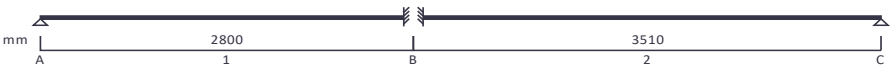
PlanningApplications.com
Summer House, Upper Court Road
Woldingham Surrey CR3 7BF
support@planningapplications.com
07922 148 701

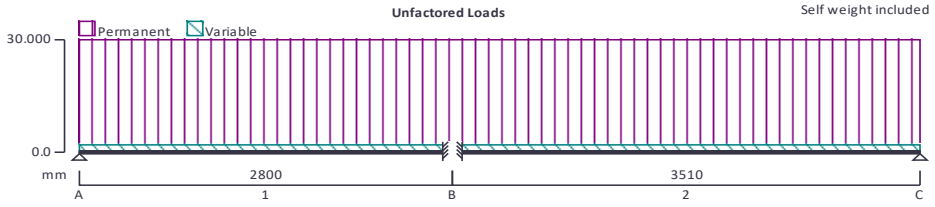
Project				Job no.	
BEAM A - 2 sections - mid support				2023-7459	
Calcs for				Start page no./Revision	
Mr Ashley Mujer 12 Avondale Gardens Hounslow TW4 5HX				1	
Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
SB	28/09/2023	DB	28/09/2023	SB	28/09/2023

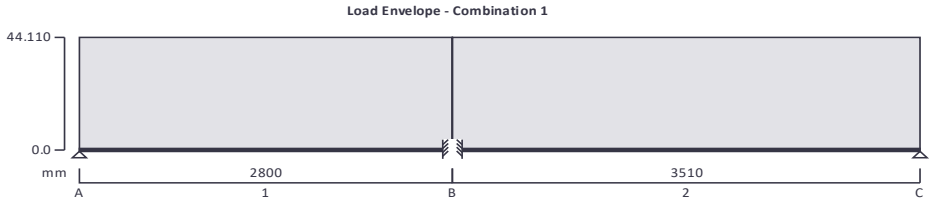
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

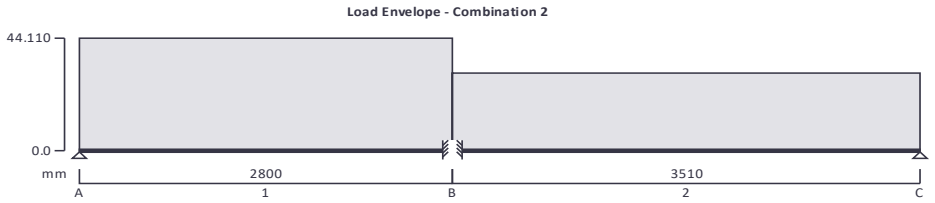
In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

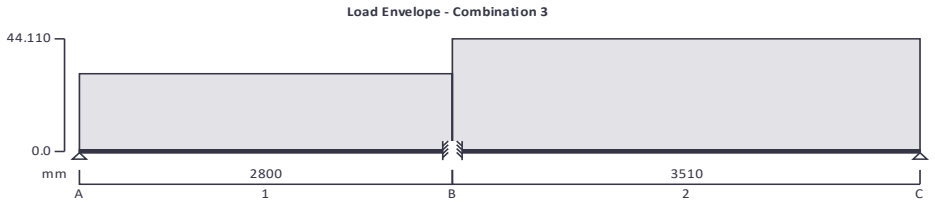
TEDDS calculation version 3.0.13











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Project				Job no.	
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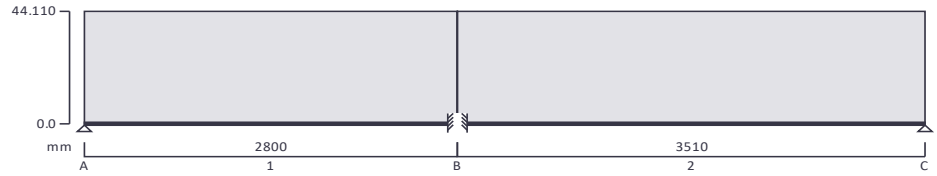
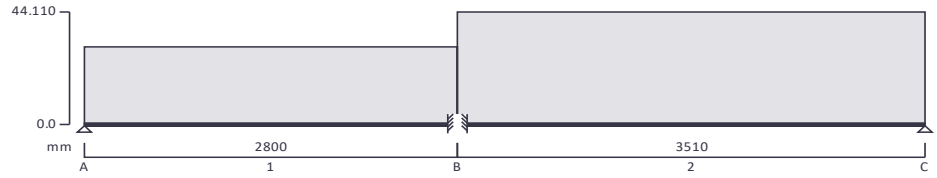
Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
SB	28/09/2023	DB	28/09/2023	SB	28/09/2023

