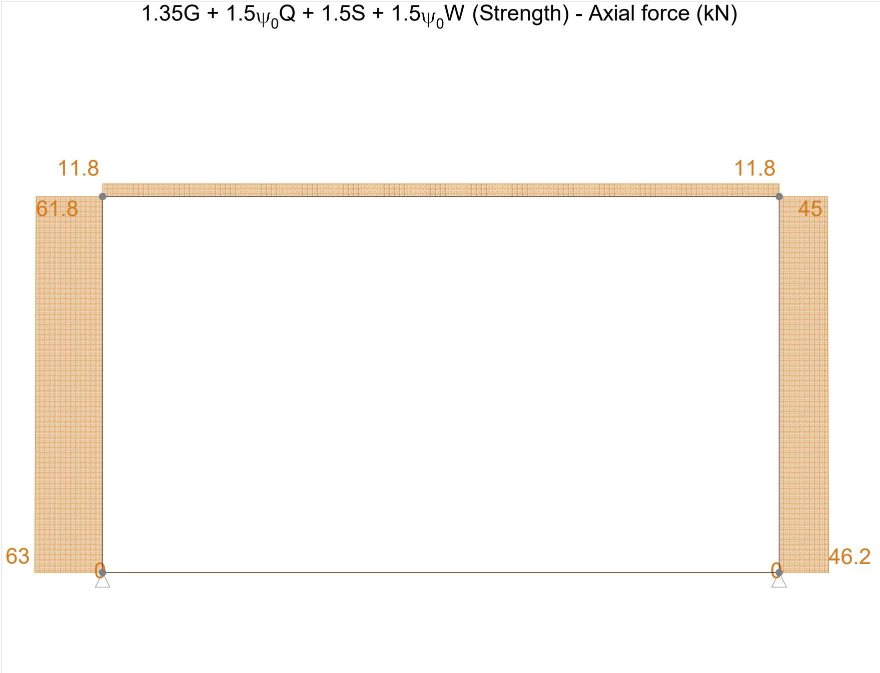
Member	Axial deflection			
	Pos (m)	Max (mm)	Pos (m)	Min (mm)
Member1	0	1.7	5.4	1.7

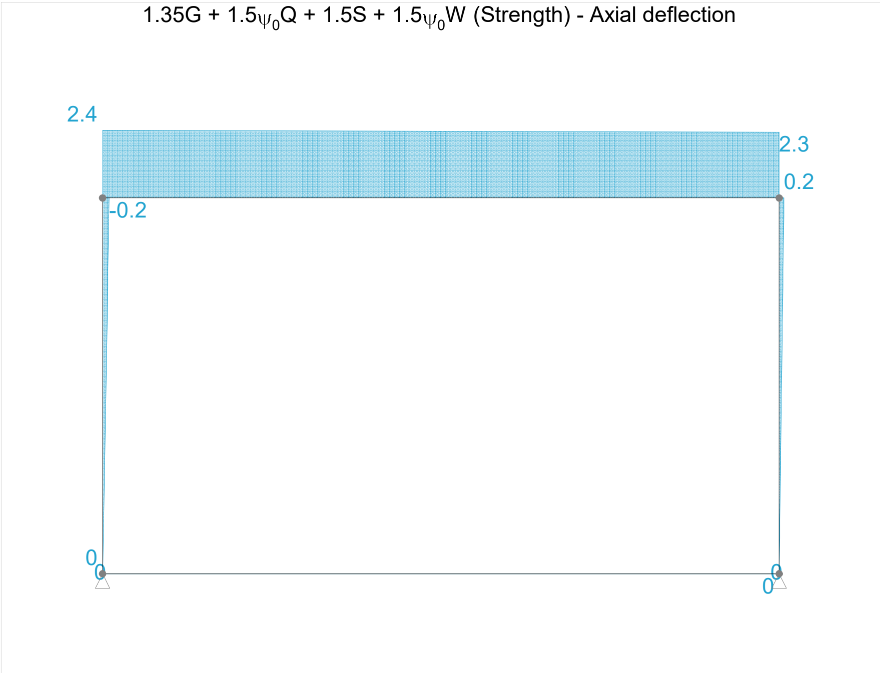
1.35G + 1.5ψ_0Q + 1.5S + 1.5ψ_0W (Strength) - Moment (kNm)



1.35G + 1.5ψ_0Q + 1.5S + 1.5ψ_0W (Strength) - Shear (kN)







Member results

Load combination: 1.35G + 1.5ψ_0Q + 1.5S + 1.5ψ_0W (Strength)

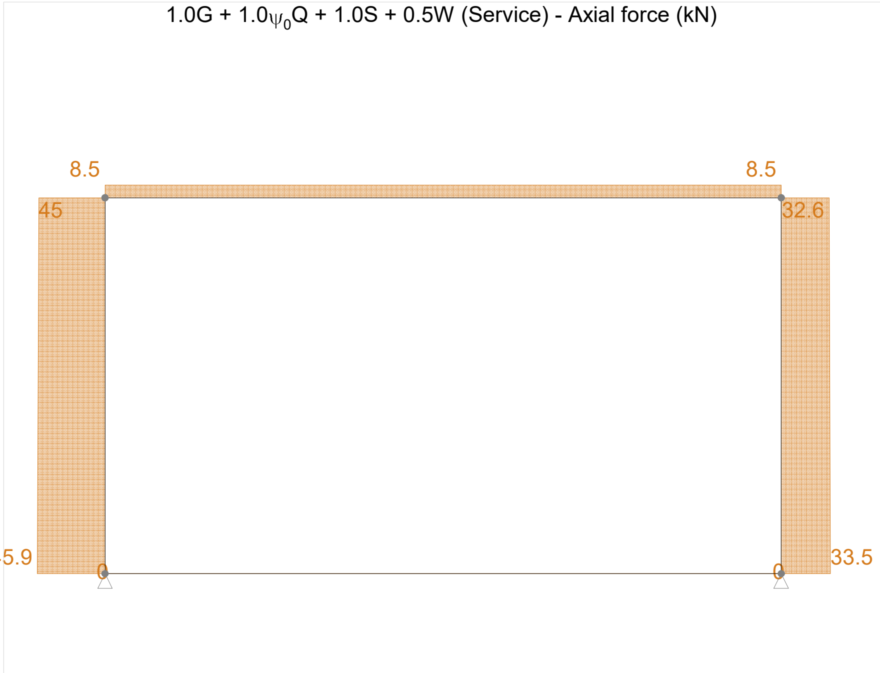
Member	Shear force		Moment			
	Pos (m)	Max abs (kN)	Pos (m)	Max (kNm)	Pos (m)	Min (kNm)
Member1	0	61.8	1.998	79.5	5.4	-29

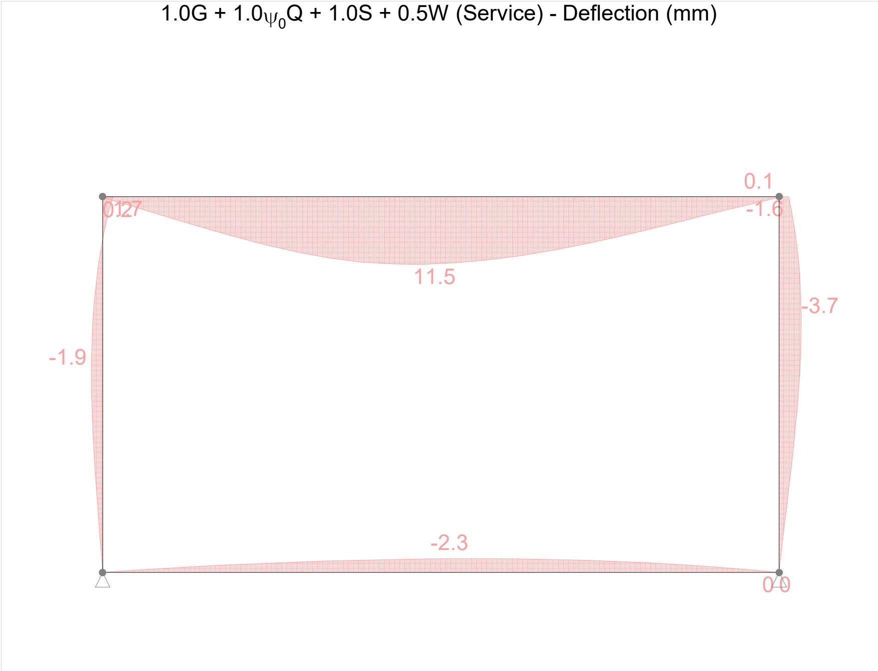
Load combination: 1.35G + 1.5ψ₀Q + 1.5S + 1.5ψ₀W (Strength)

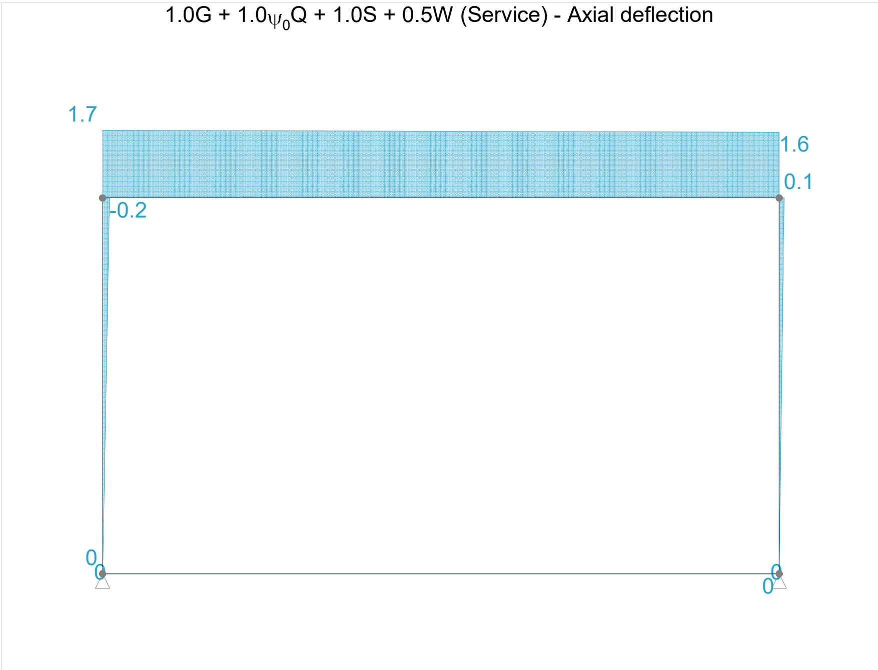
Member	Axial force			
	Pos (m)	Max (kN)	Pos (m)	Min (kN)
Member1	0	11.8	0	11.8

Load combination: 1.35G + 1.5ψ₀Q + 1.5S + 1.5ψ₀W (Strength)

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







Member results

Load combination: 1.0G + 1.0ψ_0Q + 1.0S + 0.5W (Service)

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

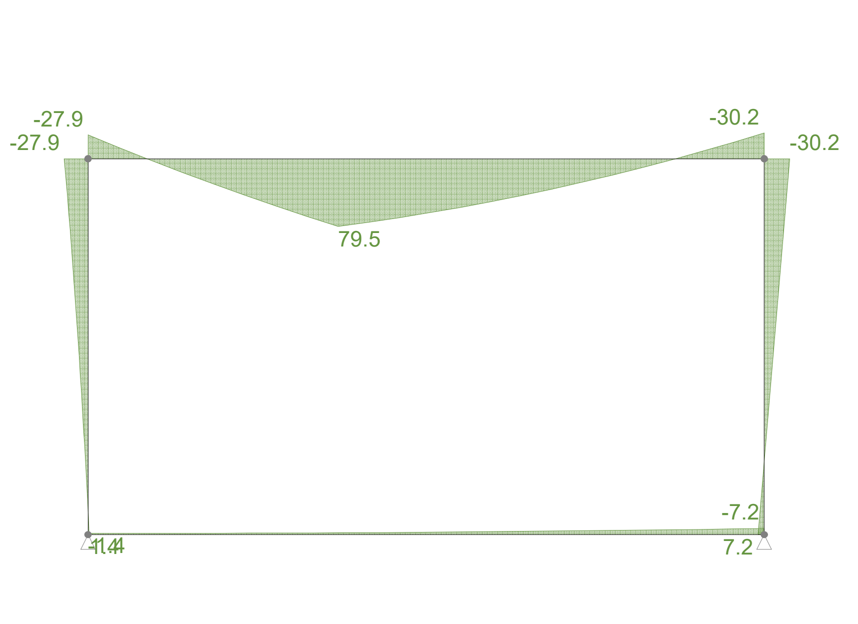
Load combination: 1.0G + 1.0ψ0Q + 1.0S + 0.5W (Service)

Member	Deflection			
	Pos (m)	Max (mm)	Pos (m)	Min (mm)
Member1	2.48	11.5	5.4	0.1

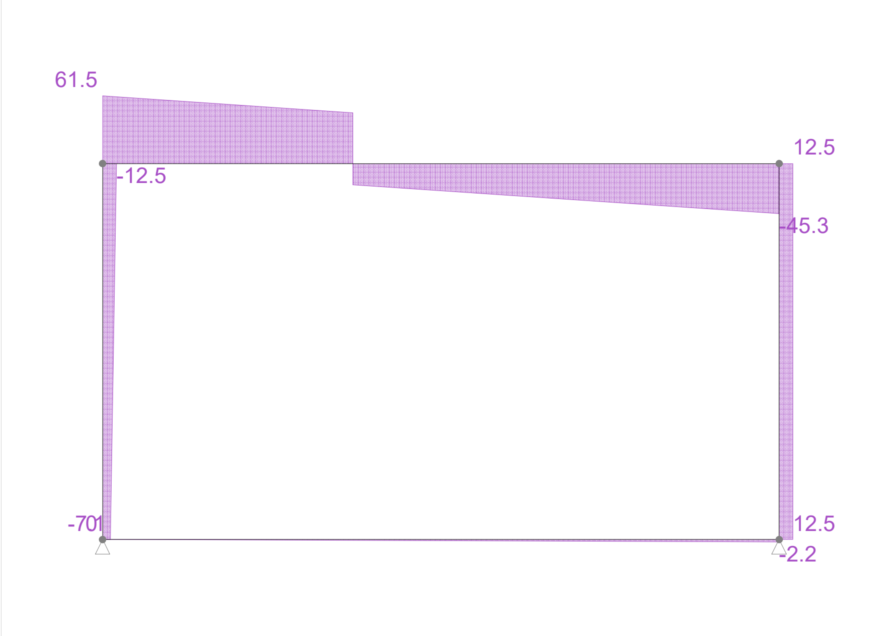
Load combination: 1.0G + 1.0ψ0Q + 1.0S + 0.5W (Service)

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

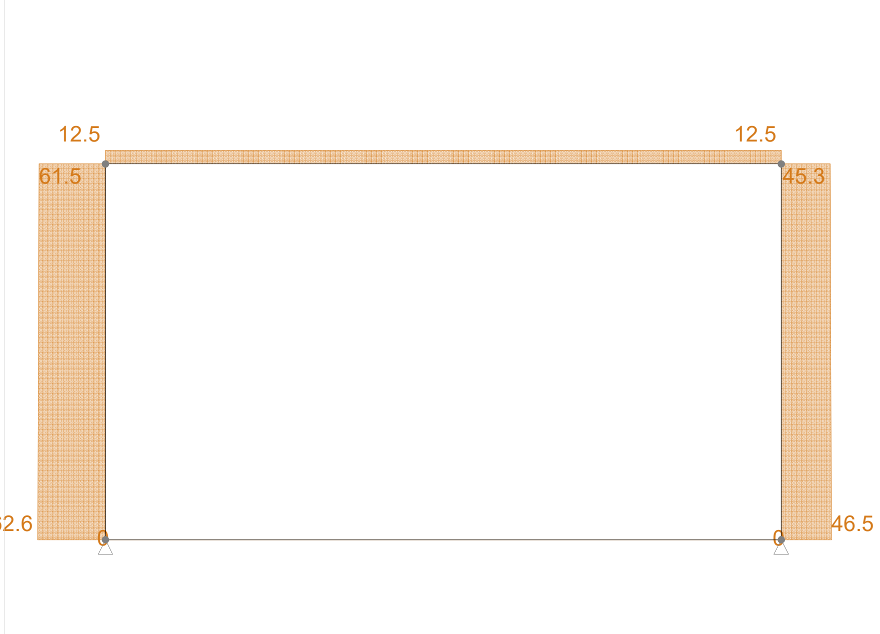
1.35G + 1.5ψ_0Q + 1.5ψ_0S + 1.5W (Strength) - Moment (kNm)



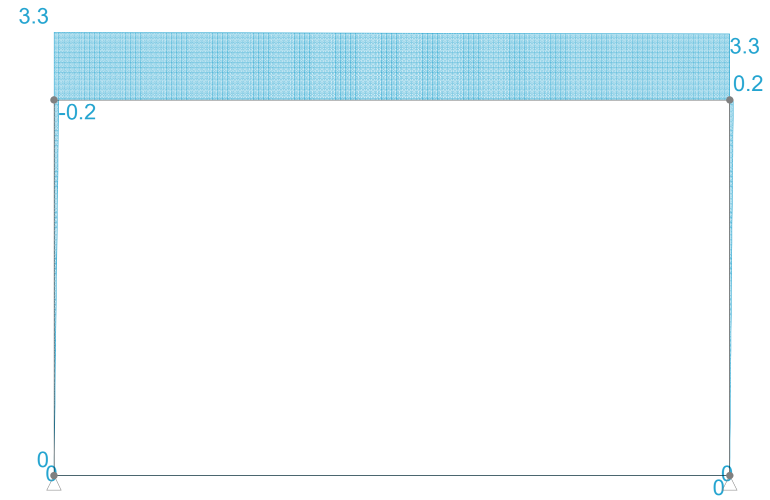
1.35G + 1.5ψ_0Q + 1.5ψ_0S + 1.5W (Strength) - Shear (kN)



1.35G + 1.5ψ_0Q + 1.5ψ_0S + 1.5W (Strength) - Axial force (kN)



1.35G + 1.5ψ_0Q + 1.5ψ_0S + 1.5W (Strength) - Axial deflection



Member results

Load combination: 1.35G + 1.5ψ0Q + 1.5ψ0S + 1.5W (Strength)

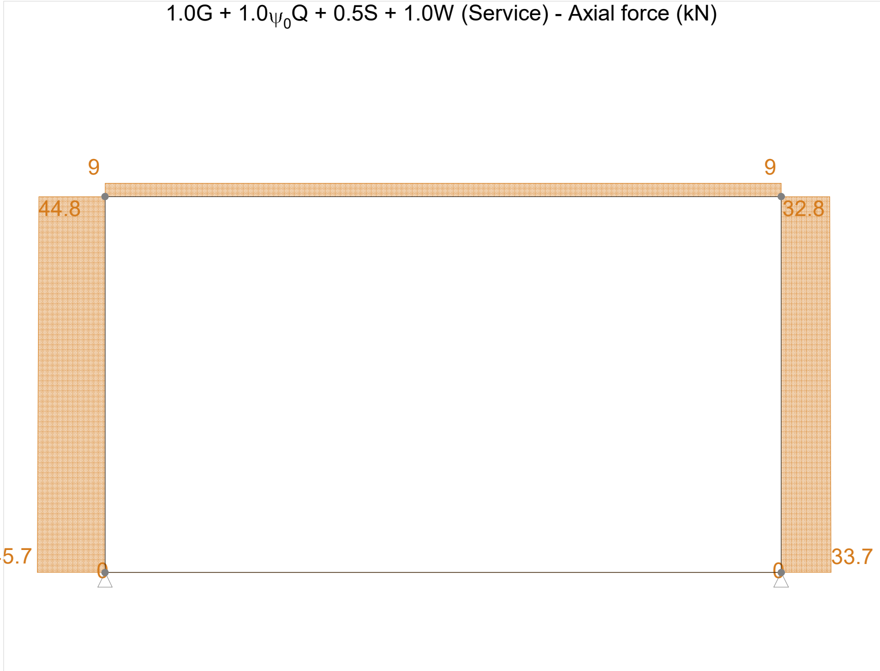
Member	Shear force		Moment			
	Pos (m)	Max abs (kN)	Pos (m)	Max (kNm)	Pos (m)	Min (kNm)
Member1	0	61.5	1.998	79.5	5.4	-30.2

Load combination: 1.35G + 1.5ψ0Q + 1.5ψ0S + 1.5W (Strength)

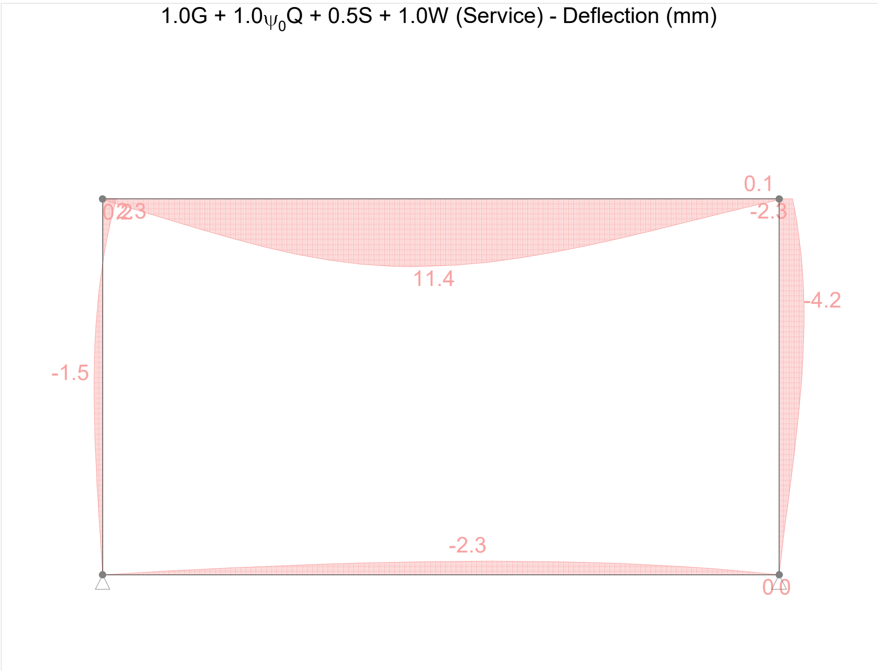
Member	Axial force			
	Pos (m)	Max (kN)	Pos (m)	Min (kN)
Member1	0	12.5	0	12.5

Load combination: 1.35G + 1.5ψ0Q + 1.5ψ0S + 1.5W (Strength)

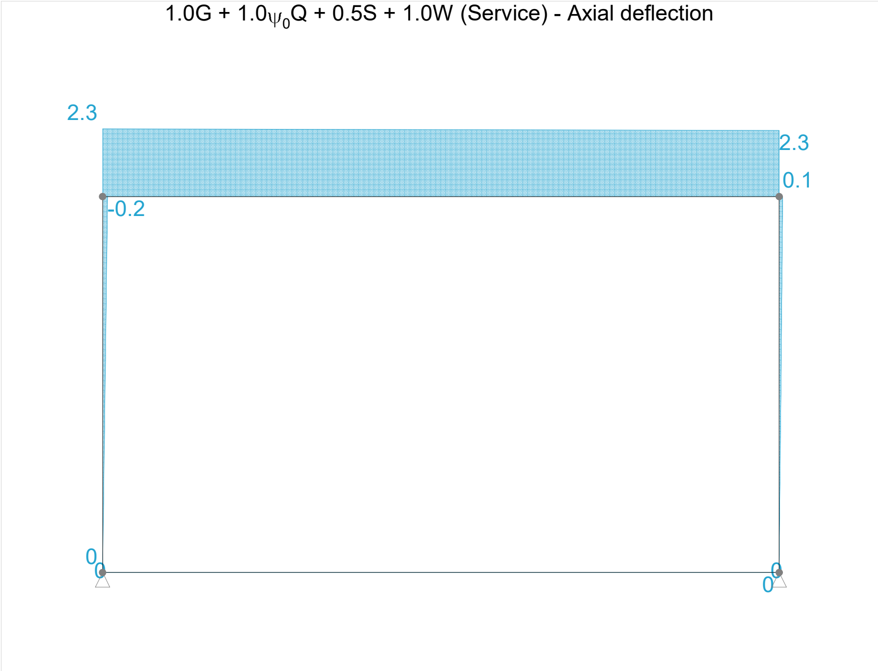
Member	Axial deflection			
	Pos (m)	Max (mm)	Pos (m)	Min (mm)
Member1	0	3.3	5.4	3.3



1.0G + 1.0ψ0Q + 0.5S + 1.0W (Service) - Axial force (kN)
9
9
44.8
32.8
45.7
33.7
0
0



1.0G + 1.0ψ0Q + 0.5S + 1.0W (Service) - Deflection (mm)
0.1
0.2.3
-2.3
11.4
-4.2
-1.5
-2.3
0.0



Member results

Load combination: 1.0G + 1.0ψ0Q + 0.5S + 1.0W (Service)

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

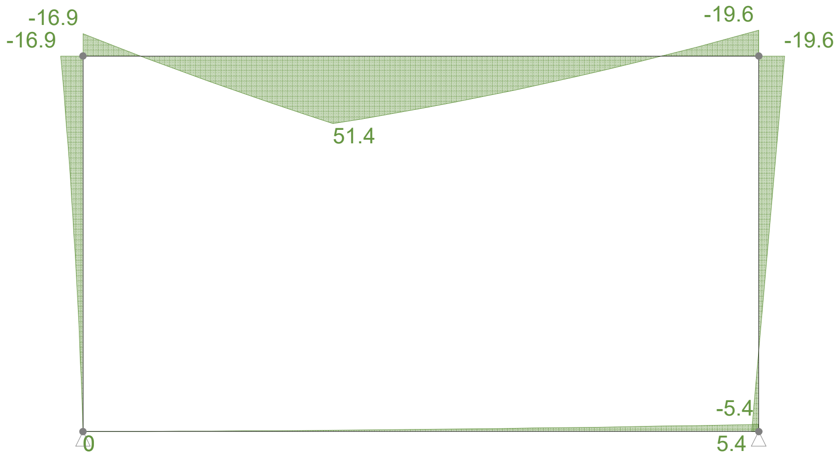
Load combination: 1.0G + 1.0ψ0Q + 0.5S + 1.0W (Service)

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

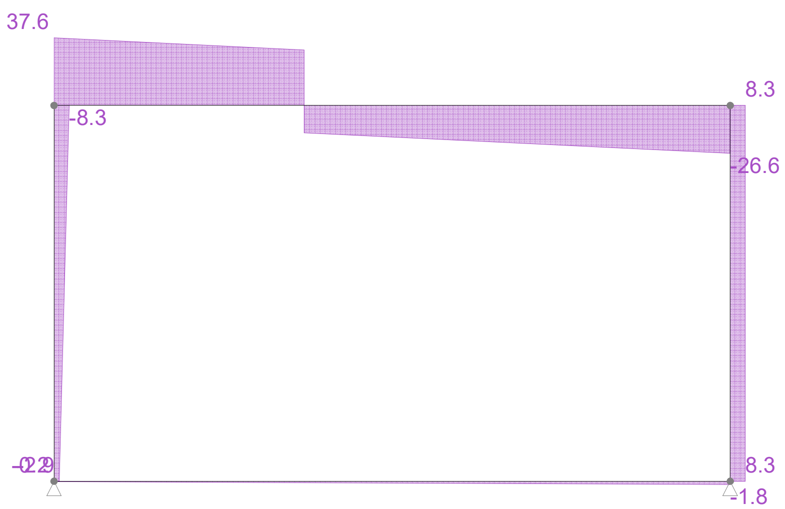
Load combination: 1.0G + 1.0ψ0Q + 0.5S + 1.0W (Service)

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

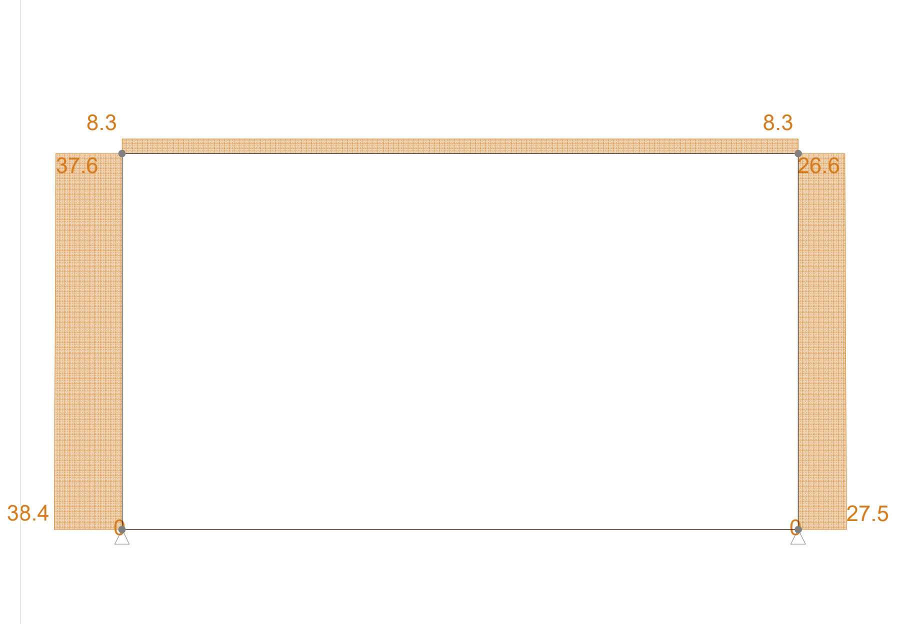
1.0G + 1.5W (Strength) - Moment (kNm)



