4th May 2023
Job No: 9306RCC

DRAFT

Building A
Chestnut Farm,
Church Lane
Dauntsey
Nr. Chippenham
SN15 4HW

Report on Structural Aspects of the Feasibility of Conversion of a single storey Barn into a Residential Property



Author: M.W.Mitchell
Director BSc CEng MICE

Structural Solutions Management Ltd
Dairy Studios, 102 Lincoln Street, Bristol BS5 0BJ tel: 0117 924 5014
Email: admin@structuralsolutions.co.uk Website: www.structuralsolutions.co.uk

Building A, Chestnut Farm, Church Lane, Dauntsey, Chippenham, Wiltshire SN15 4HW

Report on the Structural Aspects of the feasibility of conversion of a single storey barn (reference Building A) at Chestnut Farm, Church Lane, Dauntsey

Purpose of the Report

Structural Solutions was commissioned to carry out an investigation into the feasibility of converting a single storey barn (ref. Building A) at Chestnut Farm in Dauntsey into a residential property. This report considers the current condition of the structure of the barn and the likely work required as part of the conversion. This is an updated report following the successful planning application under class Q and the Client's decision to submit a full planning application. The main differences between the proposals are the increased area of mezzanine floor and the replacement of wall and roof cladding.

Introduction

Barn A is rectangular in plan approximately 15.5m x 13.5m divided into 3 'bays' of approximately 4.5m width each. Three of the four elevations are clad with a combination of concrete blockwork below vertical timber boarding supported by cladding rails fixed to structural steel columns. The fourth elevation is clad with a combination of precast concrete panels fixed to structural steel columns with corrugated metal sheeting above. The roof is similarly clad but with corrugated cement sheet spanning between purlins on a mono-pitched section connected to an 'arched' roof which is clad with preformed corrugated metal sheeting.

Investigation

Our investigation is based on a visual walk over only. However, the existing foundations were exposed (see Appendix A) and a structural analysis (see Appendix C) carried out on the existing structural steel portal frame using loads appropriate to conversion into residential use.

Barn A

The existing building is currently an agricultural structure comprised of four structural steel mono-pitched portal frames bolted together at column/beam locations. At the upper end of the frame, the beam element extends and is connected together to another column that supports an arched truss spanning to the north east elevation and a further steel column. This arrangement repeats for each frame. Each end of the building, the gable elevations, the portal frames and the arched roof have additional full height steel 'wind' posts which act to support the external wall cladding and transfer wind loads to the ground. The portal frames are designed to resist wind loads

on the front and rear elevations, lateral forces on the end gable elevations are transferred and resisted by 'knee' braces between eaves beam and posts.

The roof cladding is 'simple' corrugated, believe to be, cement based panels spanning between timber purlins which in turn span between the portal frames for the mono-pitched roof section and preformed corrugated steel sheeting spanning between steel angle purlins which in turn span between steel arched roof trusses. The external walls are constructed with concrete blockwork below vertical timber boards to the portal frame section of the building and precast concrete panels fixed to the steel columns of the 'arched' roof structural frames to a height of approximately 1.5 metres. Above is a profile metal cladding sheet supported by steel angle cladding rails. Internally, the building is divided by a concrete block wall approximately 1.4m high. The south east elevation has two separate doors to access the two different section of the building.

The building has a concrete slab throughout and this looks to be sound and no significant cracking was seen indicating that the formation below is likely to be good. There are steps in the slab which is common for cattle barns which we understand this barn was used for.

Discussion

We understand from the Client that the proposed conversion work the barns will maintain the basic plan of the building including retention of the existing roof profiles.

Generally, the structure can be incorporated into a conversion of the barn without any structural alterations. Analysis of both the existing steel frame structure and foundations (see Appendix C) show that they are capable of supporting loads from a residential building without modification. The external concrete panels and concrete blockwork are in relatively good condition. The revised planning proposal intends to upgrade external cladding of both the walls and roof.

Based on our inspection, the existing concrete floor can be re-used with the introduction of a damp proof membrane, insulation and floor finishes above the concrete.

Conclusion

This report is not intended to be a full and detailed structural survey. Its purpose is to establish in structural terms whether the conversion of the agricultural barns into usable properties is feasible without modification of the main structure. In brief, the existing structural frame, walls and foundations of the barn can be incorporated into a conversion into residential use without modification.

Insulation and breather membrane to achieve appropriate residential standards would be carried by a non-structural non-load bearing partition located inside the existing external walls constructed using a softwood timber frame or lightweight cold-formed metal stud system. The new cladding will be fixed to the new timber or metal stud partitions. Some

sections of the existing internal concrete blockwork walls would be retained for re-use in the proposed plan as shown on the Architect's drawings. Insulated roof cladding would be supported by new cold roller metal purlins.

Outline Schedule of Works

The following is an outline Schedule of Works for Plots 1 & 2 identifying anticipated elements of construction that will be retained and elements that will need to be constructed as new.

Plots 1 & 2 (Building A)
Elements for Retention

- Foundations of external walls and structural steel columns (c. 100%)
- Main structural steel frame – columns and rafters (c. 100%)
- External blockwork – modified to suit new windows and doors (c. 95%)
- External precast concrete panels (c. 100%)
- Concrete floor – modified to become one level (c. 100%)

Proposed Works/Modifications

- External metal wall cladding
- External timber cladding
- Profiled metal roof covering on purlins
- Non-load bearing wall panels to the inside face of the external walls to carry insulation, breather membrane and external cladding.
- Infill 2 No. doorway access points within the south east elevation and any localised necessary repairs as further clarified below.
- Non-load bearing internal partitions to form internal 'rooms' as shown on the Architect's drawings.
- Windows and doors
- Floor insulation and screed finish
- Free standing first floor mezzanine structure as per the specification outline below.

<u>Repairs/ Extent of Remedial Works</u>

<u>Walls</u>

- Inspect all structural steelwork (including main portal frame), clean and treat with suitable rust prevention treatment (e.g. zinc rich primer, Galvafroid or similar equivalent). Any badly corroded steelwork to be cut out and repaired. (100% of steelwork to be treated, approximately 5% requires further attention, the remainder appears to be sound and capable of reuse).

Roof

- Internally, thoroughly clean existing roof 'truss' of arched roof and paint with zinc rich primer (e.g. Galvafroid). Top coat finish to be to Architect's specification. (Ensure compatibility with primer). (100% of roof truss to be treated and painted).

Floor

- Inspect existing concrete floor slab and allow for localised repairs using either small aggregate concrete RC20 mix or proprietary concrete repair compound depending on size of repair (100% to be inspected, 10% requires attention)

New Internal Works

- New non-load bearing timber stud partitions to be constructed using 50x100 C16 grade softwood timber studs @ 600mm c/c's (or equivalent lightweight cold formed metal stud system). Partitions adjacent to existing external walls to have breather membrane and 9mm ply or OSB sheeting to external face and be insulated internally to the Architect's specification. Partitions to be constructed onto a 'dolly' block course of dense concrete masonry off the new ground bearing slabs to ensure no timber below finished floor level. Note: Window and door openings to be introduced in accordance with the Architect's elevations.

- New flat ceiling to be constructed using 50x100 C16 joists @ 400mm c/c's with plasterboard and skim attached to the underside to be supported by new timber stud partitions.

- New mezzanine floor to be constructed using 50x225 C24 grade softwood timber joists @ 400mm c/c's supported by new timber stud partitions built of existing concrete floor slab.

- Services, e.g. electricity, plumbing, drainage as well as new architectural finishes to be coordinated as part of the overall conversion. No anticipated structural affect.

- All walls and ceilings to be insulated to the Architect's specification.

