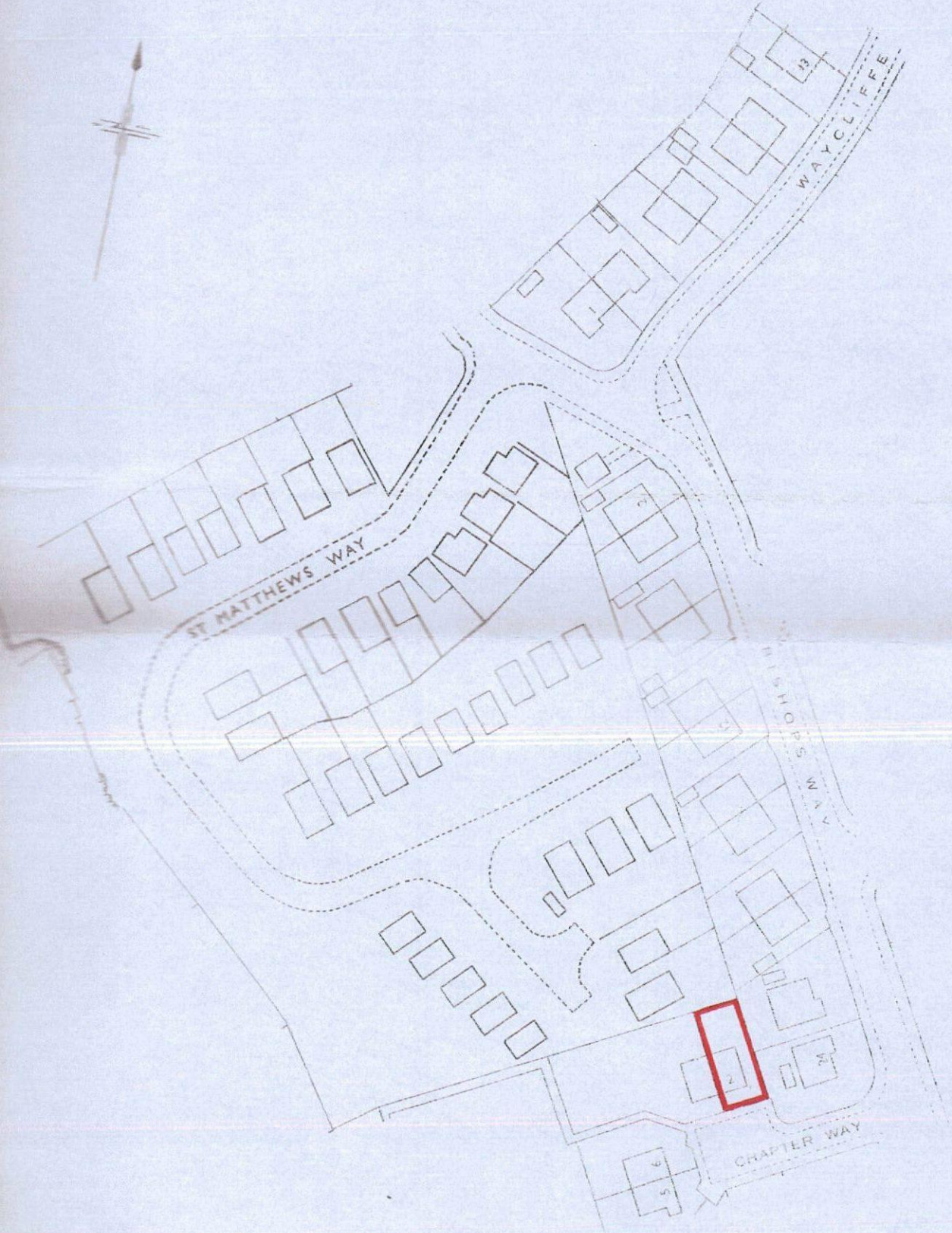
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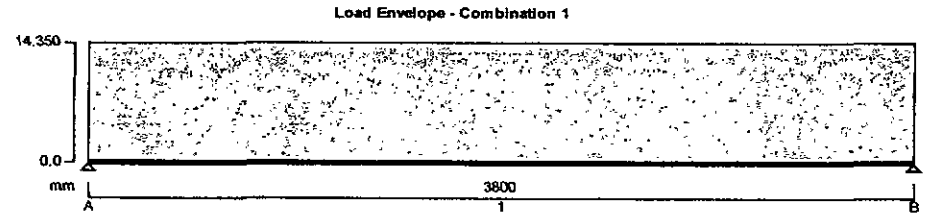
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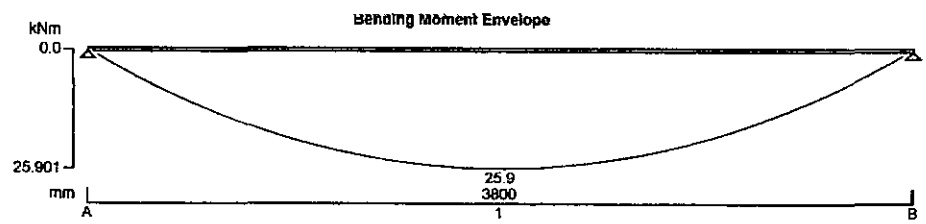
STEEL BEAM ANALYSIS & DESIGN (BS5950)