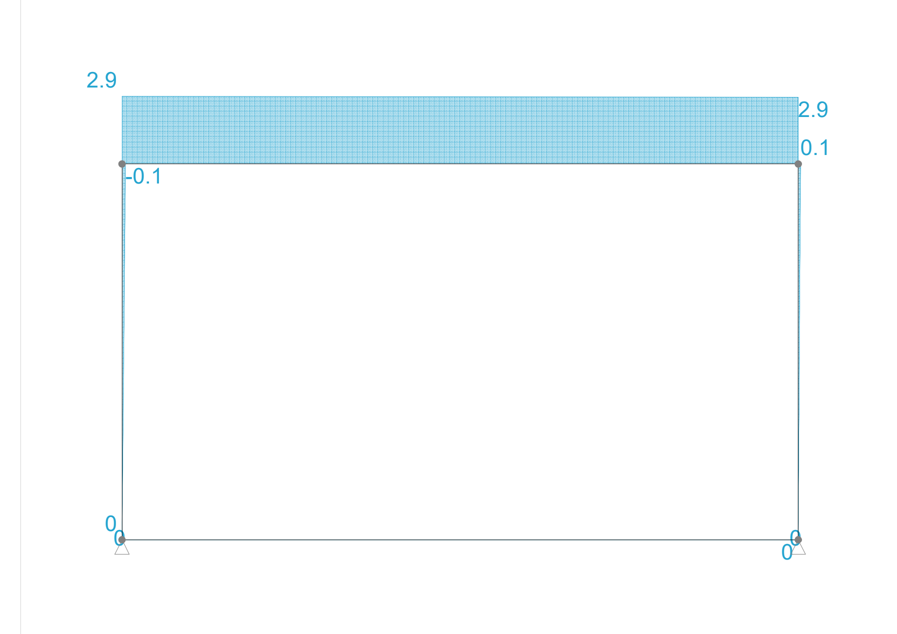
1.0G + 1.5W (Strength) - Shear (kN)



1.0G + 1.5W (Strength) - Axial force (kN)



1.0G + 1.5W (Strength) - Axial deflection



Member results

Load combination: 1.0G + 1.5W (Strength)

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

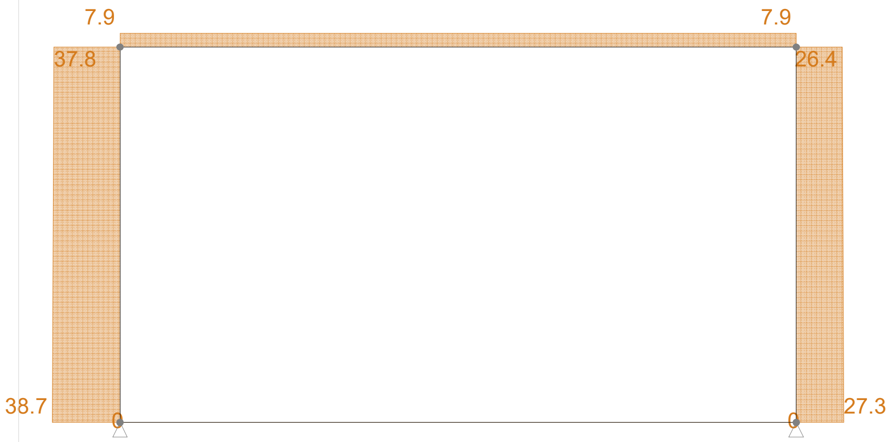
Load combination: 1.0G + 1.5W (Strength)

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

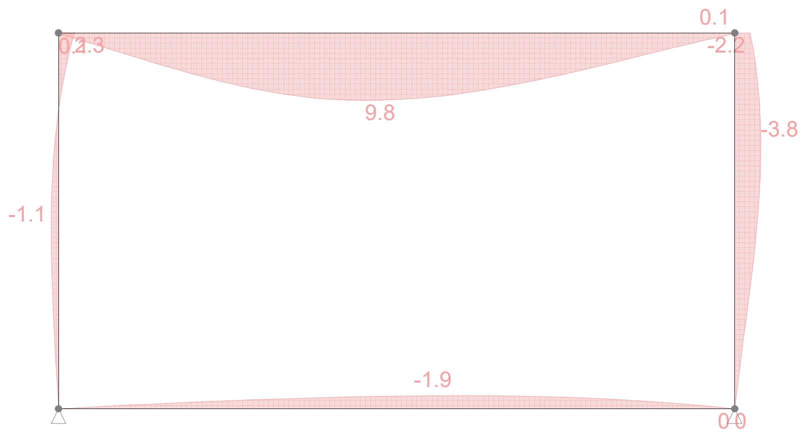
Load combination: 1.0G + 1.5W (Strength)

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

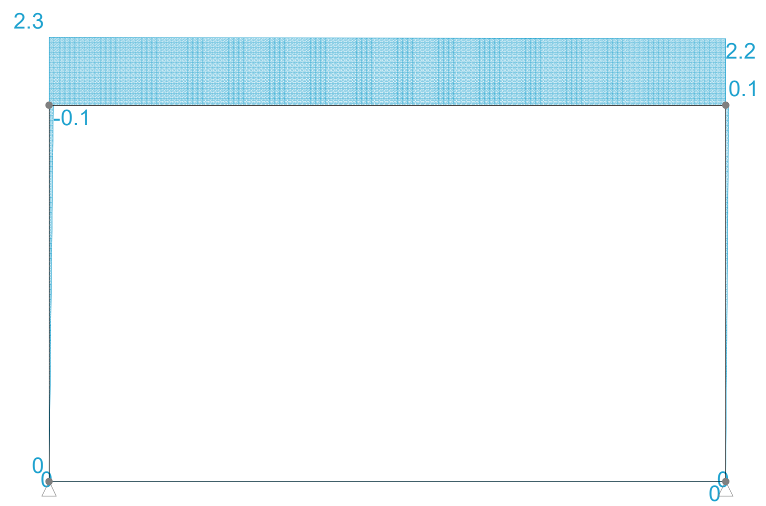
1.0G + 1.0W (Service) - Axial force (kN)



1.0G + 1.0W (Service) - Deflection (mm)



1.0G + 1.0W (Service) - Axial deflection



Member results

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 (m)	Max (mm)	Pos (m)	Min (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 6.1

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

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

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

<u>Member1 - Span 1</u>

Section details

Section type; UB 254x146x37 (BS4-1)

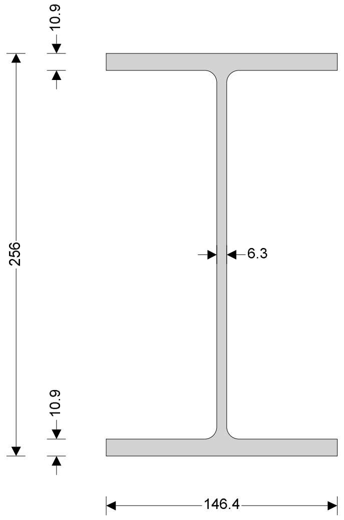
Steel grade - EN 10025-2:2004; S275

Nominal thickness of element; $t_{nom} = \max(t_f, t_w) =$ **10.9** mm

Nominal yield strength; $f_y =$ **275** N/mm^2

Nominal ultimate tensile strength; f_u = **410** N/mm^2

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



UB 254x146x37 (BS4-1)
Section depth, h, 256 mm
Section breadth, b, 146.4 mm
Mass of section, Mass, 37 kg/m
Flange thickness, t_f, 10.9 mm
Web thickness, t_w, 6.3 mm
Root radius, r, 7.6 mm
Area of section, A, 4717 mm^2
Radius of gyration about y-axis, i_y, 108 mm
Radius of gyration about z-axis, i_z, 35 mm
Elastic section modulus about y-axis, $W_{el.y}$, 432562 mm^3
Elastic section modulus about z-axis, $W_{el.z}$, 77957 mm^3
Plastic section modulus about y-axis, $W_{pl.y}$, 483231 mm^3
Plastic section modulus about z-axis, $W_{pl.z}$, 119374 mm^3
Second moment of area about y-axis, I_y, 55367902 mm^4
Second moment of area about z-axis, I_z, 5706456 mm^4

Lateral restraint

Both flanges have lateral restraint at supports only

Consider Combination 7 - 1.35G + 1.5Q + 1.5ψ0S + 1.5ψ0W (Strength)

Classification of cross sections - Section 5.5

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

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

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

$\alpha = \min([h / 2 + N_{Ed} / (2 \times t_w \times f_y) - (t_f + r)] / c, 1)$ = **0.516**

$c / t_w = 34.8 = 37.6 \times \varepsilon <= 396 \times \varepsilon / (13 \times \alpha - 1)$; Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

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

$c / t_f = 5.7 = 6.2 \times \varepsilon <= 9 \times \varepsilon$; Class 1

Section is class 1

Check compression - Section 6.2.4

Design compression force; N_{Ed} = **12.5** kN

Design resistance of section - eq 6.10; $N_{c,Rd} = N_{pl,Rd} = A \times f_y / \gamma_{M0}$ = **1297.1** kN

$N_{Ed} / N_{c,Rd}$ = **0.01**

PASS - Design compression resistance exceeds design compression

Slenderness ratio for y-y axis flexural buckling - Section 6.3.1.3

Critical buckling length; $L_{cr,y} = L_{m1\_s1}$ = **5400** mm

Critical buckling force; $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2$ = **3935.4** kN

Slenderness ratio for buckling - eq 6.50; $\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})}$ = **0.574**

Check y-y axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2; a

Imperfection factor - Table 6.1; α_y = **0.21**

Buckling reduction determination factor; $\phi_y = 0.5 \times (1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2)$ = **0.704**

Buckling reduction factor - eq 6.49; $\chi_y = \min(1 / (\phi_y + \sqrt{(\phi_y^2 - \bar{\lambda}_y^2)}), 1)$ = **0.9**

Design buckling resistance - eq 6.47; $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1}$ = **1166.8** kN

$N_{Ed} / N_{b,y,Rd}$ = **0.011**

PASS - Design buckling resistance exceeds design compression

Slenderness ratio for z-z axis flexural buckling - Section 6.3.1.3

Critical buckling length; $L_{cr,z} = L_{m1\_s1\_seg1} = \mathbf{5400}$ mm

Critical buckling force; $N_{cr,z} = \pi^2 \times E \times I_z / L_{cr,z}^2 = \mathbf{405.6}$ kN

Slenderness ratio for buckling - eq 6.50; $\bar{\lambda}_z = \sqrt{}(A \times f_y / N_{cr,z}) = \mathbf{1.788}$

Check z-z axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2; b

Imperfection factor - Table 6.1; $\alpha_z = \mathbf{0.34}$

Buckling reduction determination factor; $\phi_z = 0.5 \times (1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2) = \mathbf{2.369}$

Buckling reduction factor - eq 6.49; $\chi_z = \min(1 / (\phi_z + \sqrt{}(\phi_z^2 - \bar{\lambda}_z^2)), 1) = \mathbf{0.255}$

Design buckling resistance - eq 6.47; $N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1} = \mathbf{330.7}$ kN

$N_{Ed} / N_{b,z,Rd} = \mathbf{0.038}$

PASS - Design buckling resistance exceeds design compression

Check torsional and torsional-flexural buckling - Section 6.3.1.4

Torsional buckling length; $L_{cr,T} = L_{m1\_s1\_seg1\_R} = \mathbf{5400}$ mm

Distance from shear centre to centroid in y axis; $y_0 = \mathbf{0.0}$ mm

Distance from shear centre to centroid in z axis; $z_0 = \mathbf{0.0}$ mm

Radius of gyration; $i_0 = \sqrt{}(i_y^2 + i_z^2) = \mathbf{113.8}$ mm

Elastic critical torsional buckling force; $N_{cr,T} = 1 / i_0^2 \times (G \times I_t + \pi^2 \times E \times I_w / L_{cr,T}^2) = \mathbf{1426.8}$ kN

Torsion factor; $\beta_T = 1 - (y_0 / i_0)^2 = \mathbf{1}$

Elastic critical torsional-flexural buckling force

$N_{cr,TF} = N_{cr,y} / (2 \times \beta_T) \times [1 + N_{cr,T} / N_{cr,y} - \sqrt{}[(1 - N_{cr,T} / N_{cr,y})^2 + 4 \times (y_0 / i_0)^2 \times N_{cr,T} / N_{cr,y}]] = \mathbf{1426.8}$ kN

Elastic critical buckling force; $N_{cr} = \min(N_{cr,T}, N_{cr,TF}) = \mathbf{1426.8}$ kN

Slenderness ratio for torsional buckling - eq 6.52; $\bar{\lambda}_T = \sqrt{}[A \times f_y / N_{cr}] = \mathbf{0.953}$

Design resistance for torsional and torsional-flexural buckling - Section 6.3.1.1

Buckling curve - Table 6.2; b

Imperfection factor - Table 6.1; $\alpha_T = \mathbf{0.34}$

Buckling reduction determination factor; $\phi_T = 0.5 \times (1 + \alpha_T \times (\bar{\lambda}_T - 0.2) + \bar{\lambda}_T^2) = \mathbf{1.083}$

Buckling reduction factor - eq 6.49; $\chi_T = \min(1 / (\phi_T + \sqrt{}(\phi_T^2 - \bar{\lambda}_T^2)), 1) = \mathbf{0.627}$

Design buckling resistance - eq 6.47; $N_{b,T,Rd} = \chi_T \times A \times f_y / \gamma_{M1} = \mathbf{813}$ kN

$N_{Ed} / N_{b,T,Rd} = \mathbf{0.015}$

PASS - Design buckling resistance exceeds design compression

<u>Check design at start of span</u>

Check shear - Section 6.2.6

Height of web; $h_w = h - 2 \times t_f = \mathbf{234.2}$ mm; $\eta = \mathbf{1.000}$

$h_w / t_w = 37.2 = 40.2 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force; $V_{y,Ed} = \mathbf{66.3}$ kN

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

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

$V_{y,Ed} / V_{c,y,Rd} = \mathbf{0.237}$

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment; $M_{y,Ed} = \mathbf{30.6}$ kNm

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

$M_{y,Ed} / M_{c,y,Rd} = \mathbf{0.23}$

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6; k_c = **0.909**

$C_1 = 1 / k_c^2$ = **1.211**

Poissons ratio; ν = **0.3**

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

Unrestrained effective length; $L = 1.0 \times L_{m1\_s1\_seg1\_B}$ = **5400** mm

Elastic critical buckling moment; $M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **104.8** kNm

Slenderness ratio for lateral torsional buckling; $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **1.126**

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

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

Check buckling resistance - Section 6.3.2.1

Buckling curve - Table 6.5; b

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

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

LTB reduction determination factor; $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **1.099**

LTB reduction factor - eq 6.57; $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.623**

Modification factor; $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.964**

Modified LTB reduction factor - eq 6.58; $\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.646**

Design buckling resistance moment - eq 6.55; $M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1}$ = **85.8** kNm

$M_{y,Ed} / M_{b,y,Rd}$ = **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; $N_{y,lim} = \min(0.25 \times N_{pl,Rd}, 0.5 \times h_w \times t_w \times f_y / \gamma_{M0})$ = **202.9** kN

$N_{Ed} / N_{y,lim}$ = **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_y = -30.6 kNm / -31 kNm = **0.987**

α_y = -31 kNm / 68.7 kNm = **-0.451**

$C_{my} = 0.95 + 0.05 \times \alpha_y$ = **0.927**

; ψ_{LT} = -30.6 kNm / -31 kNm = **0.987**

α_{LT} = -31 kNm / 68.7 kNm = **-0.451**

$C_{mLT} = 0.95 + 0.05 \times \alpha_{LT}$ = **0.927**

Interaction factors k_{ij} for members susceptible to torsional deformations - Table B.2

Characteristic moment resistance; $M_{y,Rk} = W_{pl.y} \times f_y$ = **132.9** kNm

Characteristic moment resistance; $M_{z,Rk} = W_{pl.z} \times f_y$ = **32.8** kNm

Characteristic resistance to normal force; $N_{Rk} = A \times f_y$ = **1297.1** kN

Interaction factors; $k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}))$ = **0.931**

$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1})$ = **0.994**

Interaction formulae - eq 6.61 & eq 6.62; $N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **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

Check design 1998 mm along span

Check shear - Section 6.2.6

Height of web; $h_w = h - 2 \times t_f = \mathbf{234.2}$ mm; $\eta = \mathbf{1.000}$

$h_w / t_w = 37.2 = 40.2 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force; $V_{y,Ed} = \mathbf{48.2}$ kN

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

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

$V_{y,Ed} / V_{c,y,Rd} = \mathbf{0.173}$

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment; $M_{y,Ed} = \mathbf{83.8}$ kNm

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

$M_{y,Ed} / M_{c,y,Rd} = \mathbf{0.631}$

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6; $k_c = \mathbf{0.909}$

$C_1 = 1 / k_c^2 = \mathbf{1.211}$

Poissons ratio; $\nu = \mathbf{0.3}$

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

Unrestrained effective length; $L = 1.0 \times L_{m1\_s1\_seg1\_T} = \mathbf{5400}$ mm

Elastic critical buckling moment; $M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))} = \mathbf{104.8}$ kNm

Slenderness ratio for lateral torsional buckling; $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})} = \mathbf{1.126}$

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

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

Check buckling resistance - Section 6.3.2.1

Buckling curve - Table 6.5; b

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

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

LTB reduction determination factor; $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = \mathbf{1.099}$

LTB reduction factor - eq 6.57; $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = \mathbf{0.623}$

Modification factor; $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = \mathbf{0.964}$

Modified LTB reduction factor - eq 6.58; $\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2) = \mathbf{0.646}$

Design buckling resistance moment - eq 6.55; $M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1} = \mathbf{85.8}$ kNm

$M_{y,Ed} / M_{b,y,Rd} = \mathbf{0.976}$

PASS - Design buckling resistance moment exceeds design bending moment

Check bending and axial force - Section 6.2.9

Check combined bending and compression - Section 6.3.3

Equivalent uniform moment factors - Table B.3; $\psi_y = -30.6$ kNm / -31 kNm $= \mathbf{0.987}$

$\alpha_y = -31$ kNm / 68.7 kNm $= \mathbf{-0.451}$

$C_{my} = 0.95 + 0.05 \times \alpha_y = \mathbf{0.927}$

; $\psi_{LT} = -30.6$ kNm / -31 kNm $= \mathbf{0.987}$

$\alpha_{LT} = -31$ kNm / 68.7 kNm $= \mathbf{-0.451}$

$C_{mLT} = 0.95 + 0.05 \times \alpha_{LT} = \mathbf{0.927}$

Interaction factors k_{ij} for members susceptible to torsional deformations - Table B.2

Characteristic moment resistance; $M_{y,Rk} = W_{pl.y} \times f_y =$ **132.9** kNm

Characteristic moment resistance; $M_{z,Rk} = W_{pl.z} \times f_y =$ **32.8** kNm

Characteristic resistance to normal force; $N_{Rk} = A \times f_y =$ **1297.1** kN

Interaction factors; $k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1})) =$ **0.931**

$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1}) =$ **0.994**

Interaction formulae - eq 6.61 & eq 6.62; $N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) =$ **0.598**

$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) =$ **0.665**

PASS - Combined bending and compression checks are satisfied

<u>Check design at end of span</u>

Check shear - Section 6.2.6

Height of web; $h_w = h - 2 \times t_f =$ **234.2** mm; $\eta =$ **1.000**

$h_w / t_w = 37.2 = 40.2 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force; $V_{y,Ed} =$ **49.1** kN

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

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

$V_{y,Ed} / V_{c,y,Rd} =$ **0.176**

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment; $M_{y,Ed} =$ **31** kNm

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

$M_{y,Ed} / M_{c,y,Rd} =$ **0.233**

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6; $k_c =$ **0.909**

$C_1 = 1 / k_c^2 =$ **1.211**

Poissons ratio; $\nu =$ **0.3**

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

Unrestrained effective length; $L = 1.0 \times L_{m1\_s1\_seg1\_B} =$ **5400** mm

Elastic critical buckling moment; $M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))} =$ **104.8** kNm

Slenderness ratio for lateral torsional buckling; $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})} =$ **1.126**

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

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

Check buckling resistance - Section 6.3.2.1

Buckling curve - Table 6.5; b

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

Correction factor for rolled sections; $\beta =$ **0.75**

LTB reduction determination factor; $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] =$ **1.099**

LTB reduction factor - eq 6.57; $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) =$ **0.623**

Modification factor; $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) =$ **0.964**

Modified LTB reduction factor - eq 6.58; $\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2) =$ **0.646**

Design buckling resistance moment - eq 6.55; $M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1} = \mathbf{85.8}$ kNm

$M_{y,Ed} / M_{b,y,Rd} = \mathbf{0.361}$

PASS - Design buckling resistance moment exceeds design bending moment

Check bending and axial force - Section 6.2.9

Check combined bending and compression - Section 6.3.3

Equivalent uniform moment factors - Table B.3; $\psi_y = -30.6 \text{ kNm} / -31 \text{ kNm} = \mathbf{0.987}$

$\alpha_y = -31 \text{ kNm} / 68.7 \text{ kNm} = \mathbf{-0.451}$

$C_{my} = 0.95 + 0.05 \times \alpha_y = \mathbf{0.927}$

; $\psi_{LT} = -30.6 \text{ kNm} / -31 \text{ kNm} = \mathbf{0.987}$

$\alpha_{LT} = -31 \text{ kNm} / 68.7 \text{ kNm} = \mathbf{-0.451}$

$C_{mLT} = 0.95 + 0.05 \times \alpha_{LT} = \mathbf{0.927}$

Interaction factors k_{ij} for members susceptible to torsional deformations - Table B.2

Characteristic moment resistance; $M_{y,Rk} = W_{pl.y} \times f_y = \mathbf{132.9}$ kNm

Characteristic moment resistance; $M_{z,Rk} = W_{pl.z} \times f_y = \mathbf{32.8}$ kNm

Characteristic resistance to normal force; $N_{Rk} = A \times f_y = \mathbf{1297.1}$ kN

Interaction factors; $k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1})) = \mathbf{0.931}$

$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1}) = \mathbf{0.994}$

Interaction formulae - eq 6.61 & eq 6.62; $N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) = \mathbf{0.228}$

$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) = \mathbf{0.27}$

PASS - Combined bending and compression checks are satisfied

<u>**Consider Combination 5 - 1.0G + 1.0Q + 0.5S (Service)**</u>

<u>**Check design 2494 mm along span**</u>

Check y-y axis deflection - Section 7.2.1

Maximum deflection; $\delta_y = \mathbf{12.2}$ mm

Allowable deflection; $\delta_{y,Allowable} = L_{m1\_s1} / 250 = \mathbf{21.6}$ mm

$\delta_y / \delta_{y,Allowable} = \mathbf{0.566}$

PASS - Allowable deflection exceeds design deflection

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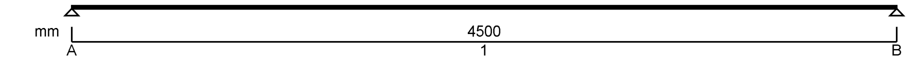
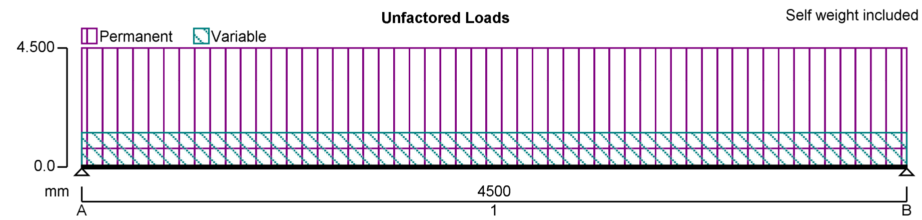
	Section Building Control Approval				Sheet no./rev. 107	
	Calc. by RC	Date 26/06/2017	Chk'd by MD	Date 26/06/2017	App'd by	Date

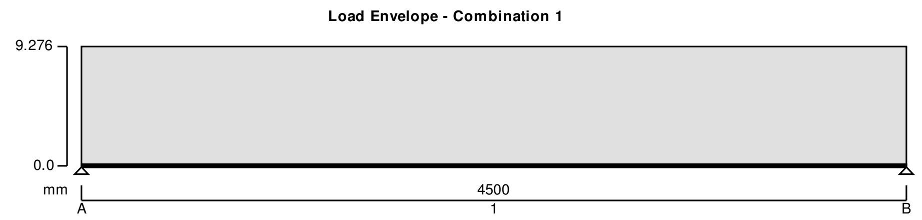
<u>NEW STEEL BEAM THAT SUPPORTS STAIRWAY WALL</u>

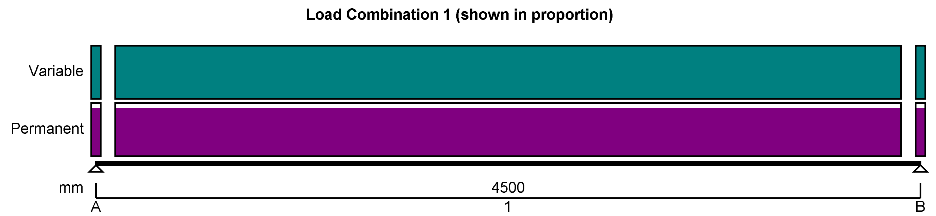
In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

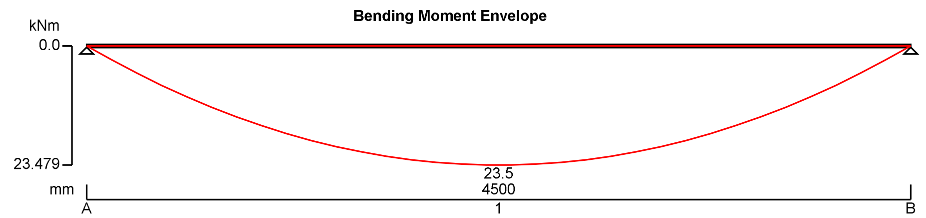
TEDDS calculation version 3.0.13

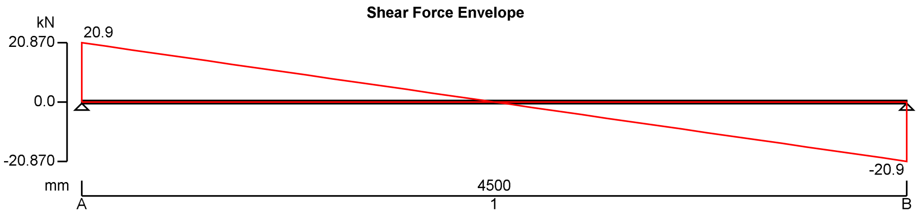












Support conditions

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

Applied loading

Beam loads

Steel self weight - Permanent self weight of beam × 1

Wall above (timber stud) - 0.5kN/m2 x height - Permanent full UDL 4.5 kN/m

Floor dead load - 0.8kN/m2 x 0.85m - Permanent full UDL 0.7 kN/m

Floor live load - 1.5kN/m2 x 0.85m - Variable full UDL 1.3 kN/m

Load combinations

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

Analysis results

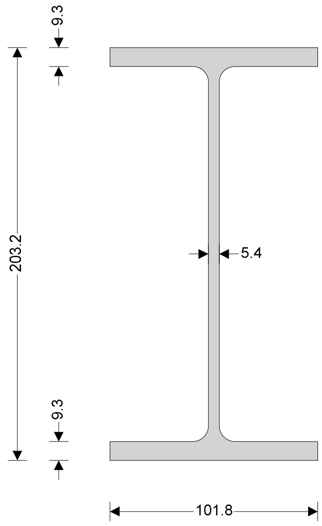
Maximum moment;	M_{max} = **23.5** kNm;	M_{min} = **0** kNm
Maximum shear;	V_{max} = **20.9** kN;	V_{min} = **-20.9** kN
Deflection;	δ_{max} = **8.1** mm;	δ_{min} = **0** mm
Maximum reaction at support A;	R_{A_max} = **20.9** kN;	R_{A_min} = **20.9** kN
Unfactored permanent load reaction at support A;	$R_{A\_Permanent}$ = **12.2** kN	
Unfactored variable load reaction at support A;	$R_{A\_Variable}$ = **2.9** kN	
Maximum reaction at support B;	R_{B_max} = **20.9** kN;	R_{B_min} = **20.9** kN
Unfactored permanent load reaction at support B;	$R_{B\_Permanent}$ = **12.2** kN	
Unfactored variable load reaction at support B;	$R_{B\_Variable}$ = **2.9** kN	

Section details

Section type;	**UB 203x102x23 (BS4-1)**
Steel grade;	**S275**

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

Nominal thickness of element;	$t = \max(t_f, t_w)$ = **9.3** mm
Nominal yield strength;	f_y = **275** N/mm^2
Nominal ultimate tensile strength;	f_u = **410** N/mm^2
Modulus of elasticity;	E = **210000** N/mm^2



Partial factors - Section 6.1

Resistance of cross-sections; γ_{M0} = **1.00**

Resistance of members to instability; γ_{M1} = **1.00**

Resistance of tensile members to fracture; γ_{M2} = **1.10**

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

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

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

Effective length factor for torsion; $K_{LT.A}$ = **1.200**; + 2 × h

$K_{LT.B}$ = **1.200**; + 2 × h

Classification of cross sections - Section 5.5

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

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

Width of section; c = d = **169.4** mm

$c / t_w = 33.9 \times \varepsilon <= 72 \times \varepsilon$; Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section; $c = (b - t_w - 2 \times r) / 2$ = **40.6** mm

