Woodside Farm, Heath Road, LE67 1DG

Structural Calculations – Analysis of critical elements

Reference: 25.1620-AMP-CAL-001-P01

AMP STRUCTURES

This page is intentionally blank.

Woodside Farm, Heath Road, LE67 1DG

Structural Calculations – Analysis of critical elements

This report has been produced on behalf of Mark Wye in order to demonstrate that the existing barn structure at the above address is capable of conversion from agricultural use in to residential use without upgrade.

The calculations are prepared for the use of Mark Wye in connection with the above property, for the specific purpose of gaining planning permission.

The calculations are not intended for, and should not be relied upon by any third party, and no responsibility is undertaken to any third party.

This report has been prepared by

DAN SMITH *MEng (Hons)*
PROJECT ENGINEER

This report has been checked by

HENRY APPLEBY *MEng(Hons) CEng MICE*
PARTNER
Henry@AMPstructures.com
07868 258600

Revision	Date	Purpose	Prepared By	Checked By
P01	December 2025	Preliminary Issue	DS	HA

1. Scope

1.1. Existing Building/Site

The existing building is a single-storey agricultural barn. The building is steel portal frame construction with masonry infill walls and ground bearing slab. The portal frame elements are 203x133x25 UB sections, haunched at the eaves and ridge. The masonry infill panels are comprised of 200mm wide hollow concrete blocks. The ground bearing slab is understood to be 200mm thick, with 2 layers of reinforcing mesh. The roof structure is supported on the primary rafters via 180x63 timber purlins. All structural elements are in satisfactory condition we have found no evidence to suggest otherwise.

1.2. Proposed Works

The building is to be converted to a two-storey residential building. The structural implications of these works are likely to include, but is not limited to the below:

1. Construction of timber first floor and framing members
2. Removal of existing roof coverings and installation of new insulated panels, such as Kingspan quadcore

Based on the information received via email on 20th November 2025 and our discussions with client, we understand that a planning application has been submitted for conversion with a structural survey attached. However, the planning officer has stated "The adequacy of the existing structure will need to be confirmed after carrying out a detailed structural analysis of the existing building, by a suitably qualified engineer".

We have therefore produced a sketch scheme equivalent to RIBA stage 2 for planning purposes. The critical existing structural elements have been checked for the proposed loads. The concept structural scheme utilises the existing structure to support the proposed loads.

2. Conclusions

The existing frame was analysed using TEDDS and Masterseries designer software. Load data was sourced from client data, site measurements and wind/snow analysis. Physical site measurements (by others) have been used to ascertain the sizes of the existing structural members.

Each existing structural element checked is listed below:

Structural Element	Design check under Proposed Loads
Frame Columns	PASS
Frame Primary Rafters	PASS
Ground Slab	PASS
Masonry Walls	PASS
Trench Foundation	PASS
Purlins	PASS

It was found that the overall deflection of the portal frames and performance under the new loads was found to be within acceptable limits. The 203x133x25 UB primary rafters and columns are sufficient for the proposed loads.

The existing ground slab is capable of supporting point loads from steel/timber posts supporting the first-floor structure as well as line loads from load-bearing timber walls. This was confirmed by analysing a 150mm thick slab (to be conservative) with TR34 checks.

The existing 200mm wide hollow blockwork walls are able to support the proposed timber floor loads and lateral loads.

The existing trench foundation was taken to be 350mm wide, based on the thickness of the walls. This is very conservative and more likely to be 450mm wide, in line with industry standard bucket widths. It was found that 350mm width is sufficient to support the existing masonry panels, with the proposed external timber cladding, timber floor and roof loads.

The existing 180x68 purlins are able to support 40mm Kingspan Quadcore KS1000RW insulated roof panels (or similar).

It can therefore be concluded that the existing structure is capable of conversion from an agricultural barn to a residential property. All existing structural elements are capable of supporting the proposed loads specified in section 3 of this report. New structural elements are required only to support new openings and the new first floor structure, which can be designed to be supported off the existing structure.

3. Calculations

See over

Project				
Woodside Farm, Heath Road, LE67 1DG				
Title			Job No	
Loading and Analysis			25.1620	
Made	Date	Checked	Date	
DS	Dec-25	HA	Dec-25	

Pitched roof (trussed or cut)

Permanent Actions			Variable Actions		
40mm KS1000RW	0.09	kN/m²	Snow	0.47	kN/m²
Trusses/Rafters & Ceiling Joists	0.25	kN/m²			
12.5mm Plasterboard	0.11	kN/m²			
	0.45	**kN/m²**		**0.47**	**kN/m²**

Floor (timber)

Permanent Actions			Variable Actions		
Finishes	0.020	kN/m²	Residential	1.50	kN/m²
22mm T&G Boarding	0.18	kN/m²	Lightweight Partitions	0.50	kN/m²
Joists	0.15	kN/m²			
Plasterboard	0.11	kN/m²			
	0.460	**kN/m²**		**2.00**	**kN/m²**

Kingspan Roof +PVs

Permanent Actions			Variable Actions		
40mm KS1000RW	0.088	kN/m²	Snow	0.47	kN/m²
PVs	0.20	kN/m²			
	0.288	**kN/m²**		**0.47**	**kN/m²**

Kingspan Roof only

Permanent Actions			Variable Actions		
40mm KS1000RW	0.09	kN/m²	Snow	0.47	kN/m²
	0.09	**kN/m²**		**0.47**	**kN/m²**

External wall (200wd Block)

Permanent Actions			Variable Actions		
Hollow Blockwork (200mm)	3.80	kN/m²			
12.5mm Plasterboard & Skim	0.15	kN/m²			
Insulation	0.10	kN/m²			
Timber Cladding & Battens	0.10	kN/m²			
Ply/OSP Board	0.10	kN/m²			
Plasterboard	0.11	kN/m²			
	4.36	**kN/m²**		**0.00**	**kN/m²**

Internal wall (Timber Stud)

Permanent Actions			Variable Actions		
Timber Stud Wall (domestic)	0.50	kN/m²			
	0.50	**kN/m²**		**0.00**	**kN/m²**

AMP STRUCTURES

Existing Purlins (with PV's)

			Dead	Live
	4.6 m Span			
Kingspan Roof +PVs		1.40 m supported	0.40 kN/m	0.66 kN/m
			0.40 kN/m	**0.66 kN/m**

Existing Purlins (without PV's)

			Dead	Live
	4.6 m Span			
Kingspan Roof only		1.40 m supported	0.124 kN/m	0.66 kN/m
			0.124 kN/m	**0.66 kN/m**

PROPOSED TIMBER STUD WALLS (SLAB CHECK)

			Dead	Live
	4.6 m Span			
Floor (timber)		4.80 m supported	2.208 kN/m	9.60 kN/m
Internal wall (Timber Stud)		2.50 m supported	1.250 kN/m	0.00 kN/m
			3.458 kN/m	**9.60 kN/m**
		TOTAL	13.058 **kN/m**	

EXISTING FRAME CHECK

			Dead	Live
	12 m Span			
Pitched roof (trussed or cut)		4.50 m supported	2.017 kN/m	2.12 kN/m
			2.01731 kN/m	**2.1150 kN/m**

EXISTING WALL CHECK

			Dead	Live
	4.6 m Span			
Floor (timber)		3.00 m supported	1.380 kN/m	6.00 kN/m
External wall (200wd Block)		2.50 m supported	10.900 kN/m	0.00 kN/m
			12.280 kN/m	**6.00 kN/m**

AMP STRUCTURES	Project: Woodside Farm, Heath Road, LE67 1DG			
	Title: Loading and Analysis			Job No: 25.1620
	Made: DS	Date: Dec-25	Checked: HA	Date: Dec-25

SLAB POINT LOAD CHECK			**Dead**	**Live**
B1.1	*6 m Span*			
Internal wall (Timber Stud)		2.50 m supported	1.25 kN/m	0.00 kN/m
			1.25 kN/m	**0.00 kN/m**
		reaction	3.75 kN	
B1.2	*2.8 m Span*			
Floor (timber)		3.00 m supported	1.38 kN/m	6.00 kN/m
			1.38 kN/m	**6.00 kN/m**
		reaction	1.932 kN	8.4 kN
Weight of masonry				
External wall (200wd Block)		1.50 m3 supported	2.85 kN*	0.00 kN/m
		Total	8.53 kN	8.4 kN

*1900kg/m3 conservatively assumed

Existing Trench Foundation Loadings

Safe Bearing Pressure = **100.00** kN/m²

(conservative assumption based on archive borehole inforamtion and trial pits

			Dead	Live
Floor (timber)		3.00 m supported	1.38 kN/m	6.00 kN/m
Kingspan Roof +PVs		3.00 m supported	0.86 kN/m	1.41 kN/m
External wall (200wd Block)		4.40 m carried	19.2 kN/m	0.00 kN/m
Strip footing SW	0.45 m dp	0.35 m wd	3.9 kN/m	
			25.4 kN/m	**7.4 kN/m**

Total load under footing = 32.78 kN/m

Width = 0.35 m

SLS dead surcharge load (slab + BU) = 3.90 kN/m²* ***Based on 200 GB, 65 screed, 0.1 for insulation**