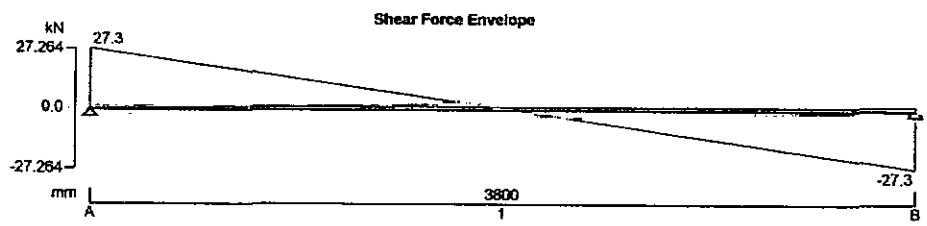
STEEL BEAM ANALYSIS & DESIGN (BS5950)

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.04







Support conditions

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

Applied loading

Beam loads	Dead self weight of beam x 1
	roof - Dead full UDL 6.5 kN/m
	roof - Dead full UDL 3 kN/m

Load combinations

Load combination 1	Support A	Dead x 1.40
		Imposed x 1.60
	Span 1	Dead x 1.40
		Imposed x 1.60

	Support B	Dead × 1.40 Imposed × 1.60

Analysis results

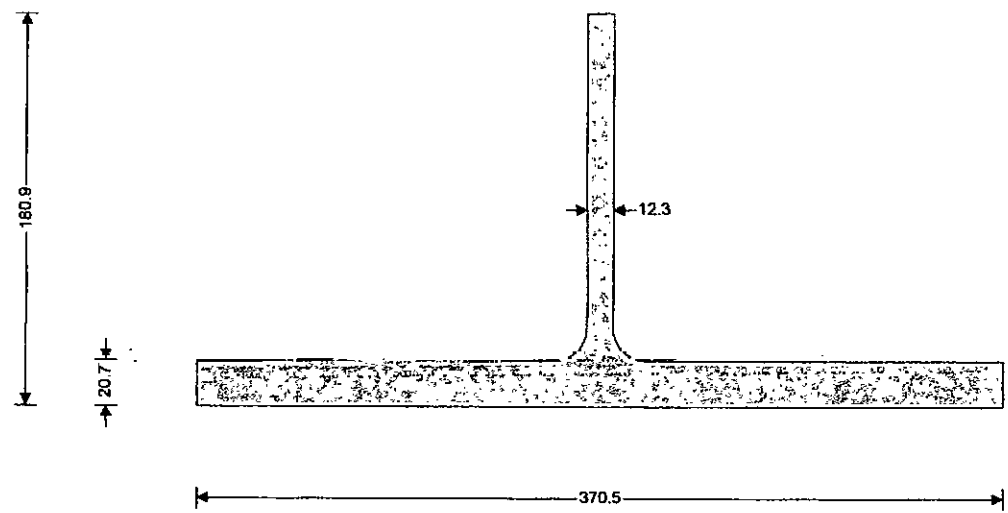
Maximum moment	M_{max} = **25.9** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **27.3** kN	V_{min} = **-27.3** kN
Deflection	δ_{max} = **0** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **27.3** kN	R_{A_min} = **27.3** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **19.5** kN	
Maximum reaction at support B	R_{B_max} = **27.3** kN	R_{B_min} = **27.3** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **19.5** kN	