Appendix A



Barn A – External north west elevation



Barn A – external south east elevation



Barn A - Internal view of building showing part of main portal frame structure



Barn A - Internal view of building showing arched roof frame



Barn A – Typical Foundation

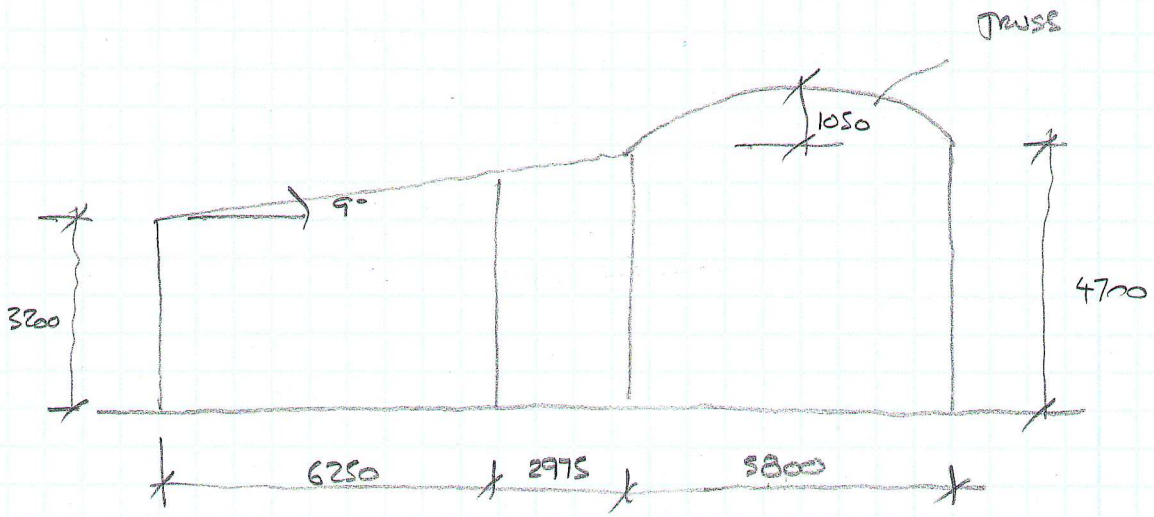
Appendix B – Existing Site Plan with Barn References



EXISTING SITE PLAN
SCALE 1:250 @ A1

Appendix C – Existing Structural Frame Analysis

Project	Sheet no./rev.
BARN A, CHESTNUT FARM, CHURCH LANE, DANBURY	RCC
FRAME ANALYSIS	By/Date PHM 13/5/21



Note: Analysis of existing frame uses loads from proposed conversion wall and roof finishes

LOADING.

DEAD Roof 0.2kN/m² × 4.5 = 0.9kN/m

LIVE (SNOW) Roof 0.6kN/m² × 4.5 = 2.7kN/m

WIND 0°

WINDWARD WALL 0.58kN/m² × 4.5 = 2.61 kN/m

LEEWARD WALL 0.01kN/m² × 4.5 = 0.05 kN/m

Roof 0.26kN/m² × 4.5 = 1.17 kN/m

WIND 180°

WINDWARD WALL 0.31kN/m² × 4.5 = 1.40 kN/m

LEEWARD WALL -0.35 kN/m² × 4.5 = -1.58 kN/m

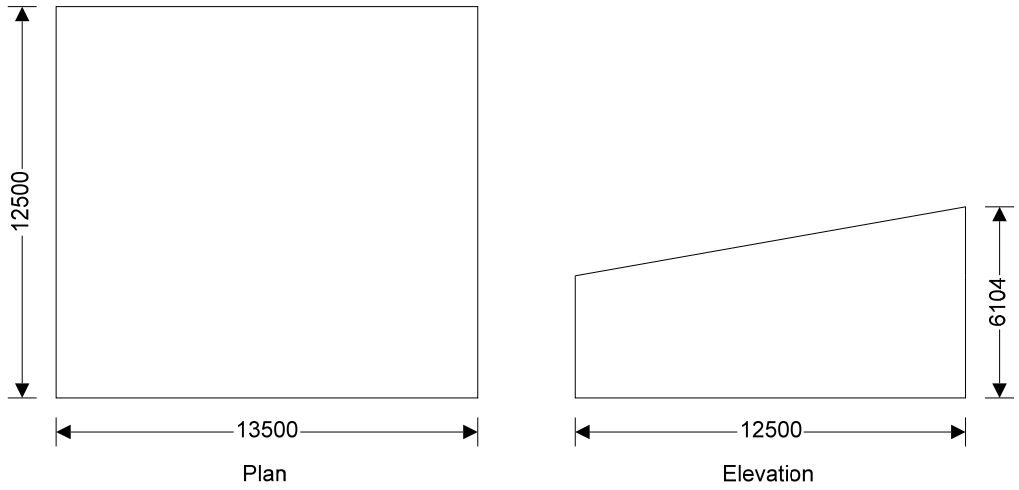
Roof -0.64 kN/m² × 4.5 = -2.88 kN/m

WIND LOADING (EN1991)

WIND LOADING

In accordance with EN1991-1-4:2005+A1:2010 and the UK national annex

Tedds calculation version 3.0.27



Building data

Type of roof	Monopitch
Length of building	L = **13500** mm
Width of building	W = **12500** mm
Height to low eaves	H = **3900** mm
Pitch of roof	α_0 = **10.0** deg
Total height	h = **6104** mm

Basic values

Location	Dauntsey
Wind speed velocity (FigureNA.1)	$v_{b,map}$ = **21.5** m/s
Distance to shore	L_{shore} = **50.00** km
Altitude above sea level	A_{alt} = **60.0**m
Altitude factor	$c_{alt} = A_{alt}/1m \times 0.001 + 1$ = **1.060**
Fundamental basic wind velocity	$v_{b,0} = v_{b,map} \times c_{alt}$ = **22.8** m/s
Direction factor	c_{dir} = **1.00**
Season factor	c_{season} = **1.00**
Shape parameter K	K = **0.2**
Exponent n	n = **0.5**
Air density	ρ = **1.226** kg/m^3
Probability factor	$c_{prob} = [(1 - K \times \ln(-\ln(1-p)))/(1 - K \times \ln(-\ln(0.98)))]^n$ = **1.00**
Basic wind velocity (Exp. 4.1)	$v_b = c_{dir} \times c_{season} \times v_{b,0} \times c_{prob}$ = **22.8** m/s
Reference mean velocity pressure	$q_b = 0.5 \times \rho \times v_b^2$ = **0.318** kN/m^2

Orography

Orography factor not significant c_o = 1.0

Terrain category Country

Displacement height (sheltering effect excluded) h_{dis} = 0mm

The velocity pressure for the windward face of the building with a 0 degree wind is to be considered as 1 part as the height h is less than b (cl.7.2.2)

The velocity pressure for the windward face of the building with a 90 degree wind is to be considered as 1 part as the height h is less than b (cl.7.2.2)

The velocity pressure for the windward face of the building with a 180 degree wind is to be considered as 1 part as the height h is less than b (cl.7.2.2)

Peak velocity pressure - windward wall - Wind 0 deg and roof

Reference height (at which q is sought) z = **3900**mm

Displacement height (sheltering effects excluded) h_{dis} = **0** mm

Exposure factor (Figure NA.7) c_e = **1.80**

Peak velocity pressure $q_p = c_e \times q_b$ = **0.57** kN/m^2

Structural factor

Structural damping δ_s = **0.050**

Height of element h_{part} = **3900** mm

Size factor (Table NA.3) c_s = **0.918**

Dynamic factor (Figure NA.9) c_d = **1.000**

Structural factor $c_{sCd} = c_s \times c_d$ = **0.918**

Peak velocity pressure - windward wall - Wind 90 deg and roof

Reference height (at which q is sought) z = **6104**mm

Displacement height (sheltering effects excluded) h_{dis} = **0** mm

Exposure factor (Figure NA.7) c_e = **2.06**

Peak velocity pressure $q_p = c_e \times q_b$ = **0.66** kN/m^2

Structural factor

Structural damping δ_s = **0.050**

Height of element h_{part} = **6104** mm

Size factor (Table NA.3) c_s = **0.914**

Dynamic factor (Figure NA.9) c_d = **1.000**

Structural factor $c_{sCd} = c_s \times c_d$ = **0.914**

Peak velocity pressure - windward wall - Wind 180 deg and roof

Reference height (at which q is sought) z = **6104**mm

Displacement height (sheltering effects excluded) h_{dis} = **0** mm

Exposure factor (Figure NA.7) c_e = **2.06**

Peak velocity pressure $q_p = c_e \times q_b$ = **0.66** kN/m^2

Structural factor

Structural damping δ_s = **0.050**

Height of element h_{part} = **6104** mm

Size factor (Table NA.3) c_s = **0.911**

Dynamic factor (Figure NA.9) c_d = **1.000**

Structural factor $c_{sCd} = c_s \times c_d$ = **0.911**

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.) $q_{p,i}$ = **0.66** kN/m^2

Pressures and forces

Net pressure $p = c_{sCd} \times q_p \times c_{pe} - q_{p,i} \times c_{pi}$

Net force $F_w = p_w \times A_{ref}$

Roof load case 1 - Wind 0, c_{pi} 0.20, -c_{pe}

Zone	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p, (kN/m^2)	Net pressure p (kN/m^2)	Area A_{ref} (m^2)	Net force F_w (kN)
F (-ve)	-1.45	0.66	-1.00	7.57	-7.55
G (-ve)	-1.00	0.66	-0.73	9.17	-6.69
H (-ve)	-0.50	0.66	-0.43	154.62	-66.52

Total vertical net force $F_{w,v}$ = **-79.53** kN

Total horizontal net force $F_{w,h}$ = **-14.02** kN

Walls load case 1 - Wind 0, c_{pi} 0.20, -c_{pe}

Zone	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p, (kN/m^2)	Net pressure p (kN/m^2)	Area A_{ref} (m^2)	Net force F_w (kN)
A	-1.20	0.66	-0.85	10.05	-8.58
B	-0.80	0.66	-0.61	50.70	-31.08
C	-0.50	0.66	-0.43	1.77	-0.77
D	0.73	0.57	0.25	52.65	13.31
E	-0.36	0.57	-0.32	82.41	-26.53

