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# Chapel Mead
# Foxham
# Nr. Chippenham
# Wiltshire

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

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# Agricultural Building, Chapel Mead, Foxham, Chippenham, Wiltshire SN15 4NH

**Report on the Structural Aspects of the feasibility of conversion of a single storey agricultural building at Chapel Mead, Foxham Nr. Chippenham**

## Purpose of the Report

Structural Solutions was commissioned to carry out an investigation into the feasibility of converting a single storey agricultural building at Chapel Mead in Foxham 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.

## Introduction

The building is rectangular in plan approximately 11.3m x 9.0m divided into 2 'bays' of approximately 4.5m width each. There is a timber framed storage area to the north elevation that is proposed to become the car port in the future. This area is excluded from this report as it is not proposed this be part of the conversion into residential accommodation. The external elevations are clad with a combination of corrugated sheet, 'Yorkshire' boarding and clad metal frames/gates. The roof is similarly clad but with corrugated sheet spanning between purlins which in turn span between the main structural steel frames

## Investigation

Our investigation is based on a visual walk over only carried out on 25th July 2023.

The existing building is currently an agricultural structure comprised of three structural steel frames forming 2 bays of approximately 4.5 metres in width. There is a frame in each 'gable' elevation and one central. The roof covering is a corrugate sheet supported by steel angle purlins spanning between the steel frames. To the north of the building, there is a shallow pitched roofed covered area of storage but this has been constructed using timber rather than the steel for the majority of the building. This is not intended to be incorporated into the proposed residential area and thus is excluded from this report. The area within the perimeter of the building is open plan but the external elevations are clad with a mixture of corrugated sheet, 'Yorkshire' boarding and clad metal frames/gate. We understand that the existing timber cladding will be retained, with the gaps between the 'Yorkshire' boarding being 'closed' with the application of an additional board to form 'castle' boarding. This will have the benefit of improving the weatherproofing of the building.

The building does not have a concrete floor but trial pits were excavated to expose the footing below both an external and an internal post of the steel frame. Foundations

were approximately 800mm x 800mm x 800mm thick of 'solid' concrete. Formation level was found to be sound.

**Discussion**

We understand from the Client that the proposed conversion work of the barn 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. External walls are of a mixture of materials and these are proposed to be retained but breather membranes and insulation measures added in the form of a non-loadbearing timber stud linings added internally. Modifications will be required in order to introduce windows and doors as shown on the architectural drawings and openings where there are no walls infilled with cladding rails spanning between existing columns and clad as the existing but this work can be done without compromising the integrity of the existing structure. Roof cladding is intended to be retained save for localised repair where required and introduction of roof lights.

A new ground bearing concrete ground slab incorporating a damp proof membrane, insulation and floor finishes above the concrete will need to be introduced.

**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 barn into a usable property is feasible without modification of the main structure. In brief, the existing roof structure, walls and foundations of the barn can be incorporated into a conversion into residential use without modification. Similarly, the existing roof cladding is capable of retention.

Insulation and breather membrane to achieve appropriate residential standards would be carried by a non-structural non-load bearing linings secured to the inside face of the existing external walls constructed using a softwood timber frame or lightweight cold-formed metal stud system

**Outline Schedule of Works**

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

Elements for Retention

- Foundation of structural frame (100%)
- Main external walls (c.95% - allowing for new openings)
- Main structural steel roof frames and purlins

Proposed Works/Modification

- Non-load bearing timber frame linings to the inside face of the existing/retained external walls to carry insulation and breather membrane.
- Infill external wall openings where there are no walls using cladding rails spanning between existing columns and clad as existing.
- Non-load bearing internal partitions to form internal 'rooms' as shown on the Architect's drawings.
- Windows and doors
- New concrete ground slab with damp proof membrane, floor insulation and screed finish

Repairs/ Extent of Remedial Works

Walls

- Inspect all cladding. Allow for localised repairs to corrugated and 'Yorkshire' boarding.

Roof

- Check fixings of the roof cladding and introduce additional fixings as necessary. (Approximately 5% of new fixings likely to be required)

- Any 'small' holes in roof cladding, thoroughly clean area around the hole and apply epoxy over hole. (Approximately 2 % requires attention, the remainder appears to be sound and capable of retention)

- Internally, inspect and thoroughly clean existing roof frames 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).

- Internally, inspect and thoroughly clean existing all steel purlins (i.e. rolled steel angles) 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).

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) to form rooms. Timber frame linings secured to the inside face of the 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 and linings 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.

- 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 – Site Photos




External west elevation




External south west corner




Internal view of east gable elevation




Internal view part of structural steel frame




Internal view part of structural steel frame




Exposed foundation of external post

Appendix B – Structural Calculations

## Introduction

Structural Solutions has been commissioned to consider the adequacy of the existing structure of an agricultural building at Chapel Mead in Foxham near Chippenham as part of a proposed conversion into a residential dwelling. The existing building is currently used for agricultural storage. We have looked at the design of the existing steel frame on the basis that it will need to support 'residential' loads in accordance with Eurocodes and the Building Regulations Approved Document A as opposed to 'agricultural' loads. Typically, the former considers a design life of 60 years and the latter 20 years. In practice, this means that the structure within a residential property must support greater loads.

As part of the proposed conversion, new insulated non-load bearing timber frame linings will be provided inside the external perimeter walls and insulated ceilings will be installed below the existing roof.

These calculations show that the existing steel frame can form the main structure of the proposed dwelling without material alteration.

## Assumptions

Location of the site is Foxham near Chippenham, Wiltshire in a country location.

Codes to be used include but not limited to:

BS EN 1991 – Eurocode 1 - Actions on Structures

BS EN 1993 – Eurocode 3 – Design of Steel Structures

## Loads

Roof ($7^o$ pitch) Dead: Assume 0.35 kN/$m^2$ normal to roof incl existing roof sheet, insulation and ceiling.

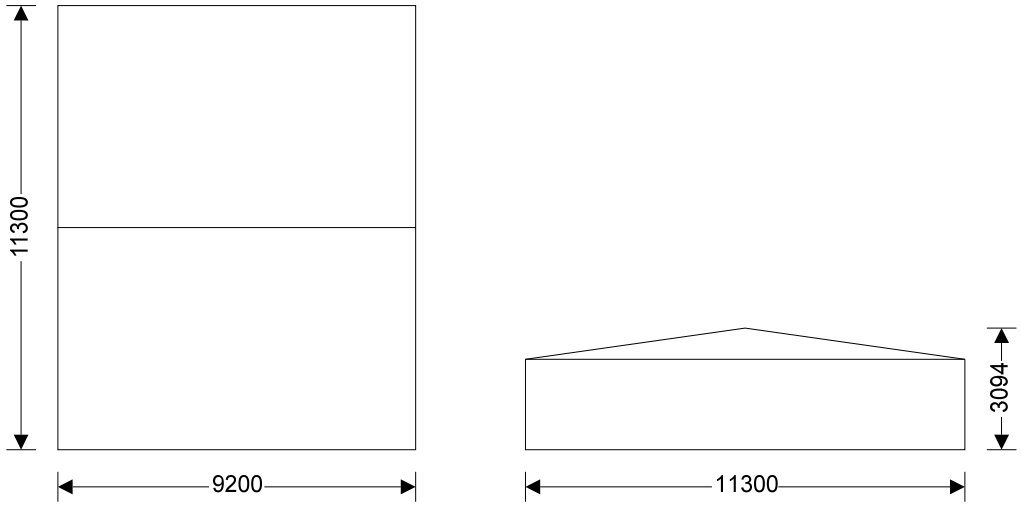
Live: ≡ 0.18kN/$m^2$ in plan (see snow load calculation)

# WIND LOADING (EN1991)

## WIND LOADING

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

Tedds calculation version 3.0.28




### Building data

Type of roof Duopitch

Length of building L = **9200** mm

Width of building W = **11300** mm

Height to eaves H = **2300** mm

Pitch of roof $\alpha_0$ = **8.0** deg

Total height h = **3094** mm

### Basic values

Location Foxham

Wind speed velocity (FigureNA.1) $v_{b,map}$ = **21.6** m/s

Distance to shore $L_{shore}$ = **47.00** 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 $v_{b,0} = v_{b,map} \times c_{alt}$ = **22.9** 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 $\rho$ = **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) $v_b = c_{dir} \times c_{season} \times v_{b,0} \times c_{prob}$ = **22.9** m/s

Reference mean velocity pressure $q_b = 0.5 \times \rho \times v_b^2$ = **0.321** 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)**

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

Reference height (at which q is sought) z = **2300**mm

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

Exposure factor (Figure NA.7) $c_e$ = **1.51**

Peak velocity pressure $q_p = c_e \times q_b$ = **0.48** kN/m$^2$

**Structural factor**

Structural damping $\delta_s$ = **0.050**

Height of element $h_{part}$ = **2300** mm

Size factor (Table NA.3) $c_s$ = **0.936**

Dynamic factor (Figure NA.9) $c_d$ = **1.024**

Structural factor $c_{sCd} = c_s \times c_d$ = **0.958**

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

Reference height (at which q is sought) z = **3094**mm

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

Exposure factor (Figure NA.7) $c_e$ = **1.67**

Peak velocity pressure $q_p = c_e \times q_b$ = **0.54** kN/m$^2$

**Structural factor**

Structural damping $\delta_s$ = **0.050**

Height of element $h_{part}$ = **3094** mm

Size factor (Table NA.3) $c_s$ = **0.927**

Dynamic factor (Figure NA.9) $c_d$ = **1.020**

Structural factor $c_{sCd} = c_s \times c_d$ = **0.945**

**Structural factor - roof 0 deg**

Structural damping $\delta_s$ = **0.050**

Height of element $h_{part}$ = **3094** mm

Size factor (Table NA.3) $c_s$ = **0.933**

Dynamic factor (Figure NA.9) $c_d$ = **1.024**

Structural factor $c_{sCd} = c_s \times c_d$ = **0.956**

**Peak velocity pressure for internal pressure**

Peak velocity pressure – internal (as roof press.) $q_{p,i}$ = **0.54** 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) | -1.59 | 0.54 | -0.92 | 1.93 | -1.78 |
| G (-ve) | -1.08 | 0.54 | -0.66 | 3.82 | -2.52 |
| H (-ve) | -0.54 | 0.54 | -0.38 | 46.74 | -17.95 |
| I (-ve) | -0.43 | 0.54 | -0.33 | 46.74 | -15.32 |
| J (-ve) | -1.02 | 0.54 | -0.63 | 5.75 | -3.62 |