$c / t_f = 4.7 \times \varepsilon <= 9 \times \varepsilon$; Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web; $h_w = h - 2 \times t_f$ = **184.6** mm

Shear area factor; η = **1.000**

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

Shear buckling resistance can be ignored

Design shear force; $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **20.9** kN

Shear area - cl 6.2.6(3); $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **1238** mm²

Design shear resistance - cl 6.2.6(2); $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0}$ = **196.6** kN

PASS - Design shear resistance exceeds design shear force

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

Design bending moment; $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **23.5** kNm

Design bending resistance moment - eq 6.13; $M_{c,Rd} = M_{pl,Rd} =$
$W_{pl.y} \times f_y / \gamma_{M0} =$ **64.4** kNm

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6; $k_c =$ **0.94**
$C_1 = 1 / k_c^2 =$ **1.132**

Curvature factor; $g = \sqrt{[1 - (I_z / I_y)]} =$ **0.96**

Poissons ratio; $\nu =$ **0.3**

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

Unrestrained length; $L = 1.2 \times L_{s1} + 2 \times h =$ **5806** mm

Elastic critical buckling moment; $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} =$ **30.4** kNm

Slenderness ratio for lateral torsional buckling; $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})} =$ **1.454**

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

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

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5; b

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

Correction factor for rolled sections; $\beta =$ **0.75**

LTB reduction determination factor; $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] =$ **1.473**

LTB reduction factor - eq 6.57; $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) =$ **0.447**

Modification factor; $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) =$ **0.996**

Modified LTB reduction factor - eq 6.58; $\chi_{LT,mod} = \min(\chi_{LT} / f, 1) =$ **0.449**

Design buckling resistance moment - eq 6.55; $M_{b,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1} =$ **28.9** kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection; $\delta_{lim} = L_{s1} / 250 =$ **18** mm

Maximum deflection span 1; $\delta = \max(abs(\delta_{max}), abs(\delta_{min})) =$ **8.125** mm

PASS - Maximum deflection does not exceed deflection limit

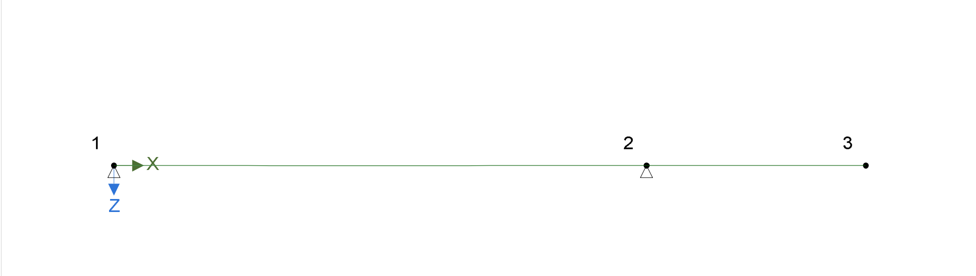
EXISTING RC CANTILEVER SLAB ANALYSIS

Tedds calculation version 1.0.13

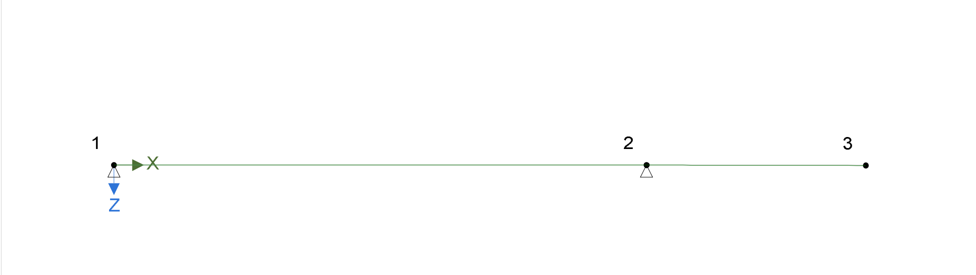
Results

Total deflection

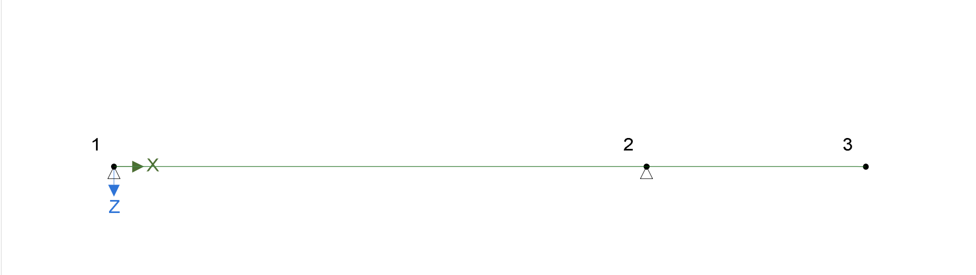
Self Weight - Total deflection @ 10x



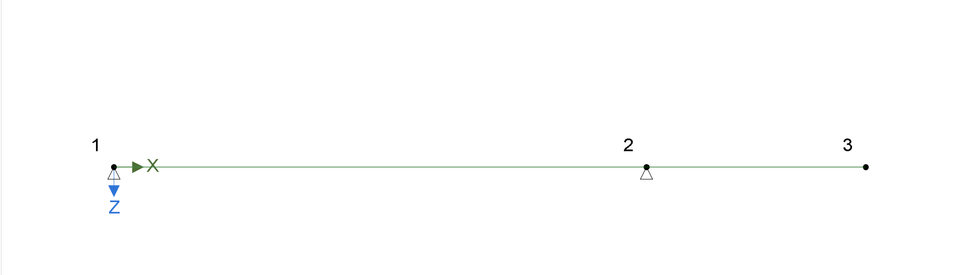
Cavity wall self weight - 7.5kN/m - Total deflection @ 10x



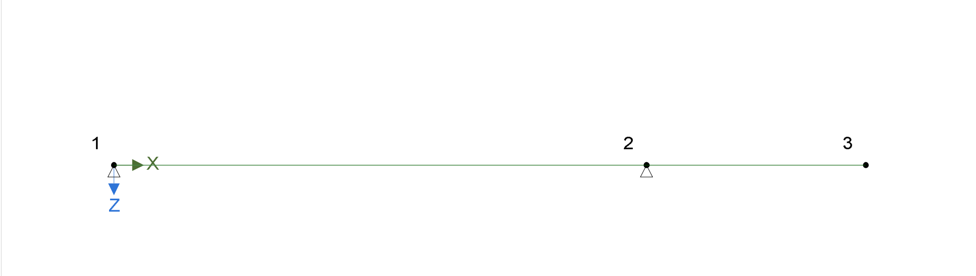
Roof dead load - 1.0kN/m2 - Total deflection @ 10x



Roof live load - 0.6kN/m2 - Total deflection @ 10x



Bath tub filled - 10kN/m3 - Total deflection @ 10x



Node deflections

Load case: Self Weight

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

Load case: Bath tub filled - 10kN/m3

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

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

Total base reactions

Load case/combination	Force	
	FX (kN)	FZ (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 (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (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

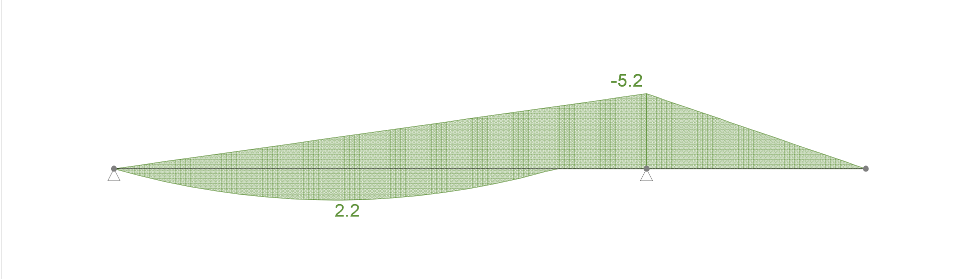
Load case: Roof live load - 0.6kN/m2

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

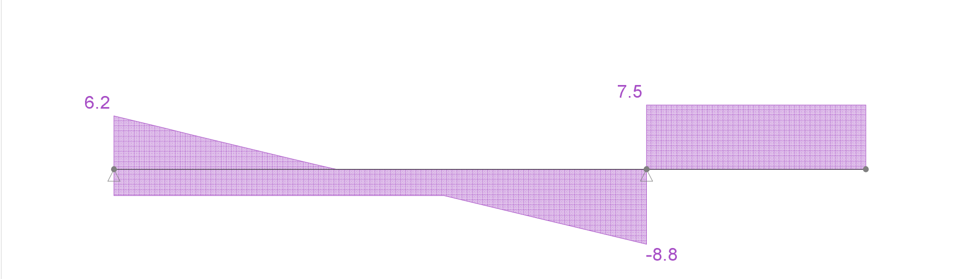
Load case: Bath tub filled - 10kN/m3

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (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



All load cases - Moment envelope (kNm)



All load cases - Shear envelope (kN)

Element results

Envelope - All load cases

Element	Shear force		Moment			
	Pos (m)	Max abs (kN)	Pos (m)	Max (kNm)	Pos (m)	Min (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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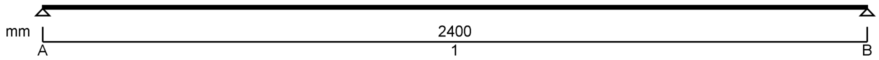
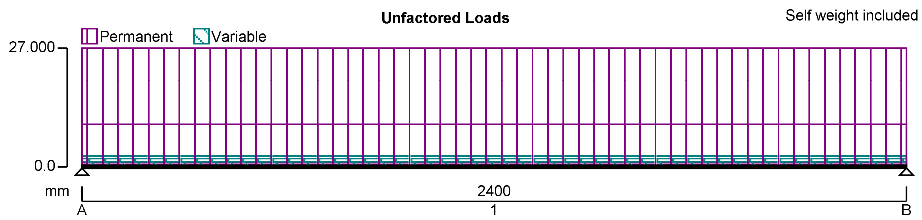
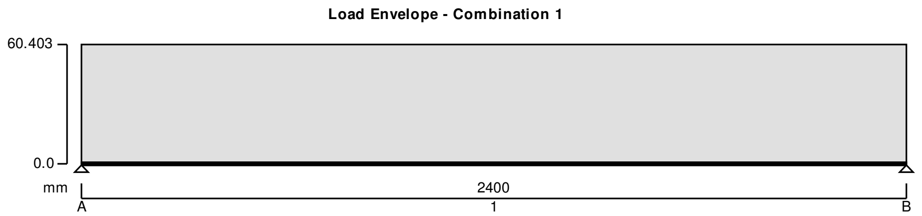
STEEL BEAM THAT SUPPORTS THE BACK WALL OF THE OUTRIGGER

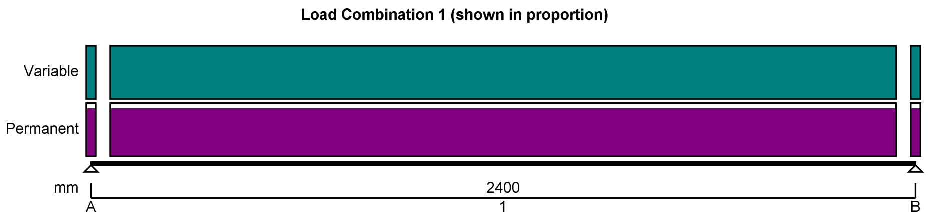
In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

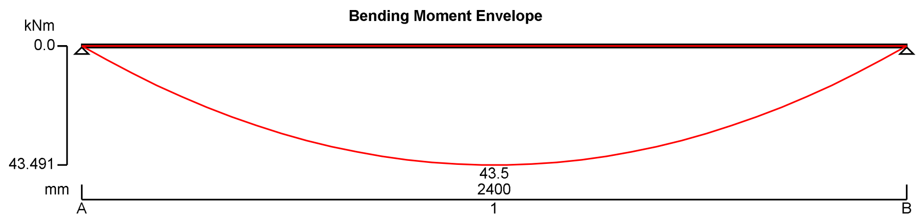
TEDDS calculation version 3.0.13

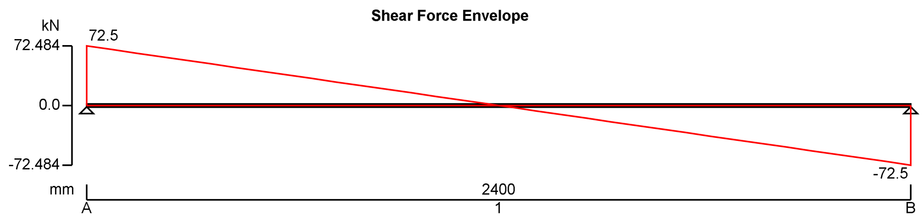












Support conditions

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

Applied loading

Beam loads

Steel self weight - Permanent self weight of beam × 1
Dead load above - Permanent full UDL 27 kN/m
Live load above - Variable full UDL 2.5 kN/m
Terrace roof dead load - 1kN/m2 x 1.25m - Permanent full UDL 1 kN/m
Terrace roof live load - 1.5kN/m2 x 1.25m - Variable full UDL 1.9 kN/m
Floor dead load - 0.8kN/m2 x 0.75m - Permanent full UDL 0.6 kN/m
Floor live load - 1.5kN/m2 x 0.75m - Variable full UDL 1.2 kN/m
Solid 215mm brick wall - 18kN/m3 x 0.215m x 3m - Permanent full UDL 9.675 kN/m

Load combinations

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

Analysis results

Maximum moment;	M_{max} = **43.5** kNm;	M_{min} = **0** kNm
Maximum shear;	V_{max} = **72.5** kN;	V_{min} = **-72.5** kN
Deflection;	δ_{max} = **3.9** mm;	δ_{min} = **0** mm
Maximum reaction at support A;	R_{A_max} = **72.5** kN;	R_{A_min} = **72.5** kN
Unfactored permanent load reaction at support A;	$R_{A\_Permanent}$ = **46.2** kN	
Unfactored variable load reaction at support A;	$R_{A\_Variable}$ = **6.7** kN	
Maximum reaction at support B;	R_{B_max} = **72.5** kN;	R_{B_min} = **72.5** kN
Unfactored permanent load reaction at support B;	$R_{B\_Permanent}$ = **46.2** kN	
Unfactored variable load reaction at support B;	$R_{B\_Variable}$ = **6.7** kN	

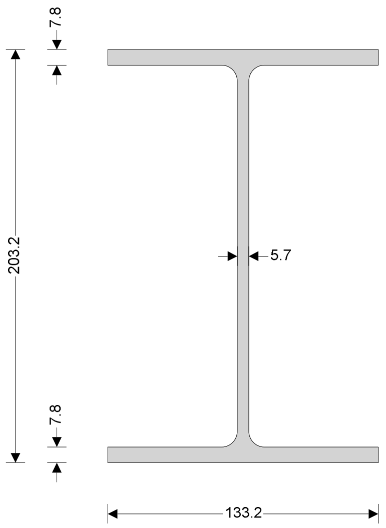
Section details

Section type;	**UB 203x133x25 (BS4-1)**
Steel grade;	**S275**

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

Nominal thickness of element;	$t = \max(t_f, t_w)$ = **7.8** mm
Nominal yield strength;	f_y = **275** N/mm^2
Nominal ultimate tensile strength;	f_u = **410** N/mm^2

Modulus of elasticity; $E = \mathbf{210000}\ N/mm^2$



Partial factors - Section 6.1

Resistance of cross-sections; $\gamma_{M0} = \mathbf{1.00}$

Resistance of members to instability; $\gamma_{M1} = \mathbf{1.00}$

Resistance of tensile members to fracture; $\gamma_{M2} = \mathbf{1.10}$

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis; $K_y = \mathbf{1.000}$

Effective length factor in minor axis; $K_z = \mathbf{1.000}$

Effective length factor for torsion; $K_{LT.A} = \mathbf{1.200}$; $+ 2 \times h$

$K_{LT.B} = \mathbf{1.200}$; $+ 2 \times h$

Classification of cross sections - Section 5.5

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

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

Width of section; $c = d = \mathbf{172.4}$ mm

$c / t_w = 32.7 \times \varepsilon <= 72 \times \varepsilon$; Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section; $c = (b - t_w - 2 \times r) / 2 = \mathbf{56.1}$ mm

$c / t_f = 7.8 \times \varepsilon <= 9 \times \varepsilon$; Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web; $h_w = h - 2 \times t_f = \mathbf{187.6}$ mm

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

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

Shear buckling resistance can be ignored

Design shear force; $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = \mathbf{72.5}$ kN

Shear area - cl 6.2.6(3); $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = \mathbf{1282}\ mm^2$

Design shear resistance - cl 6.2.6(2); $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0} = \mathbf{203.5}$ kN

PASS - Design shear resistance exceeds design shear force

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

Design bending moment; $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) =$ **43.5** kNm

Design bending resistance moment - eq 6.13; $M_{c,Rd} = M_{pl,Rd} =$

$W_{pl.y} \times f_y / \gamma_{M0} =$ **70.9** kNm

Slenderness ratio for lateral torsional buckling

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

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

Curvature factor; $g = \sqrt{[1 - (I_z / I_y)]} =$ **0.932**

Poissons ratio; $\nu =$ **0.3**

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

Unrestrained length; $L = 1.2 \times L_{s1} + 2 \times h =$ **3286** mm

Elastic critical buckling moment; $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} =$ **95.4** kNm

Slenderness ratio for lateral torsional buckling; $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})} =$ **0.862**

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

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

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5; b

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

Correction factor for rolled sections; $\beta =$ **0.75**

LTB reduction determination factor; $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] =$ **0.857**

LTB reduction factor - eq 6.57; $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) =$ **0.782**

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

Modified LTB reduction factor - eq 6.58; $\chi_{LT,mod} = \min(\chi_{LT} / f, 1) =$ **0.806**

Design buckling resistance moment - eq 6.55; $M_{b,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1} =$ **57.1** kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection; $\delta_{lim} = \min(10 \text{ mm}, L_{s1} / 500) =$ **4.8** mm

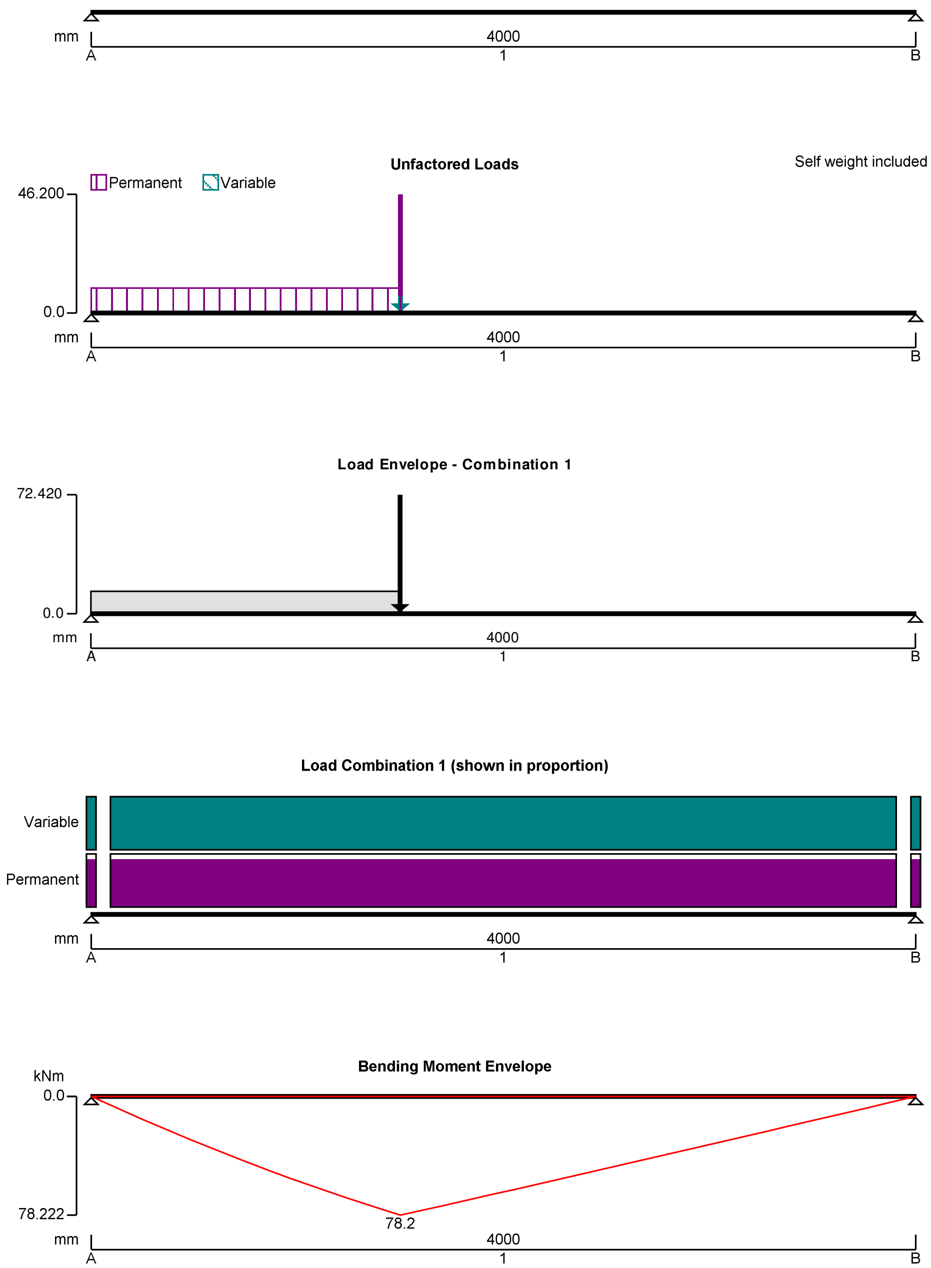
Maximum deflection span 1; $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) =$ **3.878** mm

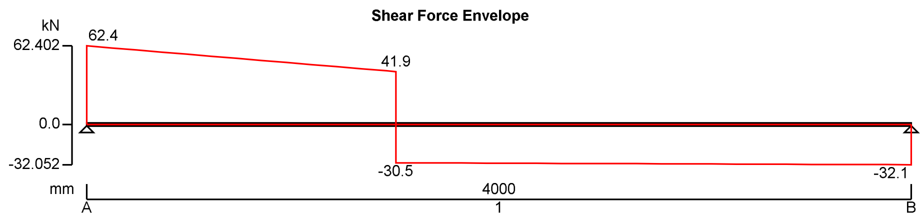
PASS - Maximum deflection does not exceed deflection limit

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





Support conditions

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

Applied loading

Beam loads	Steel self weight - Permanent self weight of beam × 1 SB2 Dead load - Permanent point load 46.2 kN at 1500 mm SB2 Live load - Variable point load 6.7 kN at 1500 mm Solid 215mm brick wall - 18kN/m3 x 0.215m x 2.5m - Permanent partial UDL 9.675 kN/m from 0 mm to 1500 mm

Load combinations

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

Analysis results

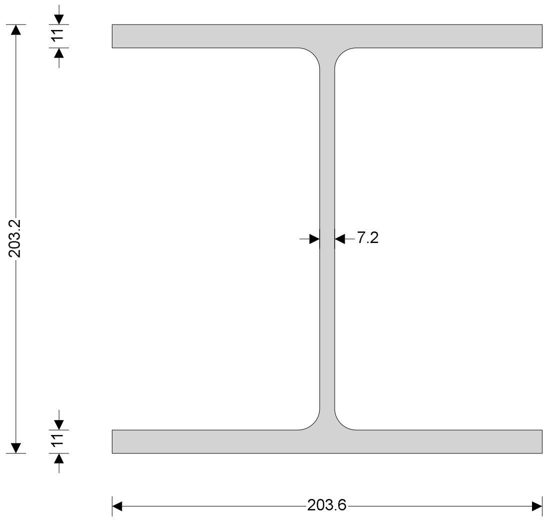
Maximum moment;	M_{max} = **78.2** kNm;	M_{min} = **0** kNm
Maximum shear;	V_{max} = **62.4** kN;	V_{min} = **-32.1** kN
Deflection;	δ_{max} = **8** mm;	δ_{min} = **0** mm
Maximum reaction at support A;	R_{A_max} = **62.4** kN;	R_{A_min} = **62.4** kN
Unfactored permanent load reaction at support A;	$R_{A\_Permanent}$ = **41.6** kN	
Unfactored variable load reaction at support A;	$R_{A\_Variable}$ = **4.2** kN	
Maximum reaction at support B;	R_{B_max} = **32.1** kN;	R_{B_min} = **32.1** kN
Unfactored permanent load reaction at support B;	$R_{B\_Permanent}$ = **21** kN	
Unfactored variable load reaction at support B;	$R_{B\_Variable}$ = **2.5** kN	

Section details

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

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

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



Partial factors - Section 6.1

Resistance of cross-sections; γ_{M0} = **1.00**

Resistance of members to instability; γ_{M1} = **1.00**

Resistance of tensile members to fracture; γ_{M2} = **1.10**

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

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

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

Effective length factor for torsion; $K_{LT.A}$ = **1.200**; + 2 × h

$K_{LT.B}$ = **1.200**; + 2 × h

Classification of cross sections - Section 5.5

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

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

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

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

Outstand flanges - Table 5.2 (sheet 2 of 3)

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

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

Section is class 1

Check shear - Section 6.2.6

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

Shear area factor; η = **1.000**

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

Shear buckling resistance can be ignored

Design shear force; $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **62.4** kN

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

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

PASS - Design shear resistance exceeds design shear force

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

Design bending moment; $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **78.2** kNm

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

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6; $k_c =$ **0.94**
$C_1 = 1 / k_c^2 =$ **1.132**

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

Poissons ratio; $\nu =$ **0.3**

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

Unrestrained length; $L = 1.2 \times L_{s1} + 2 \times h =$ **5206** mm

Elastic critical buckling moment; $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} =$ **257.1** kNm

Slenderness ratio for lateral torsional buckling; $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})} =$ **0.729**

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

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

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5; b

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

Correction factor for rolled sections; $\beta =$ **0.75**

LTB reduction determination factor; $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] =$ **0.756**

LTB reduction factor - eq 6.57; $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) =$ **0.855**

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

Modified LTB reduction factor - eq 6.58; $\chi_{LT,mod} = \min(\chi_{LT} / f, 1) =$ **0.881**

Design buckling resistance moment - eq 6.55; $M_{b,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1} =$ **120.5** kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection; $\delta_{lim} = \min(10 \text{ mm}, L_{s1} / 500) =$ **8** mm

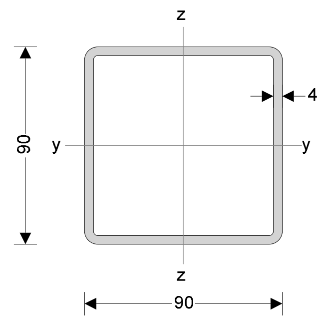
Maximum deflection span 1; $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) =$ **7.962** mm

PASS - Maximum deflection does not exceed deflection limit

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

TEDDS calculation version 1.0.11



Column and loading details

Column details

Column section;	**SHS 90x90x4.0**
System length for buckling about y axis;	L_y = ;**3000**; mm
System length for buckling about z axis;	L_z = ;**3000**; 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_{Ed} = **21** kN; (Compression)
Moment about y axis at end 1;	$M_{y,Ed1}$ = **0.0** kNm
Moment about y axis at end 2;	$M_{y,Ed2}$ = **0.0** kNm
Moment about z axis at end 1;	$M_{z,Ed1}$ = **0.0** kNm
Moment about z axis at end 2;	$M_{z,Ed2}$ = **0.0** kNm
Shear force parallel to z axis;	$V_{z,Ed}$ = **0** kN
Shear force parallel to y axis;	$V_{y,Ed}$ = **0** kN

Material details

Steel grade;	**S275**
Yield strength;	f_y = **275** N/mm^2
Ultimate strength;	f_u = **410** N/mm^2
Modulus of elasticity;	E = **210** kN/mm^2
Poisson’s ratio;	ν = **0.3**
Shear modulus;	$G = E / [2 \times (1 + \nu)]$ = **80.8** kN/mm^2

Buckling length for flexural buckling about y axis

Buckling length; L_{cr_y} = **6000** mm

Buckling length for flexural buckling about z axis

Buckling length; L_{cr_z} = **6000** mm

Section classification

Web section classification (Table 5.2)

Coefficient depending on f_y; $\varepsilon = \sqrt{(235 \text{ N/mm}^2 / f_y)}$ = **0.924**

Depth between fillets; $c_w = h - 3 \times t =$ **78.0** mm

Ratio of c/t; $ratio_w = c_w / t =$ **19.50**

Length of web taken by axial load; $l_w = \min(N_{Ed} / (2 \times f_y \times t), c_w) =$**9.5** mm

For class 1 & 2 proportion in compression; $\alpha = (c_w/2 + l_w/2) / c_w =$ **0.561**

Limit for class 1 web; $Limit_{1w} = (396 \times \varepsilon) / (13 \times \alpha - 1) =$;**58.15**

The web is class 1

Flange section classification (Table 5.2)

Depth between fillets; $c_f = b - 3 \times t =$ **78.0** mm

Ratio of c/t; $ratio_f = c_f / t =$ **19.50**

Limit for class 1 flange; $Limit_{1f} = 33 \times \varepsilon =$ **30.51**

Limit for class 2 flange; $Limit_{2f} = 38 \times \varepsilon =$ **35.13**

Limit for class 3 flange; $Limit_{3f} = 42 \times \varepsilon =$ **38.83**

The flange is class 1

Overall section classification

The section is class 1

Resistance of cross section (cl. 6.2)

Compression (cl. 6.2.4)