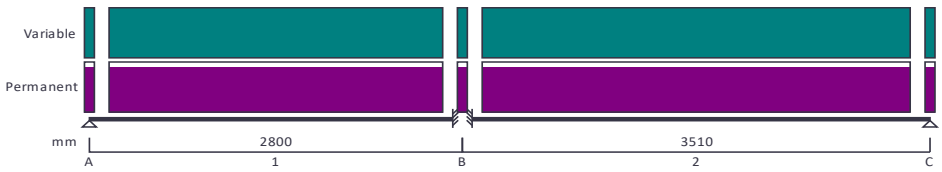
Load Envelope - Combination 4



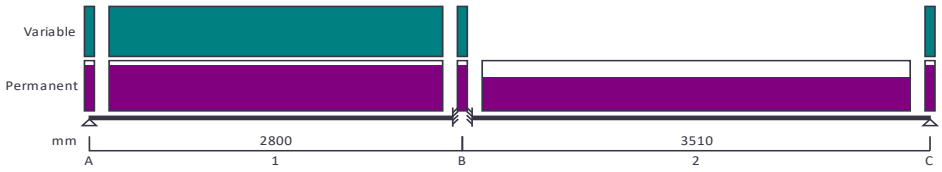
Load Envelope - Combination 5



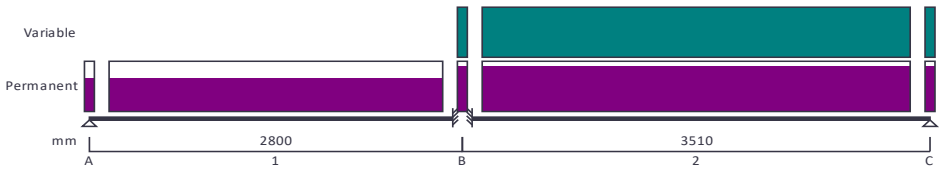
Load Combination 1 (shown in proportion)

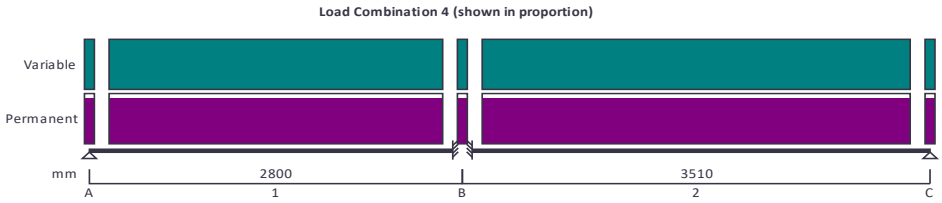


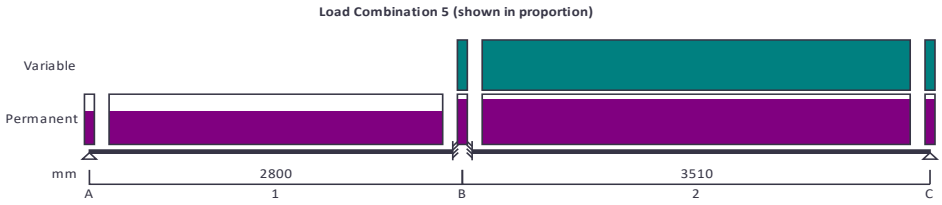
Load Combination 2 (shown in proportion)

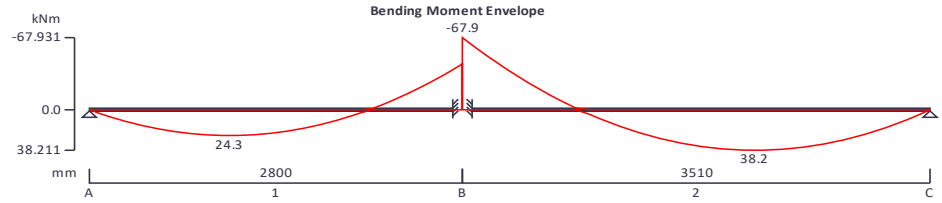


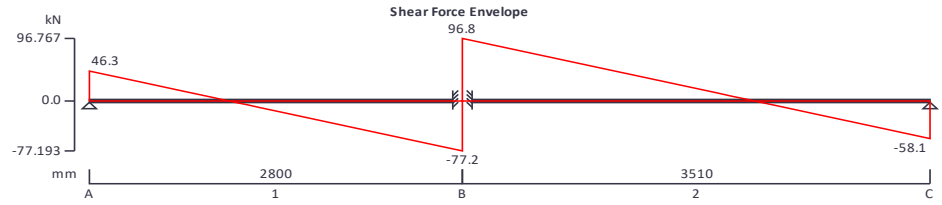
Load Combination 3 (shown in proportion)