Total vertical net force $F_{w,v}$ = **-40.79** kN

Total horizontal net force $F_{w,h}$ = **-0.46** 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 | 0.54 | -0.72 | 2.95 | -2.14 |
| B | -0.80 | 0.54 | -0.52 | 13.93 | -7.22 |
| C | -0.50 | 0.54 | -0.36 | 13.59 | -4.95 |
| D | 0.70 | 0.48 | 0.22 | 21.16 | 4.64 |

| | | | | | |
|---|---|---|---|---|---|
| E | -0.31 | 0.48 | -0.25 | 21.16 | -5.28 |

**Overall loading**

Equiv leeward net force for overall section $F_l = F_{w,wE}$ = **-5.3** kN

Net windward force for overall section $F_w = F_{w,wD}$ = **4.6** kN

Lack of correlation (cl.7.2.2(3) – Note) $f_{corr}$ = **0.85** as h/W is 0.274

Overall loading overall section $F_{w,D} = f_{corr} \times (F_w - F_l + F_{w,h})$ = **8.0** 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) | 0.06 | 0.54 | 0.19 | 1.93 | 0.37 |
| G (+ve) | 0.06 | 0.54 | 0.19 | 3.82 | 0.73 |
| H (+ve) | 0.06 | 0.54 | 0.19 | 46.74 | 8.96 |
| I (+ve) | -0.43 | 0.54 | -0.06 | 46.74 | -2.78 |
| J (+ve) | -1.02 | 0.54 | -0.36 | 5.75 | -2.08 |

Total vertical net force $F_{w,v}$ = **5.15** kN

Total horizontal net force $F_{w,h}$ = **2.08** 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 | 0.54 | -0.46 | 2.95 | -1.35 |
| B | -0.80 | 0.54 | -0.25 | 13.93 | -3.48 |
| C | -0.50 | 0.54 | -0.10 | 13.59 | -1.31 |
| D | 0.70 | 0.48 | 0.49 | 21.16 | 10.32 |
| E | -0.31 | 0.48 | 0.02 | 21.16 | 0.39 |

**Overall loading**

Equiv leeward net force for overall section $F_l = F_{w,wE}$ = **0.4** kN

Net windward force for overall section $F_w = F_{w,wD}$ = **10.3** kN

Lack of correlation (cl.7.2.2(3) – Note) $f_{corr}$ = **0.85** as h/W is 0.274

Overall loading overall section $F_{w,D} = f_{corr} \times (F_w - F_l + F_{w,h})$ = **10.2** 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) |
|---|---|---|---|---|---|
| F (-ve) | -1.88 | 0.54 | -1.06 | 1.93 | -2.05 |
| G (-ve) | -1.22 | 0.54 | -0.73 | 5.13 | -3.72 |
| H (-ve) | -0.60 | 0.54 | -0.41 | 28.25 | -11.62 |
| I (-ve) | -0.47 | 0.54 | -0.35 | 69.68 | -24.08 |

Total vertical net force $F_{w,v}$ = **-41.07** 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 | -1.20 | 0.48 | -0.66 | 2.85 | -1.87 |
| B | -0.80 | 0.48 | -0.47 | 11.39 | -5.40 |
| C | -0.50 | 0.48 | -0.34 | 6.93 | -2.33 |
| D | 0.71 | 0.54 | 0.25 | 30.48 | 7.72 |
| E | -0.32 | 0.54 | -0.27 | 30.48 | -8.26 |

**Overall loading**

Equiv leeward net force for overall section $F_l = F_{w,wE}$ = **-8.3** kN

Net windward force for overall section $F_w = F_{w,wD}$ = **7.7** kN

Lack of correlation (cl.7.2.2(3) – Note) $f_{corr}$ = **0.85** as h/L is 0.336

Overall loading overall section $F_{w,D} = f_{corr} \times (F_w - F_l + F_{w,h})$ = **13.6** 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) |
|---|---|---|---|---|---|
| F (+ve) | 0.06 | 0.54 | 0.19 | 1.93 | 0.37 |
| G (+ve) | 0.06 | 0.54 | 0.19 | 5.13 | 0.98 |
| H (+ve) | 0.06 | 0.54 | 0.19 | 28.25 | 5.40 |
| I (+ve) | 0.06 | 0.54 | 0.19 | 69.68 | 13.33 |

Total vertical net force $F_{w,v}$ = **19.89** 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 | -1.20 | 0.48 | -0.39 | 2.85 | -1.11 |
| B | -0.80 | 0.48 | -0.21 | 11.39 | -2.34 |
| C | -0.50 | 0.48 | -0.07 | 6.93 | -0.47 |
| D | 0.71 | 0.54 | 0.52 | 30.48 | 15.90 |
| E | -0.32 | 0.54 | 0.00 | 30.48 | -0.09 |

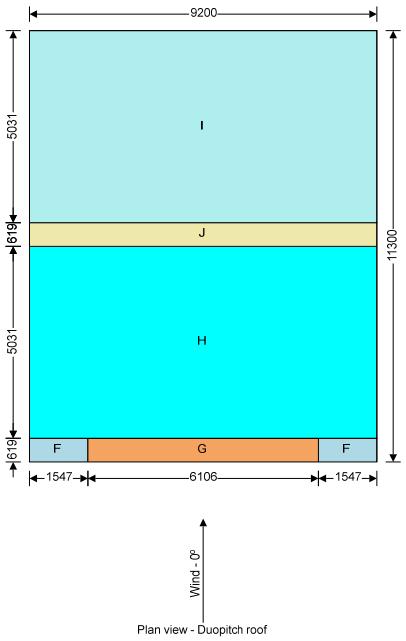
**Overall loading**

Equiv leeward net force for overall section $F_l = F_{w,wE}$ = **-0.1** kN

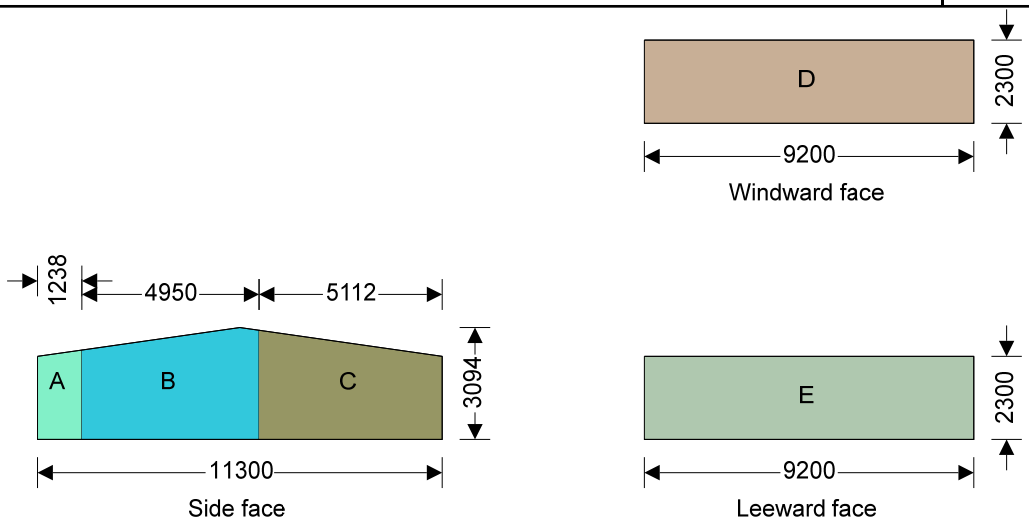
Net windward force for overall section $F_w = F_{w,wD}$ = **15.9** kN

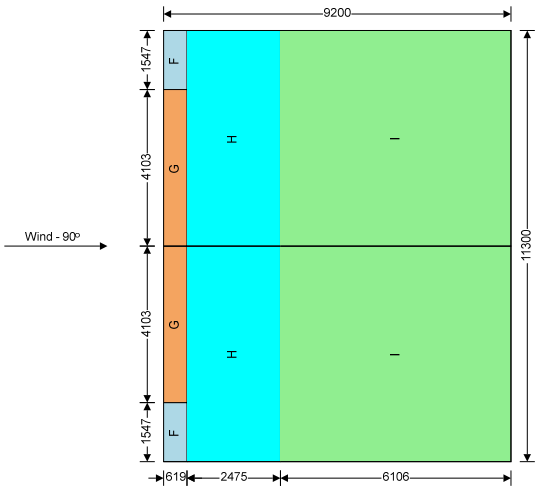
Lack of correlation (cl.7.2.2(3) – Note) $f_{corr}$ = **0.85** as h/L is 0.336

Overall loading overall section $F_{w,D} = f_{corr} \times (F_w - F_l + F_{w,h})$ = **13.6** kN

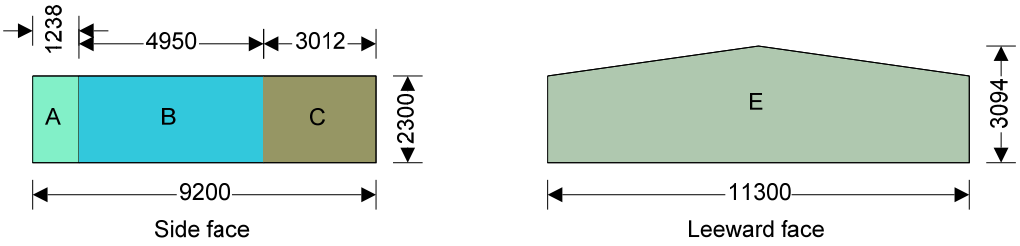



Plan view - Duopitch roof




Plan view - Duopitch roof


D
3094
11300
Windward face


1238
4950
3012
A
B
C
2300
9200
Side face


E
3094
11300
Leeward face

# SNOW LOADING (EN1991)

## 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 Foxham

Site altitude above sea level (user modified value) A = **60** m

Zone number (user modified value) Z = **1.0**

Density of snow $\gamma$ = **2.00** kN/m$^3$

Characteristic ground snow load $s_k = ((0.15 + (0.1 \times Z + 0.05)) + ((A - 100m) / 525m)) \times 1kN/m^2$ = **0.22** 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) $b_1$ = **6.32** m