SLS live surcharge load = 2.00 kN/m²

Applied pressure = 99.55 kN/m²

All other footings OK by inspection

Basic Overall Wind Load

Structure height (h)	5.0	m	
Reference height (z)	3.0	m	Fig 6.1
Fundamental basic wind velocity (V_{bMAP}	22.0	m/s	NA.1
AOD	140		
C_{ALT}	1.14		NA.2b
C_{dir}, C_{season}, C_{prob}	1.00		
Basic wind velocity V_b = V_{bMAP} C_{ALT}	25.1	m/s	NA.1
Basic velocity pressure q_b = 0.613 V_b^2	385.6	N/m²	NA2.18 & Eq4.10
Terrain	Country		
Include sheltering effects	Yes		
Manual sheltered height (if known)	0		
h_{dis}	0	m	
z-h_{dis}	3	m	
City	Leicester		
Distance from shore	100		
$C_{e(z)}$	1.9		NA.7
$C_{e(T)}$	1	(conservative)	
Peak velocity pressue $q_{p(z)}$ =	qb Ce(z)		NA.3a
	732.6	N/m²	
depth of structure	12	m	
h/d	0.417		
C_{pNET}	0.911		NA.2.27(f)
Wind Load	0.67	kN/m²	
	(conservative)		

Project Name: Woodside Farm, Heath Road, LE67 1DG		**Job No:** 25.1620
Reference: Existing Slab Check - Point Load (below Chimney)		**Date:** 02/12/2025
Made by: DS	**Checked by:** HA	**Sheet No:** 1

CONCRETE INDUSTRIAL GROUND FLOOR SLAB DESIGN

In accordance with TR34, 4th Edition 2013

Tedds calculation version 2.0.04

Design summary

Load 1 -Single edge 1200 x 500 point load

Description	Unit	Provided	Required	Utilisation	Result
Slab capacity in flexure	kN	196.9	24.2	0.123	PASS
Shear at face	kN	931.5	24.2	0.026	PASS
Shear at 2d	kN	137.3	24.2	0.176	PASS

Slab details

Reinforcement type — Fabric

Concrete class — C25/30

Slab thickness — h = **150** mm

Fabric reinforcement type — A393

Characteristic strength of reinforcement — f_{yk} = **500** N/mm^2

Area of bottom steel provided — $A_{s,prov}$ = **393** mm^2/m

Diameter of reinforcement — ϕ_s = **10** mm

Nominal cover — c_{nom_b} = **50** mm

Effective depth of reinforcement — $d = h - c_{nom\_b} - \phi_s$ = **90** mm

Partial safety factors

Concrete (with or without fibre) — γ_c = **1.50**

Reinforcement (bar or fabric) — γ_s = **1.15**

Permanent — γ_G = **1.35**

Variable — γ_Q = **1.50**

Dynamic loads — γ_D = **1.60**

Subgrade reaction

Modulus of subgrade reaction — k = **0.015** N/mm^3

Concrete details - Table 6.1. Strength properties for concrete

Characteristic compressive cylinder strength — f_{ck} = **25** N/mm^2

Characteristic compressive cube strength — f_{cu} = **30** N/mm^2

Mean value of compressive cylinder strength — $f_{cm} = f_{ck} + 8\ N/mm^2$ = **33** N/mm^2

Mean value of axial tensile strength — $f_{ctm} = 0.3\ N/mm^2 \times (f_{ck}/\ 1\ N/mm^2)^{2/3}$ = **2.6** N/mm^2

Flexural tensile strength — $f_{ctd,fl} = f_{ctm} \times (1.6 - h\ /\ 1m)\ /\ \gamma_c$ = **2.5** N/mm^2

Design concrete compressive strength (cylinder) — $f_{cd} = f_{ck}\ /\ \gamma_c$ = **16.7** N/mm^2

Secant modulus of elasticity of concrete — $E_{cm} = 22\ kN/mm^2 \times [f_{cm}/\ 10\ N/mm^2]^{0.3}$ = **31** kN/mm^2

Poisons ratio — ν = **0.2**

Radius of relative stiffness (Eqn. 20) — $l = [E_{cm} \times h^3\ /\ (12 \times (1 - \nu^2) \times k)]^{0.25}$ = **885** mm

Characteristic of system (Eqn. 33) — $\lambda = (3 \times k\ /\ (E_{cm} \times h^3))^{0.25}$ = **0.807** m^{-1}

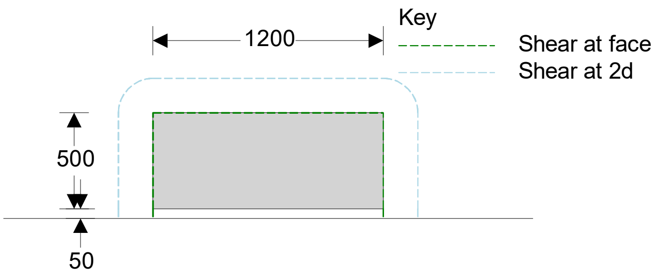
Moment capacity

Negative moment capacity (Eqn. 2) — $M_n = M_{un} = f_{ctd,fl} \times (h^2\ /\ 6)$ = **9.3** kNm/m

Positive moment capacity (Eqn. 3)	$M_p = M_{pfab} = 0.95 \times A_{s,prov} \times f_{yk} \times d / \gamma_s =$ **14.6** kNm/m
Ratio of cracked to uncracked mnt of resist (cl.7.4)	$M_p / M_n =$ **1.571**

PASS - Ratio of cracked to uncracked moment of resistance is greater or equal to 0.5

Load 1 - Single edge 1200 x 500 point load



Single edge load

Loading length	$l_l =$ **1200**mm
Loading width	$l_w =$ **500**mm
Edge distance y	$e_y =$ **50**mm
Permanent load	$G_k =$ **8.6** kN
Variable load	$Q_k =$ **8.4** kN
Dynamic load	$D_k =$ **0.0** kN

Contact radius ratio

Equivalent contact radius ratio	$a = [(l_l \times l_w) / \pi]^{0.5} =$ **437.0** mm
Radius ratio	$a / l =$ **0.494**

Ultimate capacity under single edge concentrated loads

For a/l equal to 0 (Eqn. 23)	$P_{u\_0} = [\pi \times (M_p + M_n) / 2] + 2 \times M_n =$ **56.1** kN
For a/l equal to 0.2 (Eqn. 24)	$P_{u\_0.2} = [\pi \times (M_p + M_n) + 4 \times M_n] / [1 - (2 \times a / (3 \times l))] =$ **167.4** kN
Thus for a / l equal to 0.494	$P_u = \min(P_{u\_0.2}, P_{u\_0} + (P_{u\_0.2} - P_{u\_0}) \times (a / (l \times 0.2))) =$ **167.4** kN
Percentage of aggregate transfer	$P_{agg} =$ **15** %
Total effective edge capacity (cl.7.9.1)	$P_{u,total} = \min(P_u / (1 - P_{agg}), P_u / (1 - 0.5), 4 \times \pi \times (M_p + M_n) / [1 - (a / (3 \times l))]) =$ **196.9** kN

Check ultimate load capacity of slab

Number of loads	$N =$ **1**
Loading applied to slab	$F_{uls} = N \times ((G_k \times \gamma_G) + (Q_k \times \gamma_Q) + (D_k \times \gamma_D)) =$ **24.2** kN
Utilisation	$F_{uls} / P_{u\_total} =$ **0.123**

PASS - Total slab capacity exceeds applied load

Punching shear at the face of the loaded area

Shear factor	$k_2 = 0.6 \times (1 - f_{ck} / 250N/mm^2) =$ **0.54**
Length of perimeter at face of loaded area	$u_0 = 2 \times (l_w + e_y) + l_l =$ **2300** mm
Shear stress at face of contact area	$v_{max} = 0.5 \times k_2 \times f_{cd} =$ **4.500** N/mm^2
Maximum load capacity in punching	$P_{p,max} = v_{max} \times u_0 \times d =$ **931.5** kN
Utilisation	$F_{uls} / P_{p,max} =$ **0.026**

PASS - Total slab capacity in punching at face of loaded area exceeds applied load

Punching shear at the critical perimeter

Shear factor	$k_s = \min(1 + (200mm / d)^{0.5}, 2) =$ **2.00**
Minimum shear stress at 2d from face of load	$v_{Rd,c,min} = 0.035 \times k_s^{3/2} \times (f_{ck} / 1N/mm^2)^{0.5} \times 1N/mm^2 =$ **0.495** N/mm^2
Ratio of reinforcement by area in x-direction	$\rho_x = A_{s,prov} / d =$ **0.00437**
Ratio of reinforcement by area in y-direction	$\rho_y = A_{s,prov} / d =$ **0.00437**
Reinforcement ratio	$\rho_1 = (\rho_x \times \rho_y)^{0.5} =$ **0.00437**
Maximum shear stress at 2d from face of load	$v_{Rd,c} = \max(0.18 \times k_s / \gamma_c \times (100 \times \rho_1 \times f_{ck} / 1N/mm^2)^{1/3} \times 1N/mm^2, v_{Rd,c,min}) =$ **0.532** N/mm^2
Length of perimeter at 2d from face of load	$u_1 = l_l + 2 \times (l_w + e_y + \pi \times d) =$ **2865** mm
Max. load capacity in punching at 2d from face	$P_p = v_{Rd,c} \times u_1 \times d =$ **137.3** kN
Utilisation	$F_{uls} / P_p =$ **0.176**

