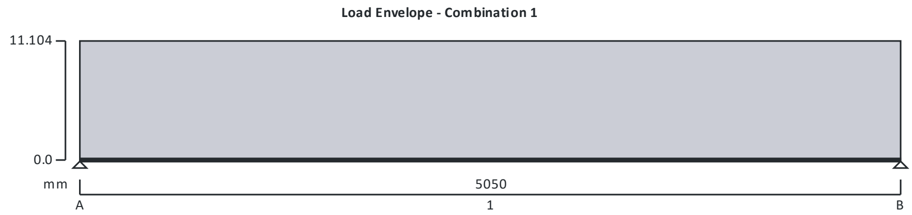
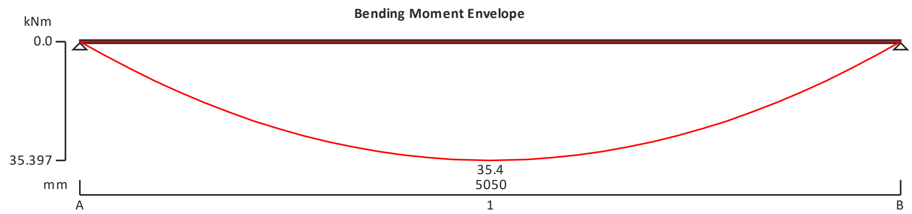
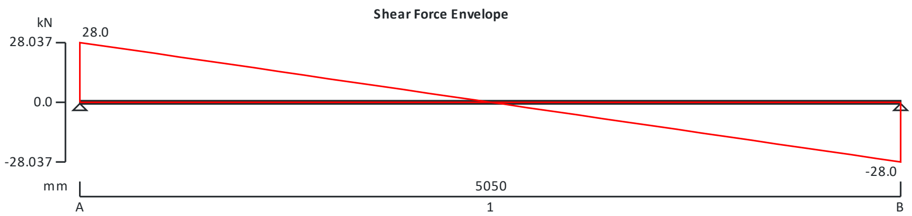
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

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

TEDDS calculation version 3.0.14







Support conditions

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

Applied loading

Beam loads	Permanent self weight of beam × 1
	FLOOR - Permanent full UDL 4 kN/m
	ROOF - Permanent full UDL 4 kN/m

Load combinations

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

Analysis results

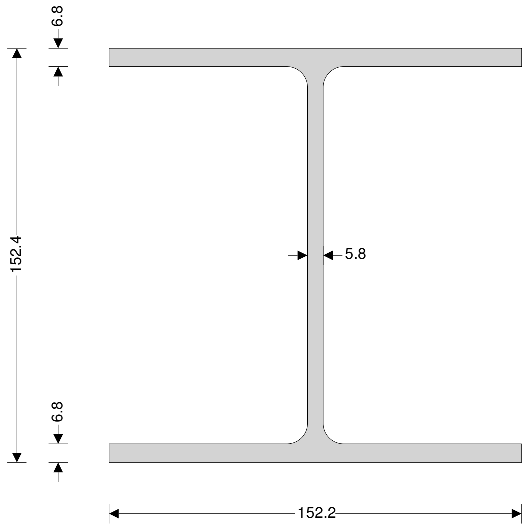
Maximum moment	M_{max} = **35.4** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **28** kN	V_{min} = **-28** kN
Deflection	δ_{max} = **0** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **28** kN	R_{A_min} = **28** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **20.8** kN	
Maximum reaction at support B	R_{B_max} = **28** kN	R_{B_min} = **28** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **20.8** kN	

Section details

Section type	**UC 152x152x23 (BS4-1)**
Steel grade	**S275**

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

Nominal thickness of element	$t = \max(t_f, t_w)$ = **6.8** mm
Nominal yield strength	f_y = **275** N/mm^2
Nominal ultimate tensile strength	f_u = **410** N/mm^2
Modulus of elasticity	E = **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 full lateral restraint

Effective length factors

Effective length factor in major axis	K_y = **1.000**
Effective length factor in minor axis	K_z = **1.000**
Effective length factor for torsion	$K_{LT.A}$ = **1.000**
	$K_{LT.B}$ = **1.000**

Classification of cross sections - Section 5.5

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

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

Width of section $c = d =$ **123.6** mm

$c / t_w = 23.1 \times \varepsilon <= 72 \times \varepsilon$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section $c = (b - t_w - 2 \times r) / 2 =$ **65.6** mm

$c / t_f = 10.4 \times \varepsilon <= 14 \times \varepsilon$ Class 3

Section is class 3

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f =$ **138.8** mm

Shear area factor $\eta =$ **1.000**

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

Shear buckling resistance can be ignored

Design shear force $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) =$ **28** kN

Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) =$ **997** mm^2

Design shear resistance - cl 6.2.6(2) $V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0} =$ **158.4** kN

PASS - Design shear resistance exceeds design shear force

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

Design bending moment $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) =$ **35.4** kNm

Design bending resistance moment - eq 6.14 $M_{c,Rd} = M_{el,Rd} = W_{el.y} \times f_y / \gamma_{M0} =$ **45.1** kNm

PASS - Design bending resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to variable loads

Limiting deflection $\delta_{lim} = L_{s1} / 360 =$ **14** mm

Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) =$ **0** mm

PASS - Maximum deflection does not exceed deflection limit