Width of roof (right on elevation) $b_2$ = **2.37** m

Slope of roof (left on elevation) $\alpha_1$ = **8.00** deg

Slope of roof (right on elevation) $\alpha_2$ = **21.00** deg

### Shape coefficients

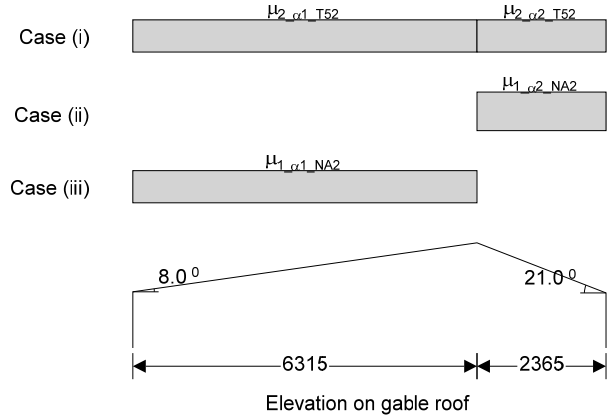
Shape coefficient roof (Table 5.2) $\mu_{2_\alpha1_T52}$ = **0.80**

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) $\mu_{1_\alpha2_NA2}$ = **0.96**

## Snow Load



Elevation on gable roof

| Shape coef | Coef | Loading (kN/m²) |
|---|---|---|
| $\mu_{2_\alpha1_T52}$ | 0.800 | 0.18 |
| $\mu_{2_\alpha2_T52}$ | 0.800 | 0.18 |
| $\mu_{1_\alpha1_NA2}$ | 0.800 | 0.18 |
| $\mu_{1_\alpha2_NA2}$ | 0.960 | 0.21 |

**Loadcase 1 Table 5.2**

Loading to roof 1 (LHS) $s_{1_1} = \mu_{2_\alpha1_T52} \times C_e \times C_t \times s_k =$ **0.18** kN/m²

Loading to roof 2 (RHS) $s_{2_1} = \mu_{2_\alpha2_T52} \times C_e \times C_t \times s_k =$ **0.18** kN/m²

**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/m²

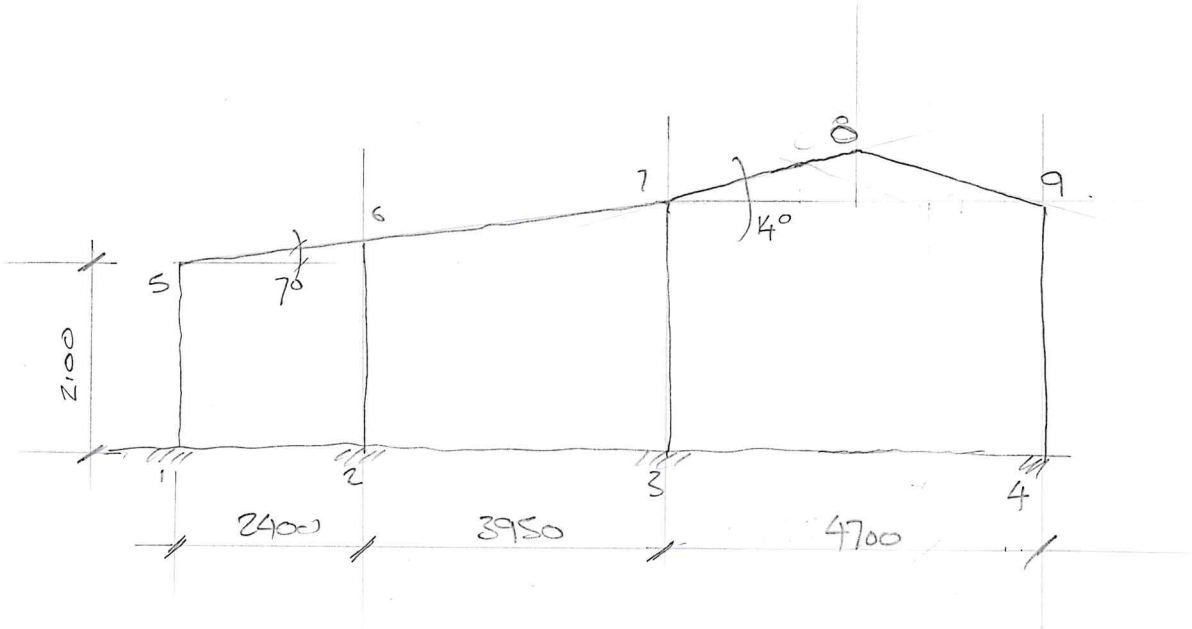
Loading to roof 2 (RHS) $s_{2_2} = \mu_{1_\alpha2_NA2} \times C_e \times C_t \times s_k =$ **0.21** kN/m²

**Loadcase 3 UK NA.2**

Loading to roof 1 (LHS) $s_{1_3} = \mu_{1_\alpha1_NA2} \times C_e \times C_t \times s_k =$ **0.18** kN/m²

Loading to roof 2 (RHS) $s_{2_3} = 0 \times C_e \times C_t \times s_k =$ **0.00** kN/m²

| Project | Sheet no./rev. |
|---|---|
| AGRICULTURAL BUILDING, CHAPEL MEAS, FORFAR | CML 20 |
| EXISTING FRAME ANALYSIS | By/Date MH 1/9/23 |




| NODES | X | Y |
|---|---|---|
| 1 | 0 | 0 |
| 2 | 2.40 | 0 |
| 3 | 6.35 | 0 |
| 4 | 11.05 | 0 |
| 5 | 0 | 2.1 |
| 6 | 2.40 | 2.395 |
| 7 | 6.35 | 2.880 |
| 8 | 8.70 | 3.466 |
| 9 | 11.05 | 2.880 |

MEMBERS

EQUIVALENT TO

IPE 160

(EUROPEAN SECTN)

SPACING OF FRAMES

≈ 4500

LOADING-

RAFTERS – DEAD LOAD (INCL. CEILING) 0.35 kN/m² × 4.5. = 1.58 kN/m

– LIVE LOAD 0.18 kN/m² × 4.5. = 0.81 kN/m

– WIND LOAD ROOF (WINDWARD) 0.19 kN/m² × 4.5

= 0.86 kN/m

STRUCTURAL SOLUTIONS

| Project | Sheet no./rev. |
| --- | --- |
| AGRICULTURAL BUILDING, CHAPEL NESS, FOXHAM | CNL 21 |
| EXISTING FRAME ANALYSIS | By/Date MH 1/9/23 |

LOADING (CONT.)

- WIND.

ROOF (LEEWARD) $-0.06\,kN/m^2 \times 4.5$

$= 0.27\,kN/m$

WALL (WINDWARD) $0.49\,kN/m^2 \times 4.5$

$= 2.21\,kN/m$

(LEEWARD) $0.02\,kN/m^2 \times 4.5$

$= 0.09\,kN/m$

# STEEL 2D ANALYSIS & DESIGN (EN1993)

## 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**

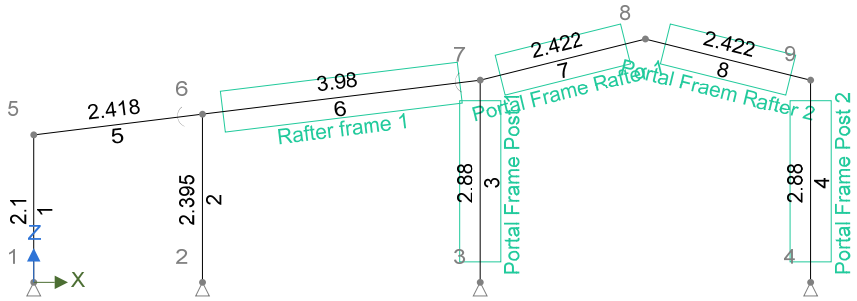
Tedds calculation version 4.4.11

## ANALYSIS

Tedds calculation version 1.0.37

### 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) |
| --- | --- | --- | --- | --- | --- |
| IPE 160 | 20.1 | 869.3 | 68.3 | 8 | 10.9 |
| RSA 90x90x8 | 13.9 | 105.1 | 105.1 | 0 | 0 |

### 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 |

| Node | Co-ordinates | | Freedom | | | Coordinate system | | Spring | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | X (m) | Z (m) | X | Z | Rot. | Name | Angle (°) | X (kN/m) | Z (kN/m) | Rot. kNm/° |
| 2 | 2.4 | 0 | Fixed | Fixed | Free | | 0 | 0 | 0 | 0 |
| 3 | 6.35 | 0 | Fixed | Fixed | Free | | 0 | 0 | 0 | 0 |
| 4 | 11.05 | 0 | Fixed | Fixed | Free | | 0 | 0 | 0 | 0 |
| 5 | 0 | 2.1 | Free | Free | Free | | 0 | 0 | 0 | 0 |
| 6 | 2.4 | 2.395 | Free | Free | Free | | 0 | 0 | 0 | 0 |
| 7 | 6.35 | 2.88 | Free | Free | Free | | 0 | 0 | 0 | 0 |
| 8 | 8.7 | 3.466 | Free | Free | Free | | 0 | 0 | 0 | 0 |
| 9 | 11.05 | 2.88 | Free | Free | Free | | 0 | 0 | 0 | 0 |

**Elements**

| Element | Length (m) | Nodes | | Section | Material | Releases | | | Rotated |
|---|---|---|---|---|---|---|---|---|---|
| | | Start | End | | | Start moment | End moment | Axial | |
| 1 | 2.1 | 1 | 5 | IPE 160 | Steel (EC3) | Fixed | Fixed | Fixed | |
| 2 | 2.395 | 2 | 6 | IPE 160 | Steel (EC3) | Fixed | Fixed | Fixed | |
| 3 | 2.88 | 3 | 7 | IPE 160 | Steel (EC3) | Fixed | Fixed | Fixed | |
| 4 | 2.88 | 4 | 9 | IPE 160 | Steel (EC3) | Fixed | Fixed | Fixed | |
| 5 | 2.418 | 5 | 6 | RSA 90x90x8 | Steel (EC3) | Fixed | Free | Fixed | |
| 6 | 3.98 | 6 | 7 | IPE 160 | Steel (EC3) | Fixed | Free | Fixed | |
| 7 | 2.422 | 7 | 8 | IPE 160 | Steel (EC3) | Fixed | Fixed | Fixed | |
| 8 | 2.422 | 8 | 9 | IPE 160 | Steel (EC3) | Fixed | Fixed | Fixed | |