PASS - Total slab capacity in punching at 2d from face of loaded area exceeds applied load

CONCRETE INDUSTRIAL GROUND FLOOR SLAB DESIGN

In accordance with TR34, 4th Edition 2013

Tedds calculation version 2.0.04

Design summary

Load 1 -Line load 10 mm from edge

Description	Unit	Provided	Required	Utilisation	Result
Slab capacity in flexure	kN/m	22.5	13.1	0.580	PASS

Slab details

Reinforcement type — Fabric

Concrete class — C25/30

Slab thickness — h = **150** mm

Fabric reinforcement type — A393

Characteristic strength of reinforcement — f_{yk} = **500** N/mm^2

Area of bottom steel provided — $A_{s,prov}$ = **393** mm^2/m

Diameter of reinforcement — ϕ_s = **10** mm

Nominal cover — c_{nom_b} = **50** mm

Effective depth of reinforcement — $d = h - c_{nom\_b} - \phi_s$ = **90** mm

Partial safety factors

Concrete (with or without fibre) — γ_c = **1.50**

Reinforcement (bar or fabric) — γ_s = **1.15**

Permanent — γ_G = **1.35**

Variable — γ_Q = **1.50**

Dynamic loads — γ_D = **1.60**

Subgrade reaction

Modulus of subgrade reaction — k = **0.015** N/mm^3

Concrete details - Table 6.1. Strength properties for concrete

Characteristic compressive cylinder strength — f_{ck} = **25** N/mm^2

Characteristic compressive cube strength — f_{cu} = **30** N/mm^2

Mean value of compressive cylinder strength — $f_{cm} = f_{ck} + 8\ N/mm^2$ = **33** N/mm^2

Mean value of axial tensile strength — $f_{ctm} = 0.3\ N/mm^2 \times (f_{ck}/\ 1\ N/mm^2)^{2/3}$ = **2.6** N/mm^2

Flexural tensile strength — $f_{ctd,fl} = f_{ctm} \times (1.6 - h / 1m) / \gamma_c$ = **2.5** N/mm^2

Design concrete compressive strength (cylinder) — $f_{cd} = f_{ck} / \gamma_c$ = **16.7** N/mm^2

Secant modulus of elasticity of concrete — $E_{cm} = 22\ kN/mm^2 \times [f_{cm}/\ 10\ N/mm^2]^{0.3}$ = **31** kN/mm^2

Poisons ratio — ν = **0.2**

Radius of relative stiffness (Eqn. 20) — $l = [E_{cm} \times h^3 / (12 \times (1 - \nu^2) \times k)]^{0.25}$ = **885** mm

Characteristic of system (Eqn. 33) — $\lambda = (3 \times k / (E_{cm} \times h^3))^{0.25}$ = **0.807** m^{-1}

Moment capacity

Negative moment capacity (Eqn. 2) — $M_n = M_{un} = f_{ctd,fl} \times (h^2 / 6)$ = **9.3** kNm/m

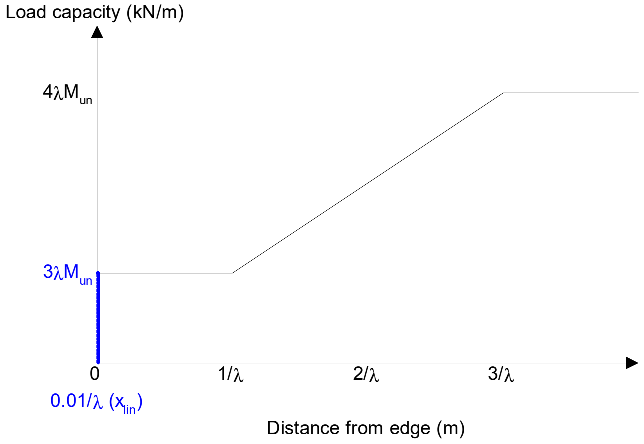
Positive moment capacity (Eqn. 3) — $M_p = M_{pfab} = 0.95 \times A_{s,prov} \times f_{yk} \times d / \gamma_s$ =**14.6** kNm/m

Ratio of cracked to uncracked mnt of resist (cl.7.4) — M_p / M_n = **1.571**

Project Name:	Woodside Farm, Heath Road, LE67 1DG		**Job No:** 25.1620
Reference:	Existing Slab Check - Line Load		**Date:** 02/12/2025
Made by:	DS	**Checked by:** HA	**Sheet No:** 2

PASS - Ratio of cracked to uncracked moment of resistance is greater or equal to 0.5

Load 1 - Line load 10 mm from edge



Working load capacity under a line load

Line load	L_k = **13.1** kN/m
Distance of line load from edge of slab	x_{lin} = **10** mm
Working load capacity of slab	$P_{lin} = 3 \times \lambda \times M_{un}$ = **22.5** kN/m
Utilisation	L_k / P_{lin} = **0.580**

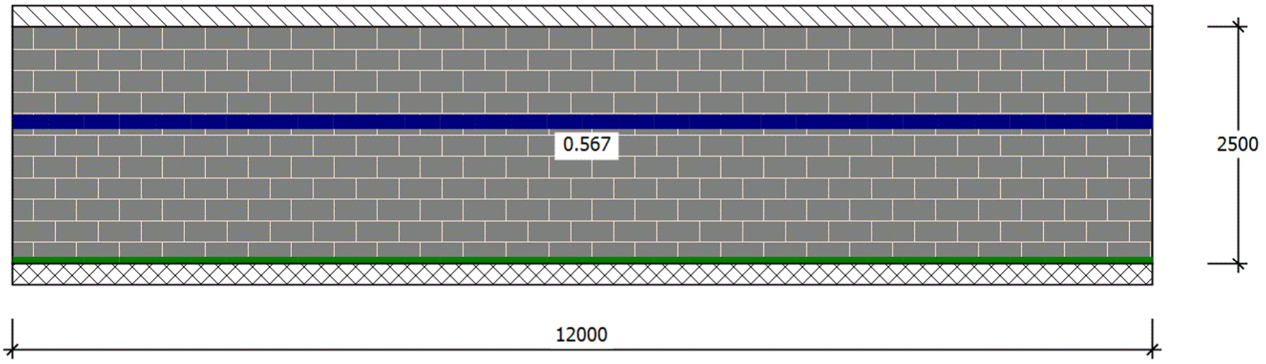
PASS - Total slab capacity exceeds applied load

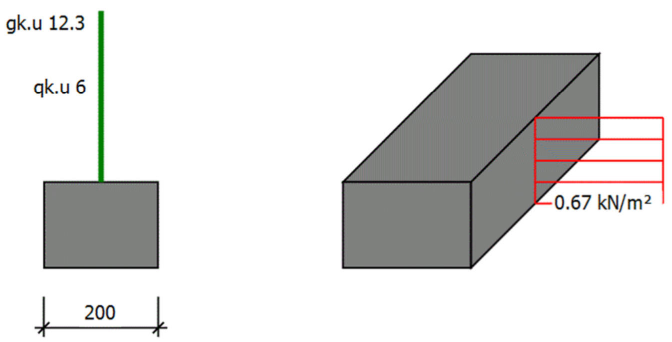
AMP Structures Ltd.

Job ref	: 25.1620
Made By	: DS
Date	**: 02/12/2025/ Version 2015.04**
Checked	**: DS**
Approved	**: HA**

VERTICALLY SPANNING, VERTICALLY AND LATERALLY LOADED, SINGLE-LEAF WALL

DESIGN TO BS EN 1996-1-1:2005+A1:2012
Brief 1





Summary of Design Data

EuroCode National Annex	Using UK valuesA1 2012		
Wall Dimensions	h=2.500 m, hef=2.500 m, L=12.000 m, Lef=12.000 m		
Support Conditions	Vertically Spanning Wall, Top Simple, Bottom Cont.		
Lateral Loads	Wx=0.67 kN/m²		
Single-leaf Wall (mm)	t=200, tef=200		
Limiting Dimensions	λ=12.5<=λ_{lim}=27, h<=30 tef	0.463	OK

Wall Design

Partial Safety Factor (γmc/γmf)	Construction Class 1, Unit Manufacture I	2.3/2.3	Table NA.1
Unit Material	Concrete Blocks, Group 1, γ=18.64 kN/m³		
	Normalised mean compressive strengfb =2.9 N/mm²		
Mortar Material	M2 fm =2 N/mm²		
Unit Ratio	Unit height=215, Least horizontal dimensions=200	1.08	
Compressive Strength (f_k)	k = 0.75, α = 0.7, β = 0.3	1.95 N/mm²	Table NA.4
Loads from above	Dead Load=12.3 kN/m, Live Load=6.0 kN/m		
Section Properties	Area=2000 cm²/m, Zb=6667 cm³/m		
Flexural Strength f_{xk1} (Parallel)	f_{xk1}=0.133, gd=0.085 N/mm²		
	f_{xk1}=f_{xk1}+min(gd, 0.15•f_k/γmc)γmf	0.328 N/mm²	Table NA.6
Critical axial compressive Case	1.25(γ.tk.h+gku)+1.5qku		
Max local stress @	X=0 m,Y=1.25 m < fk/γ_{mc}	0.18 N/mm²	OK
Critical axial buckling Case	1.25(γ.tk.h+gku)+1.5qku		