Overall loading

Equiv leeward net force for overall section $F_l = F_{w,wE}$ = **-26.5** kN

Net windward force for overall section $F_w = F_{w,wD}$ = **13.3** kN

Lack of correlation (cl.7.2.2(3) – Note) f_{corr} = **0.85** as h/W is 0.488

Overall loading overall section $F_{w,D} = f_{corr} \times (F_w - F_l + F_{w,h})$ = **21.9** kN

Roof load case 2 - Wind 0, c_{pi} -0.3, +c_{pe}

Zone	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p, (kN/m^2)	Net pressure p (kN/m^2)	Area A_{ref} (m^2)	Net force F_w (kN)
F (+ve)	0.10	0.66	0.26	7.57	1.94
G (+ve)	0.10	0.66	0.26	9.17	2.35

H (+ve)	0.10	0.66	0.26	154.62	39.68

Total vertical net force $F_{w,v}$ = **43.30** kN

Total horizontal net force $F_{w,h}$ = **7.64** kN

Walls load case 2 - Wind 0, c_{pi} -0.3, +c_{pe}

Zone	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p, (kN/m^2)	Net pressure p (kN/m^2)	Area A_{ref} (m^2)	Net force F_w (kN)
A	-1.20	0.66	-0.53	10.05	-5.28
B	-0.80	0.66	-0.28	50.70	-14.44
C	-0.50	0.66	-0.10	1.77	-0.18
D	0.73	0.57	0.58	52.65	30.58
E	-0.36	0.57	0.01	82.41	0.50

Overall loading

Equiv leeward net force for overall section $F_l = F_{w,wE}$ = **0.5** kN

Net windward force for overall section $F_w = F_{w,wD}$ = **30.6** kN

Lack of correlation (cl.7.2.2(3) – Note) f_{corr} = **0.85** as h/W is 0.488

Overall loading overall section $F_{w,D} = f_{corr} \times (F_w - F_l + F_{w,h})$ = **32.1** kN

Roof load case 3 - Wind 90, c_{pi} 0.20, -c_{pe}

Zone	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p, (kN/m^2)	Net pressure p (kN/m^2)	Area A_{ref} (m^2)	Net force F_w (kN)
FU (-ve)	-2.40	0.66	-1.57	3.78	-5.94
FL (-ve)	-1.85	0.66	-1.24	3.78	-4.70
G (-ve)	-1.10	0.66	-0.79	7.93	-6.27
H (-ve)	-0.75	0.66	-0.58	61.98	-36.02
I (-ve)	-0.75	0.66	-0.58	93.88	-54.56

Total vertical net force $F_{w,v}$ = **-105.86** kN

Total horizontal net force $F_{w,h}$ = **0.00** kN

Walls load case 3 - Wind 90, c_{pi} 0.20, -c_{pe}

Zone	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p, (kN/m^2)	Net pressure p (kN/m^2)	Area A_{ref} (m^2)	Net force F_w (kN)
A_U	-1.20	0.66	-0.85	14.90	-12.69
B_U	-0.80	0.66	-0.61	59.62	-36.43
C_U	-0.50	0.66	-0.43	7.89	-3.40
A_L	-1.20	0.57	-0.76	9.52	-7.22
B_L	-0.80	0.57	-0.55	38.09	-20.93
C_L	-0.50	0.57	-0.39	5.04	-1.98

D	0.73	0.66	0.30	62.53	19.06
E	-0.35	0.66	-0.34	62.53	-21.48

Overall loading

Equiv leeward net force for overall section $F_l = F_{w,wE}$ = **-21.5** kN

Net windward force for overall section $F_w = F_{w,wD}$ = **19.1** kN

Lack of correlation (cl.7.2.2(3) – Note) f_{corr} = **0.85** as h/L is 0.452

Overall loading overall section $F_{w,D} = f_{corr} \times (F_w - F_l + F_{w,h})$ = **34.5** kN

Roof load case 4 - Wind 90, c_{pi} -0.3, +c_{pe}

Zone	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p, (kN/m^2)	Net pressure p (kN/m^2)	Area A_{ref} (m^2)	Net force F_w (kN)
FU (+ve)	0.10	0.66	0.26	3.78	0.97
FL (+ve)	0.10	0.66	0.26	3.78	0.97
G (+ve)	0.10	0.66	0.26	7.93	2.04
H (+ve)	0.10	0.66	0.26	61.98	15.92
I (+ve)	0.10	0.66	0.26	93.88	24.11

Total vertical net force $F_{w,v}$ = **43.34** kN

Total horizontal net force $F_{w,h}$ = **0.00** kN

Walls load case 4 - Wind 90, c_{pi} -0.3, +c_{pe}

Zone	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p, (kN/m^2)	Net pressure p (kN/m^2)	Area A_{ref} (m^2)	Net force F_w (kN)
A_U	-1.20	0.66	-0.52	14.90	-7.80
B_U	-0.80	0.66	-0.28	59.62	-16.88
C_U	-0.50	0.66	-0.10	7.89	-0.81
A_L	-1.20	0.57	-0.43	9.52	-4.10
B_L	-0.80	0.57	-0.22	38.09	-8.44
C_L	-0.50	0.57	-0.06	5.04	-0.33
D	0.73	0.66	0.63	62.53	39.57
E	-0.35	0.66	-0.02	62.53	-0.97

Overall loading

Equiv leeward net force for overall section $F_l = F_{w,wE}$ = **-1.0** kN

Net windward force for overall section $F_w = F_{w,wD}$ = **39.6** kN

Lack of correlation (cl.7.2.2(3) – Note) f_{corr} = **0.85** as h/L is 0.452

Overall loading overall section $F_{w,D} = f_{corr} \times (F_w - F_l + F_{w,h})$ = **34.5** kN

Roof load case 5 - Wind 180, c_{pi} 0.20, - c_{pe}

Zone	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p, (kN/m^2)	Net pressure p (kN/m^2)	Area A_{ref} (m^2)	Net force F_w (kN)
F (-ve)	-2.50	0.66	-1.63	7.57	-12.30
G (-ve)	-1.05	0.66	-0.76	9.17	-6.96
H (-ve)	-0.85	0.66	-0.64	154.62	-98.88

Total vertical net force $F_{w,v}$ = **-116.34** kN

Total horizontal net force $F_{w,h}$ = **-20.51** kN

Walls load case 5 - Wind 180, c_{pi} 0.20, - c_{pe}

Zone	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p, (kN/m^2)	Net pressure p (kN/m^2)	Area A_{ref} (m^2)	Net force F_w (kN)
A	-1.20	0.66	-0.85	14.38	-12.20
B	-0.80	0.66	-0.61	63.82	-38.90
C	-0.50	0.66	-0.43	1.65	-0.71
D	0.73	0.66	0.31	82.41	25.25
E	-0.36	0.66	-0.35	52.65	-18.35

Overall loading

Equiv leeward net force for overall section $F_l = F_{w,wE}$ = **-18.4** kN

Net windward force for overall section $F_w = F_{w,wD}$ = **25.2** kN

Lack of correlation (cl.7.2.2(3) – Note) f_{corr} = **0.85** as h/L is 0.452

Overall loading overall section $F_{w,D} = f_{corr} \times (F_w - F_l - F_{w,h})$ = **54.5** kN

Roof load case 6 - Wind 180, c_{pi} -0.30, - c_{pe}

Zone	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p, (kN/m^2)	Net pressure p (kN/m^2)	Area A_{ref} (m^2)	Net force F_w (kN)
F (-ve)	-2.50	0.66	-1.30	7.57	-9.82
G (-ve)	-1.05	0.66	-0.43	9.17	-3.95
H (-ve)	-0.85	0.66	-0.31	154.62	-48.16

Total vertical net force $F_{w,v}$ = **-60.99** kN

Total horizontal net force $F_{w,h}$ = **-10.75** kN

Walls load case 6 - Wind 180, c_{pi} -0.30, - c_{pe}

Zone	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p, (kN/m^2)	Net pressure p (kN/m^2)	Area A_{ref} (m^2)	Net force F_w (kN)
A	-1.20	0.66	-0.52	14.38	-7.49
B	-0.80	0.66	-0.28	63.82	-17.97
C	-0.50	0.66	-0.10	1.65	-0.17