Section details

Section type	**STC 368x178x77 (BS4-1)**
Steel grade	**S275**

From table 9: Design strength p_y

Thickness of element	max(T, t) = **20.7** mm
Design strength	p_y = **265** N/mm^2
Modulus of elasticity	E = **205000** N/mm^2



Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis	K_x = **1.00**
Effective length factor in minor axis	K_y = **1.00**
Effective length factor for lateral-torsional buckling	$K_{LT,A}$ = **1.00**

Classification of cross sections - Section 3.5

$\varepsilon = \sqrt{[275\ N/mm^2 / p_y]}$ = **1.02**

Outstand flanges - Table 11

Width of section	b = D = **180.9** mm	
	b / t = 14.4 × ε <= 18 × ε	Class 3 semi-compact

Section is class 3 semi-compact

Shear capacity - Section 4.2.3

Design shear force — $F_v = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = \mathbf{27.3}$ kN

$(D - T - r) / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

Shear area — $A_v = t \times D = \mathbf{2225}$ mm^2

Design shear resistance — $P_v = 0.6 \times p_y \times A_v = \mathbf{353.8}$ kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment — $M = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = \mathbf{25.9}$ kNm

Effective plastic modulus - Section 3.5.6

Effective plastic modulus - cl.3.5.6.1 — $S_{eff} = Z_{xx} = \mathbf{113843}$ mm^3

Moment capacity low shear - cl.4.2.5.2 — $M_c = p_y \times \min(Z_{xxflange}, Z_{xxtoe}) = \mathbf{30.2}$ kNm

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling — $L_E = 1.0 \times L_{s1} = \mathbf{3800}$ mm

Slenderness ratio — $\lambda = L_E / r_{yy} = \mathbf{40.029}$

Equivalent slenderness - Section 4.3.6.7

Buckling parameter — $u = \mathbf{0.000}$

Torsional index — $x = \mathbf{8.520}$

Moment of inertia compression flange minor axis — $I_{yc} = \mathbf{0}$ mm^4

Moment of inertia tension flange minor axis — $I_{yt} = T \times B^3 / 12 = \mathbf{87731133}$ mm^4

Flange ratio — $\eta = I_{yc} / (I_{yc} + I_{yt}) = \mathbf{0.000}$

Flange ratio factor — $k_\eta = \mathbf{1.000}$

Monosymmetry index — $\psi = k_\eta \times (2 \times \eta - 1) = \mathbf{-1.000}$

Slenderness factor — $v = 1 / [(4 \times \eta \times (1 - \eta) + 0.05 \times (\lambda / x)^2 + \psi^2)^{0.5} + \psi]^{0.5} = \mathbf{1.490}$

Ratio - cl.4.3.6.9 — $\beta_W = S_{eff} / S_{xx} = \mathbf{0.526}$

Equivalent slenderness - cl.4.3.6.7 — $\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_W]} = \mathbf{0.000}$

Limiting slenderness - Annex B.2.2 — $\lambda_{LO} = 0.4 \times (\pi^2 \times E / p_y)^{0.5} = \mathbf{34.951}$

$\lambda_{LT} < \lambda_{LO}$ - No allowance need be made for lateral-torsional buckling

Buckling resistance moment - Section 4.3.6.4

Bending strength — $p_b = p_y = \mathbf{265}$ N/mm^2

Buckling resistance moment — $M_b = p_b \times S_{eff} = \mathbf{30.2}$ kNm

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection — $\delta_{lim} = L_{s1} / 360 = \mathbf{10.556}$ mm

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

PASS - Maximum deflection does not exceed deflection limit

Max Shear 28Kn Therefore bearing area min 100 x 300.