Max axial buckling force @	X=6 m, Y=1.25 m averaged over width Of 2 m	36.02kN/m	
Moments from Lateral Load	$M_{wx,top}$=0.000 kN.m, $M_{wx,mid}$=0.000 kN.m		
Capacity reduction factor top, ~F	ex=0.0 mm, hef=2500 mm, tef=200.0 mm, t=200.0 mm	0.900	
Capacity reduction factor mid, $\sim F_m$	ehm = 0.000 mm, h_{ef} = 2.500	0.796	
Fr=~F.f_k.tk/γmc	0.796x1.95x200/2.3	134.7 kN/m	
Fd/Fr	36.0/134.7	0.267	OK
Mr=f_{xk2}.Zp/γmf	0.267x6667/2.3	0.773 kN.m/m	
Mr=f_{xk1}.Zb/γmf	0.328x6667/2.3	0.952 kN.m/m	

Design For Lateral Loads

Design Lateral Load Wd	1.5 Wx	1.005 kN/m²	
Yield Line Analysis	Load Factor, λ_p	1.765	
Ut=1/λ_p	1 / 1.765	0.567	OK

AMP STRUCTURES

Calculation Sheet

Project Name:	Woodside Farm, Heath Road, LE67 1DG		**Job No:** 25.1620
Reference:	Snow Load		**Date:** 02/12/2025
Made by:	DS	**Checked by:** HA	**Sheet No:** 1

SNOW LOADING

In accordance with EN1991-1-3:2003+A1:2015 incorporating corrigenda dated December 2004 and March 2009 and the UK national annex NA+A1:2015 to BS EN 1991-1-3:2003+A1:2015 incorporating Corrigendum No.1

Tedds calculation version 1.0.14

Characteristic ground snow load

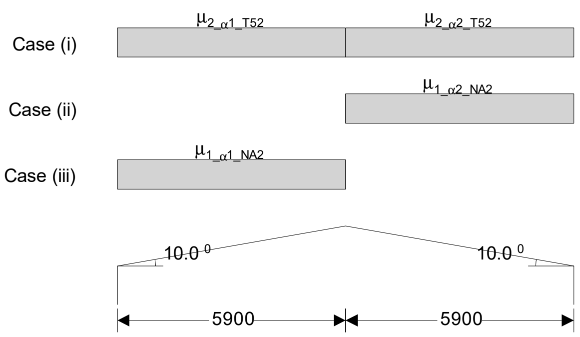
Location	Leicester
Site altitude above sea level (user modified value)	A = **145** m
Zone number	Z = **3.0**
Density of snow	γ = **2.00** kN/m^3
Characteristic ground snow load	s_k = ((0.15 + (0.1 × Z + 0.05)) + ((A - 100m) / 525m)) × 1kN/m^2 = **0.59** kN/m^2
Exposure coefficient (Normal)	C_e = **1.0**
Thermal coefficient	C_t = **1.0**
Snow fence	Not present

Building details

Roof type	Duopitch
Width of roof (left on elevation)	b_1 = **5.90** m
Width of roof (right on elevation)	b_2 = **5.90** m
Slope of roof (left on elevation)	α_1 = **10.00** deg
Slope of roof (right on elevation)	α_2 = **10.00** deg

Shape coefficients

Shape coefficient roof (Table 5.2)	$\mu_{2\_\alpha1\_T52}$ = **0.80**
Shape coefficient roof (Table 5.2)	$\mu_{2\_\alpha2\_T52}$ = **0.80**
Shape coefficient roof (Table UK NA.2)	$\mu_{1\_\alpha1\_NA2}$ = **0.80**
Shape coefficient roof (Table UK NA.2)	$\mu_{1\_\alpha2\_NA2}$ = **0.80**



Elevation on gable roof

Shape coef	Coef	Loading (kN/m^2)
$\mu_{2\_\alpha1\_T52}$	0.800	0.47
$\mu_{2\_\alpha2\_T52}$	0.800	0.47
$\mu_{1\_\alpha1\_NA2}$	0.800	0.47
$\mu_{1\_\alpha2\_NA2}$	0.800	0.47

Loadcase 1 Table 5.2

Loading to roof 1 (LHS)	$s_{1\_1} = \mu_{2\_\alpha1\_T52} \times C_e \times C_t \times s_k$ = **0.47** kN/m^2
Loading to roof 2 (RHS)	$s_{2\_1} = \mu_{2\_\alpha2\_T52} \times C_e \times C_t \times s_k$ = **0.47** kN/m^2

Loadcase 2 UK NA.2

Loading to roof 1 (LHS) $s_{1\_2} = 0 \times C_e \times C_t \times s_k =$ **0.00** kN/m^2

Loading to roof 2 (RHS) $s_{2\_2} = \mu_{1\_\alpha2\_NA2} \times C_e \times C_t \times s_k =$ **0.47** kN/m^2

Loadcase 3 UK NA.2

Loading to roof 1 (LHS) $s_{1\_3} = \mu_{1\_\alpha1\_NA2} \times C_e \times C_t \times s_k =$ **0.47** kN/m^2

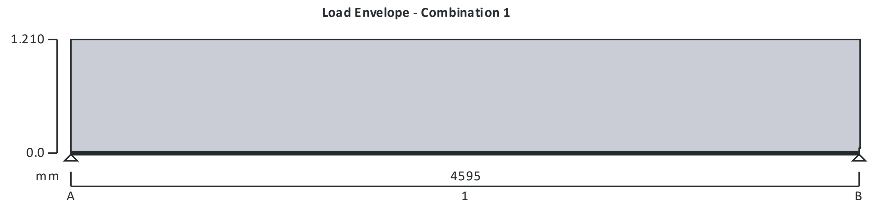
Loading to roof 2 (RHS) $s_{2\_3} = 0 \times C_e \times C_t \times s_k =$ **0.00** kN/m^2

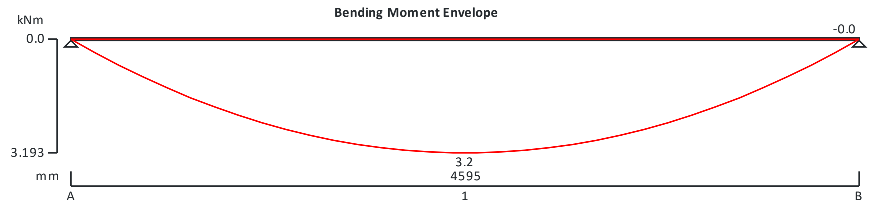
Project Name:	Woodside Farm, Heath Road, LE67 1DG		**Job No:** 25.1620
Reference:	Purlin Check (without PV's)		**Date:** 02/12/2025
Made by:	DS	**Checked by:** HA	**Sheet No:** 1

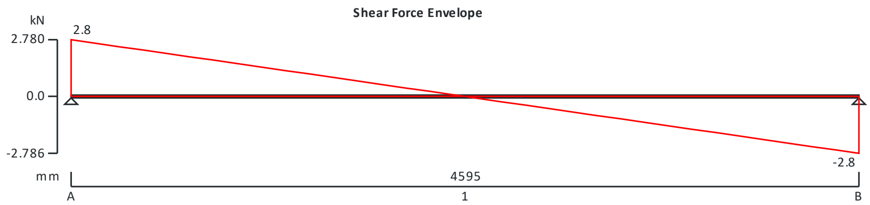
TIMBER BEAM ANALYSIS & DESIGN TO EN1995-1-1:2004

In accordance with EN1995-1-1:2004 + A2:2014 and Corrigendum No.1 and the UK National Annex incorporating National Amendment No.1

Tedds calculation version 1.7.05







Applied loading

Beam loads

Permanent self weight of beam × 1

Span 1 loads

Permanent UDL 0.124 kN/m from 0 mm to 4600 mm

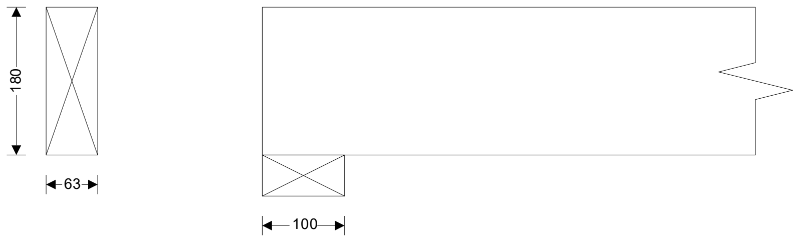
Variable UDL 0.660 kN/m from 0 mm to 4600 mm