D	0.73	0.66	0.63	82.41	52.28
E	-0.36	0.66	-0.02	52.65	-1.08

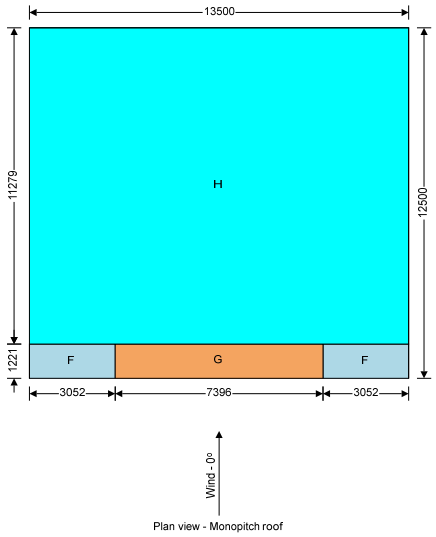
Overall loading

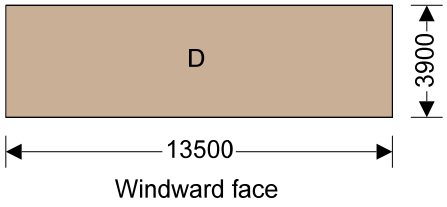
Equiv leeward net force for overall section $F_l = F_{w,wE}$ = **-1.1** kN

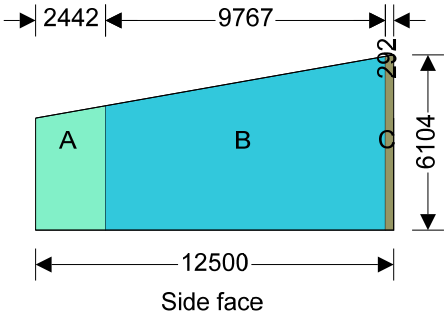
Net windward force for overall section $F_w = F_{w,wD}$ = **52.3** kN

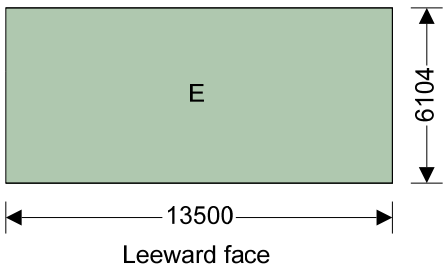
Lack of correlation (cl.7.2.2(3) – Note) f_{corr} = **0.85** as h/L is 0.452

Overall loading overall section $F_{w,D} = f_{corr} \times (F_w - F_l - F_{w,h})$ = **54.5** kN

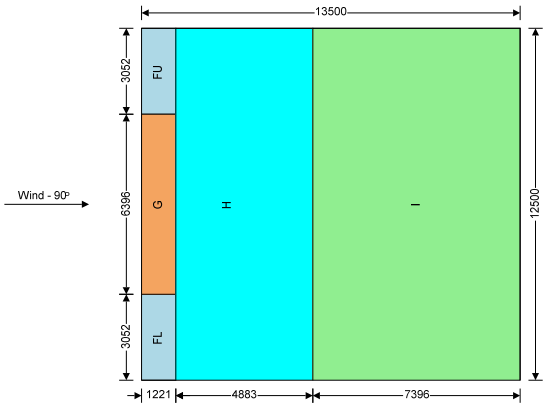




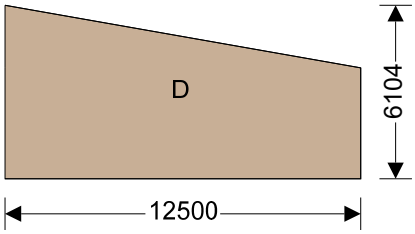




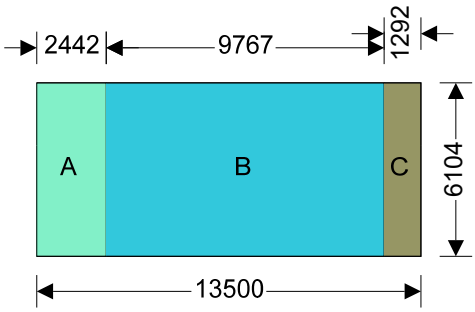
Project	Sheet no./rev.
Chestnut Farm, Church Lane, Dauntsey	RCC /
Barns A – Wind Load	By/Date MM / 12/05/21



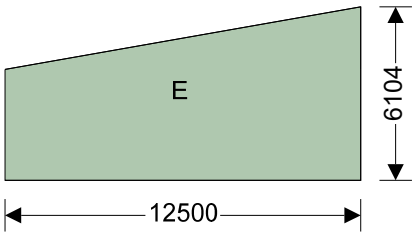
Plan view - Monopitch roof



Windward face

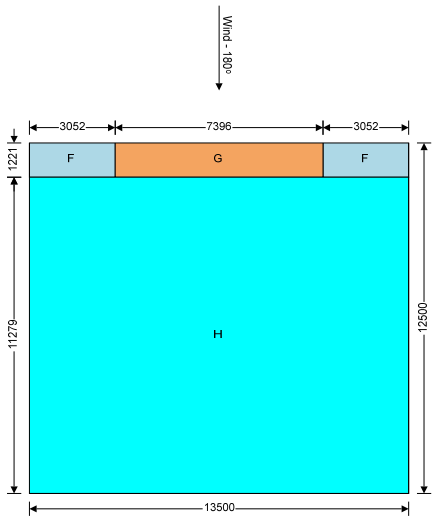


Side face

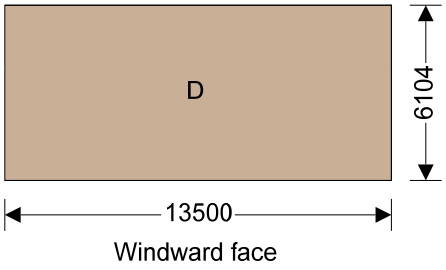


Leeward face

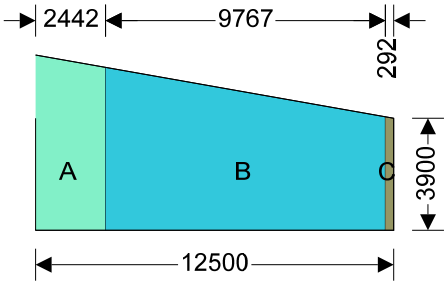
Project	Sheet no./rev.
Chestnut Farm, Church Lane, Dauntsey	RCC /
Barns A – Wind Load	By/Date MM / 12/05/21



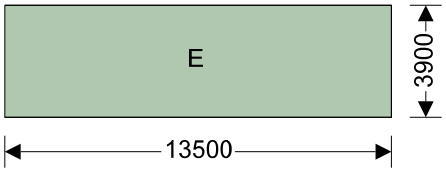
Plan view - Monopitch roof



Windward face



Side face



Leeward face

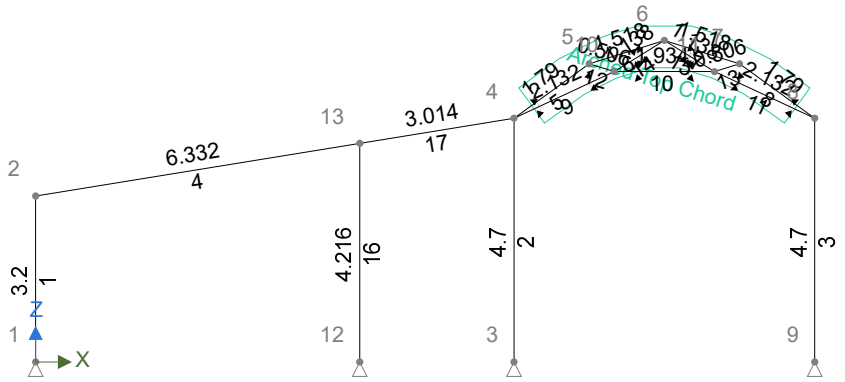
2D ANALYSIS

ANALYSIS

Tedds calculation version 1.0.36

Geometry

Geometry (m) - Steel (EC3)



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 parallel to Minor (cm²)	Shear area parallel to Major (cm²)
UB 178x102x19	24.3	1356	136.7	8.5	14.4
RSA 100x100x8	15.6	146.4	146.4	0	0
RSA 60x60x5	5.9	19.8	19.8	0	0
Dorman Long Rolled Beams 1887 6x3x15	28.8	1072.6	82.4	0	0
CHS 60.3x4.0	7.1	28.2	28.2	3.5	3.5

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	3.2	Free	Free	Free		0	0	0	0
3	9.225	0	Fixed	Fixed	Free		0	0	0	0
4	9.225	4.7	Free	Free	Free		0	0	0	0
5	10.675	5.75	Free	Free	Free		0	0	0	0
6	12.125	6.2	Free	Free	Free		0	0	0	0
7	13.575	5.75	Free	Free	Free		0	0	0	0

Node	Co-ordinates		Freedom			Coordinate system		Spring		
	X (m)	Z (m)	X	Z	Rot.	Name	Angle (°)	X (kN/m)	Z (kN/m)	Rot. kNm/°
8	15.025	4.7	Free	Free	Free		0	0	0	0
9	15.025	0	Fixed	Fixed	Free		0	0	0	0
10	11.158	5.6	Free	Free	Free		0	0	0	0
11	13.092	5.6	Free	Free	Free		0	0	0	0
12	6.25	0	Fixed	Fixed	Free		0	0	0	0
13	6.25	4.216	Free	Free	Free		0	0	0	0