**Members**

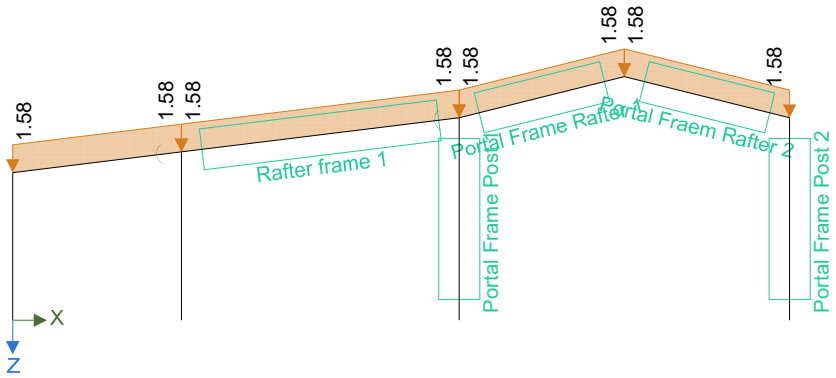
| Name | Elements | |
|---|---|---|
| | Start | End |
| Rafter frame 1 | 6 | 6 |
| Portal Frame Post 1 | 3 | 3 |
| Portal Frame Post 2 | 4 | 4 |

| Name | Elements | |
| --- | --- | --- |
| | Start | End |
| Portal Frame Rafter 1 | 7 | 7 |
| Portal Fraem Rafter 2 | 8 | 8 |

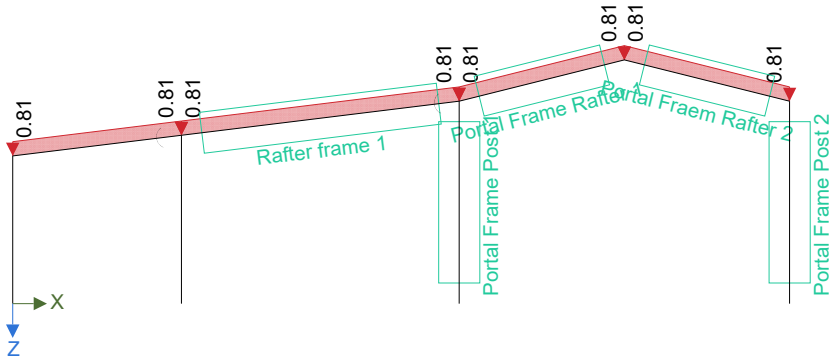
**Loading**

Self weight included

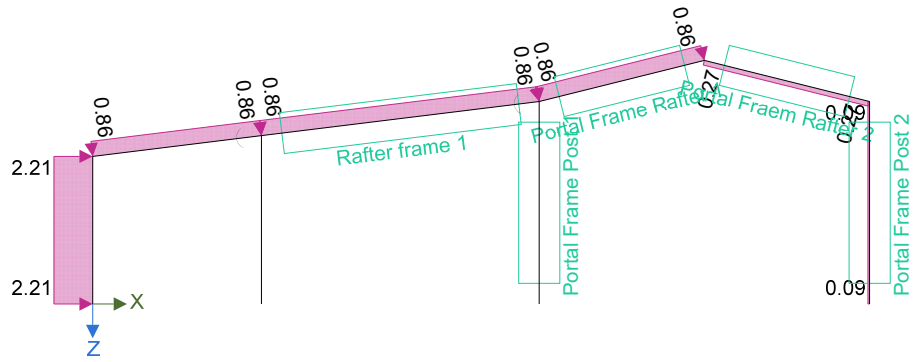
**Permanent - Loading (kN/m)**




**Imposed - Loading (kN/m)**



## 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 | |
| 1.0G + 1.0Q + 1.0RQ (Service) | 1.00 | 1.00 | 1.00 | |
| 1.35G + 1.5Q + 1.5$\psi_0$S + 1.5$\psi_0$W (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 + 1.5$\psi_0$Q + 1.5$\psi_0$S + 1.5W (Strength) | 1.35 | 1.35 | 1.05 | 1.50 |
| 1.0G + 1.0$\psi_0$Q + 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 |

### Element Loads

| Element | Load case | Load Type | Orientation | Description |
|---|---|---|---|---|
| 5 | Permanent | UDL | GlobalZ | 1.58 kN/m |
| 6 | Permanent | UDL | GlobalZ | 1.58 kN/m |
| 7 | Permanent | UDL | GlobalZ | 1.58 kN/m |
| 8 | Permanent | UDL | GlobalZ | 1.58 kN/m |
| 5 | Imposed | UDL | GlobalZ | 0.81 kN/m |
| 6 | Imposed | UDL | GlobalZ | 0.81 kN/m |
| 7 | Imposed | UDL | GlobalZ | 0.81 kN/m |
| 8 | Imposed | UDL | GlobalZ | 0.81 kN/m |
| 1 | Wind | UDL | GlobalX | 2.21 kN/m |
| 5 | Wind | UDL | LocalZ | 0.86 kN/m |
| 6 | Wind | UDL | LocalZ | 0.86 kN/m |

| Element | Load case | Load Type | Orientation | Description |
|---|---|---|---|---|
| 7 | Wind | UDL | LocalZ | 0.86 kN/m |
| 8 | Wind | UDL | LocalZ | -0.27 kN/m |
| 4 | Wind | UDL | GlobalX | 0.09 kN/m |

## Results

### Reactions

### Load case: Self Weight

| Node | Force | | Moment |
|---|---|---|---|
| | Fx (kN) | Fz (kN) | My (kNm) |
| 1 | 0 | 0.5 | 0 |
| 2 | 0 | 0.8 | 0 |
| 3 | 0 | 1.1 | 0 |
| 4 | -0.1 | 0.9 | 0 |

### Load case: Permanent

| Node | Force | | Moment |
|---|---|---|---|
| | Fx (kN) | Fz (kN) | My (kNm) |
| 1 | 0.3 | 2.3 | 0 |
| 2 | 0.4 | 5.1 | 0 |
| 3 | 0.3 | 6.2 | 0 |
| 4 | -1.1 | 4.3 | 0 |

### Load case: Imposed

| Node | Force | | Moment |
|---|---|---|---|
| | Fx (kN) | Fz (kN) | My (kNm) |
| 1 | 0.2 | 1.2 | 0 |
| 2 | 0.2 | 2.6 | 0 |
| 3 | 0.2 | 3.2 | 0 |
| 4 | -0.6 | 2.2 | 0 |

**Load case: Wind**

| Node | Force | | Moment |
|---|---|---|---|
| | Fx (kN) | Fz (kN) | My (kNm) |
| 1 | -2.6 | 1.1 | 0 |
| 2 | -1.1 | 2.2 | 0 |
| 3 | -1.1 | 2 | 0 |
| 4 | -1.4 | 1.6 | 0 |

**Load combination: 1.35G + 1.5Q + 1.5RQ (Strength)**

| Node | Force | | Moment |
|---|---|---|---|
| | Fx (kN) | Fz (kN) | My (kNm) |
| 1 | 0.8 | 5.4 | 0 |
| 2 | 0.9 | 11.8 | 0 |
| 3 | 0.7 | 14.5 | 0 |
| 4 | -2.4 | 10.3 | 0 |

**Load combination: 1.0G + 1.0Q + 1.0RQ (Service)**

| Node | Force | | Moment |
|---|---|---|---|
| | Fx (kN) | Fz (kN) | My (kNm) |
| 1 | 0.5 | 3.9 | 0 |
| 2 | 0.7 | 8.5 | 0 |
| 3 | 0.5 | 10.4 | 0 |
| 4 | -1.7 | 7.4 | 0 |

**Load combination: 1.35G + 1.5Q + 1.5$\psi_0$S + 1.5$\psi_0$W (Strength)**

| Node | Force | | Moment |
|---|---|---|---|
| | Fx (kN) | Fz (kN) | My (kNm) |
| 1 | -1.2 | 6.2 | 0 |
| 2 | 0.1 | 13.5 | 0 |

| Node | Force | | Moment |
|---|---|---|---|
| | Fx (kN) | Fz (kN) | My (kNm) |
| 3 | -0.1 | 16 | 0 |
| 4 | -3.5 | 11.4 | 0 |

**Load combination: 1.0G + 1.0Q + 0.5S + 0.5W (Service)**

| Node | Force | | Moment |
|---|---|---|---|
| | Fx (kN) | Fz (kN) | My (kNm) |
| 1 | -0.7 | 4.4 | 0 |
| 2 | 0.1 | 9.6 | 0 |
| 3 | -0.1 | 11.4 | 0 |
| 4 | -2.4 | 8.1 | 0 |

**Load combination: 1.35G + 1.5$\psi_0$Q + 1.5$\psi_0$S + 1.5W (Strength)**

| Node | Force | | Moment |
|---|---|---|---|
| | Fx (kN) | Fz (kN) | My (kNm) |
| 1 | -3.2 | 6.5 | 0 |
| 2 | -0.8 | 13.9 | 0 |
| 3 | -1.1 | 16.1 | 0 |
| 4 | -4.3 | 11.6 | 0 |

**Load combination: 1.0G + 1.0$\psi_0$Q + 0.5S + 1.0W (Service)**

| Node | Force | | Moment |
|---|---|---|---|
| | Fx (kN) | Fz (kN) | My (kNm) |
| 1 | -2.1 | 4.6 | 0 |
| 2 | -0.5 | 9.9 | 0 |
| 3 | -0.7 | 11.4 | 0 |
| 4 | -3 | 8.3 | 0 |