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} = **3.193** kNm	M_{min} = **0.000** kNm
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **3.193** kNm	
Maximum shear	F_{max} = **2.780** kN	F_{min} = **-2.786** kN
Design shear	F = max(abs(F_{max}),abs(F_{min})) = **2.786** kN	
Total load on beam	W_{tot} = **5.565** kN	
Reactions at support A	R_{A_max} = **2.780** kN	R_{A_min} = **2.780** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **0.374** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **1.516** kN	
Reactions at support B	R_{B_max} = **2.786** kN	R_{B_min} = **2.786** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **0.375** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **1.520** kN	



Timber section details

Breadth of timber sections	b = **63** mm
Depth of timber sections	h = **180** mm
Number of timber sections in member	N = **1**
Overall breadth of timber member	b_b = N × b = **63** mm
Timber strength class - EN 338:2016 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 span	L_{s1} = **4595** mm
Length of bearing	L_b = **100** mm

Section properties

Cross sectional area of member	A = N × b × h = **11340** mm^2
Section modulus	W_y = N × b × h^2 / 6 = **340200** mm^3
	W_z = h × (N × b)^2 / 6 = **119070** mm^3
Second moment of area	I_y = N × b × h^3 / 12 = **30618000** mm^4
	I_z = h × (N × b)^3 / 12 = **3750705** mm^4
Radius of gyration	r_y = √(I_y / A) = **52.0** mm
	r_z = √(I_z / A) = **18.2** 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 $k_{c.90}$ = **1.000**

System strength factor - cl.6.6 k_{sys} = **1.000**

Lateral buckling factor - cl.6.3.3(5) k_{crit} = **1.000**

Compression perpendicular to the grain - cl.6.1.5

Design compressive stress $\sigma_{c.90.d} = R_{B\_max} / (N \times b \times L_b)$ = **0.442** N/mm^2

Design compressive strength $f_{c.90.d} = k_{mod} \times k_{sys} \times k_{c.90} \times f_{c.90.k} / \gamma_M$ = **1.346** N/mm^2

$\sigma_{c.90.d} / f_{c.90.d}$ = **0.328**

PASS - Design compressive strength exceeds design compressive stress at bearing

Bending - cl 6.1.6

Design bending stress $\sigma_{m.d} = M / W_y$ = **9.387** N/mm^2

Design bending strength $f_{m.d} = k_{h.m} \times k_{mod} \times k_{sys} \times k_{crit} \times f_{m.k} / \gamma_M$ = **12.923** N/mm^2

$\sigma_{m.d} / f_{m.d}$ = **0.726**

PASS - Design bending strength exceeds design bending stress

Shear - cl.6.1.7

Applied shear stress $\tau_d = 3 \times F / (2 \times k_{cr} \times A)$ = **0.550** 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

$\tau_d / f_{v.d}$ = **0.255**

PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

Deflection limit $\delta_{lim} = \min(20\text{ mm}, 0.004 \times L_{s1})$ = **18.380** mm

Instantaneous deflection due to permanent load δ_{instG} = **2.874** mm

Final deflection due to permanent load $\delta_{finG} = \delta_{instG} \times (1 + k_{def})$ = **4.598** mm

Instantaneous deflection due to variable load δ_{instQ} = **11.642** mm

Factor for quasi-permanent variable action ψ_2 = **0.3**

Final deflection due to variable load $\delta_{finQ} = \delta_{instQ} \times (1 + \psi_2 \times k_{def})$ = **13.738** mm

Total final deflection $\delta_{fin} = \delta_{finG} + \delta_{finQ}$ = **18.336** mm

$\delta_{fin} / \delta_{lim}$ = **0.998**

PASS - Total final deflection is less than the deflection limit

STEEL MEMBER 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

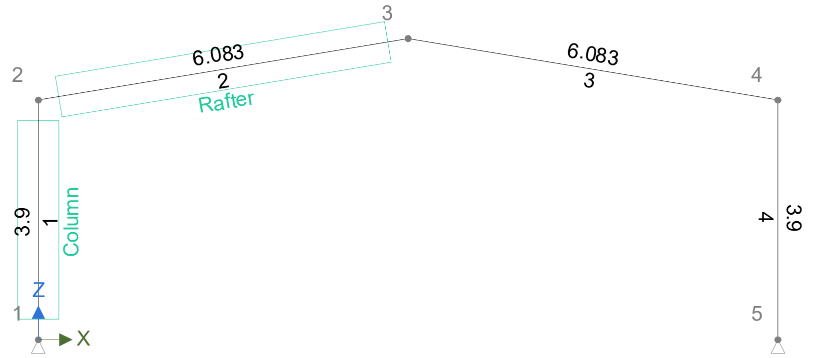
Tedds calculation version 4.5.03

ANALYSIS

Tedds calculation version 1.0.38

Geometry

Geometry (m) - Steel (EC3) - UKB 203x133x25



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	Moment of inertia		Shear area parallel to	
		Major	Minor	Minor	Major
	(cm²)	(cm⁴)	(cm⁴)	(cm²)	(cm²)
UKB 203x133x25	32	2340.2	307.6	11.6	18.7

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	3.9	Free	Free	Free		0	0	0	0
3	6	4.9	Free	Free	Free		0	0	0	0
4	12	3.9	Free	Free	Free		0	0	0	0
5	12	0	Fixed	Fixed	Free		0	0	0	0

Project Name:	Woodside Farm, Heath Road, LE67 1DG		**Job No:** 25.162
Reference:	Existing Frame Check		**Date:** 02/12/2025
Made by:	DS	**Checked by:** HA	**Sheet No:** 2

Elements

Element	Length (m)	Nodes		Section	Material	Releases			Rotated
		Start	End			Start moment	End moment	Axial	
1	3.9	1	2	UKB 203x133x25	Steel (EC3)	Fixed	Fixed	Fixed	
2	6.083	2	3	UKB 203x133x25	Steel (EC3)	Fixed	Fixed	Fixed	
3	6.083	3	4	UKB 203x133x25	Steel (EC3)	Fixed	Fixed	Fixed	
4	3.9	4	5	UKB 203x133x25	Steel (EC3)	Fixed	Fixed	Fixed	

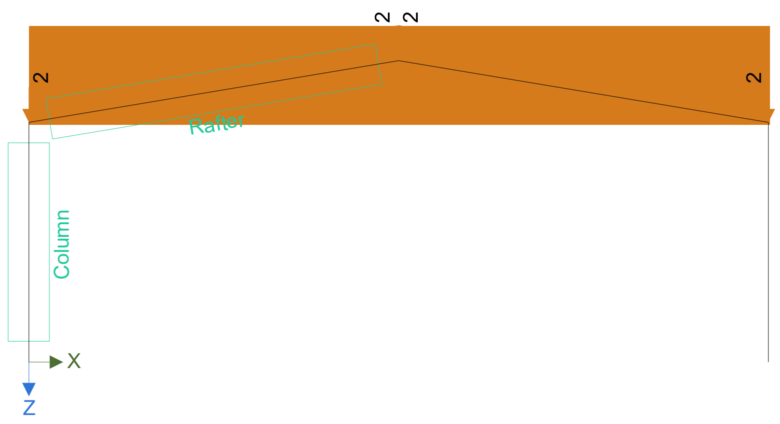
Members

Name	Elements	
	Start	End
Column	1	1
Rafter	2	2

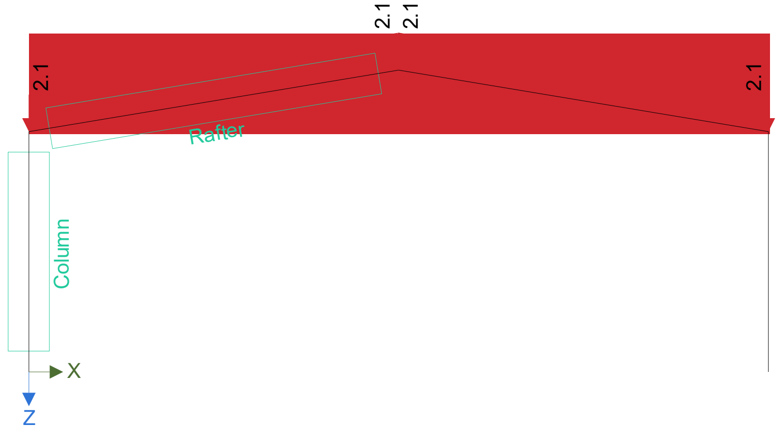
Loading

Self weight included

Permanent - Loading (kN/m)

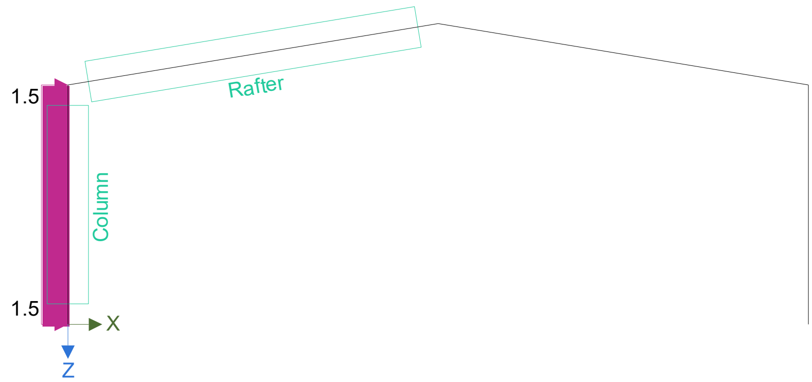


Imposed - Loading (kN/m)