Elements

Element	Length (m)	Nodes		Section	Material	Releases			Rotated
		Start	End			Start moment	End moment	Axial	
1	3.2	1	2	UB 178x102x19	Steel (EC3)	Fixed	Fixed	Fixed	
2	4.7	3	4	Dorman Long Rolled Beams 1887 6x3x15	Steel (EC3)	Fixed	Fixed	Fixed	
3	4.7	9	8	Dorman Long Rolled Beams 1887 6x3x15	Steel (EC3)	Fixed	Fixed	Fixed	
4	6.332	2	13	UB 178x102x19	Steel (EC3)	Fixed	Fixed	Fixed	
5	1.79	4	5	CHS 60.3x4.0	Steel (EC3)	Fixed	Fixed	Fixed	
6	1.518	5	6	CHS 60.3x4.0	Steel (EC3)	Fixed	Fixed	Fixed	
7	1.518	6	7	CHS 60.3x4.0	Steel (EC3)	Fixed	Fixed	Fixed	
8	1.79	7	8	CHS 60.3x4.0	Steel (EC3)	Fixed	Fixed	Fixed	
9	2.132	4	10	CHS 60.3x4.0	Steel (EC3)	Free	Free	Fixed	
10	1.934	10	11	CHS 60.3x4.0	Steel (EC3)	Free	Free	Fixed	
11	2.132	11	8	CHS 60.3x4.0	Steel (EC3)	Free	Free	Fixed	
12	0.506	5	10	CHS 60.3x4.0	Steel (EC3)	Free	Fixed	Fixed	
13	0.506	7	11	CHS 60.3x4.0	Steel (EC3)	Free	Fixed	Fixed	
14	1.138	6	10	CHS 60.3x4.0	Steel (EC3)	Free	Free	Fixed	
15	1.138	6	11	CHS 60.3x4.0	Steel (EC3)	Free	Free	Fixed	
16	4.216	12	13	UB 178x102x19	Steel (EC3)	Fixed	Fixed	Fixed	
17	3.014	13	4	UB 178x102x19	Steel (EC3)	Fixed	Fixed	Fixed	

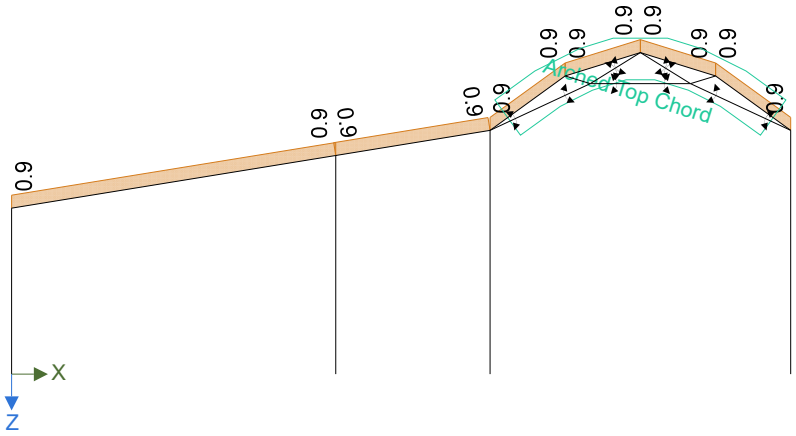
Members

Name	Elements	
	Start	End
Arched Top Chord	5	8

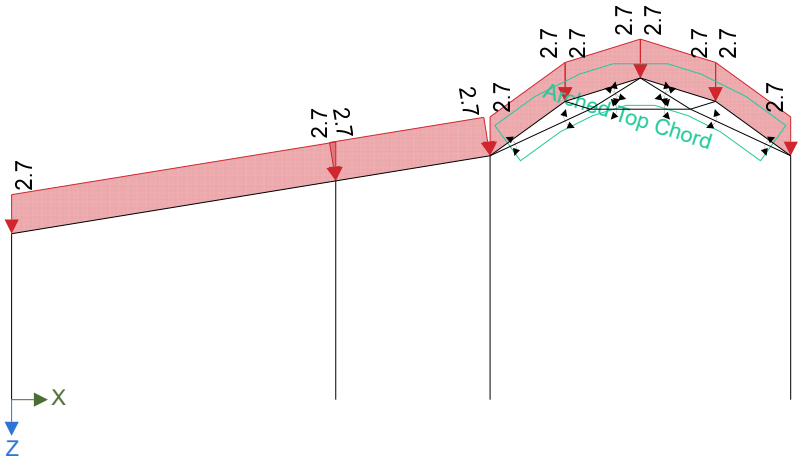
Loading

Self weight included

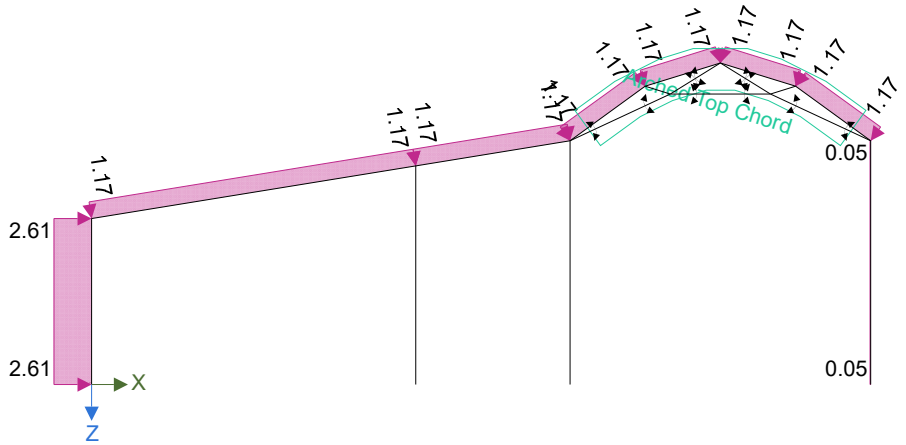
Permanent - Loading (kN/m)



Snow - Loading (kN/m)



Wind - Loading (kN/m)



Load combination factors

Load combination	Self Weight	Permanent	Snow	Wind
1.35G + 1.5Q + 1.5RQ (Strength)	1.35	1.35		
1.0G + 1.0Q + 1.0RQ (Service)	1.00	1.00		
1.0G + 1.0ψ_2Q (Quasi)	1.00	1.00		
1.35G + 1.5Q + 1.5ψ_0S (Strength)	1.35	1.35	0.75	
1.0G + 1.0Q + 0.5S (Service)	1.00	1.00	0.50	
1.35G + 1.5ψ_0Q + 1.5S (Strength)	1.35	1.35	1.50	
1.35G + 1.5Q + 1.5ψ_0S + 1.5ψ_0W (Strength)	1.35	1.35	0.75	0.75
1.0G + 1.0Q + 0.5S + 0.5W (Service)	1.00	1.00	0.50	0.50
1.35G + 1.5ψ_0Q + 1.5S + 1.5ψ_0W (Strength)	1.35	1.35	1.50	0.75
1.0G + 1.0ψ_0Q + 1.0S + 0.5W (Service)	1.00	1.00	1.00	0.50
1.35G + 1.5ψ_0Q + 1.5ψ_0S + 1.5W (Strength)	1.35	1.35	0.75	1.50
1.0G + 1.0ψ_0Q + 0.5S + 1.0W (Service)	1.00	1.00	0.50	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		
1.35G + 1.5ψ_0Q + 1.5ψ_0S (Strength)	1.35	1.35	0.75	
1.35ξG + 1.5Q + 1.5RQ (Strength)	1.25	1.25		
1.35ξG + 1.5Q + 1.5ψ_0S (Strength)	1.25	1.25	0.75	
1.35ξG + 1.5ψ_0Q + 1.5S (Strength)	1.25	1.25	1.50	
1.35G + 1.5ψ_0Q + 1.5ψ_0S + 1.5ψ_0W (Strength)	1.35	1.35	0.75	0.75
1.35ξG + 1.5Q + 1.5ψ_0S + 1.5ψ_0W (Strength)	1.25	1.25	0.75	0.75
1.35ξG + 1.5ψ_0Q + 1.5S + 1.5ψ_0W (Strength)	1.25	1.25	1.50	0.75
1.35ξG + 1.5ψ_0Q + 1.5ψ_0S + 1.5W (Strength)	1.25	1.25	0.75	1.50

Member Loads

Member	Load case	Load Type	Orientation	Description
Arched Top Chord	Permanent	UDL	GlobalZ	0.9 kN/m
Arched Top Chord	Snow	UDL	GlobalZ	2.7 kN/m
Arched Top Chord	Wind	UDL	LocalZ	1.17 kN/m

Element Loads

Element	Load case	Load Type	Orientation	Description
4	Permanent	UDL	GlobalZ	0.9 kN/m
17	Permanent	UDL	LocalZ	0.9 kN/m
4	Snow	UDL	GlobalZ	2.7 kN/m
17	Snow	UDL	LocalZ	2.7 kN/m
1	Wind	UDL	GlobalX	2.61 kN/m
3	Wind	UDL	GlobalX	0.05 kN/m
4	Wind	UDL	LocalZ	1.17 kN/m
17	Wind	UDL	LocalZ	1.17 kN/m