Design force; $N_{Ed} =$ **21** kN

Design resistance; $N_{c,Rd} = N_{pl,Rd} = A \times f_y / \gamma_{M0} =$ **374** kN

PASS - The compression design resistance exceeds the design force

Buckling resistance (cl. 6.3)

Yield strength for buckling resistance; $f_y =$ **275** N/mm^2

Flexural buckling about y axis

Elastic critical buckling force; $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr\_y}^2 =$ **96** kN

Non-dimensional slenderness; $\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})} =$ **1.976**

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

Imperfection factor (Table 6.1); $\alpha_y =$ **0.21**

Parameter Φ; $\Phi_y = 0.5 \times [1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2] =$ **2.638**

Reduction factor; $\chi_y = \min(1.0, 1 / [\Phi_y + \sqrt{(\Phi_y^2 - \bar{\lambda}_y^2)}]) =$ **0.228**

Design buckling resistance; $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1} =$ **85.2** kN

PASS - The flexural buckling resistance about the y axis exceeds the design axial load

Flexural buckling about z axis

Elastic critical buckling force; $N_{cr,z} = \pi^2 \times E \times I_z / L_{cr\_z}^2 =$ **96** kN

Non-dimensional slenderness; $\bar{\lambda}_z = \sqrt{(A \times f_y / N_{cr,z})} =$ **1.976**

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

Imperfection factor (Table 6.1); $\alpha_z =$ **0.21**

Parameter Φ; $\Phi_z = 0.5 \times [1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2] =$ **2.638**

Reduction factor; $\chi_z = \min(1.0, 1 / [\Phi_z + \sqrt{(\Phi_z^2 - \bar{\lambda}_z^2)}]) =$ **0.228**

Design buckling resistance; $N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1} =$ **85.2** kN

PASS - The flexural buckling resistance about the z axis exceeds the design axial load

Minimum buckling resistance

Minimum buckling resistance; $N_{b,Rd} = \min(N_{b,y,Rd}, N_{b,z,Rd}) =$ **85.2** kN

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

Steel connections

Note: See SD Structures plans drawings for beams annotations

SB1 AND SB3 CONNECTION TO MAIN BOX-FRAME

TEDDS calculation version 2.0.14

Section Details

Supporting Beam UKC 254x254x89;; Grade$_{supporting}$ = **"S275"**

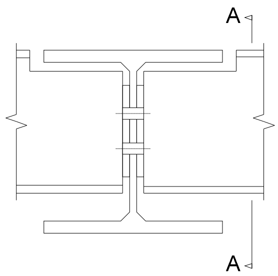
Supported Beam 1 - UB 203x102x23;; Grade$_{supportedb1}$ = **"S275"**

Supported beam 1 endplate - 130 x 190 x 10;; Grade$_{endplateb1}$ = **"S275"**

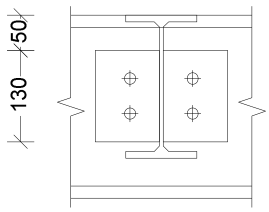
Supported Beam 2 - UKC 203x203x46;; Grade$_{supportedb2}$ = **"S275"**

Supported beam 2 endplate - 130 x 190 x 10;; Grade$_{endplateb2}$ = **"S275"**

Bolts M16 (Grade 8.8)



SECTION THROUGH SUPPORTING BEAM



SECTION A - A

Connection Details

Both beams

; Bolt pitch;; p_{bolts} = **50** mm

; Bolt gauge; g_{bolts} = **90** mm

Beam 1

; number of bolt rows; $n_{boltsb1}$ = **2**

; End plate end distance (top & bottom); $e_{1endplateb1}$ = **40** mm

End plate edge distance; $e_{2endplateb1} = (d_{endplateb1} - g_{bolts}) / 2 =$ **50** mm

End plate length; $l_{endplateb1} = p_{bolts} \times (n_{boltsb1}-1)+2 \times e_{1endplateb1} =$ **130** mm

; Weld leg length; s_{weldb1} = **6** mm

; Supported beam end reaction; Q_{b1} = **22.0** kN

Beam 2

; number of bolt rows; $n_{boltsb2}$ = **2**

; End plate end distance (top & bottom); $e_{1endplateb2}$ = **40** mm

End plate edge distance; $e_{2endplateb2} = (d_{endplateb2} - g_{bolts}) / 2 =$ **50** mm

End plate length; $l_{endplateb2} = p_{bolts} \times (n_{boltsb2}-1)+2 \times e_{1endplateb2} =$ **130** mm

; Weld leg length; s_{weldb2} = **6** mm

; Supported beam end reaction; Q_{b2} = **63.0** kN

Notch details

Beam 1

; Top notch length; $c_{topnotchb1}$ = **133** mm

; Top notch depth; $d_{ctopnotchb1}$ = **30** mm

Beam 2

; Top notch length; $c_{topnotchb2}$ = **133** mm

; Top notch depth; $d_{ctopnotchb2}$ = **30** mm

Check 1 - Essential detailing requirements

Both beams

; Bolt gauge; g_{bolts} = **90** mm; **PASS**

Beam 1

; End plate thickness; $t_{endplateb1}$ = **10** mm; **PASS**

; End plate Length; $l_{endplateb1}$ = **130** mm

; **End plate length for torsional requirements : PASS**

Beam 2

; End plate thickness; $t_{endplateb2}$ = **10** mm; **PASS**

; End plate Length; $l_{endplateb2}$ = **130** mm

; **End plate length for torsional requirements : PASS**

Check 2 - Shear capacity of bolt group connecting end plate to supporting beam

Beam 1

Shear capacity of top pair of bolts;;;;

$p_{bsendplateb1}$ = **460** N/mm^2

$P_{sbolts1b1} = \min(P_{sbolts}, 0.5 \times e_{1endplateb1} \times t_{endplateb1} \times p_{bsendplateb1})$ = **58.9** kN

Shear capacity of other bolts

P_{sbolts} = **58.9** kN

Shear capacity of bolt group - sum of bolt capacities

;$P_{sboltssumb1} = 2 \times P_{sbolts1b1} + 2 \times (n_{boltsb1} - 1) \times P_{sbolts}$ = **235.5** kN

;Shear on bolt group; Q_{b1} = **22.0** kN

Utilisation factor; $U_{check2b1} = Q_{b1} / P_{sboltssumb1}$ = **0.093**

Shear capacity of bolt group to Supporting Beam : PASS

Beam 2

Shear capacity of top pair of bolts;;;;

$p_{bsendplateb2}$ = **460** N/mm^2

$P_{sbolts1b2} = \min(P_{sbolts}, 0.5 \times e_{1endplateb2} \times t_{endplateb2} \times p_{bsendplateb2})$ = **58.9** kN

Shear capacity of other bolts

P_{sbolts} = **58.9** kN

Shear capacity of bolt group - sum of bolt capacities

;$P_{sboltssumb2} = 2 \times P_{sbolts1b2} + 2 \times (n_{boltsb2} - 1) \times P_{sbolts}$ = **235.5** kN

;Shear on bolt group; Q_{b2} = **63.0** kN

Utilisation factor; $U_{check2b2} = Q_{b2} / P_{sboltssumb2}$ = **0.268**

Shear capacity of bolt group to Supporting Beam : PASS

Check 3 - Shear and bearing capacity of end plate

Beam 1

for shear

;;

$p_{yendplateb1}$ = **275** N/mm^2

;; $e_{1aendplateb1} = e_{1endplateb1}$ = **40** mm

;; $A_{vendplateb1} = 0.9 \times (2 \times e_{1aendplateb1} + (n_{boltsb1} - 1) \times p_{bolts}) \times t_{endplateb1} = \mathbf{1170}$ mm^2

; $A_{vnetendplateb1} = A_{vendplateb1} - n_{boltsb1} \times D_{hbolts} \times t_{endplateb1} = \mathbf{810}$ mm^2

Effective net area coefficient

$K_{eendplateb1} = \mathbf{1.20}$

Plain shear capacity of end plate

$P_{vPendplateb1} = \min(0.6 \times p_{yendplateb1} \times A_{vendplateb1}, 0.7 \times K_{eendplateb1} \times p_{yendplateb1} \times A_{vnetendplateb1}) = \mathbf{187.1}$ kN

$A_{v1endplateb1} = (e_{1aendplateb1} + (n_{boltsb1} - 1) \times p_{bolts}) \times t_{endplateb1} = \mathbf{900}$ mm^2

; $A_{teffendplateb1} = (e_{2endplateb1} - 0.5 \times D_{hbolts}) \times t_{endplateb1} = \mathbf{410}$ mm^2

Block shear capacity of end plate

$P_{vBendplateb1} = 0.6 \times p_{yendplateb1} \times A_{v1endplateb1} + 0.6 \times K_{eendplateb1} \times p_{yendplateb1} \times A_{teffendplateb1} = \mathbf{229.7}$ kN

Shear capacity of the end plate; $P_{vendplateb1} = \min(P_{vPendplateb1}, P_{vBendplateb1}) = \mathbf{187.1}$ kN

;Shear force on each end plate shear plane; $Q_{b1} / 2 = \mathbf{11.0}$ kN

Utilisation factor; $U_{check3shearb1} = Q_{b1} / (2 \times P_{vendplateb1}) = \mathbf{0.059}$

Shear capacity of end plate : PASS

for bearing

$e_{endplateb1} = e_{1endplateb1} = \mathbf{40}$ mm

bearing strength of the end plate

$p_{bsendplateb1} = \mathbf{460}$ N/mm^2

For top bolt,
bearing capacity of the end plate per bolt

$P_{bsendplate1b1} = \min(d_{bolts} \times t_{endplateb1} \times p_{bsendplateb1}, 0.5 \times e_{endplateb1} \times t_{endplateb1} \times p_{bsendplateb1}) = \mathbf{73.6}$ kN

For other bolts,
bearing capacity of the end plate per bolt

$P_{bsendplateb1} = d_{bolts} \times t_{endplateb1} \times p_{bsendplateb1} = \mathbf{73.6}$ kN

Capacity of bolt group;

$P_{bsendplatesumb1} = 2 \times P_{bsendplate1b1} + 2 \times (n_{boltsb1} - 1) \times P_{bsendplateb1} = \mathbf{294.4}$ kN

Bearing force on bolt group; $Q_{b1} = \mathbf{22.0}$ kN

Utilisation factor; $U_{check3bearingb1} = Q_{b1} / P_{bsendplatesumb1} = \mathbf{0.075}$

Bearing capacity of endplate : PASS

Beam 2

for shear

;;

$p_{yendplateb2} = \mathbf{275}$ N/mm^2

;; $e_{1aendplateb2} = e_{1endplateb2} = \mathbf{40}$ mm

;; $A_{vendplateb2} = 0.9 \times (2 \times e_{1aendplateb2} + (n_{boltsb2} - 1) \times p_{bolts}) \times t_{endplateb2} = \mathbf{1170}$ mm^2

; $A_{vnetendplateb2} = A_{vendplateb2} - n_{boltsb2} \times D_{hbolts} \times t_{endplateb2} = \mathbf{810}$ mm^2

Effective net area coefficient

$K_{eendplateb2} = \mathbf{1.20}$

Plain shear capacity of end plate

$P_{vPendplateb2} = \min(0.6 \times p_{yendplateb2} \times A_{vendplateb2}, 0.7 \times K_{eendplateb2} \times p_{yendplateb2} \times A_{vnetendplateb2}) = \mathbf{187.1}$ kN

$A_{v1endplateb2} = (e_{1aendplateb2} + (n_{boltsb2} - 1) \times p_{bolts}) \times t_{endplateb2} = \mathbf{900}$ mm^2

; $A_{teffendplateb2} = (e_{2endplateb2} - 0.5 \times D_{hbolts}) \times t_{endplateb2} = \mathbf{410}$ mm^2

Block shear capacity of end plate

$P_{vBendplateb2} = 0.6 \times p_{yendplateb2} \times A_{v1endplateb2} + 0.6 \times K_{eendplateb2} \times p_{yendplateb2} \times A_{teffendplateb2} = \mathbf{229.7}$ kN

Shear capacity of the end plate; $P_{vendplateb2} = \min(P_{vPendplateb2}, P_{vBendplateb2}) = \mathbf{187.1}$ kN

;Shear force on each end plate shear plane; $Q_{b2} / 2 = \mathbf{31.5}$ kN

Utilisation factor; $U_{check3shearb2} = Q_{b2} / (2 \times P_{vendplateb2}) = \mathbf{0.168}$

Shear capacity of end plate : PASS

for bearing

$e_{endplateb2} = e_{1endplateb2} = \mathbf{40}$ mm

bearing strength of the end plate

$p_{bsendplateb2} = \mathbf{460}$ N/mm^2

For top bolt,
bearing capacity of the end plate per bolt

$P_{bsendplate1b2} = \min(d_{bolts} \times t_{endplateb2} \times p_{bsendplateb2}, 0.5 \times e_{endplateb2} \times t_{endplateb2} \times p_{bsendplateb2}) = \mathbf{73.6}$ kN

For other bolts,
bearing capacity of the end plate per bolt

$P_{bsendplateb2} = d_{bolts} \times t_{endplateb2} \times p_{bsendplateb2} = \mathbf{73.6}$ kN

Capacity of bolt group;

$P_{bsendplatesumb2} = 2 \times P_{bsendplate1b2} + 2 \times (n_{boltsb2} - 1) \times P_{bsendplateb2} = \mathbf{294.4}$ kN

Bearing force on bolt group; $Q_{b2} = \mathbf{63.0}$ kN

Utilisation factor; $U_{check3bearingb2} = Q_{b2} / P_{bsendplatesumb2} = \mathbf{0.214}$

Bearing capacity of endplate : PASS

Check 4 - shear capacity of the supported beam web at the endplate

Beam 1

;;

$p_{ysupportedb1} = \mathbf{275}$ N/mm^2

;;;

;

$A_{vsupportedb1} = 0.9 \times \min(l_{endplateb1}, d_{supportedb1}) \times t_{supportedb1} =$;**632**; mm^2;

Shear capacity of supported beam web

$P_{vsupportedb1} = 0.6 \times p_{ysupportedb1} \times A_{vsupportedb1} = \mathbf{104.2}$ kN

;Shear force on supported beam; $Q_{b1} = \mathbf{22.0}$ kN

Utilisation factor; $U_{check4shearb1} = Q_{b1} / P_{vsupportedb1} = \mathbf{0.211}$

Shear capacity of supported beam web : PASS

Beam 2

;;

$p_{ysupportedb2} = \mathbf{275}$ N/mm^2

;;;

;

$A_{vsupportedb2} = 0.9 \times \min(l_{endplateb2}, d_{supportedb2}) \times t_{supportedb2} =$;**842**; mm^2;

Shear capacity of supported beam web

$P_{vsupportedb2} = 0.6 \times p_{ysupportedb2} \times A_{vsupportedb2} = \mathbf{139.0}$ kN

;Shear force on supported beam; $Q_{b2} = \mathbf{63.0}$ kN

Utilisation factor; $U_{check4shearb2} = Q_{b2} / P_{vsupportedb2} = \mathbf{0.453}$

Shear capacity of supported beam web : PASS

Check 5 - capacity of the fillet welds connecting the end plate to the supported beam web

Beam 1

; Effective throat size of weld; $a_{weldb1} = s_{weldb1} \times 0.7 = \mathbf{4.2}$ mm

;;;

Effective length of weld; $l_{weldb1} = 2 \times (\min(l_{endplateb1}, 2 \times r_{supportedb1} + d_{supportedb1}) - 2 \times s_{weldb1}) =$;**236.0**; mm

; Design strength of weld; $p_{weldb1} = \mathbf{220}$ N/mm^2

Capacity of fillet welds; $P_{weldb1} = p_{weldb1} \times l_{weldb1} \times a_{weldb1} = \mathbf{218.1}$ kN

;Utilisation factor; $U_{check5weldb1} = Q_{b1} / P_{weldb1} = \mathbf{0.101}$

Capacity of fillet weld : PASS

Beam 2

; Effective throat size of weld; $a_{weldb2} = s_{weldb2} \times 0.7 = \mathbf{4.2}$ mm

;;;

Effective length of weld; $l_{weldb2} = 2 \times (\min(l_{endplateb2}, 2 \times r_{supportedb2} + d_{supportedb2}) - 2 \times s_{weldb2}) =$;**236.0**; mm

; Design strength of weld; $p_{weldb2} = \mathbf{220}$ N/mm^2

Capacity of fillet welds; $P_{weldb2} = p_{weldb2} \times l_{weldb2} \times a_{weldb2} = \mathbf{218.1}$ kN

;Utilisation factor; $U_{check5weldb2} = Q_{b2} / P_{weldb2} = \mathbf{0.289}$

Capacity of fillet weld : PASS

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

Beam 1

Properties of notched section ignoring root radii

;;; Depth of web; $d_{webb1} = D_{supportedb1} - d_{ctopnotchb1} - T_{supportedb1} = \mathbf{163.9}$ mm

;; Area of notched section; $A_{notchedb1} = d_{webb1} \times t_{supportedb1} + B_{supportedb1} \times T_{supportedb1} = \mathbf{1832}$ mm^2

Distance from top of notch to centroid of notched section

$$y_{barb1} = (d_{webb1}^2 \times t_{supportedb1} / 2 + B_{supportedb1} \times T_{supportedb1} \times (d_{webb1} + T_{supportedb1} / 2)) / A_{notchedb1}$$

$y_{barb1} = \mathbf{127}$ mm

Inertia of notched section

$$I_{notchedb1} = t_{supportedb1} \times d_{webb1}^3 / 12 + t_{supportedb1} \times d_{webb1} \times (y_{barb1} - d_{webb1} / 2)^2 + B_{supportedb1} \times T_{supportedb1}^3 / 12 + B_{supportedb1} \times T_{supportedb1} \times (d_{webb1} + T_{supportedb1} / 2 - y_{barb1})^2$$

$I_{notchedb1} = \mathbf{542}$ cm^4

Modulus of notched section

$Z_{notchedb1} = I_{notchedb1} / y_{barb1} = \mathbf{42.8}$ cm^3

;Moment capacity of notched section; $M_{capnotchedb1} = p_{ysupportedb1} \times Z_{notchedb1} = \mathbf{11.8}$ kNm

; Top notch length; $c_{topnotchb1} = \mathbf{133}$ mm

;;Moment applied to notched section; $M_{appnotchedb1} = Q_{b1} \times (c_{topnotchb1} + t_{endplateb1}) = \mathbf{3.1}$ kNm

Utilisation factor; $U_{check6amomentb1} = M_{appnotchedb1} / M_{capnotchedb1} = \mathbf{0.268}$

Moment capacity of notched section : PASS

Beam 2

Properties of notched section ignoring root radii

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

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

Distance from top of notch to centroid of notched section

$y_{barb2} = (d_{webb2}^2 \times t_{supportedb2} / 2 + B_{supportedb2} \times T_{supportedb2} \times (d_{webb2} + T_{supportedb2} / 2)) / A_{notchedb2}$

y_{barb2} = **138** mm

Inertia of notched section

$I_{notchedb2} = t_{supportedb2} \times d_{webb2}^3 / 12 + t_{supportedb2} \times d_{webb2} \times (y_{barb2} - d_{webb2} / 2)^2 + B_{supportedb2} \times T_{supportedb2}^3 / 12 + B_{supportedb2} \times T_{supportedb2} \times (d_{webb2} + T_{supportedb2} / 2 - y_{barb2})^2$

$I_{notchedb2}$ = **834** cm^4

Modulus of notched section

$Z_{notchedb2} = I_{notchedb2} / y_{barb2}$ = **60.4** cm^3

;Moment capacity of notched section; $M_{capnotchedb2} = p_{ysupportedb2} \times Z_{notchedb2}$ = **16.6** kNm

; Top notch length; $c_{topnotchb2}$ = **133** mm

;;Moment applied to notched section; $M_{appnotchedb2} = Q_{b2} \times (c_{topnotchb2} + t_{endplateb2})$ = **9.0** kNm

Utilisation factor; $U_{check6amomentb2} = M_{appnotchedb2} / M_{capnotchedb2}$ = **0.542**

Moment capacity of notched section : PASS

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

Beam 1

;; Depth of Notch; $d_{ctopnotchb1}$ = **30** mm

Length of notch; $c_{topnotchb1}$ = **133** mm

;Depth of top notch less than limit of D/2 : PASS

; $Grade_{supportedb1}$ = **"S275"**

;; $dot_{b1} = D_{supportedb1}/t_{supportedb1}$ = **37.6**

$c_{limitb1}$ = if(or($Grade_{supportedb1}$ == “S275”, $Grade_{supportedb1}$ == “300”), if(dot_{b1}<=54.3,$D_{supportedb1}$,160000 × $D_{supportedb1}/dot_{b1}^3$),
if(dot_{b1}<=48.0, $D_{supportedb1}$,110000 × $D_{supportedb1}/dot_{b1}^3$))

$c_{limitb1}$ = **203.2** mm

Length of top notch less than limit : PASS

Beam 2

;; Depth of Notch; $d_{ctopnotchb2}$ = **30** mm

Length of notch; $c_{topnotchb2}$ = **133** mm

;Depth of top notch less than limit of D/2 : PASS

; $Grade_{supportedb2}$ = **"S275"**

;; $dot_{b2} = D_{supportedb2}/t_{supportedb2}$ = **28.2**

$c_{limitb2}$ = if(or($Grade_{supportedb2}$ == “S275”, $Grade_{supportedb2}$ == “300”), if(dot_{b2}<=54.3,$D_{supportedb2}$,160000 × $D_{supportedb2}/dot_{b2}^3$),
if(dot_{b2}<=48.0, $D_{supportedb2}$,110000 × $D_{supportedb2}/dot_{b2}^3$))

$c_{limitb2}$ = **203.2** mm

Length of top notch less than limit : PASS

Check 7 - Local shear and bearing capacity of Supporting Beam web

for shear

;; $e_{tsupporting}$ = **90** mm

;;;; $A_{vsupporting} = (p_{bolts} + (\min(n_{boltsb1}, n_{boltsb2}) - 1) \times p_{bolts} + e_{tsupporting}) \times t_{supporting}$ = **1957** mm^2

; $A_{vnetsupporting} = A_{vsupporting} - \min(n_{boltsb1},n_{boltsb2}) \times D_{hbolts} \times t_{supporting} = \mathbf{1586}\ mm^2$

;

$p_{ysupporting} = \mathbf{265}\ N/mm^2$

Effective net area coefficient

$K_{esupporting} = \mathbf{1.20}$

$P_{vsupporting} = \min(0.6 \times p_{ysupporting} \times A_{vsupporting}, 0.7 \times K_{esupporting} \times p_{ysupporting} \times A_{vnetsupporting}) = \mathbf{311.2}\ kN$

;; $Q_v = Q_{b1} \times if(n_{boltsb2}<n_{boltsb1}, n_{boltsb2}/n_{boltsb1},1) + Q_{b2} \times if(n_{boltsb1}<n_{boltsb2}, n_{boltsb1}/n_{boltsb2},1) = \mathbf{85}\ kN$

Shear load on supporting beam; $Q_v / 2 = \mathbf{42.5}\ kN$

Utilisation factor; $U_{check7shear} = Q_v / (2 \times P_{vsupporting}) = \mathbf{0.137}$

Local shear capacity of Supporting Beam web: PASS

for bearing

$p_{bssupporting} = \mathbf{460}\ N/mm^2$

$P_{bssupporting} = d_{bolts} \times t_{supporting} \times p_{bssupporting} = \mathbf{75.8}\ kN$

$F_{bs} = Q_{b1} / (2 \times n_{boltsb1}) + Q_{b2} / (2 \times n_{boltsb2}) = \mathbf{21.3}\ kN$

Utilisation factor; $U_{check7bearing} = F_{bs} / P_{bssupporting} = \mathbf{0.280}$

Local bearing capacity of Supporting Beam web : PASS

SUMMARY OF RESULTS

Check 2 - Shear capacity of bolt group connecting end plate to supporting Beam

Beam 1

Shear utilisation factor; $U_{check2b1} = \mathbf{0.093}$; **PASS**

Beam 2

Shear utilisation factor; $U_{check2b2} = \mathbf{0.268}$; **PASS**

Check 3 - Capacity of end plate

Beam 1

Shear utilisation factor; $U_{check3shearb1} = \mathbf{0.059}$; **PASS**

Bearing utilisation factor; $U_{check3bearingb1} = \mathbf{0.075}$; **PASS**

Beam 2

Shear utilisation factor; $U_{check3shearb2} = \mathbf{0.168}$; **PASS**

Bearing utilisation factor; $U_{check3bearingb2} = \mathbf{0.214}$; **PASS**

Check 4 - Shear capacity of the supported beam web at the endplate

Beam 1

Shear utilisation factor; $U_{check4shearb1} = \mathbf{0.211}$; **PASS**

Beam 2

Shear utilisation factor; $U_{check4shearb2} = \mathbf{0.453}$; **PASS**

Check 5 - Capacity of the fillet welds connecting the end plate to the supported beam web

Beam 1

Shear utilisation factor; $U_{check5weldb1} = \mathbf{0.101}$; **PASS**

Beam 2

Shear utilisation factor; $U_{check5weldb2} = \mathbf{0.289}$; **PASS**

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

Beam 1

Utilisation factor; $U_{check6amomentb1}$ = **0.268**; **PASS**

Beam 2

Utilisation factor; $U_{check6amomentb2}$ = **0.542**; **PASS**

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

Beam 1

Depth of top notch less than limit of D/2 : PASS

Length of top notch less than limit : PASS

Beam 2

Depth of top notch less than limit of D/2 : PASS

Length of top notch less than limit : PASS

Check 7 - Local shear and bearing capacity of Supporting Beam web

Shear utilisation factor; $U_{check7shear}$ = **0.137**; **PASS**

Bearing utilisation factor; $U_{check7bearing}$ = **0.280**; **PASS**

SB3 TO SB4 CONNECTION

TEDDS calculation version 2.0.14

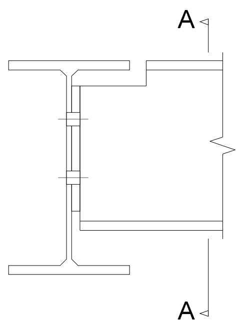
Section Details

Supporting Beam UKB 254x146x37;; Grade$_{supporting}$ = **"S275"**

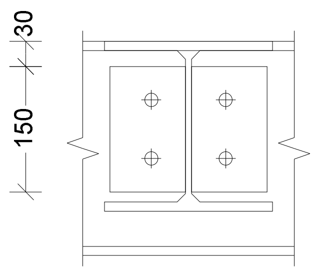
Supported Beam UKC 203x203x46;; Grade$_{supported}$ = **"S275"**

Endplate - 150 x 190 x 10;; Grade$_{endplate}$ = **"S275"**

Bolts M16 (Grade 8.8)



SECTION THROUGH SUPPORTING BEAM



SECTION A - A

Connection Details

; number of bolt rows; n_{bolts} = **2**

; Bolt pitch;; p_{bolts} = **70** mm

; Bolt gauge; g_{bolts} = **90** mm

; Endplate end distance (top & bottom); $e_{1endplate}$ = **40** mm

Endplate edge distance; $e_{2endplate} = (d_{endplate} - g_{bolts}) / 2$ = **50** mm

Endplate length; $l_{endplate} = p_{bolts} \times (n_{bolts}-1)+2 \times e_{1endplate}$ = **150** mm

; Weld leg length; s_{weld} = **6** mm

; supported beam end reaction; Q = **67.0** kN

Notch details

; Top notch length; $c_{topnotch}$ = **80** mm

; Top notch depth; $d_{ctopnotch}$ = **30** mm

Check 1 - Essential detailing requirements

; Endplate thickness; $t_{endplate}$ = **10** mm; **PASS**

; Bolt gauge; g_{bolts} = **90** mm; **PASS**

; Endplate Length; $l_{endplate}$ = **150** 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;;;;

$p_{bsendplate}$ = **460** N/mm^2

$P_{sbolts1} = \min(P_{sbolts}, 0.5 \times e_{1endplate} \times t_{endplate} \times p_{bsendplate})$ = **58.9** kN

Shear capacity of other bolts

P_{sbolts} = **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** kN

;Shear on bolt group; Q = **67.0** kN

Utilisation factor; $U_{check2} = Q / P_{sboltssum} = \mathbf{0.285}$

Shear capacity of bolt group to Supporting Beam : PASS

Check 3 - Shear and bearing capacity of endplate

for shear

;;

$p_{yendplate} = \mathbf{275}$ N/mm^2

;; $e_{1aendplate} = e_{1endplate} = \mathbf{40}$ mm

;; $A_{vendplate} = 0.9 \times (2 \times e_{1aendplate} + (n_{bolts} - 1) \times p_{bolts}) \times t_{endplate} = \mathbf{1350}$ mm^2

; $A_{vnetendplate} = A_{vendplate} - n_{bolts} \times D_{hbolts} \times t_{endplate} = \mathbf{990}$ mm^2

Effective net area coefficient

$K_{eendplate} = \mathbf{1.20}$

Plain shear capacity of endplate

$P_{vPendplate} = \min(0.6 \times p_{yendplate} \times A_{vendplate}, 0.7 \times K_{eendplate} \times p_{yendplate} \times A_{vnetendplate}) = \mathbf{222.8}$ kN

$A_{v1endplate} = (e_{1aendplate} + (n_{bolts} - 1) \times p_{bolts}) \times t_{endplate} = \mathbf{1100}$ mm^2

; $A_{teffendplate} = (e_{2endplate} - 0.5 \times D_{hbolts}) \times t_{endplate} = \mathbf{410}$ mm^2