Project Name:	Woodside Farm, Heath Road, LE67 1DG		**Job No:** 25.162
Reference:	Existing Frame Check		**Date:** 02/12/2025
Made by:	DS	**Checked by:** HA	**Sheet No:** 3

Wind - Loading (kN/m)



Load combination factors

Load combination	Self Weight	Permanent	Imposed	Wind
1.35G + 1.5Q + 1.5RQ (Strength)	1.35	1.35	1.50	0.75
1.0G + 1.0Q + 1.0RQ (Service)	1.00	1.00	1.00	
1.35G + 1.5Q + 1.5ψ_0S (Strength)	1.35	1.35	1.50	
1.0G + 1.0Q + 0.5S (Service)	1.00	1.00	1.00	
1.35G + 1.5ψ_0Q + 1.5S (Strength)	1.35	1.35	1.05	
1.35G + 1.5Q + 1.5ψ_0S + 1.5ψ_0W (Strength)	1.35	1.35	1.50	0.75
1.0G + 1.0Q + 0.5S + 0.5W (Service)	1.00	1.00	1.00	0.50
1.35G + 1.5ψ_0Q + 1.5S + 1.5ψ_0W (Strength)	1.35	1.35	1.05	0.75
1.0G + 1.0ψ_0Q + 1.0S + 0.5W (Service)	1.00	1.00	0.70	0.50
1.35G + 1.5ψ_0Q + 1.5ψ_0S + 1.5W (Strength)	1.35	1.35	1.05	1.50
1.0G + 1.0ψ_0Q + 0.5S + 1.0W (Service)	1.00	1.00	0.70	1.00
1.0G + 1.5W (Strength)	1.00	1.00		1.50
1.0G + 1.0W (Service)	1.00	1.00		1.00
1.35G + 1.5ψ_0Q + 1.5ψ_0RQ (Strength)	1.35	1.35	1.05	
1.35G + 1.5ψ_0Q + 1.5ψ_0S (Strength)	1.35	1.35	1.05	
1.35ξG + 1.5Q + 1.5RQ (Strength)	1.25	1.25	1.50	
1.35ξG + 1.5Q + 1.5ψ_0S (Strength)	1.25	1.25	1.50	
1.35ξG + 1.5ψ_0Q + 1.5S (Strength)	1.25	1.25	1.05	
1.35G + 1.5ψ_0Q + 1.5ψ_0S + 1.5ψ_0W (Strength)	1.35	1.35	1.05	0.75
1.35ξG + 1.5Q + 1.5ψ_0S + 1.5ψ_0W (Strength)	1.25	1.25	1.50	0.75
1.35ξG + 1.5ψ_0Q + 1.5S + 1.5ψ_0W (Strength)	1.25	1.25	1.05	0.75

Project Name:	Woodside Farm, Heath Road, LE67 1DG		**Job No:** 25.162
Reference:	Existing Frame Check		**Date:** 02/12/2025
Made by:	DS	**Checked by:** HA	**Sheet No:** 4

Load combination	Self Weight	Permanent	Imposed	Wind
$1.35\xi G + 1.5\psi_0 Q + 1.5\psi_0 S + 1.5W$ (Strength)	1.25	1.25	1.05	1.50

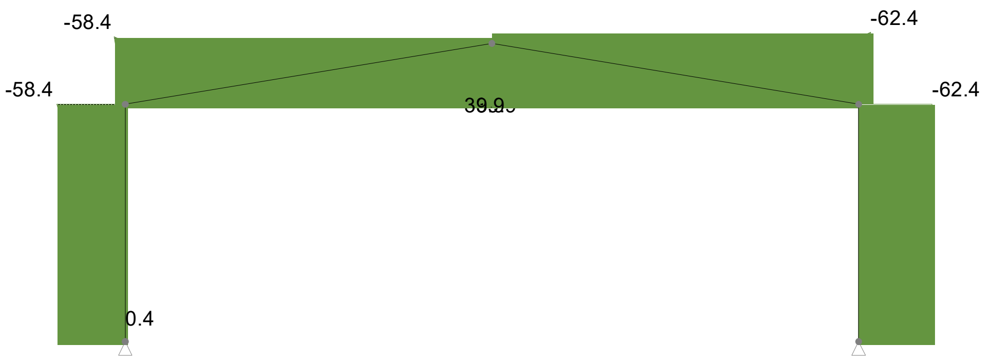
Element Loads

Element	Load case	Load Type	Orientation	Description
2	Permanent	UDL	GlobalZ	2 kN/m
3	Permanent	UDL	GlobalZ	2 kN/m
2	Imposed	UDL	GlobalZ	2.1 kN/m
3	Imposed	UDL	GlobalZ	2.1 kN/m
1	Wind	UDL	GlobalX	1.5 kN/m

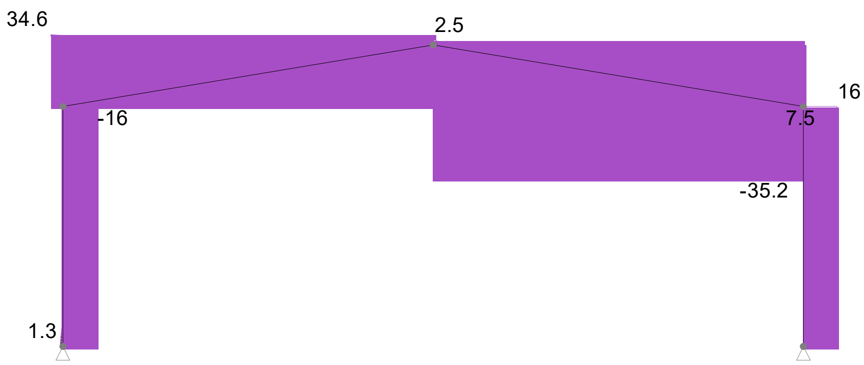
Results

Forces

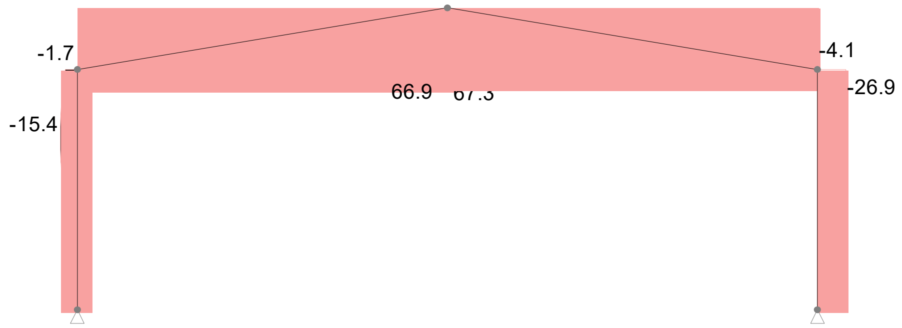
Strength combinations - Moment envelope (kNm)



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**

Column design

Section details

Section type UKB 203x133x25 (Tata Steel Advance)

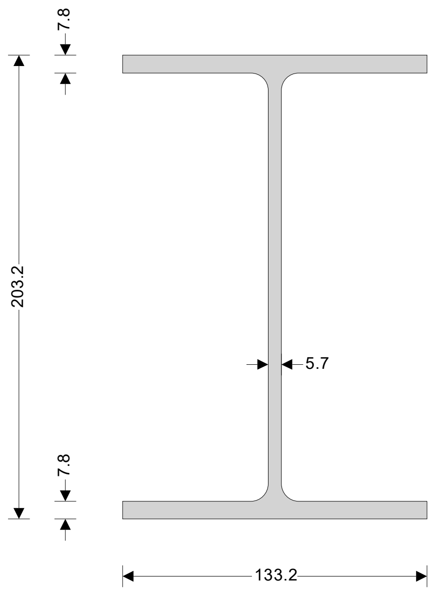
Steel grade - EN 10025-2:2004 S355

Nominal thickness of element t_{nom} = max(t_f, t_w) = **7.8** mm

Nominal yield strength f_y = **355** N/mm^2

Nominal ultimate tensile strength f_u = **470** N/mm^2

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



UKB 203x133x25 (Tata Steel Advance)
Section depth, h, 203.2 mm
Section breadth, b, 133.2 mm
Mass of section, Mass, 25.1 kg/m
Flange thickness, t_f, 7.8 mm
Web thickness, t_w, 5.7 mm
Root radius, r, 7.6 mm
Area of section, A, 3197 mm^2
Radius of gyration about y-axis, i_y, 85.559 mm
Radius of gyration about z-axis, i_z, 31.021 mm
Elastic section modulus about y-axis, $W_{el.y}$, 230332 mm^3
Elastic section modulus about z-axis, $W_{el.z}$, 46190 mm^3
Plastic section modulus about y-axis, $W_{pl.y}$, 257731 mm^3
Plastic section modulus about z-axis, $W_{pl.z}$, 70944 mm^3
Second moment of area about y-axis, I_y, 23401694 mm^4
Second moment of area about z-axis, I_z, 3076268 mm^4