Results

Total base reactions

Load case/combination	Force	
	FX (kN)	FZ (kN)
Self Weight	0	6.1
Permanent	-0.4	14.3
Snow	-1.3	43
Wind	-10.3	17.6

Reactions

Load case: Self Weight

Node	Force		Moment
	Fx (kN)	Fz (kN)	My (kNm)
1	0.1	1.1	0
3	0	1.7	0
9	0	1.5	0
12	-0.1	1.8	0

Load case: Permanent

Node	Force		Moment
	Fx (kN)	Fz (kN)	My (kNm)
1	0.3	2.5	0
3	-0.1	4.1	0
9	-0.1	3	0
12	-0.5	4.7	0

Load case: Snow

Node	Force		Moment
	Fx (kN)	Fz (kN)	My (kNm)
1	0.9	7.4	0
3	-0.4	12.4	0
9	-0.3	9	0
12	-1.5	14.2	0

Load case: Wind

Node	Force		Moment
	Fx (kN)	Fz (kN)	My (kNm)
1	-5.9	1.9	0
3	-1.3	8	0
9	-0.3	3.5	0
12	-2.8	4.2	0

Element end forces

Load case: Self Weight

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	3.2	1	-1.1	0.1	0
		2	0.5	-0.1	-0.3
2	4.7	3	-1.7	0	0
		4	0.6	0	0.1
3	4.7	9	-1.5	0	0
		8	0.4	0	0.1
4	6.332	2	-0.2	-0.5	0.3
		13	0	-0.6	-0.7
5	1.79	4	-1.4	-0.1	0
		5	1.4	0	0
6	1.518	5	-1.9	0	0
		6	1.9	-0.1	0
7	1.518	6	-1.8	-0.1	0
		7	1.9	0	0
8	1.79	7	-1.4	0	0
		8	1.4	-0.1	-0.1
9	2.132	4	1.2	-0.1	0
		10	-1.3	-0.1	0
10	1.934	10	1	-0.1	0
		11	-1	-0.1	0
11	2.132	11	1.2	-0.1	0
		8	-1.2	-0.1	0
12	0.506	5	0.7	0	0
		10	-0.7	0	0
13	0.506	7	0.7	0	0
		11	-0.7	0	0
14	1.138	6	0.9	0	0
		10	-0.9	0	0
15	1.138	6	0.9	0	0
		11	-0.9	0	0
16	4.216	12	-1.8	-0.1	0
		13	1	0.1	0.3
17	3.014	13	-0.1	-0.4	0.4
		4	0	-0.2	-0.1

Load case: Permanent

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	3.2	1	-2.5	0.3	0
		2	2.5	-0.3	-0.9
2	4.7	3	-4.1	-0.1	0
		4	4.1	0.1	0.6

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
3	4.7	9	-3	-0.1	0
		8	3	0.1	0.5
4	6.332	2	-0.7	-2.4	0.9
		13	-0.2	-3.2	-3.5
5	1.79	4	-9.7	-0.8	0.3
		5	8.8	-0.5	-0.1
6	1.518	5	-11.1	-0.6	0.1
		6	10.7	-0.7	-0.2
7	1.518	6	-10.5	-0.8	0.2
		7	10.9	-0.6	0
8	1.79	7	-8.6	-0.4	0
		8	9.5	-0.9	-0.5
9	2.132	4	8.1	0	0
		10	-8.1	0	0
10	1.934	10	6.3	0	0
		11	-6.3	0	0
11	2.132	11	7.8	0	0
		8	-7.8	0	0
12	0.506	5	3.1	0	0
		10	-3.1	0	0
13	0.506	7	3.2	0	0
		11	-3.2	0	0
14	1.138	6	4.7	0	0
		10	-4.7	0	0
15	1.138	6	4.5	0	0
		11	-4.5	0	0
16	4.216	12	-4.7	-0.5	0
		13	4.7	0.5	2.1
17	3.014	13	0	-1.5	1.4
		4	0	-1.2	-1

Load case: Snow

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	3.2	1	-7.4	0.9	0
		2	7.4	-0.9	-2.8
2	4.7	3	-12.4	-0.4	0
		4	12.4	0.4	1.8
3	4.7	9	-9	-0.3	0
		8	9	0.3	1.4
4	6.332	2	-2.1	-7.2	2.8
		13	-0.7	-9.7	-10.6
5	1.79	4	-29.2	-2.4	1
		5	26.4	-1.5	-0.2
6	1.518	5	-33.2	-1.7	0.2

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
		6	32	-2.2	-0.6
7	1.518	6	-31.4	-2.3	0.6
		7	32.6	-1.7	-0.1
8	1.79	7	-25.7	-1.3	0.1
		8	28.6	-2.7	-1.4
9	2.132	4	24.2	0	0
		10	-24.2	0	0
10	1.934	10	18.9	0	0
		11	-18.9	0	0
11	2.132	11	23.5	0	0
		8	-23.5	0	0
12	0.506	5	9.4	0	0
		10	-9.4	0	0
13	0.506	7	9.5	0	0
		11	-9.5	0	0
14	1.138	6	14.1	0	0
		10	-14.1	0	0
15	1.138	6	13.4	0	0
		11	-13.4	0	0
16	4.216	12	-14.2	-1.5	0
		13	14.2	1.5	6.3
17	3.014	13	-0.1	-4.5	4.3
		4	0.1	-3.6	-2.9

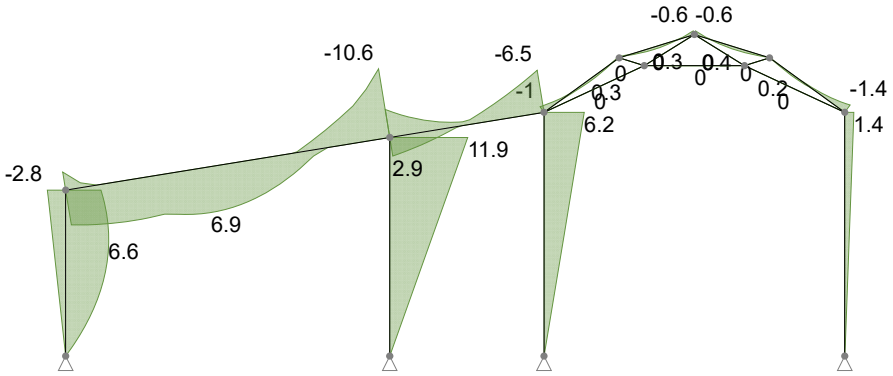
Load case: Wind

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	3.2	1	-1.9	-5.9	0
		2	1.9	-2.5	5.4
2	4.7	3	-8	-1.3	0
		4	8	1.3	6.2
3	4.7	9	-3.5	-0.3	0
		8	3.5	0.1	1.1
4	6.332	2	-2.8	-1.4	-5.4
		13	2.8	-6	-9
5	1.79	4	-9.5	-1.1	0.3
		5	9.5	-1	-0.2
6	1.518	5	-11.2	-0.8	0.2
		6	11.2	-0.9	-0.3
7	1.518	6	-9.8	-1.1	0.3
		7	9.8	-0.7	0
8	1.79	7	-8	-0.4	0
		8	8	-1.6	-1.1
9	2.132	4	7.6	0	0
		10	-7.6	0	0

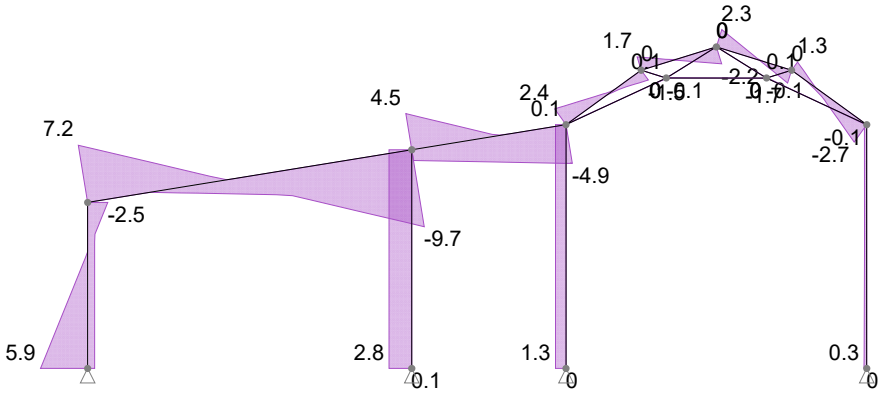
Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
10	1.934	10	5	0	0
		11	-5	0	0
11	2.132	11	6	0	0
		8	-6	0	0
12	0.506	5	2.3	0	0
		10	-2.3	0	0
13	0.506	7	2.5	0	0
		11	-2.5	0	0
14	1.138	6	4.8	0	0
		10	-4.8	0	0
15	1.138	6	3.4	0	0
		11	-3.4	0	0
16	4.216	12	-4.2	-2.8	0
		13	4.2	2.8	11.9
17	3.014	13	-0.6	1.3	-2.9
		4	0.6	-4.9	-6.5