**Load combination: 1.0G + 1.5W (Strength)**

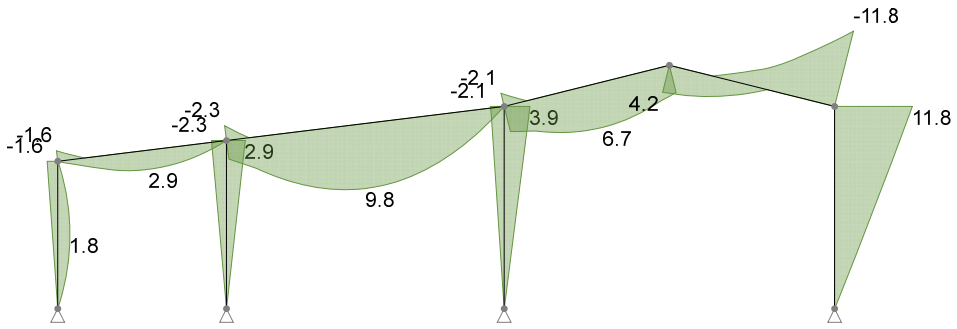
| Node | Force | | Moment |
|---|---|---|---|
| | Fx (kN) | Fz (kN) | My (kNm) |
| 1 | -3.5 | 4.3 | 0 |
| 2 | -1.2 | 9.2 | 0 |
| 3 | -1.4 | 10.2 | 0 |
| 4 | -3.3 | 7.5 | 0 |

**Load combination: 1.0G + 1.0W (Service)**

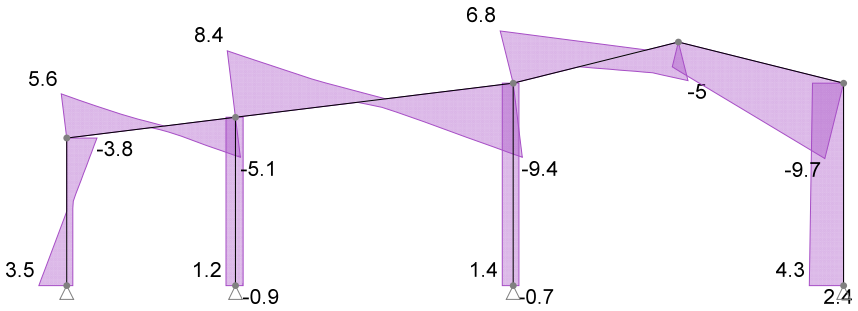
| Node | Force | | Moment |
|---|---|---|---|
| | Fx (kN) | Fz (kN) | My (kNm) |
| 1 | -2.2 | 3.8 | 0 |
| 2 | -0.6 | 8.1 | 0 |
| 3 | -0.8 | 9.2 | 0 |
| 4 | -2.6 | 6.7 | 0 |

**Forces**

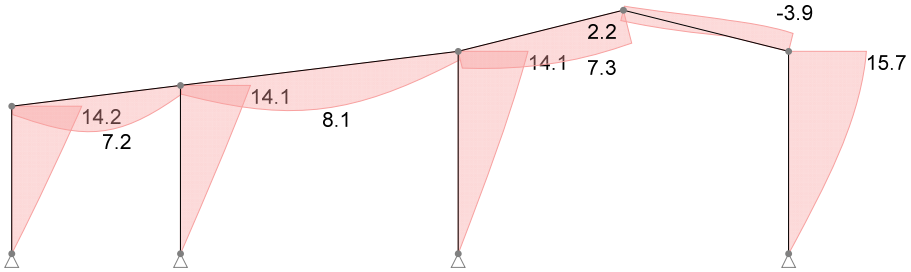
**Strength combinations - Moment envelope (kNm)**



**Strength combinations - Shear envelope (kN)**




**Service combinations - Deflection envelope (mm)**

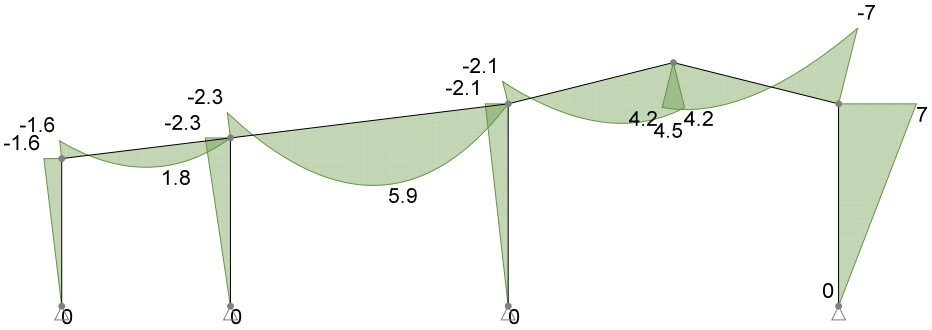



## Member results

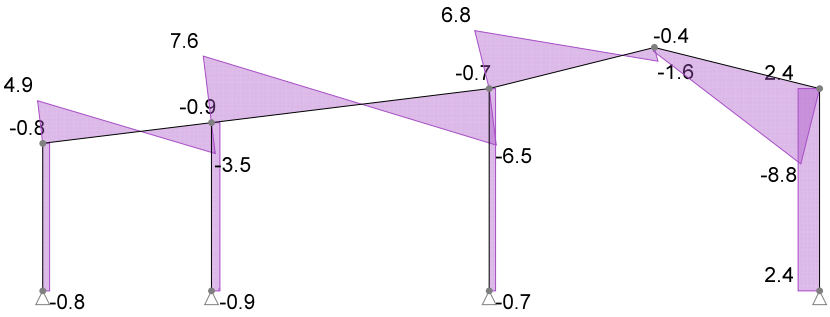
### Envelope - Strength combinations

| Member | Shear force | | Moment | | | |
|---|---|---|---|---|---|---|
| | Pos (m) | Max abs (kN) | Pos (m) | Max (kNm) | Pos (m) | Min (kNm) |
| Rafter frame 1 | 3.98 | -9.4 | 1.881 | 9.8 | 0 | -2.3 |
| Portal Frame Post 1 | 0 | 1.4 | 2.88 | 3.9 | 2.88 | -2.1 |
| Portal Frame Post 2 | 0 | 4.3 | 2.88 | 11.8 (max) | 0 | 0 |
| Portal Frame Rafter 1 | 0 | 6.8 | 1.285 | 6.7 | 0 | -2.1 |
| Portal Frame Rafter 2 | 2.422 | -9.7 (max abs) | 0 | 4.2 | 2.422 | -11.8 (min) |

**1.35G + 1.5Q + 1.5RQ (Strength) - Moment (kNm)**




**1.35G + 1.5Q + 1.5RQ (Strength) - Shear (kN)**

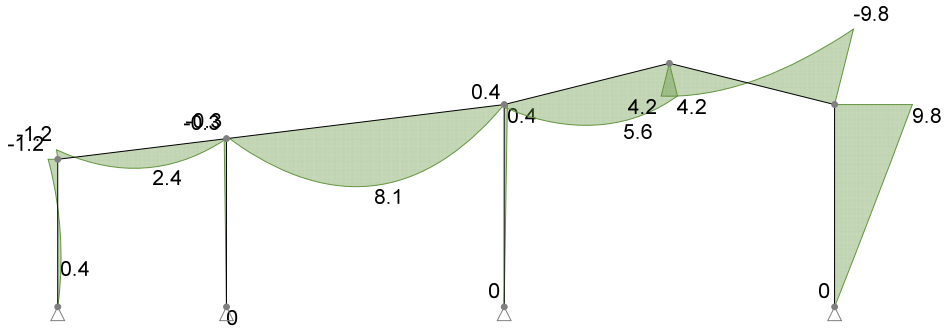



**Member results**

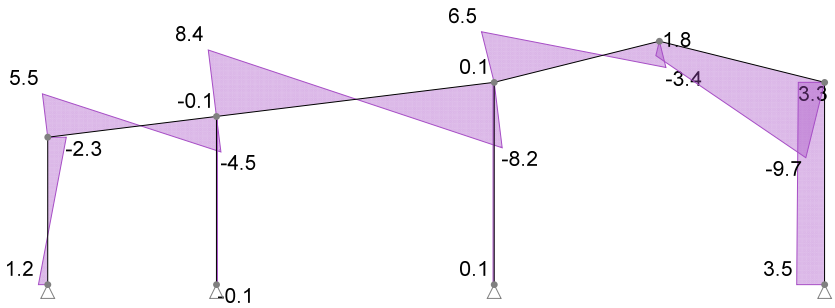
**Load combination: 1.35G + 1.5Q + 1.5RQ (Strength)**

| Member | Shear force | | Moment | | | |
|---|---|---|---|---|---|---|
| | Pos (m) | Max abs (kN) | Pos (m) | Max (kNm) | Pos (m) | Min (kNm) |
| Rafter frame 1 | 0 | 7.6 | 2.151 | 5.9 | 0 | -2.3 |
| Portal Frame Post 1 | 0 | -0.7 | 0 | 0 | 2.88 | -2.1 |
| Portal Frame Post 2 | 0 | 2.4 | 2.88 | 7 (max) | 0 | 0 |
| Portal Frame Rafter 1 | 0 | 6.8 | 1.956 | 4.5 | 0 | -2.1 |
| Portal Fraem Rafter 2 | 2.422 | -8.8 (max abs) | 0 | 4.2 | 2.422 | -7 (min) |