Block shear capacity of endplate

$P_{vBendplate} = 0.6 \times p_{yendplate} \times A_{v1endplate} + 0.6 \times K_{eendplate} \times p_{yendplate} \times A_{teffendplate} = \mathbf{262.7}$ kN

Shear capacity of the endplate; $P_{vendplate} = \min(P_{vPendplate}, P_{vBendplate}) = \mathbf{222.8}$ kN

;Shear force on each endplate shear plane; $Q / 2 = \mathbf{33.5}$ kN

Utilisation factor; $U_{check3shear} = Q / (2 \times P_{vendplate}) = \mathbf{0.150}$

Shear capacity of endplate : PASS

for bearing

$e_{endplate} = e_{1endplate} = \mathbf{40}$ mm

bearing strength of the endplate

$p_{bsendplate} = \mathbf{460}$ N/mm^2

For top bolt,
bearing capacity of the endplate per bolt

$P_{bsendplate1} = \min(d_{bolts} \times t_{endplate} \times p_{bsendplate}, 0.5 \times e_{endplate} \times t_{endplate} \times p_{bsendplate}) = \mathbf{73.6}$ kN

For other bolts,
bearing capacity of the endplate per bolt

$P_{bsendplate} = d_{bolts} \times t_{endplate} \times p_{bsendplate} = \mathbf{73.6}$ kN

Capacity of bolt group;

$P_{bsendplatesum} = 2 \times P_{bsendplate1} + 2 \times (n_{bolts} - 1) \times P_{bsendplate} = \mathbf{294.4}$ kN

Bearing force on bolt group; $Q = \mathbf{67.0}$ kN

Utilisation factor; $U_{check3bearing} = Q / P_{bsendplatesum} = \mathbf{0.228}$

Bearing capacity of endplate : PASS

Check 4 - shear capacity of the supported beam web at the endplate

;;

$p_{ysupported} = \mathbf{275}$ N/mm^2

;;;

;

$A_{vsupported} = 0.9 \times \min(l_{endplate}, d_{supported}) \times t_{supported}$ = ;**972**; mm²;

Shear capacity of supported beam web

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

;Shear force on supported beam; Q = **67.0** kN

Utilisation factor; $U_{check4shear} = Q / P_{vsupported}$ = **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** mm

;;;

Effective length of weld; $l_{weld} = 2 \times (\min(l_{endplate}, 2 \times r_{supported} + d_{supported}) - 2 \times s_{weld})$ = ;**276.0**; mm

; Design strength of weld; p_{weld} = **220** N/mm²

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

;Utilisation factor; $U_{check5weld} = Q / P_{weld}$ = **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; $d_{web} = D_{supported} - d_{ctopnotch} - T_{supported}$

;; Area of notched section; $A_{notched} = d_{web} \times t_{supported} + B_{supported} \times T_{supported}$ = **3407** mm²

Distance from top of notch to centroid of notched section

$y_{bar} = (d_{web}^2 \times t_{supported} / 2 + B_{supported} \times T_{supported} \times (d_{web} + T_{supported} / 2)) / A_{notched}$ = **138** mm

Inertia of notched section

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

$I_{notched}$ = **834** cm⁴

Modulus of notched section

$Z_{notched} = I_{notched} / y_{bar}$ = **60.4** cm³

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

;;;Moment applied to notched section; $M_{appnotched} = Q \times (c_{topnotch} + t_{endplate})$ = **6.0** kNm

Utilisation factor; $U_{check6amoment} = M_{appnotched} / M_{capnotched}$ = **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; $d_{ctopnotch}$ = **30** mm

; Length of notch; $c_{topnotch}$ = **80** mm

;Depth of top notch less than limit of D/2 : PASS

; $Grade_{supported}$ = **"S275"**

; dot = $D_{supported}/t_{supported}$ = **28.2**

c_{limit} = if(or($Grade_{supported}$ == “S275”, $Grade_{supported}$ == “300”), if(dot<=54.3,$D_{supported}$,160000 × $D_{supported}$/dot³), if(dot<=48.0,$D_{supported}$,110000 × $D_{supported}$/dot³))

c_{limit} = **203.2** mm

Length of top notch less than limit : PASS

Check 7 - Local shear and bearing capacity of Supporting Beam web

for shear

;; $e_{tsupporting}$ = **70** mm

;;; $A_{vsupporting} = (p_{bolts} + (n_{bolts} - 1) \times p_{bolts} + e_{tsupporting}) \times t_{supporting}$ = **1323** mm^2

; $A_{vnetsupporting} = A_{vsupporting} - n_{bolts} \times D_{hbolts} \times t_{supporting}$ = **1096** mm^2

;

$p_{ysupporting}$ = **275** N/mm^2

Effective net area coefficient

$K_{esupporting}$ = **1.20**

$P_{vsupporting} = \min(0.6 \times p_{ysupporting} \times A_{vsupporting}, 0.7 \times K_{esupporting} \times p_{ysupporting} \times A_{vnetsupporting})$ = **218.3** kN

;Shear load on Supporting Beam; Q / 2 = **33.5** kN

Utilisation factor; $U_{check7shear} = Q / (2 \times P_{vsupporting})$ = **0.153**

Local shear capacity of Supporting Beam web: PASS

for bearing

$p_{bssupporting}$ = **460** N/mm^2

$P_{bssupporting} = d_{bolts} \times t_{supporting} \times p_{bssupporting}$ = **46.4** kN

$Q / (2 \times n_{bolts})$ = **16.8** kN

Utilisation factor; $U_{check7bearing} = Q / (2 \times n_{bolts} \times P_{bssupporting})$ = **0.361**

Local bearing capacity of Supporting Beam web : PASS

<u>SUMMARY OF RESULTS</u>

Check 2 - Shear capacity of bolt group connecting endplate to supporting Beam

Shear utilisation factor; U_{check2} = **0.285**; **PASS**

Check 3 - Capacity of endplate

Shear utilisation factor; $U_{check3shear}$ = **0.150**; **PASS**

Bearing utilisation factor; $U_{check3bearing}$ = **0.228**; **PASS**

Check 4 - Shear capacity of the supported beam web at the endplate

Shear utilisation factor; $U_{check4shear}$ = **0.418**; **PASS**

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

Shear utilisation factor; $U_{check5weld}$ = **0.263**; **PASS**

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

Utilisation factor; $U_{check6amoment}$ = **0.363**; **PASS**

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

Depth of top notch less than limit of D/2 : PASS

Length of top notch less than limit : PASS

Check 7 - Local shear and bearing capacity of Supporting Beam web

Shear utilisation factor; $U_{check7shear}$ = **0.153**; **PASS**

Bearing utilisation factor; $U_{check7bearing}$ = **0.361**; **PASS**

SD structures Chartered Engineers SD Structures 107 Fleet Street, London EC4A 2AB www.sd-structures.com	Project 28 Burghley Road				Job Ref. SDS187		
	Section Building Control Approval				Sheet no./rev. 138		
	Calc. by RC	Date 26/06/2017	Chk'd by MD	Date 26/06/2017	App'd by	Date	

SB3 TO SB3 CONNECTION

TEDDS calculation version 2.0.14

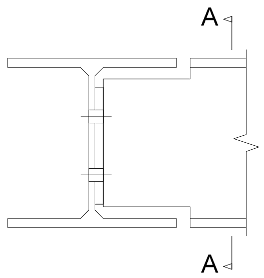
Section Details

Supporting Beam UKC 203x203x46;; $Grade_{supporting}$ = **"S355"**

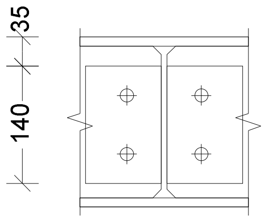
Supported Beam UKC 203x203x46;; $Grade_{supported}$ = **"S355"**

Endplate - 140 x 190 x 10;; $Grade_{endplate}$ = **"S275"**

Bolts M16 (Grade 8.8)



SECTION THROUGH SUPPORTING BEAM



SECTION A - A

Connection Details

; number of bolt rows; n_{bolts} = **2**

; Bolt pitch;; p_{bolts} = **70** mm

; Bolt gauge; g_{bolts} = **90** mm

; Endplate end distance (top & bottom); $e_{1endplate}$ = **35** mm

Endplate edge distance; $e_{2endplate} = (d_{endplate} - g_{bolts}) / 2$ = **50** mm

Endplate length; $l_{endplate} = p_{bolts} \times (n_{bolts}-1)+2\times e_{1endplate}$ = **140** mm

; Weld leg length; s_{weld} = **6** mm

; supported beam end reaction; Q = **73.0** kN

Notch details

; Top notch length; $c_{topnotch}$ = **105** mm

; Top notch depth; $d_{ctopnotch}$ = **25** mm

; Bottom notch length; $c_{bottomnotch}$ = **105** mm

; Bottom notch depth; $d_{cbottomnotch}$ = **25** mm

Check 1 - Essential detailing requirements

; Endplate thickness; $t_{endplate}$ = **10** mm; **PASS**

; Bolt gauge; g_{bolts} = **90** mm; **PASS**

; Endplate Length; $l_{endplate}$ = **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;;;;

$p_{bsendplate}$ = **460** N/mm^2

$P_{sbolts1} = \min(P_{sbolts}, 0.5 \times e_{1endplate} \times t_{endplate} \times p_{bsendplate})$ = **58.9** kN

Shear capacity of other bolts

P_{sbolts} = **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** kN

;Shear on bolt group; Q = **73.0** kN

Utilisation factor; U_{check2} = Q / $P_{sboltssum}$ = **0.310**

Shear capacity of bolt group to Supporting Beam : PASS

Check 3 - Shear and bearing capacity of endplate

for shear

;;

$p_{yendplate}$ = **275** N/mm^2

;; $e_{1aendplate} = e_{1endplate}$ = **35** mm

;; $A_{vendplate} = 0.9 \times (2 \times e_{1aendplate} + (n_{bolts} - 1) \times p_{bolts}) \times t_{endplate}$ = **1260** mm^2

; $A_{vnetendplate} = A_{vendplate} - n_{bolts} \times D_{hbolts} \times t_{endplate}$ = **900** mm^2

Effective net area coefficient

$K_{eendplate}$ = **1.20**

Plain shear capacity of endplate

$P_{vPendplate} = \min(0.6 \times p_{yendplate} \times A_{vendplate}, 0.7 \times K_{eendplate} \times p_{yendplate} \times A_{vnetendplate})$ = **207.9** kN

$A_{v1endplate} = (e_{1aendplate} + (n_{bolts} - 1) \times p_{bolts}) \times t_{endplate}$ = **1050** mm^2

; $A_{teffendplate} = (e_{2endplate} - 0.5 \times D_{hbolts}) \times t_{endplate}$ = **410** mm^2

Block shear capacity of endplate

$P_{vBendplate} = 0.6 \times p_{yendplate} \times A_{v1endplate} + 0.6 \times K_{eendplate} \times p_{yendplate} \times A_{teffendplate}$ = **254.4** kN

Shear capacity of the endplate; $P_{vendplate} = \min(P_{vPendplate}, P_{vBendplate})$ = **207.9** kN

;Shear force on each endplate shear plane; Q / 2 = **36.5** kN

Utilisation factor; $U_{check3shear} = Q / (2 \times P_{vendplate})$ = **0.176**

Shear capacity of endplate : PASS

for bearing

$e_{endplate} = e_{1endplate}$ = **35** mm

bearing strength of the endplate

$p_{bsendplate}$ = **460** N/mm^2

For top bolt,
bearing capacity of the endplate per bolt

$P_{bsendplate1} = \min(d_{bolts} \times t_{endplate} \times p_{bsendplate}, 0.5 \times e_{endplate} \times t_{endplate} \times p_{bsendplate})$ = **73.6** kN

For other bolts,
bearing capacity of the endplate per bolt

$P_{bsendplate} = d_{bolts} \times t_{endplate} \times p_{bsendplate}$ = **73.6** kN

Capacity of bolt group;

$P_{bsendplatesum} = 2 \times P_{bsendplate1} + 2 \times (n_{bolts} - 1) \times P_{bsendplate}$ = **294.4** kN

Bearing force on bolt group; Q = **73.0** kN

Utilisation factor; $U_{check3bearing} = Q / P_{bsendplatesum}$ = **0.248**

Bearing capacity of endplate : PASS

Check 4 - shear capacity of the supported beam web at the endplate

;;

$p_{ysupported}$ = **355** N/mm^2

;;;

;

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

Shear capacity of supported beam web

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

;Shear force on supported beam; Q = **73.0** kN

Utilisation factor; $U_{check4shear} = Q / P_{vsupported}$ = **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** mm

;;;

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

; Design strength of weld; p_{weld} = **220** N/mm^2

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

;Utilisation factor; $U_{check5weld} = Q / P_{weld}$ = **0.309**

Capacity of fillet weld : PASS

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

;;;; $Z_{notched} = t_{supported} \times (D_{supported} - d_{ctopnotch} - d_{cbottomnotch})^2 / 6$ = **28164** mm^3

;Moment capacity of notched section; $M_{capnotched} = p_{ysupported} \times Z_{notched}$ = **10.0** kNm

;;;;Moment applied to notched section; $M_{appnotched} = Q \times (\max(c_{topnotch}, c_{bottomnotch}) + t_{endplate})$= **8.4** kNm

Utilisation factor; $U_{check6amoment} = M_{appnotched} / M_{capnotched}$ = **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; $d_{ctopnotch}$ = **25** mm

; Length of top notch; $c_{topnotch}$ = **105** mm

; Depth of bottom notch; $d_{cbottomnotch}$ = **25** mm

; Length of bottom notch; $c_{bottomnotch}$ = **105** mm

$Check_{depth} = \max(d_{ctopnotch}, d_{cbottomnotch})$ = **25** mm

$Check_{length} = \max(c_{topnotch}, c_{bottomnotch})$ = **105** mm

;Depth of both notches less than limit of D/5 : PASS

; $Grade_{supported}$ = **"S355"**

; $dot = D_{supported}/t_{supported}$ = **28.2**

c_{limit} = if(or($Grade_{supported}$ == “S275”, $Grade_{supported}$ == “300”), if(dot<=54.3,$D_{supported}$,160000 × $D_{supported}$/dot^3), if(dot<=48.0,$D_{supported}$,110000 × $D_{supported}$/dot^3))

c_{limit} = **203.2** mm

Length of both notches less than limit : PASS

Check 7 - Local shear and bearing capacity of Supporting Beam web

for shear

;; $e_{tsupporting}$ = **70** mm

;;; $A_{vsupporting} = (p_{bolts} + (n_{bolts} - 1) \times p_{bolts} + e_{tsupporting}) \times t_{supporting}$ = **1512** mm^2

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

;

$p_{ysupporting}$ = **355** N/mm^2

Effective net area coefficient

$K_{esupporting}$ = **1.10**

$P_{vsupporting} = \min(0.6 \times p_{ysupporting} \times A_{vsupporting}, 0.7 \times K_{esupporting} \times p_{ysupporting} \times A_{vnetsupporting})$ = **322.1** kN

;Shear load on Supporting Beam; Q / 2 = **36.5** kN

Utilisation factor; $U_{check7shear} = Q / (2 \times P_{vsupporting})$ = **0.113**

Local shear capacity of Supporting Beam web: PASS

for bearing

$p_{bssupporting}$ = **550** N/mm^2

$P_{bssupporting} = d_{bolts} \times t_{supporting} \times p_{bssupporting}$ = **63.4** kN

$Q / (2 \times n_{bolts})$ = **18.3** kN

Utilisation factor; $U_{check7bearing} = Q / (2 \times n_{bolts} \times P_{bssupporting})$ = **0.288**

Local bearing capacity of Supporting Beam web : PASS

SUMMARY OF RESULTS

Check 2 - Shear capacity of bolt group connecting endplate to supporting Beam

Shear utilisation factor; U_{check2} = **0.310**; **PASS**

Check 3 - Capacity of endplate

Shear utilisation factor; $U_{check3shear}$ = **0.176**; **PASS**

Bearing utilisation factor; $U_{check3bearing}$ = **0.248**; **PASS**

Check 4 - Shear capacity of the supported beam web at the endplate

Shear utilisation factor; $U_{check4shear}$ = **0.378**; **PASS**

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

Shear utilisation factor; $U_{check5weld}$ = **0.309**; **PASS**

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

Utilisation factor; $U_{check6amoment}$ = **0.840**; **PASS**

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

Depth of both notches less than limit of D/5 : PASS

Length of both notches less than limit : PASS

Check 7 - Local shear and bearing capacity of Supporting Beam web

Shear utilisation factor; $U_{check7shear}$ = **0.113**; **PASS**

Bearing utilisation factor; $U_{check7bearing}$ = **0.288**; **PASS**

;

MOMENT CONNECT – BOX-FRAME ORIGINAL WALL

Simplified calculation methods

STEEL PROPERTIES

Steel section:

;Yield Strength; f_y = **275.0** N/mm^2;

;Ultimate Tensile Strength; f_u = **410.0** N/mm^2;

$\varepsilon=\sqrt{(235N/mm^2/ f_y)}$ =**0.924**

Try UC 254 x 254 x 89;

Section depth; D=**260.300** mm

Section width; B=**256.300** mm

Flange thickness; T=**17.300** mm

Radii; r= **12.700** mm

Cross-Section Area; A=**11331.141** mm^2

Web thickness; t=**10.300** mm

Bolts information:

Try M24 (Grade 8.8);

f_{ub}=800 N/mm^2

Partial safety factor; γ_{M0}=1.00

Partial safety factor; γ_{M2}=1.25

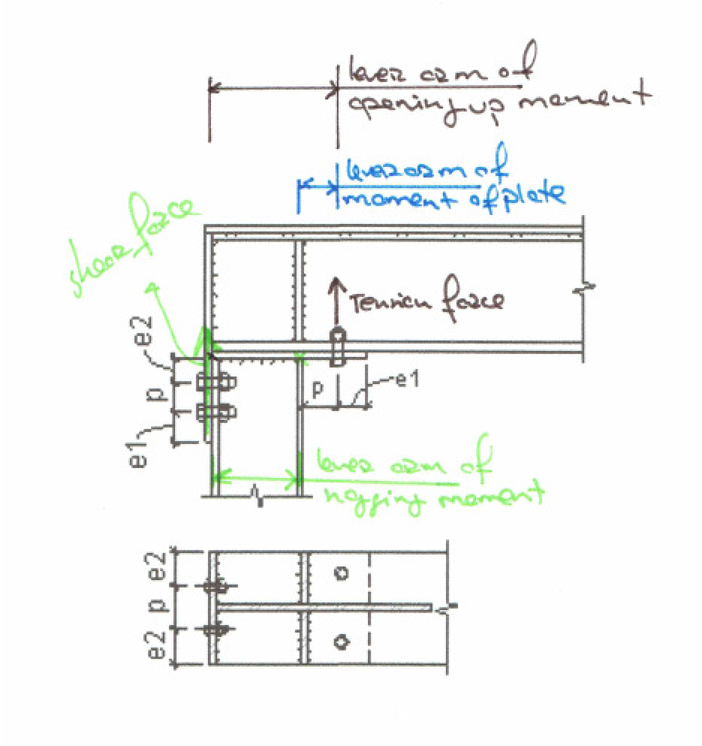
d=**24.000** mm

D_h=**26.000** mm

Edge distance; e_2=1.5*d=**36.000** mm

End distance; e_1=2.0* d=**48.000** mm

Pitch/gauge distance; p=3.0*d=**72.000** mm



Steel plate information:

Initial plate thickness; t_p= max(T,t) = **17.300** mm

Connection Design

;Design Shear Force; V_{Ed} = **300.0** kN;

;Design Hogging Moment; M_{Ed_h} = **220.000** kNm;

;Design Opening up Moment; M_{Ed_ou} = **0.000** kNm;

Shear capacity - Section 6.2.6

Height of web; h_w= D-2*T=**225.700** mm

Shear area factor; η = **1.000**

Shear buckling verification; if ($h_w/t<72*\varepsilon/\eta$, "***Shear buckling resistance can be ignored***",

"Account **for shear buckling**")= **"Shear buckling resistance can be ignored"**

Shear area - cl 6.2.6(3); A_v = max(A-2* B* T+ (t+ 2* r)* T,η* h_w*t)= **3080.771** mm^2

Design shear resistance - cl 6.2.6(2) $V_{c,Rd} = V_{pl,Rd} = A_v* (f_y/\sqrt{(3)})/\gamma_{M0}$=**489.138** kN

Diagonal stiffner requierement; $ds=if(M_{Ed\_h}/D<V_{pl,Rd}$, "**Not required**", "**Required**")= **"Required"**

Hogging moment design:

;Number of bolts under shear; n_h = **8.000** ;

Shear force on bolts; $V_{Ed,b} = M_{Ed\_h}/D$ = **845.179** kN

Shear force per bolt; $F_{b,Ed} = V_{Ed,b}/n_h$=**105.647** kN

k_1=min(2.8*(e_2/D_h)-1.7,1.4*(p/D_h)-1.7,2.5)= **2.177**

α_v=min(e_1/(3*D_h),p/(3*D_h)-0.25,f_{ub}/f_u,1)=**0.615**

Shear/Bearing capacity per bolt; $F_{v,Rd}=F_{b,Rd}$=min(k_1*α_v*f_u*d*t_p/γ_{M2},0.6*f_{ub}*A_s/γ_{M2})=**135.552** kN

Design check; hc=if($F_{b,Ed}<F_{b,Rd}$,"PASS","FAIL")=**"PASS"**

Utilization; $F_{b,Ed}/F_{b,Rd}$=**0.779**

Opening up moment design:

;Number of bolts under tension; n_{ou} = **2.000** ;

Tension force on bolts; $T_{Ed,b} = M_{Ed\_ou}/(D+p)$ = **0.000** kN

Tension force per bolt; $T_{b,Ed} = T_{Ed,b}/n_{ou}$=**0.000** kN

k_2=0.63

Tension/Punching capacity per bolt; $F_{t,Rd}=B_{p,Rd}$=min(k_2*f_{ub}*A_s/γ_{M2},0.6*π*D_h*t_p*f_u/γ_{M2})=**142.330** kN

Design check; om= if($T_{b,Ed} < F_{t,Rd}$,"PASS","FAIL")=**"PASS"**

Utilization; $T_{b,Ed} / F_{t,Rd}$= **0.000**

Bending of plate subjected to tension from bolts:

Bending moment on plate; M_{Ed_plate}= $T_{Ed,b}$*p=**0.000** kNm

Plate thicknesss required; 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 N/mm^2))))= **17.300** mm

Tension check with reduced flange:

Tension force on bolts; $V_{Ed,b} = M_{Ed\_h}/D$ = **845.179** kN

A_{net}; A_{net} = (B-(n_{ou}*d))*T=**3603.590**mm^2

Tension capacity of reduced flange; $F_{ft,Rd}$ =(A_{net} * f_u)/γ_{M2}=**1181.978**kN

Design check; trf= if($V_{Ed,b} < F_{ft,Rd}$,"PASS","FAIL")=**"PASS"**

Utilization; $V_{Ed,b} / F_{ft,Rd}$ = **0.715**

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

MOMENT CONNECTION – BOX-FRAME AT THE REAR

Simplified calculation methods

STEEL PROPERTIES

Steel section:

;Yield Strength; f_y = **275.0** N/mm^2;

;Ultimate Tensile Strength; f_u = **410.0** N/mm^2;

$\varepsilon=\sqrt{(235N/mm^2/f_y)}$ =**0.924**

Try UC 152 x 152 x 30;

Section depth; D=**157.600** mm

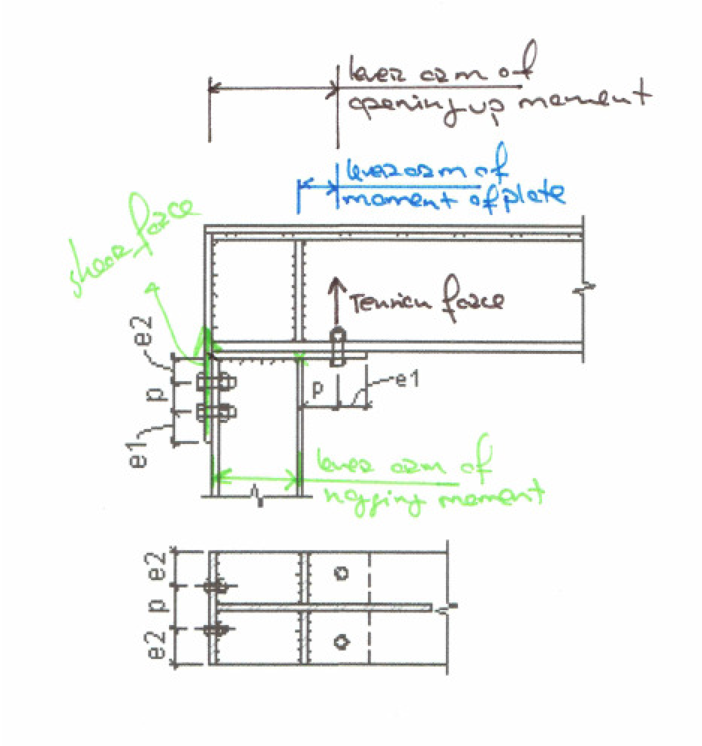
Section width; B=**152.900** mm

Flange thickness; T=**9.400** mm

Radii; r= **7.600** mm

Cross-Section Area; A=**3826.301** mm^2

Web thickness; t=**6.500** mm



Bolts information:

Try M16 (Grade 8.8);

f_{ub}=800 N/mm^2

Partial safety factor; γ_{M0}=1.00

Partial safety factor; γ_{M2}=1.25

d=**16.000** mm

D_h=**18.000** mm

Edge distance; e_2=1.5*d=**24.000** mm

End distance; e_1=2.0* d=**32.000** mm

Pitch/gauge distance; p=3.0*d=**48.000** mm

Steel plate information:

Initial plate thickness; t_p= max(T,t) = **9.400** mm

Connection Design

;Design Shear Force; V_{Ed} = **66.0** kN;

;Design Hogging Moment; M_{Ed_h} = **30.000** kNm;

;Design Opening up Moment; M_{Ed_ou} = **0.000** kNm;

Shear capacity - Section 6.2.6

Height of web; h_w= D-2*T=**138.800** mm

Shear area factor; η = **1.000**

Shear buckling verification; if ($h_w/t<72*\varepsilon/\eta$, "***Shear buckling resistance can be ignored***",

"Account **for shear buckling**")= **"Shear buckling resistance can be ignored"**

Shear area - cl 6.2.6(3); A_v = max(A-2* B* T+ (t+ 2* r)* T,η* h_w*t)= **1155.761** mm^2

Design shear resistance - cl 6.2.6(2) $V_{c,Rd} = V_{pl,Rd} = A_v* (f_y/\sqrt{(3)})/\gamma_{M0}$=**183.502** kN

Diagonal stiffner requierement; ds=if(M_{Ed_h}/D<$V_{pl,Rd}$, "**Not required**", "**Required**")= **"Required"**

Hogging moment design:

;Number of bolts under shear; n_h = **4.000** ;

Shear force on bolts; $V_{Ed,b} = M_{Ed\_h}/D$ = **190.355** kN

Shear force per bolt; $F_{b,Ed} = V_{Ed,b}/n_h$=**47.589** kN

k_1=min(2.8*(e_2/D_h)-1.7,1.4*(p/D_h)-1.7,2.5)= **2.033**

α_v=min(e_1/(3*D_h),p/(3*D_h)-0.25,f_{ub}/f_u,1)=**0.593**

Shear/Bearing capacity per bolt; $F_{v,Rd}$=$F_{b,Rd}$=min(k_1*α_v*f_u*d*t_p/γ_{M2},0.6*f_{ub}*A_s/γ_{M2})=**59.441** kN

Design check; hc=if($F_{b,Ed}$<$F_{b,Rd}$,"PASS","FAIL")=**"PASS"**

Utilization; $F_{b,Ed}$/ $F_{b,Rd}$=**0.801**

Opening up moment design:

;Number of bolts under tension; n_{ou} = **2.000** ;

Tension force on bolts; $T_{Ed,b} = M_{Ed\_ou}/(D+p)$ = **0.000** kN

Tension force per bolt; $T_{b,Ed} = T_{Ed,b}/n_{ou}$=**0.000** kN

k_2=0.63

Tension/Punching capacity per bolt; $F_{t,Rd}$=$B_{p,Rd}$=min(k_2*f_{ub}*A_s/γ_{M2},0.6*π*D_h*t_p*f_u/γ_{M2})=**63.302** kN

Design check; om= if($T_{b,Ed}$ < $F_{t,Rd}$,"PASS","FAIL")=**"PASS"**

Utilization; $T_{b,Ed}$ / $F_{t,Rd}$= **0.000**

Bending of plate subjected to tension from bolts:

Bending moment on plate; M_{Ed_plate}= $T_{Ed,b}$*p=**0.000** kNm

Plate thicknesss required; t_p=if(t_p>√((M_{Ed_plate}*4) / (B* 275 N/mm^2)), t_p, (√((M_{Ed_plate} × 4) / (B* 275 N/mm^2))))= **9.400** mm

Tension check with reduced flange:

Tension force on bolts; $V_{Ed,b} = M_{Ed\_h}/D$ = **190.355** kN

A_{net}; A_{net} = (B-(n_{ou}*d))*T=**1136.460**mm^2

Tension capacity of reduced flange; $F_{ft,Rd}$ =(A_{net} * f_u)/γ_{M2}=**372.759**kN