Lateral restraint

Both flanges have lateral restraint at supports only

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235\ N/mm^2 / f_y]} = \mathbf{0.81}$

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

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

$\alpha = \min([h / 2 + N_{Ed} / (2 \times t_w \times f_y) - (t_f + r)] / c, 1) = \mathbf{0.556}$

$c / t_w = 30.2 = 37.2 \times \varepsilon <= 396 \times \varepsilon / (13 \times \alpha - 1)$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

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

$c / t_f = 7.2 = 8.8 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Section is class 1

Check compression - Section 6.2.4

Design compression force — $N_{Ed} = \mathbf{38.9}$ kN

Design resistance of section - eq 6.10 — $N_{c,Rd} = A \times f_y / \gamma_{M0} = \mathbf{1134.9}$ kN

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

PASS - Design compression resistance exceeds design compression

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

Critical buckling length — $L_{cr,y} = L_{m1\_s1} = \mathbf{3900}$ mm

Critical buckling force — $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2 = \mathbf{3188.9}$ kN

Slenderness ratio for buckling - eq 6.50 — $\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})} = \mathbf{0.597}$

Check y-y axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2 — a

Imperfection factor - Table 6.1 — $\alpha_y = \mathbf{0.21}$

Buckling reduction determination factor — $\Phi_y = 0.5 \times (1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2) = \mathbf{0.72}$

Buckling reduction factor - eq 6.49 — $\chi_y = \min(1 / (\Phi_y + \sqrt{(\Phi_y^2 - \bar{\lambda}_y^2)}), 1) = \mathbf{0.891}$

Design buckling resistance - eq 6.47 — $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1} = \mathbf{1011.5}$ kN

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

PASS - Design buckling resistance exceeds design compression

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

Critical buckling length — $L_{cr,z} = L_{m1\_s1\_seg1} = \mathbf{3900}$ mm

Critical buckling force — $N_{cr,z} = \pi^2 \times E \times I_z / L_{cr,z}^2 = \mathbf{419.2}$ kN

Slenderness ratio for buckling - eq 6.50 — $\bar{\lambda}_z = \sqrt{(A \times f_y / N_{cr,z})} = \mathbf{1.645}$

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 — $\Phi_z = 0.5 \times (1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2) = \mathbf{2.099}$

Buckling reduction factor - eq 6.49 — $\chi_z = \min(1 / (\Phi_z + \sqrt{(\Phi_z^2 - \bar{\lambda}_z^2)}), 1) = \mathbf{0.294}$

Design buckling resistance - eq 6.47 — $N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1} = \mathbf{333.5}$ kN

$N_{Ed} / N_{b,z,Rd} = \mathbf{0.117}$

PASS - Design buckling resistance exceeds design compression

Check torsional and torsional-flexural buckling - Section 6.3.1.4

Torsional buckling length — $L_{cr,T} = L_{m1\_s1\_seg1\_R}$ = **3900** mm

Distance from shear centre to centroid in y axis — y_0 = **0.0** mm

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

Radius of gyration — $i_0 = \sqrt{(i_y^2 + i_z^2)}$ = **91.0** mm

Elastic critical torsional buckling force — $N_{cr,T} = (G \times I_t + \pi^2 \times E \times I_w / L_{cr,T}^2) / i_0^2$ = **1064.7** kN

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

Elastic critical torsional-flexural buckling force — $N_{cr,TF}$ = **780.2** kN

Elastic critical buckling force — $N_{cr} = \min(N_{cr,T}, N_{cr,TF})$ = **780.2** kN

Slenderness ratio for torsional buckling - eq 6.52 — $\bar{\lambda}_T = \sqrt{(A \times f_y / N_{cr})}$ = **1.206**

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

Buckling curve - Table 6.2 — b

Imperfection factor - Table 6.1 — α_T = **0.34**

Buckling reduction determination factor — $\Phi_T = 0.5 \times (1 + \alpha_T \times (\bar{\lambda}_T - 0.2) + \bar{\lambda}_T^2)$ = **1.398**

Buckling reduction factor - eq 6.49 — $\chi_T = \min(1 / (\Phi_T + \sqrt{(\Phi_T^2 - \bar{\lambda}_T^2)}), 1)$ = **0.475**

Design buckling resistance - eq 6.47 — $N_{b,T,Rd} = \chi_T \times A \times f_y / \gamma_{M1}$ = **538.9** kN

$N_{Ed} / N_{b,T,Rd}$ = **0.072**

PASS - Design buckling resistance exceeds design compression

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

Check shear - Section 6.2.6

Height of web — $h_w = h - 2 \times t_f$ = **187.6** mm — η = **1.000**

$h_w / t_w = 32.9 = 40.5 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force — $V_{y,Ed}$ = **15** kN

Shear area - cl 6.2.6(3) — $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **1282** mm²

Design shear resistance - cl 6.2.6(2) — $V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **262.7** kN

$V_{y,Ed} / V_{c,y,Rd}$ = **0.057**

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment — $M_{y,Ed}$ = **58.4** kNm

Design bending resistance moment - eq 6.13 — $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **91.5** kNm

$M_{y,Ed} / M_{c,y,Rd}$ = **0.638**

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 — k_c = **0.77**

$C_1 = 1 / k_c^2$ = **1.687**

Poissons ratio — ν = **0.3**

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

Unrestrained effective length — $L = 1.0 \times L_{m1\_s1\_seg1\_B}$ = **3900** mm

Elastic critical buckling moment — $M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **102.5** kNm

Slenderness ratio for lateral torsional buckling — $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **0.945**

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 — $\Phi_{LT} = 0.5 \times (1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2)$ = **0.927**

LTB reduction factor - eq 6.57 — $\chi_{LT} = \min(1 / (\Phi_{LT} + \sqrt{(\Phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}), 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.733**

Modification factor — $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.890**

Modified LTB reduction factor - eq 6.58 — $\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.824**

Design buckling resistance moment - eq 6.55 — $M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1}$ = **75.4** kNm

$M_{y,Ed} / M_{b,y,Rd}$ = **0.774**

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 — $N_{y,lim} = \min(0.25 \times N_{pl,Rd}, 0.5 \times h_w \times t_w \times f_y / \gamma_{M0})$ = **189.8** kN

$N_{Ed} / N_{y,lim}$ = **0.198**

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 = 0 kNm / -58.401 kNm = **0.000**

α_y = -29.2 kNm / -58.401 kNm = **0.500**

$C_{my} = \max(0.6 + 0.4 \times \psi_y)$ = **0.600**

ψ_{LT} = 0 kNm / -58.401 kNm = **0.000**

α_{LT} = -29.2 kNm / -58.401 kNm = **0.500**

$C_{mLT} = \max(0.6 + 0.4 \times \psi_{LT})$ = **0.600**

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

Characteristic moment resistance — $M_{y,Rk} = W_{pl.y} \times f_y$ = **91.5** kNm

Characteristic moment resistance — $M_{z,Rk} = W_{pl.z} \times f_y$ = **25.2** kNm

Characteristic resistance to normal force — $N_{Rk} = A \times f_y$ = **1134.9** kN

Interaction factors — $k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}))$ = **0.609**

$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1})$ = **0.968**

Interaction formulae - eq 6.61 & eq 6.62 — $N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.567**

$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.955**

PASS - Combined bending and compression checks are satisfied

AMP STRUCTURES

Calculation Sheet

Project Name:	Woodside Farm, Heath Road, LE67 1DG		**Job No:** 25.162
Reference:	Existing Frame Check		**Date:** 02/12/2025
Made by:	DS	**Checked by:** HA	**Sheet No:** 9

Rafter design

Section details

Section type — UKB 203x133x25 (Tata Steel Advance)

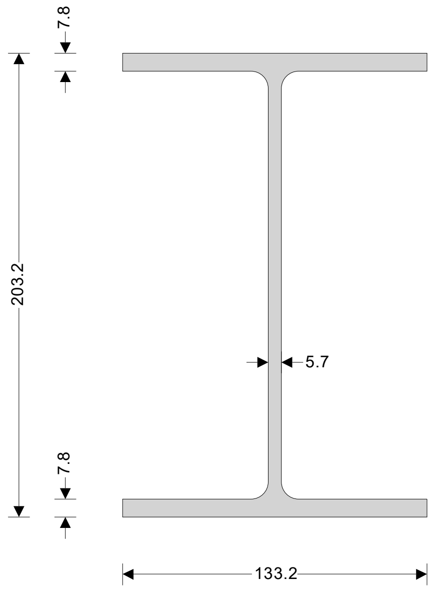
Steel grade - EN 10025-2:2004 — S355

Nominal thickness of element — $t_{nom} = \max(t_f, t_w) = \mathbf{7.8}$ mm

Nominal yield strength — $f_y = \mathbf{355}$ N/mm^2

Nominal ultimate tensile strength — $f_u = \mathbf{470}$ N/mm^2

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



UKB 203x133x25 (Tata Steel Advance)
Section depth, h, 203.2 mm
Section breadth, b, 133.2 mm
Mass of section, Mass, 25.1 kg/m
Flange thickness, t_f, 7.8 mm
Web thickness, t_w, 5.7 mm
Root radius, r, 7.6 mm
Area of section, A, 3197 mm^2
Radius of gyration about y-axis, i_y, 85.559 mm
Radius of gyration about z-axis, i_z, 31.021 mm
Elastic section modulus about y-axis, $W_{el.y}$, 230332 mm^3
Elastic section modulus about z-axis, $W_{el.z}$, 46190 mm^3
Plastic section modulus about y-axis, $W_{pl.y}$, 257731 mm^3
Plastic section modulus about z-axis, $W_{pl.z}$, 70944 mm^3
Second moment of area about y-axis, I_y, 23401694 mm^4
Second moment of area about z-axis, I_z, 3076268 mm^4