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	0	5.9	2.245	6.6	3.2	-2.8
2	0	1.3	4.7	6.2	0	0
3	0	0.3	4.7	1.4	0	0
4	6.332	-9.7 (max abs)	2.702	6.9	6.332	-10.6 (min)
5	0	2.4	1.103	0.3	0	-1
6	1.518	-2.2	0.667	0.3	1.518	-0.6
7	0	2.3	0.874	0.4	0	-0.6
8	1.79	-2.7	0.576	0.2	1.79	-1.4
9	0	0.1	1.066	0	0	0
10	1.934	-0.1	0.967	0	1.934	0
11	0	0.1	1.066	0	0	0
12	0	0	0.253	0	0	0
13	0.506	0	0.253	0	0.506	0
14	1.138	0	0.569	0	1.138	0
15	1.138	0	0.569	0	1.138	0
16	0	2.8	4.216	11.9 (max)	0	0
17	3.014	-4.9	0	2.9	3.014	-6.5

STEEL MEMBER DESIGN (EN1993)

STEEL MEMBER 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 4.4.08

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**

Design section 1

Section details

Section type UB 178x102x19 (BS4-1)

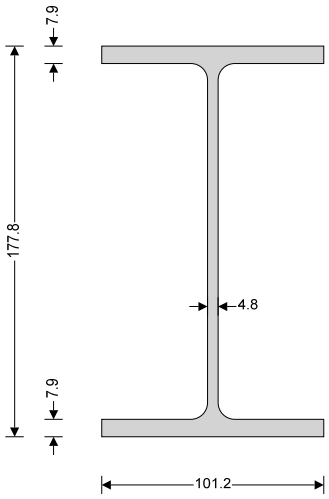
Steel grade - EN 10025-2:2004 S275

Nominal thickness of element $t_{nom} = \max(t_f, t_w)$ = **7.9** 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 178x102x19 (BS4-1)
Section depth, h, 177.8 mm
Section breadth, b, 101.2 mm
Mass of section, Mass, 19 kg/m
Flange thickness, t_f, 7.9 mm
Web thickness, t_w, 4.8 mm
Root radius, r, 7.6 mm
Area of section, A, 2426 mm^2
Radius of gyration about y-axis, i_y, 74.76 mm
Radius of gyration about z-axis, i_z, 23.738 mm
Elastic section modulus about y-axis, $W_{el,y}$, 152528 mm^3
Elastic section modulus about z-axis, $W_{el,z}$, 27017 mm^3
Plastic section modulus about y-axis, $W_{pl,y}$, 171256 mm^3
Plastic section modulus about z-axis, $W_{pl,z}$, 41590 mm^3
Second moment of area about y-axis, I_y, 13559739 mm^4
Second moment of area about z-axis, I_z, 1367064 mm^4

Analysis results

Design bending moment - Major axis $M_{y,Ed}$ = **11.9** kNm

Design shear force - Major axis $V_{y,Ed}$ = **9.7** kN

Restraint spacing

Major axis lateral restraint L_y = **6250** mm

Minor axis lateral restraint L_z = **6250** mm

Torsional restraint L_T = **6250** mm

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 $c = d$ = **146.8** mm

$c / t_w = 30.6 = 33.1 \times \varepsilon <= 72 \times \varepsilon$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

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

$c / t_f = 5.1 = 5.6 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f$ = **162** mm η = **1.000**

$h_w / t_w = 33.8 = 36.5 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force $V_{y,Ed}$ = **9.7** kN

Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **985** mm^2

Design shear resistance - cl 6.2.6(2) $V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **156.4** kN

$V_{y,Ed} / V_{c,y,Rd}$ = **0.062**

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment $M_{y,Ed}$ = **11.9** kNm

Design bending resistance moment - eq 6.13 $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **47.1** kNm

$M_{y,Ed} / M_{c,y,Rd}$ = **0.253**

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - User defined k_c = **1**

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

Poissons ratio ν = **0.3**

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

Unrestrained effective length $L = 1.0 \times L_{z_s1}$ = **6250** mm

Elastic critical buckling moment $M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **17.2** kNm

Slenderness ratio for lateral torsional buckling $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **1.654**

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 $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **1.739**

LTB reduction factor - eq 6.57 $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.365**

Modification factor $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **1.000**

Modified LTB reduction factor - eq 6.58 $\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.365**

Design buckling resistance moment - eq 6.55 $M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1}$ = **17.2** kNm

$M_{y,Ed} / M_{b,y,Rd}$ = **0.691**

PASS - Design buckling resistance moment exceeds design bending moment

STEEL MEMBER DESIGN (EN1993)

STEEL MEMBER 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 4.4.08

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**

Design section 1

Section details

Section type UB 152x89x16 (BS4-1) (Equivalent size)

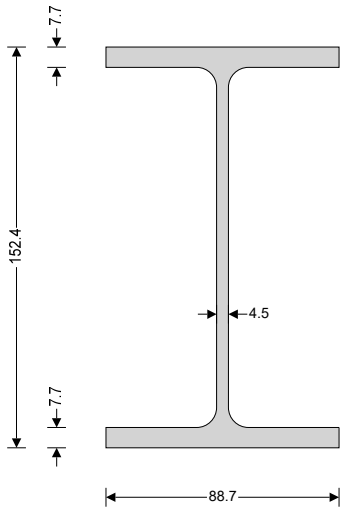
Steel grade - EN 10025-2:2004 S275

Nominal thickness of element $t_{nom} = \max(t_f, t_w)$ = **7.7** 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 152x89x16 (BS4-1)
Section depth, h, 152.4 mm
Section breadth, b, 88.7 mm
Mass of section, Mass, 16 kg/m
Flange thickness, t_f, 7.7 mm
Web thickness, t_w, 4.5 mm
Root radius, r, 7.6 mm
Area of section, A, 2032 mm^2
Radius of gyration about y-axis, i_y, 64.074 mm
Radius of gyration about z-axis, i_z, 21.016 mm
Elastic section modulus about y-axis, $W_{el,y}$, 109483 mm^3
Elastic section modulus about z-axis, $W_{el,z}$, 20237 mm^3
Plastic section modulus about y-axis, $W_{pl,y}$, 123256 mm^3
Plastic section modulus about z-axis, $W_{pl,z}$, 31180 mm^3
Second moment of area about y-axis, I_y, 8342621 mm^4
Second moment of area about z-axis, I_z, 897506 mm^4

Analysis results

Design bending moment - Major axis $M_{y,Ed}$ = **6.2** kNm

Design shear force - Major axis $V_{y,Ed}$ = **1.3** kN

Restraint spacing

Major axis lateral restraint L_y = **4700** mm

Minor axis lateral restraint L_z = **4700** mm

Torsional restraint L_T = **4700** mm

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 $c = d$ = **121.8** mm

$c / t_w = 27.1 = 29.3 \times \varepsilon <= 72 \times \varepsilon$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section $c = (b - t_w - 2 \times r) / 2$ = **34.5** mm

$c / t_f = 4.5 = 4.8 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f$ = **137** mm η = **1.000**

$h_w / t_w = 30.4 = 32.9 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force $V_{y,Ed}$ = **1.3** kN

Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **818** mm^2

Design shear resistance - cl 6.2.6(2) $V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **129.8** kN

$V_{y,Ed} / V_{c,y,Rd}$ = **0.01**

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment $M_{y,Ed}$ = **6.2** kNm

Design bending resistance moment - eq 6.13 $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **33.9** kNm

$M_{y,Ed} / M_{c,y,Rd}$ = **0.183**

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - User defined k_c = **1**

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

Poissons ratio ν = **0.3**

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

Unrestrained effective length $L = 1.0 \times L_{z_s1}$ = **4700** mm

Elastic critical buckling moment $M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **16.7** kNm

Slenderness ratio for lateral torsional buckling $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **1.424**

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 $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **1.435**

LTB reduction factor - eq 6.57 $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.461**

Modification factor $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **1.000**

Modified LTB reduction factor - eq 6.58 $\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.461**

Design buckling resistance moment - eq 6.55 $M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1}$ = **15.6** kNm