**1.35G + 1.5Q + 1.5ψ0S + 1.5ψ0W (Strength) - Moment (kNm)**




**1.35G + 1.5Q + 1.5ψ0S + 1.5ψ0W (Strength) - Shear (kN)**

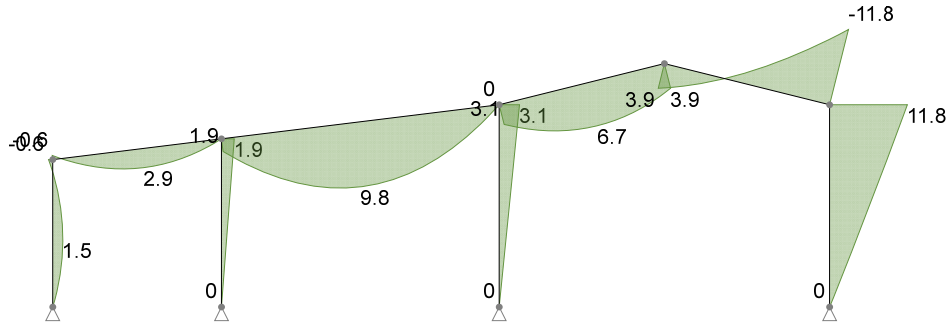



**Member results**

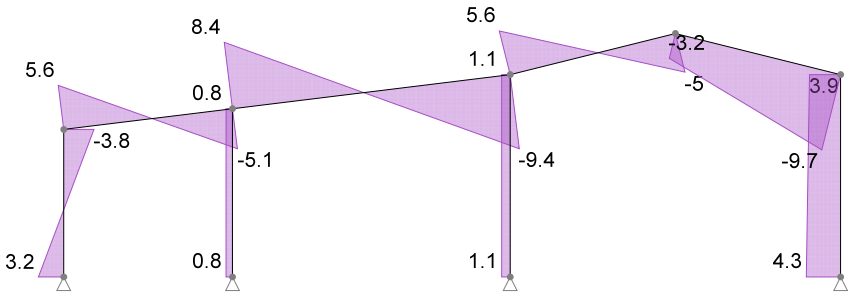
**Load combination: 1.35G + 1.5Q + 1.5ψ0S + 1.5ψ0W (Strength)**

| Member | Shear force | | Moment | | | |
|---|---|---|---|---|---|---|
| | Pos (m) | Max abs (kN) | Pos (m) | Max (kNm) | Pos (m) | Min (kNm) |
| Rafter frame 1 | 0 | 8.4 | 2.007 | 8.1 | 0 | -0.3 |
| Portal Frame Post 1 | 0 | 0.1 | 2.88 | 0.4 | 0 | 0 |
| Portal Frame Post 2 | 0 | 3.5 | 2.88 | 9.8 (max) | 0 | 0 |
| Portal Frame Rafter 1 | 0 | 6.5 | 1.597 | 5.6 | 0 | 0.4 |
| Portal Fraem Rafter 2 | 2.422 | -9.7 (max abs) | 0 | 4.2 | 2.422 | -9.8 (min) |

**1.35G + 1.5ψ0Q + 1.5ψ0S + 1.5W (Strength) - Moment (kNm)**




**1.35G + 1.5ψ0Q + 1.5ψ0S + 1.5W (Strength) - Shear (kN)**

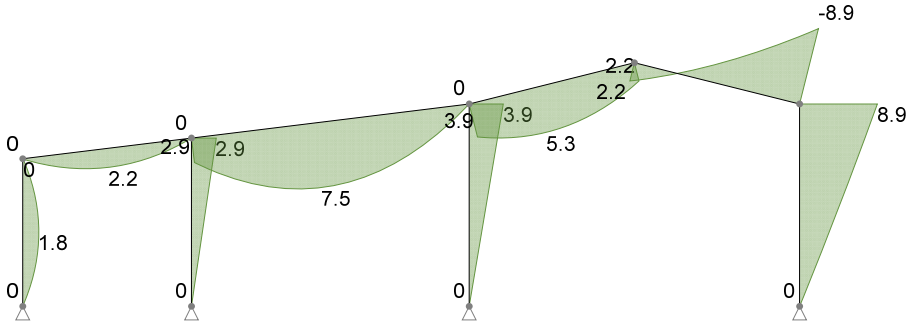



**Member results**

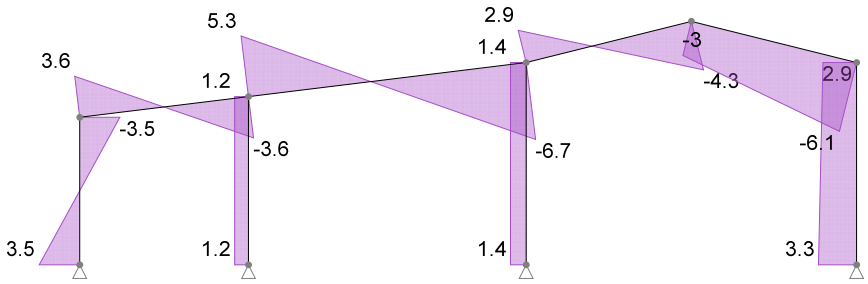
**Load combination: 1.35G + 1.5ψ0Q + 1.5ψ0S + 1.5W (Strength)**

| Member | Shear force | | Moment | | | |
|---|---|---|---|---|---|---|
| | Pos (m) | Max abs (kN) | Pos (m) | Max (kNm) | Pos (m) | Min (kNm) |
| Rafter frame 1 | 3.98 | -9.4 | 1.881 | 9.8 | 3.98 | 0 |
| Portal Frame Post 1 | 0 | 1.1 | 2.88 | 3.1 | 0 | 0 |
| Portal Frame Post 2 | 0 | 4.3 | 2.88 | 11.8 (max) | 0 | 0 |
| Portal Frame Rafter 1 | 0 | 5.6 | 1.285 | 6.7 | 0 | 3.1 |
| Portal Fraem Rafter 2 | 2.422 | -9.7 (max abs) | 0 | 3.9 | 2.422 | -11.8 (min) |

**1.0G + 1.5W (Strength) - Moment (kNm)**




**1.0G + 1.5W (Strength) - Shear (kN)**




**Member results**

**Load combination: 1.0G + 1.5W (Strength)**

| Member | Shear force | | Moment | | | |
| --- | --- | --- | --- | --- | --- | --- |
| | Pos (m) | Max abs (kN) | Pos (m) | Max (kNm) | Pos (m) | Min (kNm) |
| Rafter frame 1 | 3.98 | -6.7 (max abs) | 1.752 | 7.5 | 3.98 | 0 |
| Portal Frame Post 1 | 0 | 1.4 | 2.88 | 3.9 | 0 | 0 |
| Portal Frame Post 2 | 0 | 3.3 | 2.88 | 8.9 (max) | 0 | 0 |
| Portal Frame Rafter 1 | 2.422 | -4.3 | 0.967 | 5.3 | 2.422 | 2.2 |
| Portal Fraem Rafter 2 | 2.422 | -6.1 | 0 | 2.2 | 2.422 | -8.9 (min) |

**Partial factors - Section 6.1**

Resistance of cross-sections $\gamma_{M0}$ = **1**

Resistance of members to instability $\gamma_{M1}$ = **1**

Resistance of tensile members to fracture $\gamma_{M2}$ = **1.1**

Library item: Partial factors out

## Rafter frame 1 design

### Section details

Section type IPE 160 (Arcelor)

Steel grade - EN 10025-2:2004 S235

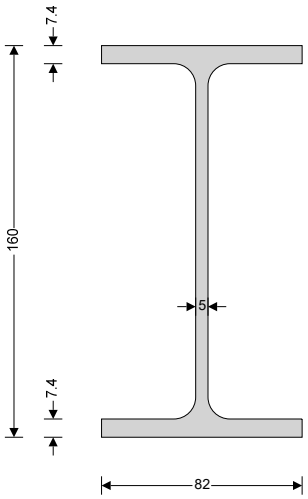
Nominal thickness of element $t_{nom} = \max(t_f, t_w)$ = **7.4** mm

Nominal yield strength $f_y$ = **235** N/mm$^2$

Nominal ultimate tensile strength $f_u$ = **360** N/mm$^2$

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

Library item: Section details out




**IPE 160 (Arcelor)**
Section depth, h, 160 mm
Section breadth, b, 82 mm
Mass of section, Mass, 15.8 kg/m
Flange thickness, $t_f$, 7.4 mm
Web thickness, $t_w$, 5 mm
Root radius, r, 9 mm
Area of section, A, 2009 mm$^2$
Radius of gyration about y-axis, $i_y$, 65.778 mm
Radius of gyration about z-axis, $i_z$, 18.44 mm
Elastic section modulus about y-axis, $W_{el,y}$, 108662 mm$^3$
Elastic section modulus about z-axis, $W_{el,z}$, 16662 mm$^3$
Plastic section modulus about y-axis, $W_{pl,y}$, 123860 mm$^3$
Plastic section modulus about z-axis, $W_{pl,z}$, 26100 mm$^3$
Second moment of area about y-axis, $I_y$, 8692929 mm$^4$
Second moment of area about z-axis, $I_z$, 683146 mm$^4$

### Lateral restraint

Both flanges have lateral restraint at supports only

### Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235\ \text{N/mm}^2 / f_y]}$ = **1.00**

Library item: Class heading out

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

Width of section c = d = **127.2** mm

$\alpha = \min([h / 2 + N_{Ed} / (2 \times t_w \times f_y) - (t_f + r)] / c, 1)$ = **0.515**

$c / t_w = 25.4 = 25.4 \times \varepsilon <= 396 \times \varepsilon / (13 \times \alpha - 1)$ Class 1

Library item: Int bend comp class out

### Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section $c = (b - t_w - 2 \times r) / 2$ = **29.5** mm

$c / t_f = 4 = 4 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Library item: Out flange class out

***Section is class 1***

**Check compression - Section 6.2.4**

Design compression force $N_{Ed}$ = **4.4** kN

Design resistance of section - eq 6.10 $N_{c,Rd} = N_{pl,Rd} = A \times f_y / \gamma_{M0}$ = **472.1** kN

$N_{Ed} / N_{c,Rd}$ = **0.009**

***PASS - Design compression resistance exceeds design compression***

Library item: Compression out

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

Critical buckling length $L_{cr,y} = L_{m1_s1}$ = **3980** mm

Critical buckling force $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2$ = **1137.6** kN

Slenderness ratio for buckling - eq 6.50 $\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})}$ = **0.644**

**Check y-y axis flexural buckling resistance - Section 6.3.1.1**

Buckling curve - Table 6.2 a

Imperfection factor - Table 6.1 $\alpha_y$ = **0.21**

Buckling reduction determination factor $\phi_y = 0.5 \times (1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2)$ = **0.754**

Buckling reduction factor - eq 6.49 $\chi_y = \min(1 / (\phi_y + \sqrt{(\phi_y^2 - \bar{\lambda}_y^2)}), 1)$ = **0.872**

Design buckling resistance - eq 6.47 $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1}$ = **411.9** kN