Design check; trf= if($V_{Ed,b}$ < $F_{ft,Rd}$,"PASS","FAIL")=**"PASS"**

Utilization; $V_{Ed,b}$ / $F_{ft,Rd}$ = **0.511**

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

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; $L = \mathbf{2000}$ mm

Panel height; $h = \mathbf{2500}$ mm

Thickness of load bearing leaf; $t = \mathbf{215}$ mm

Effective height; $h_{ef} = \mathbf{2500}$ mm

Effective thickness; $t_{ef} = \mathbf{215}$ mm

Masonry material details

Unit type; **Clay - Group 2**

Compressive strength of masonry unit; $f_c = \mathbf{20}$ N/mm^2

Height of unit; $h_u = \mathbf{65}$ mm

Width of unit; $w_u = \mathbf{215}$ mm

Conditioning factor; $k = \mathbf{1.0}$

- Conditioning to the air dry condition in accordance with cl.7.3.2

Shape factor - Table A.1; $d_{sf} = \mathbf{0.685}$

Mean compressive strength of masonry unit; $f_b = f_c \times k \times d_{sf} = \mathbf{13.7}$ N/mm^2

Specific weight of units; $\gamma = \mathbf{18}$ kN/m^3

Mortar type; **M4 - General Purpose**

Compressive strength of mortar; $f_m = \mathbf{4.0}$ N/mm^2

Compressive strength factor - Tbl. NA 4; $K = \mathbf{0.40}$

Characteristic compressive strength of the masonry - eq. 3.1

$f_k = K \times f_b^{0.7} \times f_m^{0.3} = \mathbf{3.79}$ N/mm^2

Short term secant modulus of elasticity factor; $K_E = \mathbf{1000}$

Modulus of elasticity - cl.3.7.2; $E_w = K_E \times f_k = \mathbf{3788}$ N/mm^2

Design compressive strength of masonry

Category of manufacturing control; **Category II**

Class of execution control; **Class 2**

Partial factor for material strength in direct or flexural compression

$\gamma_M = \mathbf{3.00}$

Cross-sectional area of wall; $A = L \times t = \mathbf{0.43}$ m^2;

Design compressive strength of masonry; $f_d = f_k / \gamma_M = \mathbf{1.26}$ N/mm^2

Partial safety factors for design loads

Partial safety factor for permanent load; $\gamma_{fG} = \mathbf{1.35}$

Partial safety factor for variable load; $\gamma_{fQ} = \mathbf{1.50}$

Superimposed vertical loading details

Permanent UDL at top of wall; $g_k = \mathbf{2.00}$ kN/m

Variable UDL at top of wall; $q_k = \mathbf{1.30}$ kN/m

Eccentricity of permanent UDL load; $e_{gu} = \mathbf{0}$ mm

Eccentricity of variable UDL load; $e_{qu} = \mathbf{0}$ mm

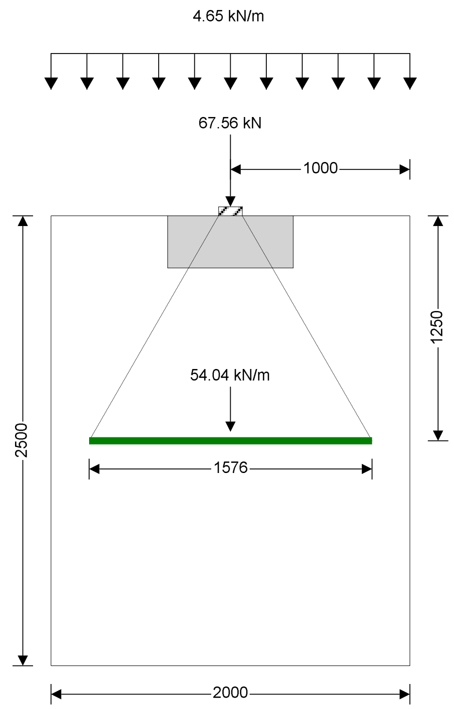
Slenderness ratio of masonry wall - Section 5.5.1.4

Slenderness ratio limit; $\lambda_{lim} = \mathbf{27}$

Slenderness ratio; $\lambda = h_{ef} / t_{ef} = \mathbf{11.6}$

PASS - Slenderness ratio is less than slenderness limit

Concentrated Load 1 details - SB2 Reaction



Permanent concentrated load; G_{kc1} = **42.60** kN
Variable concentrated load; Q_{kc1} = **6.70** kN
Eccentricity of concentrated load; e_{c1} = **54** mm
Length of concentrated load; L_{c1} = **133** mm
Width of concentrated load; w_{c1} = **100** mm
Height of concentrated load; h_{c1} = **2500** mm
Distance of load to right vertical edge; r_{11} = **934** mm
Distance of load to nearest vertical edge; a_{11} = **934** mm

Walls subjected to concentrated loads - Section 6.1.3

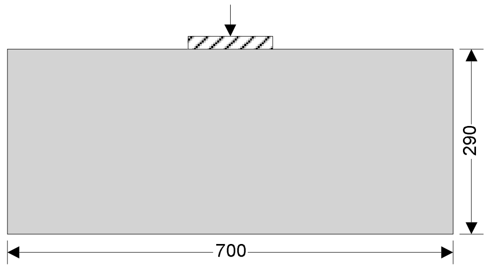
Eccentricity check; $e_{c1} <= t / 4$

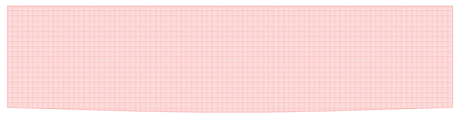
PASS - Eccentricity of load is less than t/4

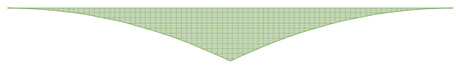
Area of bearing; $A_{b1} = L_{c1} \times w_{c1}$ = **13300** mm^2
Effective length of bearing at mid-height; $l_{efm1} = L_{c1} + h_{c1} \times \tan(30)$ = **1576** mm
Effective bearing area; $A_{ef1} = l_{efm1} \times t$ = **338920.77** mm^2
Bearing area ratio check; $A_{ratio1} = \text{Min}(A_{b1} / A_{ef1}, 0.45)$ = **0.04**
Enhancement factor - cl.6.1.3(3); β_1 = **1.00**
Design value of the concentrated load; $N_{Edc1} = G_{kc1} \times \gamma_{fG} + Q_{kc1} \times \gamma_{fQ}$ = **67.56** kN
Design value concentrated load resistance; $N_{Rdc1} = \beta_1 \times A_{b1} \times f_d$ = **16.79** kN

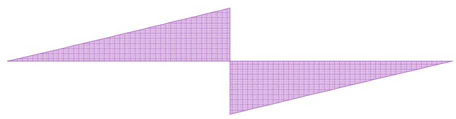
Applied concentrated load exceeds design resistance, spreader required!

Design of spreader beam




Max. stress, 1.20 N/mm^2


5.86 kNm
Bending Moment


-33.78 kN
Shear Force

Type of spreader;	**Concrete padstone**
Type of bearing onto spreader;	**Point load**
Location of load from RHS of spreader;	P_{11} = **350** mm
Length of spreader;	L_{sp1} = **700** mm
Height of spreader;	h_{sp1} = **290** mm
Width of spreader;	w_{sp1} = **100** mm
Eccentricity of load on spreader;	e_{sp1} = **4** mm
Modulus of elasticity;	E_{sp1} = **29962** N/mm^2
Second moment of area;	$I_{sp1} = 1/12 \times w_{sp1} \times h_{sp1}^3$ = **203241667** mm^4
Modulus of the wall;	$k_0 = E_w / h$ = **1.52** N/mm^2/mm
Winkler's constant;	$K_{c1} = k_0 \times w_{sp1}$ = **151.51** N/mm/mm
Characteristic of the system;	$\alpha_1 = (K_{c1} / (4 \times E_{sp1} \times I_{sp1}))^{1/4}$ = **0.00158** mm^{-1}
Classification of spreader;	$\alpha L_1 = \alpha_1 \times L_{sp1}$ = **1.11**; **Medium**
Krilov's functions for the spreader length;	$B_{\alpha l1} = 1/2 \times (\cosh(\alpha L_1) \times \sin(180 \times \alpha L_1 / \pi) + \sinh(\alpha L_1) \times \cos(180 \times \alpha L_1 / \pi))$ = **1.05**
	$C_{\alpha l1} = 1/2 \times \sinh(\alpha L_1) \times \sin(180 \times \alpha L_1 / \pi)$ = **0.60**
	$D_{\alpha l1} = 1/4 \times (\cosh(\alpha L_1) \times \sin(180 \times \alpha L_1 / \pi) - \sinh(\alpha L_1) \times \cos(180 \times \alpha L_1 / \pi))$ = **0.22**
Krilov's functions at the point load;	$A_{\alpha P11} = \cosh(\alpha_1 \times P_{11}) \times \cos(180 \times \alpha_1 \times P_{11} / \pi)$ = **0.98**
	$B_{\alpha P11} = 1/2 \times (\cosh(\alpha_1 \times P_{11}) \times \sin(180 \times \alpha_1 \times P_{11} / \pi) + \sinh(\alpha_1 \times P_{11}) \times \cos(180 \times \alpha_1 \times P_{11} / \pi))$ = **0.55**

Using method of initial conditions

Initial moment of LH edge; M_{01} = **0** kNm

Initial shear of LH edge; V_{01} = **0** kN

Which gives; $(4 \times \alpha_1^2 \times C_{\alpha l1} \times \delta_{01} + 4 \times \alpha_1 \times D_{\alpha l1} \times \Phi_{01}) \times E_{sp1} \times I_{sp1} - B_{\alpha P11} / \alpha_1 \times N_{Edc1}$ = **0.00** kNm

and; $(4 \times \alpha_1^3 \times B_{\alpha l1} \times \delta_{01} + 4 \times \alpha_1^2 \times C_{\alpha l1} \times \Phi_{01}) \times E_{sp1} \times I_{sp1} - A_{\alpha P11} \times N_{Edc1}$ = **0.00** kN

Therefore,

Initial deflection of LH edge; δ_{01} = **0.61939** mm

Initial rotationof LH edge; Φ_{01} = **0.000112**

Location of maximum deflection; x_{def1} = **350** mm

Krilov's functions at the spreader length; $A_{\alpha xdef1} = \cosh(\alpha_1 \times x_{def1}) \times \cos(180 \times \alpha_1 \times x_{def1} / \pi)$ = **0.98**

$B_{\alpha xdef1} = 1/2 \times (\cosh(\alpha_1 \times x_{def1}) \times \sin(180 \times \alpha_1 \times x_{def1} / \pi) + \sinh(\alpha_1 \times x_{def1}) \times \cos(180 \times \alpha_1 \times x_{def1} / \pi))$ = **0.55**

Distance of point load right of loaction; p_{1def1} = **0** mm

Krilov's functions at the spreader length; $D_{\alpha p1def1} = 1/4 \times (\cosh(\alpha_1 \times p_{1def1}) \times \sin(180 \times \alpha_1 \times p_{1def1} / \pi) - \sinh(\alpha_1 \times p_{1def1}) \times \cos(180 \times \alpha_1 \times p_{1def1} / \pi))$ = **0.00**

Particular integral due to load; $\delta'_1 = D_{\alpha p1def1} / \alpha_1^3 \times N_{Edc1} / (I_{sp1} \times E_{sp1})$ = **0.000** mm

Maximum deflection; $\delta_{max1} = A_{\alpha xdef1} \times \delta_{01} + B_{\alpha xdef1} \times \Phi_{01} / \alpha_1 + \delta'_1$ = **0.649** mm

Location of maximum moment; x_{M1} = **350** mm

Krilov's functions at the spreader length; $C_{\alpha xM1} = 1/2 \times \sinh(\alpha_1 \times x_{M1}) \times \sin(180 \times \alpha_1 \times x_{M1} / \pi)$ = **0.15**

$D_{\alpha xM1} = 1/4 \times (\cosh(\alpha_1 \times x_{M1}) \times \sin(180 \times \alpha_1 \times x_{M1} / \pi) - \sinh(\alpha_1 \times x_{M1}) \times \cos(180 \times \alpha_1 \times x_{M1} / \pi))$ = **0.03**

Distance of point load right of loaction; p_{1M1} = **0** mm

Krilov's functions at the spreader length; $B_{\alpha p1M1} = 1/2 \times (\cosh(\alpha_1 \times p_{1M1}) \times \sin(180 \times \alpha_1 \times p_{1M1} / \pi) + \sinh(\alpha_1 \times p_{1M1}) \times \cos(180 \times \alpha_1 \times p_{1M1} / \pi))$ = **0.00**

Particular integral due to load; $M'_1 = -B_{\alpha p1M1} / \alpha_1 \times N_{Edc1}$ = **0.00** kNm

Maximum moment; $M_{Edsp1} = (4 \times \alpha_1^2 \times C_{\alpha xM1} \times \delta_{01} + 4 \times \alpha_1 \times D_{\alpha xM1} \times \Phi_{01}) \times (I_{sp1} \times E_{sp1}) + M'_1$ = **5.86** kNm

Location of maximum shear; x_{V1} = **350** mm

Krilov's functions at the spreader length; $B_{\alpha xV1} = 1/2 \times (\cosh(\alpha_1 \times x_{V1}) \times \sin(180 \times \alpha_1 \times x_{V1} / \pi) + \sinh(\alpha_1 \times x_{V1}) \times \cos(180 \times \alpha_1 \times x_{V1} / \pi))$ = **0.55**

$C_{\alpha xV1} = 1/2 \times \sinh(\alpha_1 \times x_{V1}) \times \sin(180 \times \alpha_1 \times x_{V1} / \pi)$ = **0.15**

Distance of point load right of loaction; p_{1V1} = **0** mm

Krilov's functions at the spreader length; $A_{\alpha p1V1} = \cosh(\alpha_1 \times p_{1V1}) \times \cos(180 \times \alpha_1 \times p_{1V1} / \pi)$ = **1.00**

Particular integral due to load; $V'_1 = -A_{\alpha p1V1} \times N_{Edc1}$ = **-67.56** kN

Shear at concentrated point load; $V_1 = (4 \times \alpha_1^3 \times B_{\alpha xV1} \times \delta_{01} + 4 \times \alpha_1^2 \times C_{\alpha xV1} \times \Phi_{01}) \times (I_{sp1} \times E_{sp1}) + V'_1$ = **-33.78** kN

Maximum shear; $V_{Edsp1} = \mathrm{Max}(\mathrm{Abs}(V_1), N_{Edc1} - \mathrm{Abs}(V_1))$ = **33.78** kN

Maximum allowable stress under spreader; $\sigma_{Rdsp1} = \beta_1 \times f_d$ = **1.26** N/mm^2

Maximum reaction; $N_{Edsp1} = K_{c1} \times \delta_{max1}$ = **98.30** kN/m

Design stress; $\sigma_{Edsp1} = N_{Edsp1} / w_{sp1} \times (1 + 6 \times e_{sp1} / w_{sp1})$ = **1.20** N/mm^2

PASS - Design stress under spreader is less than the allowable bearing stress

Walls subjected to mainly vertical loading - Section6.1.2

Eccentricity of permanent UDL at mid-height below concentrated load

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

Eccentricity of variable UDL at mid-height below concentrated load

$e_{qmu1} = e_{qu} \times h_{c1} / (2 \times h) = \mathbf{0.0}$ mm

Eccentricity of concentrated load at mid-height; $e_{mc1} = e_{c1} / 2 = \mathbf{26.9}$ mm

Initial eccentricity - cl.5.5.1.1(4); $e_{init} = h / 450 = \mathbf{5.6}$ mm

Concentrated load at mid-height as UDL; $N_{mc1} = N_{Edc1} / l_{efm1} = \mathbf{42.86}$ kN/m

Vertical load at mid-height; $N_{Ed1} = (g_k + \gamma \times t \times (h - h_{c1} / 2)) \times \gamma_{fG} + q_k \times \gamma_{fQ} + N_{mc1} = \mathbf{54.04}$ kN/m

Design moment at mid-height; $M_{Ed1} = g_k \times \gamma_{fG} \times e_{gmu1} + q_k \times \gamma_{fQ} \times e_{qmu1} + N_{mc1} \times e_{mc1} = \mathbf{1.15}$ kNm/m

Eccentricities due to loads - eq. 6.7; $e_{m1} = Abs(M_{Ed1}) / N_{Ed1} + e_{init} = \mathbf{26.9}$ mm

Slenderness ratio limit for creep eccentricity; $\lambda_c = \mathbf{27}$

Eccentricity due to creep; $e_{k1} = \mathbf{0.0}$ mm

Eccentricity at mid-height - eq. 6.6; $e_{mk1} = Max(e_{m1} + e_{k1}, 0.05 \times t) = \mathbf{26.9}$ mm

From eq. G2; $A_{11} = 1 - 2 \times e_{mk1} / t = \mathbf{0.75}$

From eq. G3; $u_1 = (h_{ef} / t_{ef} \times (1 / K_E)^{1/2} - 0.063) / (0.73 - 1.17 \times e_{mk1} / t) = \mathbf{0.52}$

Capacity reduction factor - eq. G1; $\Phi_{m1} = A_{11} \times exp(-(u_1^2) / 2) = \mathbf{0.65}$

Design vertical resistance of panel - eq.6.2; $N_{Rd1} = \Phi_{m1} \times t \times f_d = \mathbf{177.68}$ kN/m

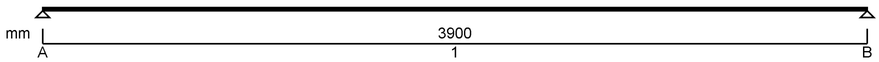
PASS - Design value of vertical resistance exceeds applied vertical load

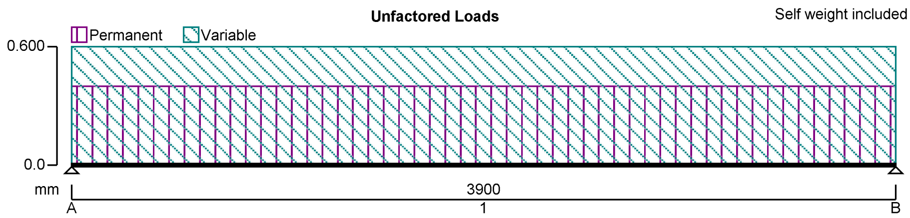
Timber structural members

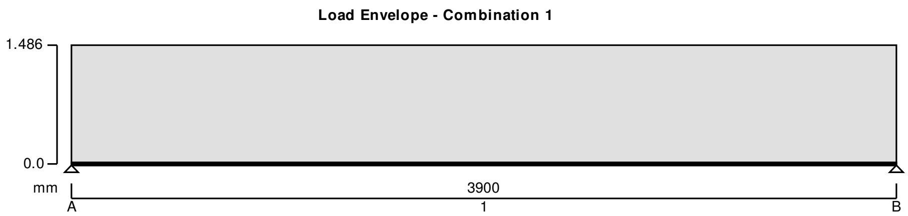
NEW ROOF TERRACE JOISTS

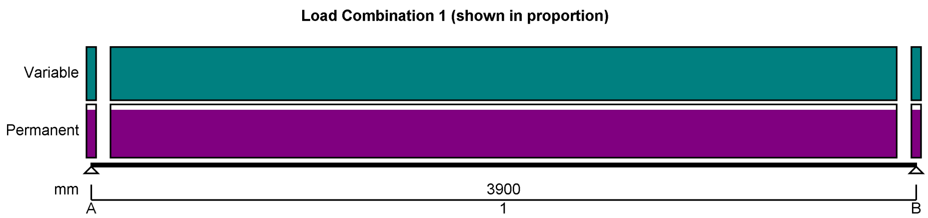
In accordance with EN1995-1-1:2004 + A1:2008 and Corrigendum No.1 and the UK National Annex incorporating National Amendment No.1

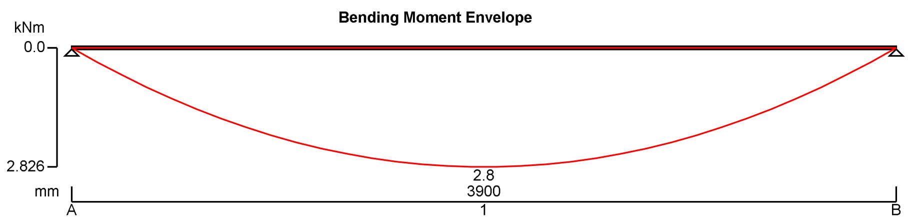
Tedds calculation version 1.5.11

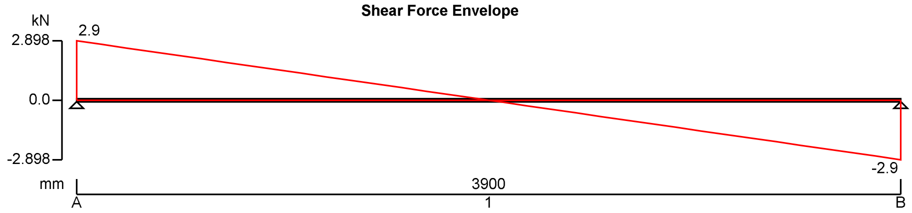












Applied loading

Beam loads

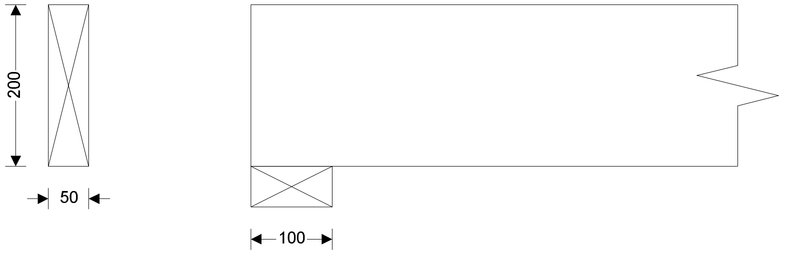
Timber self weight	Permanent self weight of beam × 1
Terrace roof dead load - 1.0kN/m2 x 0.4m	Permanent full UDL 0.400 kN/m
Terrace roof live load - 1.5kN/m2 x 0.4m	Variable full UDL 0.600 kN/m

Load combinations

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

Analysis results

Maximum moment;	M_{max} = **2.826** kNm;	M_{min} = **0.000** kNm
Design moment;	M = max(abs(M_{max}),abs(M_{min})) = **2.826** kNm	
Maximum shear;	F_{max} = **2.898** kN;	F_{min} = **-2.898** kN
Design shear;	F = max(abs(F_{max}),abs(F_{min})) = **2.898** kN	
Total load on beam;	W_{tot} = **5.797** kN	
Reactions at support A;	R_{A_max} = **2.898** kN;	R_{A_min} = **2.898** kN
Unfactored permanent load reaction at support A;	$R_{A\_Permanent}$ = **0.847** kN	
Unfactored variable load reaction at support A;	$R_{A\_Variable}$ = **1.170** kN	
Reactions at support B;	R_{B_max} = **2.898** kN;	R_{B_min} = **2.898** kN
Unfactored permanent load reaction at support B;	$R_{B\_Permanent}$ = **0.847** kN	
Unfactored variable load reaction at support B;	$R_{B\_Variable}$ = **1.170** kN	



Timber section details

Breadth of timber sections;	b = **50** mm
Depth of timber sections;	h = **200** mm
Number of timber sections in member;	N = **1**
Overall breadth of timber member;	b_b = N × b = **50** mm
Timber strength class - EN 338:2009 Table 1;	**C24**

Member details

Load duration - cl.2.3.1.2; **Long-term**

Service class of timber - cl.2.3.1.3; **1**

Length of bearing; L_b = **100** mm

Section properties

Cross sectional area of member; $A = N \times b \times h$ = **10000** mm^2

Section modulus; $W_y = N \times b \times h^2 / 6$ = **333333** mm^3

$W_z = h \times (N \times b)^2 / 6$ = **83333** mm^3

Second moment of area; $I_y = N \times b \times h^3 / 12$ = **33333333** mm^4

$I_z = h \times (N \times b)^3 / 12$ = **2083333** mm^4

Radius of gyration; $r_y = \sqrt{(I_y / A)}$ = **57.7** mm

$r_z = \sqrt{(I_z / A)}$ = **14.4** mm

Partial factor for material properties and resistances

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

Modification factors

Modification factor for load duration and moisture content - Table 3.1

k_{mod} = **0.700**

Deformation factor for service classes - Table 3.2; k_{def} = **0.600**

Depth factor for bending - exp.3.1; $k_{h.m}$ = **1.000**

Depth factor for tension - exp.3.1; $k_{h.t}$ = **1.000**

Bending stress re-distribution factor - cl.6.1.6(2); k_m = **0.700**

Crack factor for shear resistance - cl.6.1.7(2); k_{cr} = **0.670**

Load configuration factor - exp.6.4; $k_{c.90}$ = **1.500**

System strength factor - cl.6.6; k_{sys} = **1.000**

Effective length - Table 6.1; $L_{ef} = 1.0 \times L_{s1}$ = **3900** mm

Critical bending stress - exp.6.32; $\sigma_{m.crit} = 0.78 \times (N \times b)^2 \times E_{0.05} / (h \times L_{ef})$ = **18.500** N/mm^2

Relative slenderness for bending - exp.6.30; $\lambda_{rel.m} = \sqrt{[f_{m.k} / \sigma_{m.crit}]}$ = **1.139**

Lateral buckling factor - exp.6.34; $k_{crit} = 1.56 - 0.75 \times \lambda_{rel.m}$ = **0.706**

Compression perpendicular to the grain - cl.6.1.5

Design compressive stress; $\sigma_{c.90.d} = R_{A\_max} / (N \times b \times (L_b + \min(L_b, 30\text{ mm})))$ = **0.446** N/mm^2

Design compressive strength; $f_{c.90.d} = k_{mod} \times k_{sys} \times k_{c.90} \times f_{c.90.k} / \gamma_M$ = **2.019** N/mm^2

$\sigma_{c.90.d} / f_{c.90.d}$ = **0.221**

PASS - Design compressive strength exceeds design compressive stress at bearing

Bending - cl 6.1.6

Design bending stress; $\sigma_{m.d} = M / W_y$ = **8.478** N/mm^2

Design bending strength; $f_{m.d} = k_{h.m} \times k_{mod} \times k_{sys} \times k_{crit} \times f_{m.k} / \gamma_M$ = **9.121** N/mm^2

$\sigma_{m.d} / f_{m.d}$ = **0.930**

PASS - Design bending strength exceeds design bending stress

Shear - cl.6.1.7

Applied shear stress; $\tau_d = 3 \times F / (2 \times k_{cr} \times A)$ = **0.649** N/mm^2

Permissible shear stress; $f_{v.d} = k_{mod} \times k_{sys} \times f_{v.k} / \gamma_M$ = **2.154** N/mm^2

$\tau_d / f_{v.d}$ = **0.301**

PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

Deflection limit; $\delta_{lim} = \min(14\text{ mm}, 0.004 \times L_{s1})$ = **14.000** mm

Instantaneous deflection due to permanent load; δ_{instG} = **3.712** mm

Final deflection due to permanent load; $\delta_{finG} = \delta_{instG} \times (1 + k_{def})$ = **5.939** mm

Instantaneous deflection due to variable load; δ_{instQ} = **5.128** mm

Factor for quasi-permanent variable action; ψ_2 = **0.3**

Final deflection due to variable load; $\delta_{finQ} = \delta_{instQ} \times (1 + \psi_2 \times k_{def})$ = **6.051** mm

Total final deflection; $\delta_{fin} = \delta_{finG} + \delta_{finQ}$ = **11.989** mm

$\delta_{fin} / \delta_{lim}$ = **0.856**

PASS - Total final deflection is less than the deflection limit

Foundations

FOUNDATION OF BOX-FRAME UNDER ORIGINAL BACK WALL

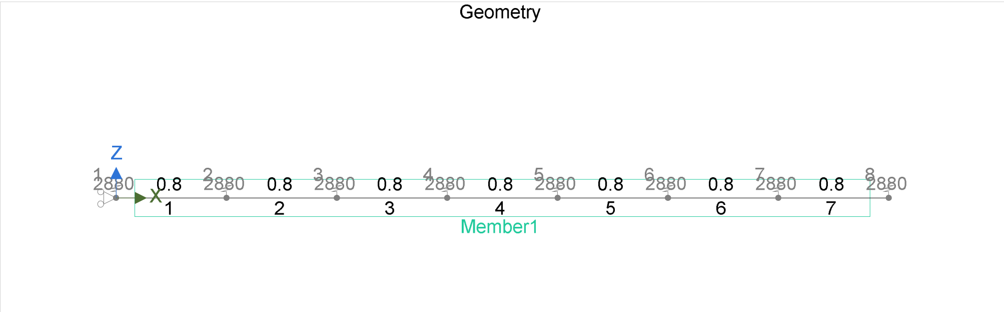
In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

Tedds calculation version 4.0.02

Analysis

Tedds calculation version 1.0.13

Geometry



Nodes

Node	Co-ordinates X (m)	Co-ordinates Z (m)	Freedom X	Freedom Z	Freedom Rot.	Coordinate system Name	Coordinate system Angle (°)	Spring X (kN/m)	Spring Z (kN/m)	Spring Rot. 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/mm²	Shear Modulus kN/mm²	Thermal Coefficient °C⁻¹
Steel (EC3)	7850	210	80.8	0.000012

Sections

Name	Area (cm²)	Moment of inertia Major (cm⁴)	Moment of inertia Minor (cm⁴)	Shear area A_y (cm²)	Shear area A_z (cm²)
UC 254x254x89	113	14268	4857	80	27

Elements

Element	Length (m)	Nodes Start	Nodes End	Section	Material	Releases Start moment	Releases End moment	Releases Axial	Rotated
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	