Lateral restraint

Upper flange has full lateral restraint

Lower flange has lateral restraint at supports only

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]} = \mathbf{0.81}$

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

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

$\alpha = \min([h / 2 + N_{Ed} / (2 \times t_w \times f_y) - (t_f + r)] / c, 1) = \mathbf{0.530}$

$c / t_w = 30.2 = 37.2 \times \varepsilon <= 396 \times \varepsilon / (13 \times \alpha - 1)$ — Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

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

$c / t_f = 7.2 = 8.8 \times \varepsilon <= 9 \times \varepsilon$ — Class 1

Section is class 1

Check compression - Section 6.2.4

Design compression force — $N_{Ed} = \mathbf{21}$ kN

Design resistance of section - eq 6.10 — $N_{c,Rd} = A \times f_y / \gamma_{M0} = \mathbf{1134.9}$ kN

$N_{Ed} / N_{c,Rd}$ = **0.018**

PASS - Design compression resistance exceeds design compression

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

Critical buckling length — $L_{cr,y} = L_{m2\_s1}$ = **6083** mm

Critical buckling force — $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2$ = **1310.9** kN

Slenderness ratio for buckling - eq 6.50 — $\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})}$ = **0.93**

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 — $\Phi_y = 0.5 \times (1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2)$ = **1.01**

Buckling reduction factor - eq 6.49 — $\chi_y = \min(1 / (\Phi_y + \sqrt{(\Phi_y^2 - \bar{\lambda}_y^2)}), 1)$ = **0.714**

Design buckling resistance - eq 6.47 — $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1}$ = **809.9** kN

$N_{Ed} / N_{b,y,Rd}$ = **0.026**

PASS - Design buckling resistance exceeds design compression

Check torsional and torsional-flexural buckling - Section 6.3.1.4

Torsional buckling length — $L_{cr,T} = L_{m2\_s1\_seg1\_R}$ = **6083** mm

Distance from shear centre to centroid in y axis — y_0 = **0.0** mm

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

Radius of gyration — $i_0 = \sqrt{(i_y^2 + i_z^2)}$ = **91.0** mm

Elastic critical torsional buckling force — $N_{cr,T} = (G \times I_t + \pi^2 \times E \times I_w / L_{cr,T}^2) / i_0^2$ = **780.2** kN

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

Elastic critical torsional-flexural buckling force — $N_{cr,TF}$ = **780.2** kN

Elastic critical buckling force — $N_{cr} = \min(N_{cr,T}, N_{cr,TF})$ = **780.2** kN

Slenderness ratio for torsional buckling - eq 6.52 — $\bar{\lambda}_T = \sqrt{(A \times f_y / N_{cr})}$ = **1.206**

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

Buckling curve - Table 6.2 — b

Imperfection factor - Table 6.1 — α_T = **0.34**

Buckling reduction determination factor — $\Phi_T = 0.5 \times (1 + \alpha_T \times (\bar{\lambda}_T - 0.2) + \bar{\lambda}_T^2)$ = **1.398**

Buckling reduction factor - eq 6.49 — $\chi_T = \min(1 / (\Phi_T + \sqrt{(\Phi_T^2 - \bar{\lambda}_T^2)}), 1)$ = **0.475**

Design buckling resistance - eq 6.47 — $N_{b,T,Rd} = \chi_T \times A \times f_y / \gamma_{M1}$ = **538.9** kN

$N_{Ed} / N_{b,T,Rd}$ = **0.039**

PASS - Design buckling resistance exceeds design compression

Check design at start of span

Check shear - Section 6.2.6

Height of web — $h_w = h - 2 \times t_f$ = **187.6** mm — η = **1.000**

$h_w / t_w = 32.9 = 40.5 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force — $V_{y,Ed}$ = **34.6** kN

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

Design shear resistance - cl 6.2.6(2) — $V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **262.7** kN

$V_{y,Ed} / V_{c,y,Rd}$ = **0.132**

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment $M_{y,Ed}$ = **58.4** kNm

Design bending resistance moment - eq 6.13 $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **91.5** kNm

$M_{y,Ed} / M_{c,y,Rd}$ = **0.638**

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 k_c = **0.655**

$C_1 = 1 / k_c^2$ = **2.332**

Poissons ratio ν = **0.3**

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

Unrestrained effective length $L = 1.0 \times L_{m2\_s1\_seg1\_B}$ = **6083** mm

Elastic critical buckling moment $M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **77.8** kNm

Slenderness ratio for lateral torsional buckling $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **1.084**

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 $\Phi_{LT} = 0.5 \times (1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2)$ = **1.057**

LTB reduction factor - eq 6.57 $\chi_{LT} = \min(1 / (\Phi_{LT} + \sqrt{(\Phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}), 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.648**

Modification factor $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.855**

Modified LTB reduction factor - eq 6.58 $\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.758**

Design buckling resistance moment - eq 6.55 $M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1}$ = **69.3** kNm

$M_{y,Ed} / M_{b,y,Rd}$ = **0.842**

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 $N_{y,lim} = \min(0.25 \times N_{pl,Rd}, 0.5 \times h_w \times t_w \times f_y / \gamma_{M0})$ = **189.8** kN

$N_{Ed} / N_{y,lim}$ = **0.11**

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 = 39.44 kNm / -58.401 kNm = **-0.675**

α_y = 18.724 kNm / -58.401 kNm = **-0.321**

$C_{my} = \max(0.2 + 0.8 \times \alpha_y, 0.4)$ = **0.400**

ψ_{LT} = 39.44 kNm / -58.401 kNm = **-0.675**

α_{LT} = 18.724 kNm / -58.401 kNm = **-0.321**

$C_{mLT} = \max(0.2 + 0.8 \times \alpha_{LT}, 0.4)$ = **0.400**

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

Characteristic moment resistance — $M_{y,Rk} = W_{pl.y} \times f_y =$ **91.5** kNm

Characteristic moment resistance — $M_{z,Rk} = W_{pl.z} \times f_y =$ **25.2** kNm

Characteristic resistance to normal force — $N_{Rk} = A \times f_y =$ **1134.9** kN

Interaction factors — $k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1})) =$ **0.408**

$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1}) =$ **0.958**

Interaction formulae - eq 6.61 & eq 6.62 — $N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) =$ **0.427**

$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) =$ **1.006**

PASS - Combined bending and compression checks are satisfied

Check design 5679 mm along span

Check bending moment - Section 6.2.5

Design bending moment — $M_{y,Ed} =$ **39.9** kNm

Design bending resistance moment - eq 6.13 — $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl.y} \times f_y / \gamma_{M0} =$ **91.5** kNm

$M_{y,Ed} / M_{c,y,Rd} =$ **0.436**

PASS - Design bending resistance moment exceeds design bending moment

Check combined bending and compression - Section 6.3.3

Equivalent uniform moment factors - Table B.3 — $\psi_y = 39.44\ \text{kNm} / -58.401\ \text{kNm} =$ **-0.675**

$\alpha_y = 18.724\ \text{kNm} / -58.401\ \text{kNm} =$ **-0.321**

$C_{my} = \max(0.2 + 0.8 \times \alpha_y, 0.4) =$ **0.400**

$\psi_{LT} = 39.44\ \text{kNm} / -58.401\ \text{kNm} =$ **-0.675**

$\alpha_{LT} = 18.724\ \text{kNm} / -58.401\ \text{kNm} =$ **-0.321**

$C_{mLT} = \max(0.2 + 0.8 \times \alpha_{LT}, 0.4) =$ **0.400**

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

Characteristic moment resistance — $M_{y,Rk} = W_{pl.y} \times f_y =$ **91.5** kNm

Characteristic moment resistance — $M_{z,Rk} = W_{pl.z} \times f_y =$ **25.2** kNm

Characteristic resistance to normal force — $N_{Rk} = A \times f_y =$ **1134.9** kN

Interaction factors — $k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1})) =$ **0.405**

$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1}) =$ **0.970**

Interaction formulae - eq 6.61 & eq 6.62 — $N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) =$ **0.196**

$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) =$ **0.469**

PASS - Combined bending and compression checks are satisfied

AMP
STRUCTURES

Unit 9 The Energy Centre,
Bowling Green Walk,
London
N1 6AL

mail@ampstructures.com
ampstructures.com

Registered office - Unit 9 The Energy Centre, Bolwing Green Walk, London, N1 6AL.
Registered in England & Wales, Company Number 12303473
VAT No. 384 7668 39