$N_{Ed} / N_{b,y,Rd}$ = **0.011**

***PASS - Design buckling resistance exceeds design compression***

Library item: Compression flexural out

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

Critical buckling length $L_{cr,z} = L_{m1_s1_seg1}$ = **3980** mm

Critical buckling force $N_{cr,z} = \pi^2 \times E \times I_z / L_{cr,z}^2$ = **89.4** kN

Slenderness ratio for buckling - eq 6.50 $\bar{\lambda}_z = \sqrt{(A \times f_y / N_{cr,z})}$ = **2.298**

**Check z-z axis flexural buckling resistance - Section 6.3.1.1**

Buckling curve - Table 6.2 b

Imperfection factor - Table 6.1 $\alpha_z$ = **0.34**

Buckling reduction determination factor $\phi_z = 0.5 \times (1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2)$ = **3.497**

Buckling reduction factor - eq 6.49 $\chi_z = \min(1 / (\phi_z + \sqrt{(\phi_z^2 - \bar{\lambda}_z^2)}), 1)$ = **0.163**

Design buckling resistance - eq 6.47 $N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1}$ = **77** kN

$N_{Ed} / N_{b,z,Rd}$ = **0.057**

***PASS - Design buckling resistance exceeds design compression***

Library item: Compression flexural out

**Check torsional and torsional-flexural buckling - Section 6.3.1.4**

Torsional buckling length $L_{cr,T} = L_{m1_s1_seg1_R}$ = **3980** 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 $i_0 = \sqrt{(i_y^2 + i_z^2)}$ = **68.3** mm

Elastic critical torsional buckling force $N_{cr,T} = 1 / i_0^2 \times (G \times I_t + \pi^2 \times E \times I_w / L_{cr,T}^2)$ = **724.3** kN

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

Elastic critical torsional-flexural buckling force

$N_{cr,TF} = N_{cr,y} / (2 \times \beta_T) \times [1 + N_{cr,T} / N_{cr,y} - \sqrt{[(1 - N_{cr,T} / N_{cr,y})^2 + 4 \times (y_0 / i_0)^2 \times N_{cr,T} / N_{cr,y}]}]$ = **724.3** kN

Elastic critical buckling force $N_{cr} = \min(N_{cr,T}, N_{cr,TF})$ = **724.3** kN

Slenderness ratio for torsional buckling - eq 6.52 $\bar{\lambda}_T = \sqrt{[A \times f_y / N_{cr}]}$ = **0.807**

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

Buckling curve - Table 6.2 b

Imperfection factor - Table 6.1 $\alpha_T$ = **0.34**

Buckling reduction determination factor $\phi_T = 0.5 \times (1 + \alpha_T \times (\bar{\lambda}_T - 0.2) + \bar{\lambda}_T^2)$ = **0.929**

Buckling reduction factor - eq 6.49 $\chi_T = \min(1 / (\phi_T + \sqrt{(\phi_T^2 - \bar{\lambda}_T^2)}), 1)$ = **0.72**

Design buckling resistance - eq 6.47 $N_{b,T,Rd} = \chi_T \times A \times f_y / \gamma_{M1}$ = **339.9** kN

$N_{Ed} / N_{b,T,Rd}$ = **0.013**

***PASS - Design buckling resistance***

***exceeds design compression***

Library item: Compression torsional out

**Check design 1881 mm along span**

**Check bending moment - Section 6.2.5**

Design bending moment $M_{y,Ed}$ = **9.8** kNm

Design bending resistance moment - eq 6.13 $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **29.1** kNm

$M_{y,Ed} / M_{c,y,Rd}$ = **0.337**

***PASS - Design bending resistance moment exceeds design bending moment***

Library item: Bending resistance out

**Slenderness ratio for lateral torsional buckling**

Correction factor - Table 6.6 $k_c$ = **0.944**

$C_1 = 1 / k_c^2$ = **1.121**

Poissons ratio $\nu$ = **0.3**

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

Unrestrained effective length $L = 1.0 \times L_{m1_s1_seg1_T}$ = **3980** mm

Elastic critical buckling moment $M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **19.5** kNm

Slenderness ratio for lateral torsional buckling $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **1.222**

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

***$\bar{\lambda}_{LT} > \bar{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored***

Library item: Bending LTB slenderness out

**Check buckling resistance - Section 6.3.2.1**

Buckling curve - Table 6.5 b

Imperfection factor - Table 6.3 $\alpha_{LT}$ = **0.34**

Correction factor for rolled sections $\beta$ = **0.75**

LTB reduction determination factor $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **1.200**

LTB reduction factor - eq 6.57 $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.567**

Modification factor $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) =$ **0.982**

Modified LTB reduction factor - eq 6.58 $\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2) =$ **0.577**

Design buckling resistance moment - eq 6.55 $M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1} =$ **16.8** kNm

$M_{y,Ed} / M_{b,y,Rd} =$ **0.585**

***PASS - Design buckling resistance moment exceeds design bending moment***

Library item: Bending buckling out

**Check bending and axial force - Section 6.2.9**

Bending and axial force check - eq.6.33 & eq.6.34 $N_{y,lim} = \min(0.25 \times N_{pl,Rd}, 0.5 \times h_w \times t_w \times f_y / \gamma_{M0}) =$ **85.3** kN

$N_{Ed} / N_{y,lim} =$ **0.043**

***Allowance need not be made for the effect of the axial force on the plastic resistance moment about the y-y axis***

Library item: Combined axial check out

**Check combined bending and compression - Section 6.3.3**

Equivalent uniform moment factors - Table B.3 $\psi_y = 0$ kNm / 1.928 kNm = **0.000**

$\alpha_y = 1.928$ kNm / 9.79 kNm = **0.197**

$C_{my} = 0.95 + 0.05 \times \alpha_y =$ **0.960**

Library item: Comb check factor calc out

$\psi_{LT} = 0$ kNm / 1.928 kNm = **0.000**

$\alpha_{LT} = 1.928$ kNm / 9.79 kNm = **0.197**

$C_{mLT} = 0.95 + 0.05 \times \alpha_{LT} =$ **0.960**

Library item: Comb check factor calc out

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

Characteristic moment resistance $M_{y,Rk} = W_{pl.y} \times f_y =$ **29.1** kNm

Characteristic moment resistance $M_{z,Rk} = W_{pl.z} \times f_y =$ **6.1** kNm

Characteristic resistance to normal force $N_{Rk} = A \times f_y =$ **472.1** kN

Library item: Combined forces out

Interaction factors

$k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1})) =$ **0.964**

$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1}) =$ **0.993**

Library item: Combined factors out

Interaction formulae - eq 6.61 & eq 6.62

$N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) =$ **0.583**

$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) =$ **0.639**

***PASS - Combined bending and compression checks are satisfied***

Library item: Combined check out

**Check design at end of span**

**Check shear - Section 6.2.6**

Height of web $h_w = h - 2 \times t_f =$ **145.2** mm $\eta =$ **1.000**

$h_w / t_w = 29 = 29 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

***Shear buckling resistance can be ignored***

Library item: Shear slenderness out

Design shear force $V_{y,Ed} =$ **9.4** kN

Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) =$ **966** $mm^2$

Design shear resistance - cl 6.2.6(2) $V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0} =$ **131** kN

$V_{y,Ed} / V_{c,y,Rd} =$ **0.071**

***PASS - Design shear resistance exceeds design shear force***

Library item: Shear resistance out

**Consider Combination 6 - 1.0G + 1.0$\psi_0$Q + 0.5S + 1.0W (Service)**

**Check design 1963 mm along span**

**Check y-y axis deflection - Section 7.2.1**

Maximum deflection $\delta_y =$ **8.1** mm

Allowable deflection $\delta_{y,Allowable} = L_{m1_s1} / 360 =$ **11.1** mm

$\delta_y / \delta_{y,Allowable} =$ **0.737**

***PASS - Allowable deflection exceeds design deflection***

Library item: Deflection out

## Portal Frame Post 1 design

### Section details

Section type IPE 160 (Arcelor)

Steel grade - EN 10025-2:2004 S235

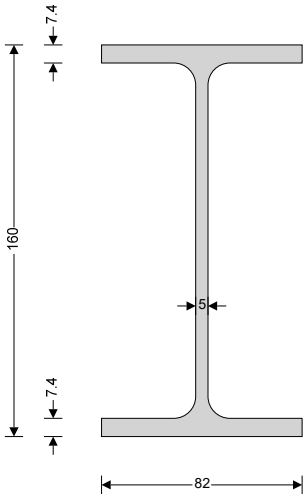
Nominal thickness of element $t_{nom} = \max(t_f, t_w) =$ **7.4** mm

Nominal yield strength $f_y =$ **235** N/mm$^2$

Nominal ultimate tensile strength $f_u =$ **360** N/mm$^2$

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

Library item: Section details out




**IPE 160 (Arcelor)**
Section depth, h, 160 mm
Section breadth, b, 82 mm
Mass of section, Mass, 15.8 kg/m
Flange thickness, $t_f$, 7.4 mm
Web thickness, $t_w$, 5 mm
Root radius, r, 9 mm
Area of section, A, 2009 mm$^2$
Radius of gyration about y-axis, $i_y$, 65.778 mm
Radius of gyration about z-axis, $i_z$, 18.44 mm
Elastic section modulus about y-axis, $W_{el,y}$, 108662 mm$^3$
Elastic section modulus about z-axis, $W_{el,z}$, 16662 mm$^3$
Plastic section modulus about y-axis, $W_{pl,y}$, 123860 mm$^3$
Plastic section modulus about z-axis, $W_{pl,z}$, 26100 mm$^3$
Second moment of area about y-axis, $I_y$, 8692929 mm$^4$
Second moment of area about z-axis, $I_z$, 683146 mm$^4$

### Lateral restraint

Both flanges have lateral restraint at supports only

### Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]} =$ **1.00**

Library item: Class heading out

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

Width of section $c = d =$ **127.2** mm

$\alpha = \min([h / 2 + N_{Ed} / (2 \times t_w \times f_y) - (t_f + r)] / c, 1) =$ **0.534**

$c / t_w = 25.4 = 25.4 \times \varepsilon <= 396 \times \varepsilon / (13 \times \alpha - 1)$ Class 1

Library item: Int bend comp class out

**Outstand flanges - Table 5.2 (sheet 2 of 3)**

Width of section $c = (b - t_w - 2 \times r) / 2 =$ **29.5** mm

$c / t_f = 4 = 4 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Library item: Out flange class out