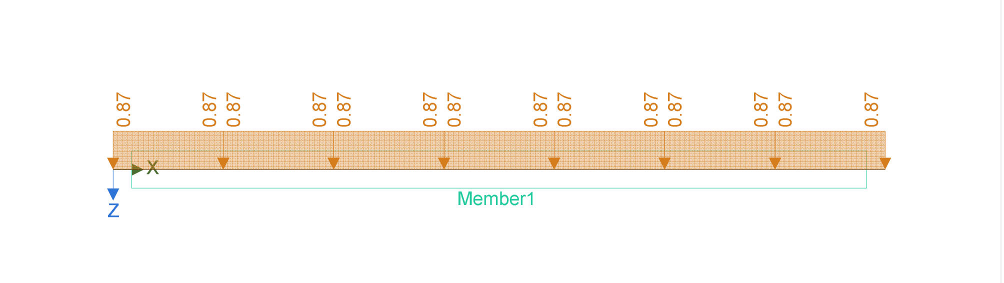
Element	Length (m)	Nodes		Section	Material	Releases			Rotated
		Start	End			Start moment	End moment	Axial	
5	0.8	5	6	UC 254x254x89	Steel (EC3)	Fixed	Fixed	Fixed	
6	0.8	6	7	UC 254x254x89	Steel (EC3)	Fixed	Fixed	Fixed	
7	0.8	7	8	UC 254x254x89	Steel (EC3)	Fixed	Fixed	Fixed	

Members

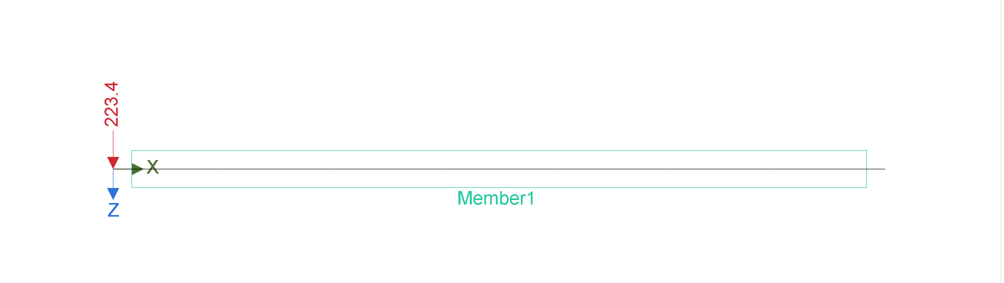
Name	Elements	
	Start	End
Member1	1	7

Loading

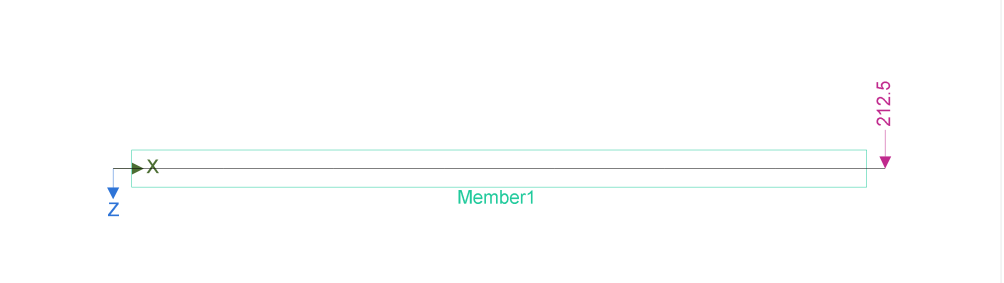
Self Weight - Loading



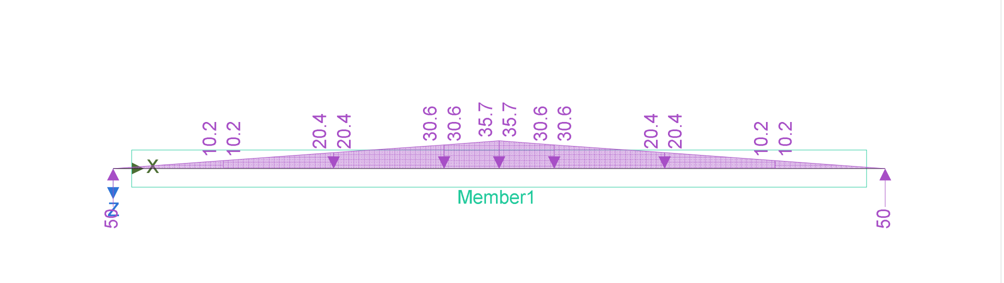
Left Column1 - Loading



Right Column1 - Loading



Pre-Deflection - Loading



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	Type	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	Force		Moment
		X (kN)	Z (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

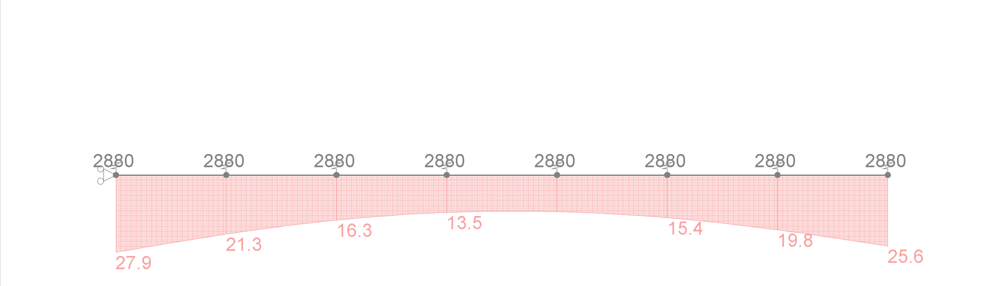
Member VDL loads

Member	Load case	Position			Load		Orientation
		Type	Start	End	Start (kN/m)	End (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

Service combinations - Deflection envelope (mm)



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Partial factors - Section 6.1

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

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

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

Library item: Partial factors out

Member1 - Span 1

Section details

Section type; UC 254x254x89 (BS4-1)

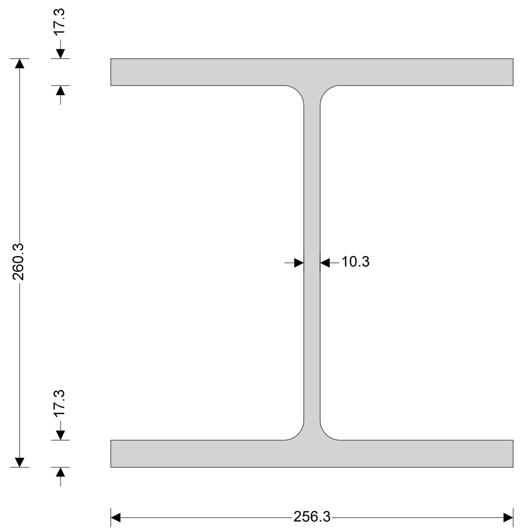
Steel grade - EN 10025-2:2004; S275

Nominal thickness of element; $t_{nom} = max(t_f, t_w) = \mathbf{17.3}$ mm

Nominal yield strength; $f_y = \mathbf{265}$ N/mm^2

Nominal ultimate tensile strength; $f_u = \mathbf{410}$ N/mm^2

Modulus of elasticity; $E = \mathbf{210000}$ N/mm^2



UC 254x254x89 (BS4-1)
Section depth, h, 260.3 mm
Section breadth, b, 256.3 mm
Mass of section, Mass, 88.9 kg/m
Flange thickness, t_f, 17.3 mm
Web thickness, t_w, 10.3 mm
Root radius, r, 12.7 mm
Area of section, A, 11331 mm^2
Radius of gyration about y-axis, i_y, 112 mm
Radius of gyration about z-axis, i_z, 65 mm
Elastic section modulus about y-axis, $W_{el.y}$, 1096254 mm^3
Elastic section modulus about z-axis, $W_{el.z}$, 379046 mm^3
Plastic section modulus about y-axis, $W_{pl.y}$, 1223863 mm^3
Plastic section modulus about z-axis, $W_{pl.z}$, 575307 mm^3
Second moment of area about y-axis, I_y, 142677465 mm^4
Second moment of area about z-axis, I_z, 48574741 mm^4

Lateral restraint

Both flanges have lateral restraint at supports only

Check design at start of span

Check y-y axis deflection - Section 7.2.1

Maximum deflection; $\delta_y = \mathbf{23.9}$ mm

Allowable deflection; $\delta_{y,Allowable} = Min(L_{m1\_s1} / 180, 25\ mm) = \mathbf{25}$ mm

$\delta_y / \delta_{y,Allowable} = \mathbf{0.956}$

PASS - Design deflection does not exceed allowable deflection

<u>FOUNDATION UNDER REAR BOX-FRAME</u>

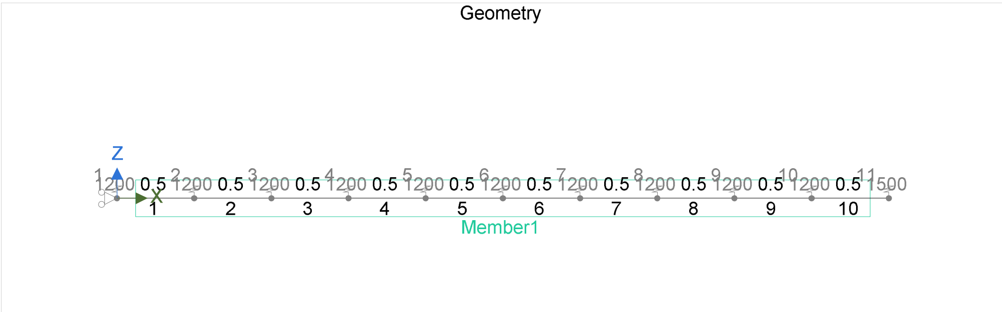
In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

Tedds calculation version 4.0.02

Analysis

Tedds calculation version 1.0.13

Geometry



Nodes

Node	Co-ordinates		Freedom			Coordinate system		Spring		
	X (m)	Z (m)	X	Z	Rot.	Name	Angle (°)	X (kN/m)	Z (kN/m)	Rot. 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 (kg/m^3)	Youngs Modulus kN/mm^2	Shear Modulus kN/mm^2	Thermal Coefficient °C^{-1}
Steel (EC3)	7850	210	80.8	0.000012

Sections

Name	Area	Moment of inertia		Shear area	
	(cm^2)	Major (cm^4)	Minor (cm^4)	A_y (cm^2)	A_z (cm^2)
UC 152x152x30	38	1748	560	26	10

Elements

Element	Length (m)	Nodes		Section	Material	Releases			Rotated
		Start	End			Start moment	End moment	Axial	
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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	Section Building Control Approval				Sheet no./rev. 161	
	Calc. by RC	Date 26/06/2017	Chk'd by MD	Date 26/06/2017	App'd by	Date

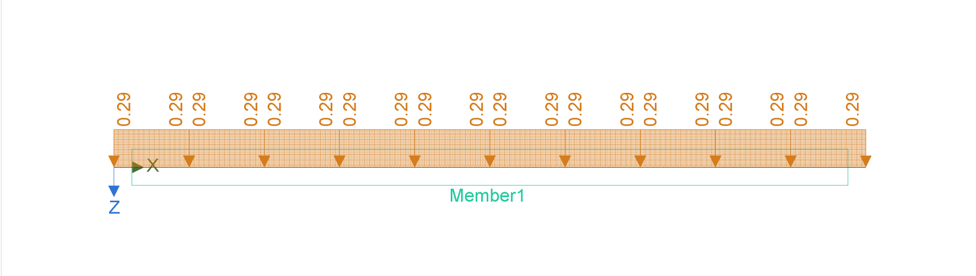
Element	Length (m)	Nodes		Section	Material	Releases			Rotated
		Start	End			Start moment	End moment	Axial	
4	0.5	4	5	UC 152x152x30	Steel (EC3)	Fixed	Fixed	Fixed	
5	0.5	5	6	UC 152x152x30	Steel (EC3)	Fixed	Fixed	Fixed	
6	0.5	6	7	UC 152x152x30	Steel (EC3)	Fixed	Fixed	Fixed	
7	0.5	7	8	UC 152x152x30	Steel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

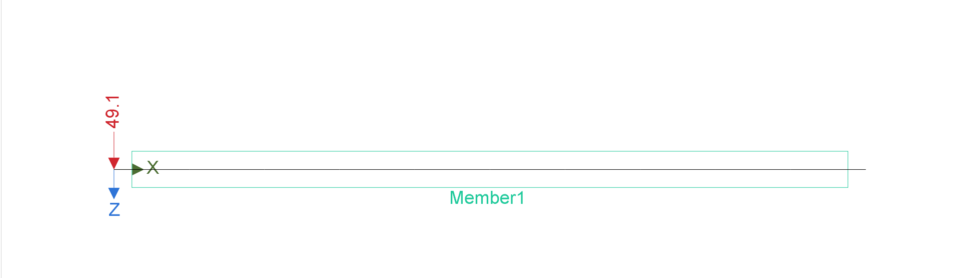
Name	Elements	
	Start	End
Member1	1	10

Loading

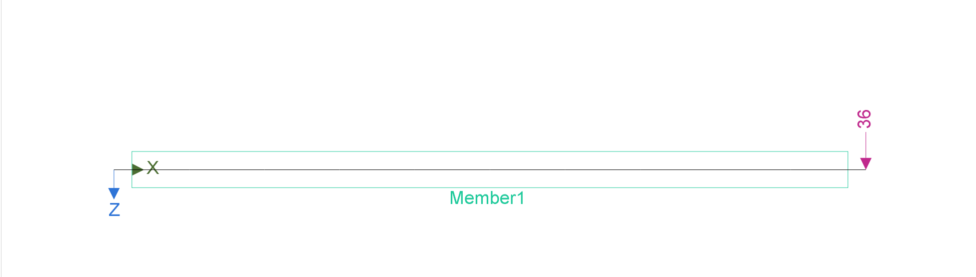
Self Weight - Loading



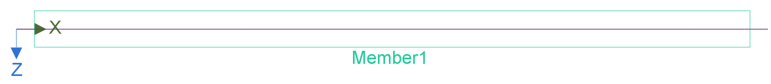
Left Column - Loading



Right Column - Loading



Pre-Deflection - Loading



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	Type	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 X (kN)	Force Z (kN)	Moment (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

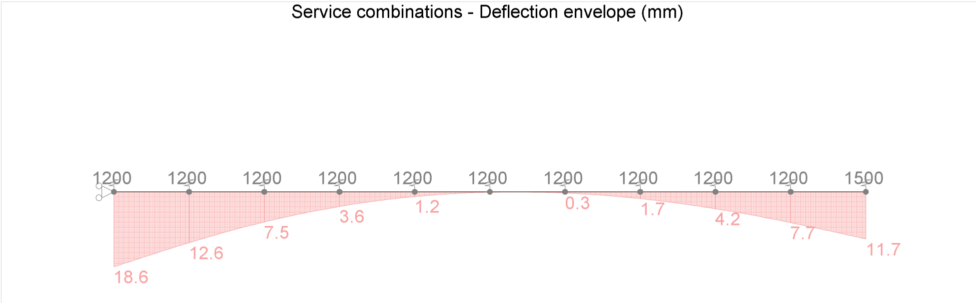
Member VDL loads

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

Results

SD structures Chartered Engineers SD Structures 107 Fleet Street, London EC4A 2AB www.sd-structures.com	Project 28 Burghley Road				Job Ref. SDS187	
	Section Building Control Approval				Sheet no./rev. 163	
	Calc. by RC	Date 26/06/2017	Chk'd by MD	Date 26/06/2017	App'd by	Date

Forces



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Partial factors - Section 6.1

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

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

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

Library item: Partial factors out

Member1 - Span 1

Section details

Section type; UC 152x152x30 (BS4-1)

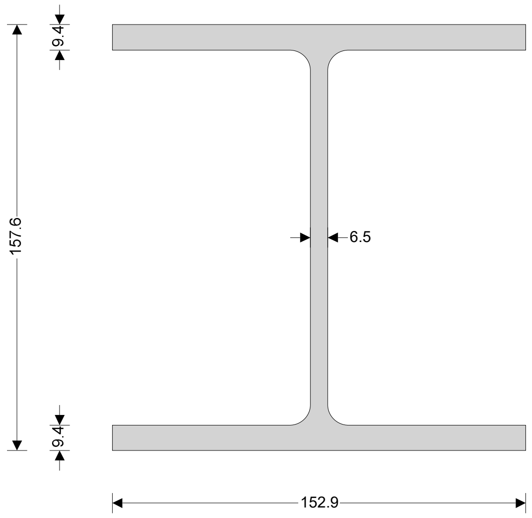
Steel grade - EN 10025-2:2004; S275

Nominal thickness of element; t_{nom} = max(t_f, t_w) = **9.4** mm

Nominal yield strength; f_y = **275** N/mm^2

Nominal ultimate tensile strength; f_u = **410** N/mm^2

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



UC 152x152x30 (BS4-1)
Section depth, h, 157.6 mm
Section breadth, b, 152.9 mm
Mass of section, Mass, 30 kg/m
Flange thickness, t_f, 9.4 mm
Web thickness, t_w, 6.5 mm
Root radius, r, 7.6 mm
Area of section, A, 3826 mm^2
Radius of gyration about y-axis, i_y, 68 mm
Radius of gyration about z-axis, i_z, 38 mm
Elastic section modulus about y-axis, $W_{el.y}$, 221833 mm^3
Elastic section modulus about z-axis, $W_{el.z}$, 73311 mm^3
Plastic section modulus about y-axis, $W_{pl.y}$, 247665 mm^3
Plastic section modulus about z-axis, $W_{pl.z}$, 111590 mm^3
Second moment of area about y-axis, I_y, 17480413 mm^4
Second moment of area about z-axis, I_z, 5604633 mm^4

Lateral restraint

Both flanges have lateral restraint at supports only

Check design at start of span

Check y-y axis deflection - Section 7.2.1

Maximum deflection; δ_y = **18.6** mm

Allowable deflection; $\delta_{y,Allowable}$ = Min(L_{m1_s1} / 180, 25 mm) = **25** mm

δ_y / $\delta_{y,Allowable}$ = **0.745**

PASS - Allowable deflection exceeds design deflection

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 version 2.9.00

Retaining wall details

Stem type;	Cantilever
Stem height;	h_{stem} = **900** mm
Stem thickness;	t_{stem} = **150** mm
Angle to rear face of stem;	α = **90** deg
Stem density;	γ_{stem} = **25** kN/m^3
Toe length;	l_{toe} = **1000** mm
Base thickness;	t_{base} = **150** mm
Base density;	γ_{base} = **25** kN/m^3
Height of retained soil;	h_{ret} = **900** mm
Angle of soil surface;	β = **0** deg
Depth of cover;	d_{cover} = **0** mm
Height of water;	h_{water} = **900** mm
Water density;	γ_w = **9.8** kN/m^3

Retained soil properties

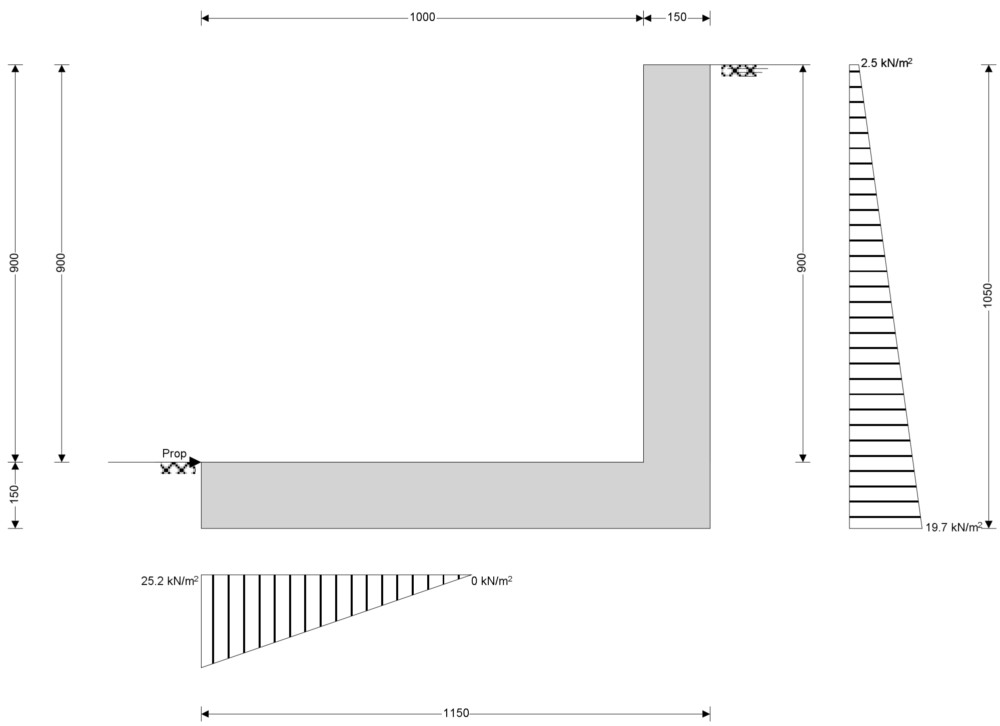
Soil type;	Medium dense well graded sand
Moist density;	γ_{mr} = **21** kN/m^3
Saturated density;	γ_{sr} = **23** kN/m^3
Characteristic effective shear resistance angle;	$\phi'_{r.k}$ = **30** deg
Characteristic wall friction angle;	$\delta_{r.k}$ = **0** deg

Base soil properties

Soil type;	Medium dense well graded sand
Soil density;	γ_b = **18** kN/m^3
Characteristic effective shear resistance angle;	$\phi'_{b.k}$ = **30** deg
Characteristic wall friction angle;	$\delta_{b.k}$ = **15** deg
Characteristic base friction angle;	$\delta_{bb.k}$ = **30** deg
Presumed bearing capacity;	$P_{bearing}$ = **100** kN/m^2

Loading details

Variable surcharge load;	$Surcharge_Q$ = **5** kN/m^2



General arrangement

Calculate retaining wall geometry

Base length;	$l_{base} = l_{toe} + t_{stem} =$ **1150** mm
Saturated soil height;	$h_{sat} = h_{water} + d_{cover} =$ **900** mm
Moist soil height;	$h_{moist} = h_{ret} - h_{water} =$ **0** mm
Length of surcharge load;	$l_{sur} = l_{heel} =$ **0** mm
- Distance to vertical component;	$x_{sur\_v} = l_{base} - l_{heel} / 2 =$ **1150** mm
Effective height of wall;	$h_{eff} = h_{base} + d_{cover} + h_{ret} =$ **1050** mm
- Distance to horizontal component;	$x_{sur\_h} = h_{eff} / 2 =$ **525** mm
Area of wall stem;	$A_{stem} = h_{stem} \times t_{stem} =$ **0.135** m^2
- Distance to vertical component;	$x_{stem} = l_{toe} + t_{stem} / 2 =$ **1075** mm
Area of wall base;	$A_{base} = l_{base} \times t_{base} =$ **0.173** m^2
- Distance to vertical component;	$x_{base} = l_{base} / 2 =$ **575** mm

Using Rankine theory

At rest pressure coefficient;	$K_0 = 1 - \sin(\phi'_{r.k}) =$ **0.500**
Passive pressure coefficient;	$K_P = (1 + \sin(\phi'_{b.k})) / (1 - \sin(\phi'_{b.k})) =$ **3.000**

Bearing pressure check

Vertical forces on wall

Wall stem;	$F_{stem} = A_{stem} \times \gamma_{stem} =$ **3.4** kN/m
Wall base;	$F_{base} = A_{base} \times \gamma_{base} =$ **4.3** kN/m
Total;	$F_{total\_v} = F_{stem} + F_{base} + F_{water\_v} =$ **7.7** kN/m

Horizontal forces on wall

Surcharge load;	$F_{sur\_h} = K_0 \times Surcharge_Q \times h_{eff} =$ **2.6** kN/m

Saturated retained soil;	$F_{sat\_h} = K_0 \times (\gamma_{sr}' - \gamma_w') \times (h_{sat} + h_{base})^2 / 2 = \mathbf{3.6}$ kN/m
Water;	$F_{water\_h} = \gamma_w' \times (h_{water} + d_{cover} + h_{base})^2 / 2 = \mathbf{5.4}$ kN/m
Base soil;	$F_{pass\_h} = -K_P \times \gamma_b' \times (d_{cover} + h_{base})^2 / 2 = \mathbf{-0.6}$ kN/m
Total;	$F_{total\_h} = F_{sat\_h} + F_{moist\_h} + F_{pass\_h} + F_{water\_h} + F_{sur\_h} = \mathbf{11.1}$ kN/m

Moments on wall

Wall stem;	$M_{stem} = F_{stem} \times x_{stem} = \mathbf{3.6}$ kNm/m
Wall base;	$M_{base} = F_{base} \times x_{base} = \mathbf{2.5}$ kNm/m
Surcharge load;	$M_{sur} = -F_{sur\_h} \times x_{sur\_h} = \mathbf{-1.4}$ kNm/m
Saturated retained soil;	$M_{sat} = -F_{sat\_h} \times x_{sat\_h} = \mathbf{-1.3}$ kNm/m
Water;	$M_{water} = -F_{water\_h} \times x_{water\_h} = \mathbf{-1.9}$ kNm/m
Total;	$M_{total} = M_{stem} + M_{base} + M_{sat} + M_{moist} + M_{water} + M_{sur} = \mathbf{1.6}$ kNm/m

Check bearing pressure

Propping force;	$F_{prop\_base} = F_{total\_h} = \mathbf{11.1}$ kN/m
Distance to reaction;	$\bar{x} = M_{total} / F_{total\_v} = \mathbf{204}$ mm
Eccentricity of reaction;	$e = \bar{x} - l_{base} / 2 = \mathbf{-371}$ mm
Loaded length of base;	$l_{load} = 3 \times \bar{x} = \mathbf{611}$ mm
Bearing pressure at toe;	$q_{toe} = 2 \times F_{total\_v} / l_{load} = \mathbf{25.2}$ kN/m^2
Bearing pressure at heel;	$q_{heel} = \mathbf{0}$ kN/m^2
Factor of safety;	$FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel}) = \mathbf{3.971}$

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

Retaining wall design

In accordance with EN1992-1-1:2004 incorporating Corrigendum dated January 2008 and the UK National Annex incorporating National Amendment No.1 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;	$f_{ck} = \mathbf{30}$ N/mm^2
Characteristic compressive cube strength;	$f_{ck,cube} = \mathbf{37}$ N/mm^2
Mean value of compressive cylinder strength;	$f_{cm} = f_{ck} + 8$ N/mm^2 $= \mathbf{38}$ N/mm^2
Mean value of axial tensile strength;	$f_{ctm} = 0.3$ N/mm$^2 \times (f_{ck} / 1$ N/mm$^2)^{2/3} = \mathbf{2.9}$ N/mm^2
5% fractile of axial tensile strength;	$f_{ctk,0.05} = 0.7 \times f_{ctm} = \mathbf{2.0}$ N/mm^2
Secant modulus of elasticity of concrete;	$E_{cm} = 22$ kN/mm$^2 \times (f_{cm} / 10$ N/mm$^2)^{0.3} = \mathbf{32837}$ N/mm^2
Partial factor for concrete - Table 2.1N;	$\gamma_C = \mathbf{1.50}$
Compressive strength coefficient - cl.3.1.6(1);	$\alpha_{cc} = \mathbf{0.85}$
Design compressive concrete strength - exp.3.15;	$f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C = \mathbf{17.0}$ N/mm^2
Maximum aggregate size;	$h_{agg} = \mathbf{20}$ mm

Reinforcement details

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

Cover to reinforcement

Top face of base;	$c_{bt} = \mathbf{50}$ mm
Bottom face of base;	$c_{bb} = \mathbf{75}$ mm

Masonry details - Section 3.1

Masonry type; Autoclaved aerated concrete - Group 1

Normalised mean compressive strength; f_b = **10.4** N/mm^2

Characteristic flexural strength - cl.6.3.4(1); f_{xk} = **0** N/mm^2

Initial shear strength - Table NA.5; f_{vko} = **0.2** N/mm^2

Mortar details - Section 3.2

Mortar type; General purpose - M12, prescribed mix

Compressive strength of mortar; f_m = **12** N/mm^2

Ultimate limit states - Table NA.1

Class of execution control; 1

Category of manufacture control; 2

Partial factor for direct or flexural compression; γ_{Mc} = **2.3**

Partial factor for flexural tension; γ_{Mt} = **2.3**

Partial factor for shear; γ_{Mv} = **2.0**

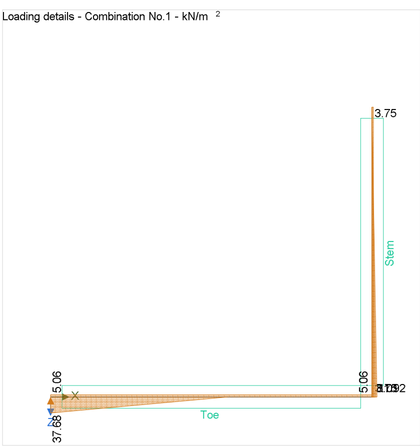
Characteristic strengths of concrete infill - Table 3.2