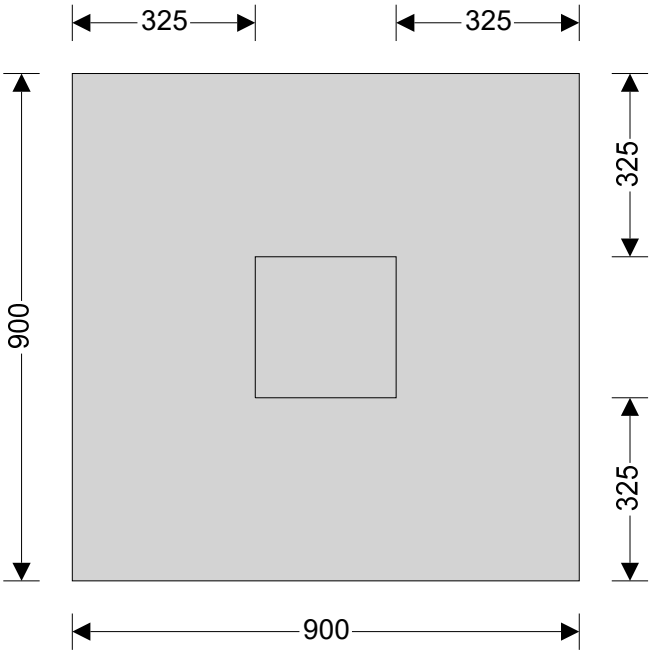
$M_{y,Ed} / M_{b,y,Rd}$ = **0.396**

PASS - Design buckling resistance moment exceeds design bending moment

PAD FOOTING ANALYSIS & DESIGN (BS8110)

PAD FOOTING ANALYSIS AND DESIGN (BS8110-1:1997)

Tedds calculation version 2.0.07



Pad footing details

Length of pad footing L = **900** mm

Width of pad footing B = **900** mm

Area of pad footing $A = L \times B =$ **0.810** m^2

Depth of pad footing h = **500** mm

Depth of soil over pad footing h_{soil} = **400** mm

Density of concrete ρ_{conc} = **23.6** kN/m^3

Column details

Column base length l_A = **250** mm

Column base width b_A = **250** mm

Column eccentricity in x e_{PxA} = **0** mm

Column eccentricity in y e_{PyA} = **0** mm

Soil details

Density of soil ρ_{soil} = **20.0** kN/m^3

Design shear strength ϕ' = **25.0** deg

Design base friction δ = **19.3** deg

Allowable bearing pressure $P_{bearing}$ = **100** kN/m^2

Axial loading on column

Dead axial load on column P_{GA} = **8.0** kN

Imposed axial load on column P_{QA} = **17.8** kN

Wind axial load on column P_{WA} = **9.4** kN

Total axial load on column P_A = **35.2** kN

Foundation loads

Dead surcharge load F_{Gsur} = **0.000** kN/m^2

Imposed surcharge load F_{Qsur} = **2.500** kN/m^2

Pad footing self weight $F_{swt} = h \times \rho_{conc}$ = **11.800** kN/m^2

Soil self weight $F_{soil} = h_{soil} \times \rho_{soil}$ = **8.000** kN/m^2

Total foundation load $F = A \times (F_{Gsur} + F_{Qsur} + F_{swt} + F_{soil})$ = **18.1** kN

Calculate pad base reaction

Total base reaction $T = F + P_A$ = **53.3** kN

Eccentricity of base reaction in x $e_{Tx} = (P_A \times e_{PxA} + M_{xA} + H_{xA} \times h) / T$ = **0** mm

Eccentricity of base reaction in y $e_{Ty} = (P_A \times e_{PyA} + M_{yA} + H_{yA} \times h) / T$ = **0** mm

Check pad base reaction eccentricity

$abs(e_{Tx}) / L + abs(e_{Ty}) / B$ = **0.000**

Base reaction acts within middle third of base

Calculate pad base pressures

$q_1 = T / A - 6 \times T \times e_{Tx} / (L \times A) - 6 \times T \times e_{Ty} / (B \times A)$ = **65.757** kN/m^2

$q_2 = T / A - 6 \times T \times e_{Tx} / (L \times A) + 6 \times T \times e_{Ty} / (B \times A)$ = **65.757** kN/m^2

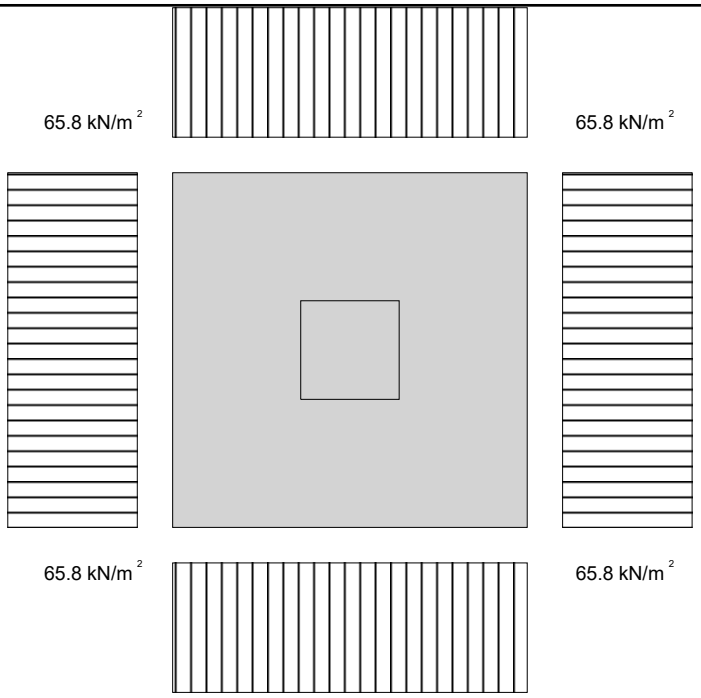
$q_3 = T / A + 6 \times T \times e_{Tx} / (L \times A) - 6 \times T \times e_{Ty} / (B \times A)$ = **65.757** kN/m^2

$q_4 = T / A + 6 \times T \times e_{Tx} / (L \times A) + 6 \times T \times e_{Ty} / (B \times A)$ = **65.757** kN/m^2

Minimum base pressure $q_{min} = min(q_1, q_2, q_3, q_4)$ = **65.757** kN/m^2

Maximum base pressure $q_{max} = max(q_1, q_2, q_3, q_4)$ = **65.757** kN/m^2

PASS - Maximum base pressure is less than allowable bearing pressure



Material details

Characteristic strength of concrete f_{cu} = **25** N/mm^2

Calculate pad lengths in x direction

Left hand length $L_L = L / 2 + e_{PxA}$ = **450** mm

Right hand length $L_R = L / 2 - e_{PxA}$ = **450** mm

Calculate rate of change of base pressure in x direction

Length of base reaction $L_x = L$ = **900** mm

Rate of change of base pressure $C_x = [(q_1 + q_2) - (q_3 + q_4)] / (2 \times L_x)$ = **0.000** kN/m^2/m

Calculate pad lengths in y direction

Top length $L_T = B / 2 - e_{PyA}$ = **450** mm

Bottom length $L_B = B / 2 + e_{PyA}$ = **450** mm

Calculate rate of change of base pressure in y direction

Length of base reaction $L_y = B$ = **900** mm

Rate of change of base pressure $C_y = [(q_1 + q_3) - (q_2 + q_4)] / (2 \times L_y)$ = **0.000** kN/m^2/m

Calculate minimum depth of unreinforced pad footing

Average pressure to left of pad footing $q_L = (q_1 + q_2) / 2 - C_x \times (L_L - l_A / 2) / 2$ = **65.757** kN/m^2

Minimum depth to left of pad footing $h_{Lmin} = (L_L - l_A/2) \times \max(0.15 \times [(q_L/1\text{kN/m}^2)^2 / (f_{cu}/1\text{N/mm}^2)]^{1/4}, 1)$ = **325** mm

Average pressure to right of pad footing $q_R = (q_3 + q_4) / 2 - C_x \times (L_R - l_A / 2) / 2$ = **65.757** kN/m^2

Minimum depth to right of pad footing $h_{Rmin} = (L_R - l_A/2) \times \max(0.15 \times [(q_R/1kN/m^2)^2 / (f_{cu}/1N/mm^2)]^{1/4}, 1) =$ **325** mm

Average pressure to top of pad footing $q_T = (q_2 + q_4) / 2 - C_y \times (L_T - b_A / 2) / 2 =$ **65.757** kN/m^2

Minimum depth to top of pad footing $h_{Tmin} = (L_T - b_A/2) \times \max(0.15 \times [(q_T/1kN/m^2)^2 / (f_{cu}/1N/mm^2)]^{1/4}, 1) =$ **325** mm

Average pressure to bottom of pad footing $q_B = (q_1 + q_3) / 2 - C_y \times (L_B - b_A / 2) / 2 =$ **65.757** kN/m^2

Minimum depth to bottom of pad footing $h_{Bmin} = (L_B - b_A/2) \times \max(0.15 \times [(q_B/1kN/m^2)^2 / (f_{cu}/1N/mm^2)]^{1/4}, 1) =$ **325** mm

Minimum depth of unreinforced pad footing $h_{min} = \max(h_{Lmin}, h_{Rmin}, h_{Tmin}, h_{Bmin}, 300\ mm) =$ **325** mm

PASS - Unreinforced pad footing depth is greater than minimum