Support conditions

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

Applied loading

Beam loads	Permanent self weight of beam × 1
	Permanent full UDL 30 kN/m
	Variable full UDL 2 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
		Permanent × 1.35
		Variable × 1.50
	Support C	Permanent × 1.35
		Variable × 1.50
Load combination 2	Support A	Permanent × 1.35
		Variable × 1.50
		Permanent × 1.35
		Variable × 1.50
	Support B	Permanent × 1.35
		Variable × 1.50
		Permanent × 1.00
	Support C	Permanent × 1.35
		Variable × 1.50
Load combination 3	Support A	Permanent × 1.00
		Permanent × 1.00
	Support B	Permanent × 1.35
		Variable × 1.50
		Permanent × 1.35
		Variable × 1.50
	Support C	Permanent × 1.35
		Variable × 1.50
Load combination 4	Support A	Permanent × 1.35
		Variable × 1.50
		Permanent × 1.35
		Variable × 1.50
	Support B	Permanent × 1.35
		Variable × 1.50
		Permanent × 1.35
		Variable × 1.50
	Support C	Permanent × 1.35
		Variable × 1.50
Load combination 5	Support A	Permanent × 1.00
		Permanent × 1.00
	Support B	Permanent × 1.35
		Variable × 1.50
		Permanent × 1.35
		Variable × 1.50

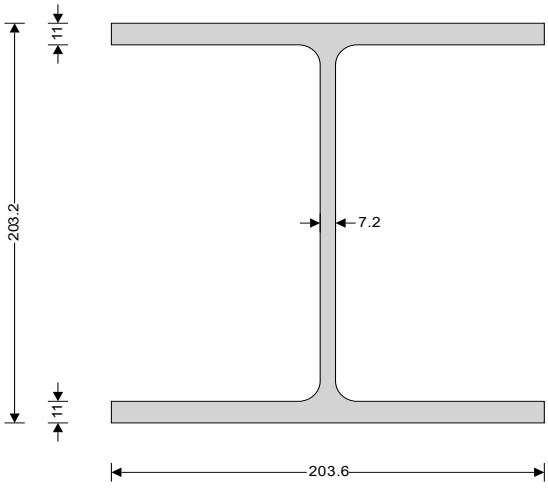
	Support C	Permanent × 1.35 Variable × 1.50
Analysis results		
Maximum moment	M_{max} = **38.2** kNm	M_{min} = **-67.9** kNm
Maximum moment span 1	M_{s1_max} = **24.3** kNm	M_{s1_min} = **-43.2** kNm
Maximum moment span 2	M_{s2_max} = **38.2** kNm	M_{s2_min} = **-67.9** kNm
Maximum shear	V_{max} = **96.8** kN	V_{min} = **-77.2** kN
Maximum shear span 1	V_{s1_max} = **46.3** kN	V_{s1_min} = **-77.2** kN
Maximum shear span 2	V_{s2_max} = **96.8** kN	V_{s2_min} = **-58.1** kN
Deflection	δ_{max} = **0.2** mm	δ_{min} = **0** mm
Deflection span 1	δ_{s1_max} = **0.1** mm	δ_{s1_min} = **0** mm
Deflection span 2	δ_{s2_max} = **0.2** mm	δ_{s2_min} = **0** mm
Maximum reaction at support A	R_{A_max} = **46.3** kN	R_{A_min} = **32** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **32** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **2.1** kN	
Maximum reaction at support B	R_{B_max} = **174** kN	R_{B_min} = **144** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **120.1** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **7.9** kN	
Maximum reaction at support C	R_{C_max} = **58.1** kN	R_{C_min} = **40.1** kN
Unfactored permanent load reaction at support C	$R_{C\_Permanent}$ = **40.1** kN	
Unfactored variable load reaction at support C	$R_{C\_Variable}$ = **2.6** 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

Span 2 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 $K_{LT.A}$ = **1.000**

$K_{LT.B}$ = **1.000**

$K_{LT.C}$ = **1.000**

Classification of cross sections - Section 5.5

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

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

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

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

Outstand flanges - Table 5.2 (sheet 2 of 3)

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

$c / t_f = 8.7 \times \varepsilon <= 9 \times \varepsilon$ 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 $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **96.8** 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 at span 2 major (y-y) axis - Section 6.2.5

Design bending moment $M_{Ed} = \max(\text{abs}(M_{s2\_max}), \text{abs}(M_{s2\_min}))$ = **67.9** 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 = **0.66**

$C_1 = 1 / k_c^2$ = **2.296**

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

Poissons ratio ν = **0.3**

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

Unrestrained length $L = 1.0 \times L_{s2}$ = **3510** mm

Elastic critical buckling moment $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]}$ = **933.3** kNm

Slenderness ratio for lateral torsional buckling $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **0.383**

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

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

PASS - Design bending resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to variable loads

Limiting deflection $\delta_{lim} = L_{s2} / 360 =$ **9.8** mm

Maximum deflection span 2 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) =$ **0.171** mm

PASS - Maximum deflection does not exceed deflection limit