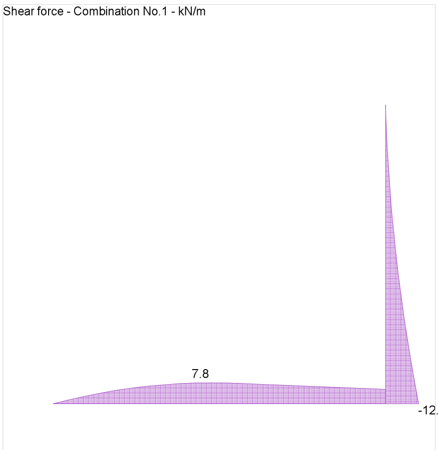
Concrete infill strength class; C30/37

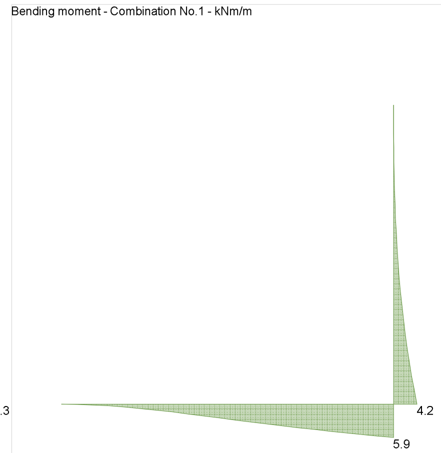
Characteristic compressive strength; $f_{ck,infill}$ = **25** N/mm^2

Characteristic shear strength; $f_{cvk,infill}$ = **0.45** N/mm^2

Design shear strength; $f_{cvd,infill} = f_{cvk,infill} / \gamma_{Mv}$ = **0.225** N/mm^2







Check stem design at base of stem

Depth of section; t = **150** mm

Hollow wall details

Face shell thickness; t_{shell} = **44** mm

Web shell thickness; t_{web} = **44** mm

Masonry characteristics

Compressive strength constants - Table NA.4; K = **0.55**

Characteristic compressive strength - cl.3.6.1.2(1); $f_k = K \times f_b^{0.7} \times \min(f_m, f_b, 12\text{ N/mm}^2)^{0.3}$ = **5.72** N/mm^2

Design compressive strength; $f_d = \min(f_k, f_{ck,infill}) / \gamma_{Mc}$ = **2.487** N/mm^2

Design flexural strength; $f_{xd} = f_{xk} / \gamma_{Mt}$ = **0** N/mm^2

Height of masonry; $h_{wt} = h_{stem}$ = **900** mm

Compressive axial force combination 0; $F_x = \gamma_{Gf} \times \gamma_{stem} \times h_{wt} \times t$ = **3.4** kN/m

Moment combination 0; $M_x = \gamma_{Gf} \times \gamma_{stem} \times h_{wt} \times t^2 / 2$ = **0.3** kNm/m

Eccentricity of axial load; $e = \max(\text{abs}(t / 2 - M_x / F_x), 0.05 \times t)$ = **7.5** mm

Capacity reduction factor - exp.6.4; $\Phi = 1 - 2 \times e / t =$ **0.9**

Design vertical resistance - exp.6.2; $N_{Rd} = \Phi \times t \times f_d =$ **335.7** kN/m

Design vertical compressive stress; $\sigma_d = \min(F_x / t, 0.15 \times N_{Rd} / t) =$ **0.023** N/mm^2

Apparent design flexural strength - exp.6.16; $f_{xd,app} = f_{xd} + \sigma_d =$ **0.023** N/mm^2

Characteristic shear strength - exp.3.5; $f_{vk} = \min(f_{vko} + 0.4 \times \sigma_d, 0.065 \times f_b) =$ **0.209** N/mm^2

Design shear strength; $f_{vd} = f_{vk} / \gamma_{Mv} =$ **0.105** N/mm^2

Reinforced masonry members subjected to bending, bending and axial loading, or axial loading - Section 6.6

Design bending moment combination 1; $M =$ **4.2** kNm/m

Tension reinforcement provided; 12 dia.bars @ 220 c/c

Area of tension reinforcement provided; $A_{sr.prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr}) =$ **514** mm^2/m

Depth to tension reinforcement; $d =$ **80** mm

Minimum area of reinforcement - cl.8.2.3(1); $A_{sr.min} = 0.0005 \times d =$ **40** mm^2/m

Lever arm - exp.6.23; $z = d \times \min(1 - 0.5 \times A_{sr.prov} \times f_{yd} / (d \times f_d), 0.95) =$ **35** mm

Moment of resistance - exp.6.22 and exp.6.24a; $M_{Rd} = \min(A_{sr.prov} \times f_{yd} \times z, 0.4 \times f_d \times d^2) =$ **6.4** kNm/m

$M / M_{Rd} =$ **0.661**

PASS - Moment of resistance exceeds applied design moment

Reinforced masonry members subjected to shear loading - Section 6.7

Design shear force $V =$ **12.344** kN/m

Design shear resistance - exp.6.40; $V_{Rd} = \min(f_{vd}, f_{cvd,infill}) \times d =$ **8.36** kN/m

$V / V_{Rd} =$ **1.477**

FAIL - Design shear resistance is less than applied design shear force

Horizontal reinforcement parallel to face of stem

Minimum area of reinforcement - cl.8.2.3(4); $A_{sx.req} = 0.0005 \times d =$ **40** mm^2/m

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

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

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

Check base design at toe

Depth of section; $h =$ **150** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1; $M =$ **5.9** kNm/m

Depth to tension reinforcement; $d = h - c_{bb} - \phi_{bb} / 2 =$ **69** mm

$K = M / (d^2 \times f_{ck}) =$ **0.042**

$K' =$ **0.207**

K' > K - No compression reinforcement is required

Lever arm; $z = \min(0.5 + 0.5 \times (1 - 3.53 \times K)^{0.5}, 0.95) \times d =$ **66** mm

Depth of neutral axis; $x = 2.5 \times (d - z) =$ **9** mm

Area of tension reinforcement required; $A_{bb.req} = M / (f_{yd} \times z) =$ **208** mm^2/m

Tension reinforcement provided; 12 dia.bars @ 200 c/c

Area of tension reinforcement provided; $A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb}) =$ **565** mm^2/m

Minimum area of reinforcement - exp.9.1N; $A_{bb.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d =$ **104** mm^2/m

Maximum area of reinforcement - cl.9.2.1.1(3); $A_{bb.max} = 0.04 \times h =$ **6000** mm^2/m

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

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

Crack control - Section 7.3

Limiting crack width; $w_{max} =$ **0.3** mm

Variable load factor - EN1990 – Table A1.1; $\psi_2 =$ **0.6**

Serviceability bending moment; M_{sls} = **4.2** kNm/m

Tensile stress in reinforcement; $\sigma_s = M_{sls} / (A_{bb.prov} \times z)$ = **114.6** N/mm^2

Load duration; Long term

Load duration factor; k_t = **0.4**

Effective area of concrete in tension; $A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$ = **47125** mm^2/m

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

Reinforcement ratio; $\rho_{p.eff} = A_{bb.prov} / A_{c.eff}$ = **0.012**

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

Bond property coefficient; k_1 = **0.8**

Strain distribution coefficient; k_2 = **0.5**

k_3 = **3.4**

k_4 = **0.425**

Maximum crack spacing - exp.7.11; $s_{r.max} = k_3 \times c_{bb} + k_1 \times k_2 \times k_4 \times \phi_{bb} / \rho_{p.eff}$ = **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_k = **0.146** mm

w_k / w_{max} = **0.487**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force; V = **7.8** kN/m

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

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

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

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

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

$V_{Rd.c}$ = **48.2** kN/m

$V / V_{Rd.c}$ = **0.162**

PASS - Design shear resistance exceeds design shear force

Secondary transverse reinforcement to base - Section 9.3

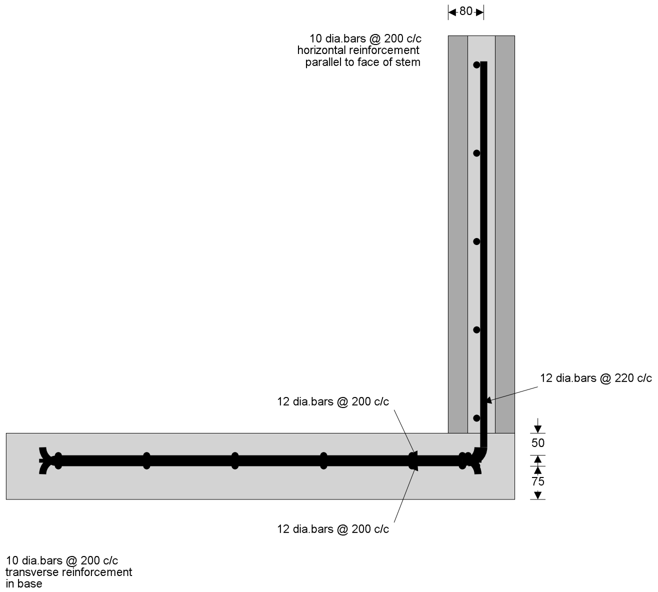
Minimum area of reinforcement – cl.9.3.1.1(2); $A_{bx.req} = 0.2 \times A_{bb.prov}$ = **113** mm^2/m

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

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

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

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



Reinforcement details

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_{stem} = **1650** mm
Stem thickness;	t_{stem} = **210** mm
Angle to rear face of stem;	α = **90** deg
Stem density;	γ_{stem} = **25** kN/m^3
Toe length;	l_{toe} = **1500** mm
Base thickness;	t_{base} = **150** mm
Base density;	γ_{base} = **25** kN/m^3
Height of retained soil;	h_{ret} = **1650** mm
Angle of soil surface;	β = **0** deg
Depth of cover;	d_{cover} = **0** mm
Height of water;	h_{water} = **1650** mm
Water density;	γ_w = **9.8** kN/m^3

Retained soil properties

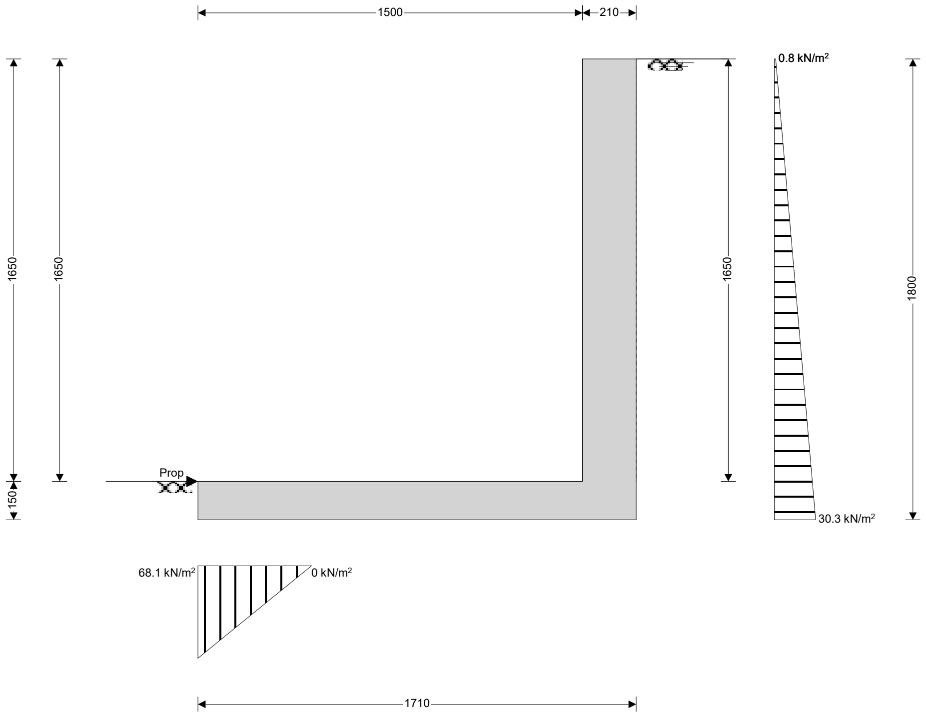
Soil type;	Medium dense well graded sand
Moist density;	γ_{mr} = **21** kN/m^3
Saturated density;	γ_{sr} = **23** kN/m^3
Characteristic effective shear resistance angle;	$\phi'_{r.k}$ = **30** deg
Characteristic wall friction angle;	$\delta_{r.k}$ = **0** deg

Base soil properties

Soil type;	Medium dense well graded sand
Soil density;	γ_b = **18** kN/m^3
Characteristic effective shear resistance angle;	$\phi'_{b.k}$ = **30** deg
Characteristic wall friction angle;	$\delta_{b.k}$ = **15** deg
Characteristic base friction angle;	$\delta_{bb.k}$ = **30** deg
Presumed bearing capacity;	$P_{bearing}$ = **100** kN/m^2

Loading details

Variable surcharge load;	$Surcharge_Q$ = **1.5** kN/m^2



General arrangement

Calculate retaining wall geometry

Base length;	$l_{base} = l_{toe} + t_{stem} = \mathbf{1710}$ mm
Saturated soil height;	$h_{sat} = h_{water} + d_{cover} = \mathbf{1650}$ mm
Moist soil height;	$h_{moist} = h_{ret} - h_{water} = \mathbf{0}$ mm
Length of surcharge load;	$l_{sur} = l_{heel} = \mathbf{0}$ mm
- Distance to vertical component;	$x_{sur\_v} = l_{base} - l_{heel} / 2 = \mathbf{1710}$ mm
Effective height of wall;	$h_{eff} = h_{base} + d_{cover} + h_{ret} = \mathbf{1800}$ mm
- Distance to horizontal component;	$x_{sur\_h} = h_{eff} / 2 = \mathbf{900}$ mm
Area of wall stem;	$A_{stem} = h_{stem} \times t_{stem} = \mathbf{0.347}$ m^2
- Distance to vertical component;	$x_{stem} = l_{toe} + t_{stem} / 2 = \mathbf{1605}$ mm
Area of wall base;	$A_{base} = l_{base} \times t_{base} = \mathbf{0.257}$ m^2
- Distance to vertical component;	$x_{base} = l_{base} / 2 = \mathbf{855}$ mm

Using Rankine theory

At rest pressure coefficient;	$K_0 = 1 - \sin(\phi'_{r.k}) = \mathbf{0.500}$
Passive pressure coefficient;	$K_P = (1 + \sin(\phi'_{b.k})) / (1 - \sin(\phi'_{b.k})) = \mathbf{3.000}$

Bearing pressure check

Vertical forces on wall

Wall stem;	$F_{stem} = A_{stem} \times \gamma_{stem} = \mathbf{8.7}$ kN/m
Wall base;	$F_{base} = A_{base} \times \gamma_{base} = \mathbf{6.4}$ kN/m
Total;	$F_{total\_v} = F_{stem} + F_{base} + F_{water\_v} = \mathbf{15.1}$ kN/m

Horizontal forces on wall

Surcharge load;	$F_{sur\_h} = K_0 \times Surcharge_Q \times h_{eff} = \mathbf{1.4}$ kN/m

Saturated retained soil; $F_{sat\_h} = K_0 \times (\gamma_{sr}' - \gamma_w') \times (h_{sat} + h_{base})^2 / 2 = \mathbf{10.7}$ kN/m

Water; $F_{water\_h} = \gamma_w' \times (h_{water} + d_{cover} + h_{base})^2 / 2 = \mathbf{15.9}$ kN/m

Moist retained soil; $F_{moist\_h} = K_0 \times \gamma_{mr}' \times ((h_{eff} - h_{sat} - h_{base})^2 / 2 + (h_{eff} - h_{sat} - h_{base}) \times (h_{sat} + h_{base})) = \mathbf{0}$ kN/m

Base soil; $F_{pass\_h} = -K_P \times \gamma_b' \times (d_{cover} + h_{base})^2 / 2 = \mathbf{-0.6}$ kN/m

Total; $F_{total\_h} = F_{sat\_h} + F_{moist\_h} + F_{pass\_h} + F_{water\_h} + F_{sur\_h} = \mathbf{27.3}$ kN/m

Moments on wall

Wall stem; $M_{stem} = F_{stem} \times x_{stem} = \mathbf{13.9}$ kNm/m

Wall base; $M_{base} = F_{base} \times x_{base} = \mathbf{5.5}$ kNm/m

Surcharge load; $M_{sur} = -F_{sur\_h} \times x_{sur\_h} = \mathbf{-1.2}$ kNm/m

Saturated retained soil; $M_{sat} = -F_{sat\_h} \times x_{sat\_h} = \mathbf{-6.4}$ kNm/m

Water; $M_{water} = -F_{water\_h} \times x_{water\_h} = \mathbf{-9.5}$ kNm/m

Moist retained soil; $M_{moist} = -F_{moist\_h} \times x_{moist\_h} = \mathbf{0}$ kNm/m

Total; $M_{total} = M_{stem} + M_{base} + M_{sat} + M_{moist} + M_{water} + M_{sur} = \mathbf{2.2}$ kNm/m

Check bearing pressure

Propping force; $F_{prop\_base} = F_{total\_h} = \mathbf{27.3}$ kN/m

Distance to reaction; $\bar{x} = M_{total} / F_{total\_v} = \mathbf{148}$ mm

Eccentricity of reaction; $e = \bar{x} - l_{base} / 2 = \mathbf{-707}$ mm

Loaded length of base; $l_{load} = 3 \times \bar{x} = \mathbf{443}$ mm

Bearing pressure at toe; $q_{toe} = 2 \times F_{total\_v} / l_{load} = \mathbf{68.1}$ kN/m^2

Bearing pressure at heel; $q_{heel} = \mathbf{0}$ kN/m^2

Factor of safety; $FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel}) = \mathbf{1.469}$

PASS - Allowable bearing pressure exceeds maximum applied bearing 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; $f_{ck} = \mathbf{30}$ N/mm^2

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

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

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

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

Secant modulus of elasticity of concrete; $E_{cm} = 22$ kN/mm$^2 \times (f_{cm} / 10$ N/mm$^2)^{0.3} = \mathbf{32837}$ N/mm^2

Partial factor for concrete - Table 2.1N; $\gamma_C = \mathbf{1.50}$

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

Design compressive concrete strength - exp.3.15; $f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C = \mathbf{17.0}$ N/mm^2

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

Reinforcement details

Characteristic yield strength of reinforcement; $f_{yk} = \mathbf{500}$ N/mm^2

Modulus of elasticity of reinforcement; $E_s = \mathbf{200000}$ N/mm^2

Partial factor for reinforcing steel - Table 2.1N; $\gamma_S = \mathbf{1.15}$

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

Cover to reinforcement

Top face of base; c_{bt} = **35** mm

Bottom face of base; c_{bb} = **35** mm

Masonry details - Section 3.1

Masonry type; Autoclaved aerated concrete - Group 1

Normalised mean compressive strength; f_b = **10.4** N/mm^2

Characteristic flexural strength - cl.6.3.4(1); f_{xk} = **0** N/mm^2

Initial shear strength - Table NA.5; f_{vko} = **0.2** N/mm^2

Mortar details - Section 3.2

Mortar type; General purpose - M12, prescribed mix

Compressive strength of mortar; f_m = **12** N/mm^2

Ultimate limit states - Table NA.1

Class of execution control; 1

Category of manufacture control; 2

Partial factor for direct or flexural compression; γ_{Mc} = **2.3**

Partial factor for flexural tension; γ_{Mt} = **2.3**

Partial factor for shear; γ_{Mv} = **2.0**

Characteristic strengths of concrete infill - Table 3.2

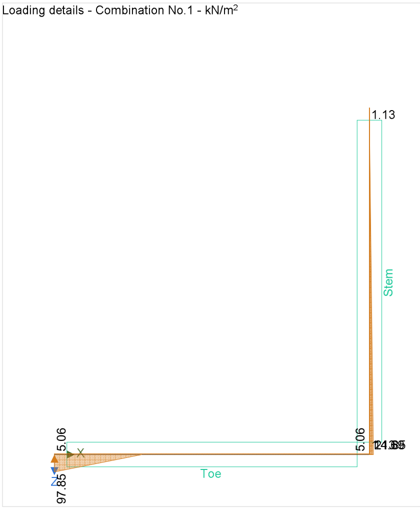
Concrete infill strength class; C30/37

Characteristic compressive strength; $f_{ck,infill}$ = **25** N/mm^2

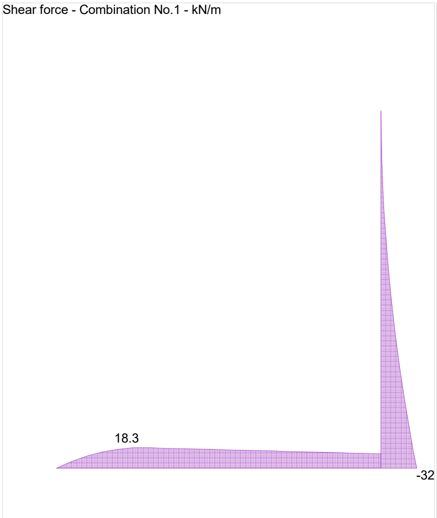
Characteristic shear strength; $f_{cvk,infill}$ = **0.45** N/mm^2

Design shear strength; $f_{cvd,infill} = f_{cvk,infill} / \gamma_{Mv}$ = **0.225** N/mm^2

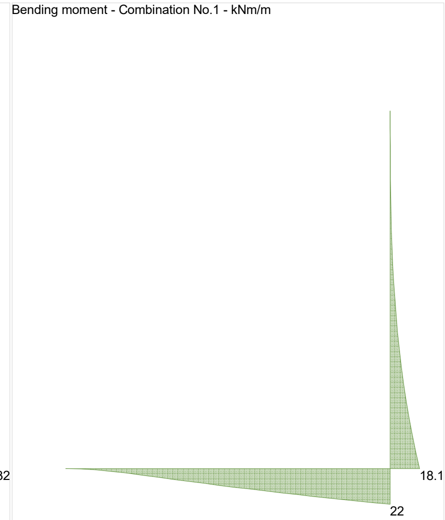
Loading details - Combination No.1 - kN/m^2



Shear force - Combination No.1 - kN/m



Bending moment - Combination No.1 - kNm/m



Check stem design at base of stem

Depth of section; t = **210** mm

Hollow wall details

Face shell thickness; t_{shell} = **44** mm

Web shell thickness; t_{web} = **44** mm

Masonry characteristics

Compressive strength constants - Table NA.4; K = **0.55**

Characteristic compressive strength - cl.3.6.1.2(1); $f_k = K \times f_b^{0.7} \times \min(f_m, f_b, 12\text{ N/mm}^2)^{0.3}$ = **5.72** N/mm^2

Design compressive strength; $f_d = \min(f_k, f_{ck,infill}) / \gamma_{Mc}$ = **2.487** N/mm^2

Design flexural strength; $f_{xd} = f_{xk} / \gamma_{Mt} = \mathbf{0}$ N/mm^2
Height of masonry; $h_{wt} = h_{stem} = \mathbf{1650}$ mm
Compressive axial force combination 0; $F_x = \gamma_{Gf} \times \gamma_{stem} \times h_{wt} \times t = \mathbf{8.7}$ kN/m
Moment combination 0; $M_x = \gamma_{Gf} \times \gamma_{stem} \times h_{wt} \times t^2 / 2 = \mathbf{0.9}$ kNm/m
Eccentricity of axial load; $e = \max(\text{abs}(t / 2 - M_x / F_x), 0.05 \times t) = \mathbf{10.5}$ mm
Capacity reduction factor - exp.6.4; $\Phi = 1 - 2 \times e / t = \mathbf{0.9}$
Design vertical resistance - exp.6.2; $N_{Rd} = \Phi \times t \times f_d = \mathbf{470}$ kN/m
Design vertical compressive stress; $\sigma_d = \min(F_x / t, 0.15 \times N_{Rd} / t) = \mathbf{0.041}$ N/mm^2
Apparent design flexural strength - exp.6.16; $f_{xd,app} = f_{xd} + \sigma_d = \mathbf{0.041}$ N/mm^2
Characteristic shear strength - exp.3.5; $f_{vk} = \min(f_{vko} + 0.4 \times \sigma_d, 0.065 \times f_b) = \mathbf{0.217}$ N/mm^2
Design shear strength; $f_{vd} = f_{vk} / \gamma_{Mv} = \mathbf{0.108}$ N/mm^2

Reinforced masonry members subjected to bending, bending and axial loading, or axial loading - Section 6.6

Design bending moment combination 1; $M = \mathbf{18.1}$ kNm/m
Tension reinforcement provided; 12 dia.bars @ 220 c/c
Area of tension reinforcement provided; $A_{sr.prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr}) = \mathbf{514}$ mm^2/m
Depth to tension reinforcement; $d = \mathbf{140}$ mm
Minimum area of reinforcement - cl.8.2.3(1); $A_{sr.min} = 0.0005 \times d = \mathbf{70}$ mm^2/m
Lever arm - exp.6.23; $z = d \times \min(1 - 0.5 \times A_{sr.prov} \times f_{yd} / (d \times f_d), 0.95) = \mathbf{95}$ mm
Moment of resistance - exp.6.22 and exp.6.24a; $M_{Rd} = \min(A_{sr.prov} \times f_{yd} \times z, 0.4 \times f_d \times d^2) = \mathbf{19.5}$ kNm/m
$M / M_{Rd} = \mathbf{0.929}$

PASS - Moment of resistance exceeds applied design moment

Reinforced masonry members subjected to shear loading - Section 6.7

Design shear force $V = \mathbf{32.004}$ kN/m
Design shear resistance - exp.6.40; $V_{Rd} = \min(f_{vd}, f_{cvd,infill}) \times d = \mathbf{15.155}$ kN/m
$V / V_{Rd} = \mathbf{2.112}$

FAIL - Design shear resistance is less than applied design shear force

Horizontal reinforcement parallel to face of stem

Minimum area of reinforcement - cl.8.2.3(4); $A_{sx.req} = 0.0005 \times d = \mathbf{70}$ mm^2/m
Transverse reinforcement provided; 10 dia.bars @ 200 c/c
Area of transverse reinforcement provided; $A_{sx.prov} = \pi \times \phi_{sx}^2 / (4 \times s_{sx}) = \mathbf{393}$ mm^2/m

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

Check base design at toe

Depth of section; $h = \mathbf{150}$ mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1; $M = \mathbf{22}$ kNm/m
Depth to tension reinforcement; $d = h - c_{bb} - \phi_{bb} / 2 = \mathbf{109}$ mm
$K = M / (d^2 \times f_{ck}) = \mathbf{0.062}$
$K' = \mathbf{0.207}$

K' > K - No compression reinforcement is required

Lever arm; $z = \min(0.5 + 0.5 \times (1 - 3.53 \times K)^{0.5}, 0.95) \times d = \mathbf{103}$ mm
Depth of neutral axis; $x = 2.5 \times (d - z) = \mathbf{16}$ mm
Area of tension reinforcement required; $A_{bb.req} = M / (f_{yd} \times z) = \mathbf{493}$ mm^2/m
Tension reinforcement provided; 12 dia.bars @ 200 c/c
Area of tension reinforcement provided; $A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb}) = \mathbf{565}$ mm^2/m
Minimum area of reinforcement - exp.9.1N; $A_{bb.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = \mathbf{164}$ mm^2/m
Maximum area of reinforcement - cl.9.2.1.1(3); $A_{bb.max} = 0.04 \times h = \mathbf{6000}$ mm^2/m

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

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

Crack control - Section 7.3

Limiting crack width; w_{max} = **0.3** mm

Variable load factor - EN1990 – Table A1.1; ψ_2 = **0.6**

Serviceability bending moment; M_{sls} = **16.2** kNm/m

Tensile stress in reinforcement; $\sigma_s = M_{sls} / (A_{bb.prov} \times z)$ = **278.4** 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** N/mm^2

Reinforcement ratio; $\rho_{p.eff} = A_{bb.prov} / A_{c.eff}$ = **0.013**

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

Bond property coefficient; k_1 = **0.8**

Strain distribution coefficient; k_2 = **0.5**

k_3 = **3.4**

k_4 = **0.425**

Maximum crack spacing - exp.7.11; $s_{r.max} = k_3 \times c_{bb} + k_1 \times k_2 \times k_4 \times \phi_{bb} / \rho_{p.eff}$ = **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_k = **0.252** mm

w_k / w_{max} = **0.84**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force; V = **18.3** kN/m

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

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

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

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

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

$V_{Rd.c}$ = **65.3** kN/m

$V / V_{Rd.c}$ = **0.280**

PASS - Design shear resistance exceeds design shear force

Secondary transverse reinforcement to base - Section 9.3

Minimum area of reinforcement – cl.9.3.1.1(2); $A_{bx.req} = 0.2 \times A_{bb.prov}$ = **113** mm^2/m

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

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

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

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

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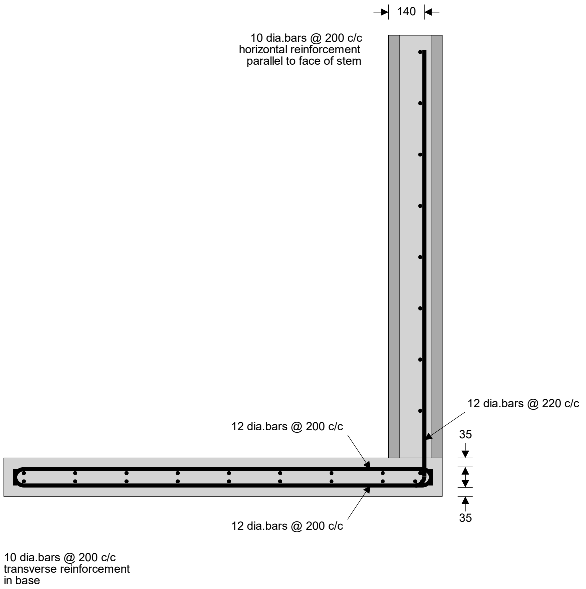
	Calc. by RC	Date 26/06/2017	Chk'd by MD	Date 26/06/2017	App'd by	Date



Reinforcement details