***Section is class 1***

**Check compression - Section 6.2.4**

Design compression force $N_{Ed} =$ **10.2** kN

Design resistance of section - eq 6.10 $N_{c,Rd} = N_{pl,Rd} = A \times f_y / \gamma_{M0} =$ **472.1** kN

$N_{Ed} / N_{c,Rd} =$ **0.022**

***PASS - Design compression resistance exceeds design compression***

Library item: Compression out

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

Critical buckling length $L_{cr,y} = L_{m2_s1} =$ **2880** mm

Critical buckling force $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2 =$ **2172.2** kN

Slenderness ratio for buckling - eq 6.50 $\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})} =$ **0.466**

**Check y-y axis flexural buckling resistance - Section 6.3.1.1**

Buckling curve - Table 6.2 a

Imperfection factor - Table 6.1 $\alpha_y =$ **0.21**

Buckling reduction determination factor $\phi_y = 0.5 \times (1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2) =$ **0.637**

Buckling reduction factor - eq 6.49 $\chi_y = \min(1 / (\phi_y + \sqrt{(\phi_y^2 - \bar{\lambda}_y^2)}), 1) =$ **0.934**

Design buckling resistance - eq 6.47 $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1} =$ **441.2** kN

$N_{Ed} / N_{b,y,Rd} =$ **0.023**

***PASS - Design buckling resistance exceeds design compression***

Library item: Compression flexural out

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

Critical buckling length $L_{cr,z} = L_{m2_s1_seg1} =$ **2880** mm

Critical buckling force $N_{cr,z} = \pi^2 \times E \times I_z / L_{cr,z}^2 =$ **170.7** kN

Slenderness ratio for buckling - eq 6.50 $\bar{\lambda}_z = \sqrt{(A \times f_y / N_{cr,z})} =$ **1.663**

**Check z-z axis flexural buckling resistance - Section 6.3.1.1**

Buckling curve - Table 6.2 b

Imperfection factor - Table 6.1 $\alpha_z =$ **0.34**

Buckling reduction determination factor $\phi_z = 0.5 \times (1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2) =$ **2.132**

Buckling reduction factor - eq 6.49 $\chi_z = \min(1 / (\phi_z + \sqrt{(\phi_z^2 - \bar{\lambda}_z^2)}), 1) =$ **0.289**

Design buckling resistance - eq 6.47 $N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1} =$ **136.3** kN

$N_{Ed} / N_{b,z,Rd} =$ **0.075**

***PASS - Design buckling resistance exceeds design compression***

Library item: Compression flexural out

**Check torsional and torsional-flexural buckling - Section 6.3.1.4**

Torsional buckling length $L_{cr,T} = L_{m2_s1_seg1_R} =$ **2880** 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 $i_0 = \sqrt{(i_y^2 + i_z^2)} =$ **68.3** mm

Elastic critical torsional buckling force $N_{cr,T} = 1 / i_0^2 \times (G \times I_t + \pi^2 \times E \times I_w / L_{cr,T}^2) =$ **825.7** kN

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

Elastic critical torsional-flexural buckling force

$N_{cr,TF} = N_{cr,y} / (2 \times \beta_T) \times [1 + N_{cr,T} / N_{cr,y} - \sqrt{[(1 - N_{cr,T} / N_{cr,y})^2 + 4 \times (y_0 / i_0)^2 \times N_{cr,T} / N_{cr,y}]}] =$ **825.7** kN

Elastic critical buckling force $N_{cr} = \min(N_{cr,T}, N_{cr,TF}) =$ **825.7** kN

Slenderness ratio for torsional buckling - eq 6.52 $\bar{\lambda}_T = \sqrt{[A \times f_y / N_{cr}]} =$ **0.756**

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

Buckling curve - Table 6.2 b

Imperfection factor - Table 6.1 $\alpha_T =$ **0.34**

Buckling reduction determination factor $\phi_T = 0.5 \times (1 + \alpha_T \times (\bar{\lambda}_T - 0.2) + \bar{\lambda}_T^2) =$ **0.88**

Buckling reduction factor - eq 6.49 $\chi_T = \min(1 / (\phi_T + \sqrt{(\phi_T^2 - \bar{\lambda}_T^2)}), 1) =$ **0.751**

Design buckling resistance - eq 6.47 $N_{b,T,Rd} = \chi_T \times A \times f_y / \gamma_{M1} =$ **354.6** kN

$N_{Ed} / N_{b,T,Rd} =$ **0.029**

***PASS - Design buckling***

***resistance exceeds design compression***

Library item: Compression torsional out

**Check design at start of span**

**Check shear - Section 6.2.6**

Height of web $h_w = h - 2 \times t_f$ = **145.2** mm $\eta$ = **1.000**

$h_w / t_w = 29 = 29 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

***Shear buckling resistance can be ignored***

Library item: Shear slenderness out

Design shear force $V_{y,Ed}$ = **1.4** kN

Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **966** $mm^2$

Design shear resistance - cl 6.2.6(2) $V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **131** kN

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

***PASS - Design shear resistance exceeds design shear force***

Library item: Shear resistance out

**Check design at end of span**

**Check shear - Section 6.2.6**

Height of web $h_w = h - 2 \times t_f$ = **145.2** mm $\eta$ = **1.000**

$h_w / t_w = 29 = 29 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

***Shear buckling resistance can be ignored***

Library item: Shear slenderness out

Design shear force $V_{y,Ed}$ = **1.4** kN

Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **966** $mm^2$

Design shear resistance - cl 6.2.6(2) $V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **131** kN

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

***PASS - Design shear resistance***

***exceeds design shear force***

Library item: Shear resistance out

**Check bending moment - Section 6.2.5**

Design bending moment $M_{y,Ed}$ = **3.9** kNm

Design bending resistance moment - eq 6.13 $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **29.1** kNm

$M_{y,Ed} / M_{c,y,Rd}$ = **0.135**

***PASS - Design bending resistance moment exceeds design bending moment***

Library item: Bending resistance out

**Slenderness ratio for lateral torsional buckling**

Correction factor - For cantilever beams $k_c$ = **1**

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

Poissons ratio $\nu$ = **0.3**

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

Unrestrained effective length $L = 1.0 \times L_{m2_s1_seg1_T}$ = **2880** mm

Elastic critical buckling moment $M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **25.6** kNm

Slenderness ratio for lateral torsional buckling $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **1.065**

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

***$\bar{\lambda}_{LT} > \bar{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored***

Library item: Bending LTB slenderness out

**Check buckling resistance - Section 6.3.2.1**

Buckling curve - Table 6.5 b

Imperfection factor - Table 6.3 $\alpha_{LT}$ = **0.34**

Correction factor for rolled sections $\beta$ = **0.75**

LTB reduction determination factor $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **1.039**

LTB reduction factor - eq 6.57 $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.660**

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 $\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \overline{\lambda}_{LT}^2) =$ **0.660**

Design buckling resistance moment - eq 6.55 $M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1} =$ **19.2** kNm

$M_{y,Ed} / M_{b,y,Rd} =$ **0.204**

***PASS - Design buckling resistance moment exceeds design bending moment***

Library item: Bending buckling out

**Check bending and axial force - Section 6.2.9**

Bending and axial force check - eq.6.33 & eq.6.34 $N_{y,lim} = \min(0.25 \times N_{pl,Rd}, 0.5 \times h_w \times t_w \times f_y / \gamma_{M0}) =$ **85.3** kN

$N_{Ed} / N_{y,lim} =$ **0.115**

***Allowance need not be made for the effect of the axial force on the plastic resistance moment about the y-y axis***

Library item: Combined axial check out

**Check combined bending and compression - Section 6.3.3**

Equivalent uniform moment factors - Table B.3 $\psi_y =$ 0 kNm / 3.92 kNm = **0.000**

$\alpha_y =$ 1.96 kNm / 3.92 kNm = **0.500**

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

Library item: Comb check factor calc out

$\psi_{LT} =$ 0 kNm / 3.92 kNm = **0.000**

$\alpha_{LT} =$ 1.96 kNm / 3.92 kNm = **0.500**

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

Library item: Comb check factor calc out

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

Characteristic moment resistance $M_{y,Rk} = W_{pl.y} \times f_y =$ **29.1** kNm

Characteristic moment resistance $M_{z,Rk} = W_{pl.z} \times f_y =$ **6.1** kNm

Characteristic resistance to normal force $N_{Rk} = A \times f_y =$ **472.1** kN

Library item: Combined forces out

Interaction factors $k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1})) =$ **0.604**

$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1}) =$ **0.979**

Library item: Combined factors out

Interaction formulae - eq 6.61 & eq 6.62 $N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) =$ **0.145**

$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) =$ **0.272**

***PASS - Combined bending and compression checks are satisfied***

Library item: Combined check out

**Consider Combination 6 - 1.0G + 1.0$\psi_0$Q + 0.5S + 1.0W (Service)**

**Check design at end of span**

**Check y-y axis deflection - Section 7.2.1**

Maximum deflection $\delta_y =$ **14.1** mm

Allowable deflection $\delta_{y,Allowable} = L_{m2_s1} / 180 =$ **16** mm

$\delta_y / \delta_{y,Allowable} =$ **0.883**

***PASS - Allowable deflection exceeds design deflection***

Library item: Deflection out

**Portal Frame Post 2 design**

**Section details**

Section type IPE 160 (Arcelor)

Steel grade - EN 10025-2:2004 S235

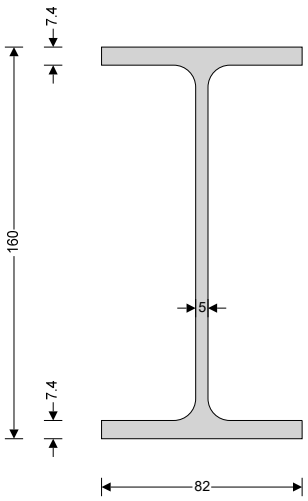
Nominal thickness of element $t_{nom} = \max(t_f, t_w) =$ **7.4** mm

Nominal yield strength $f_y =$ **235** N/mm$^2$

Nominal ultimate tensile strength $f_u =$ **360** N/mm$^2$

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

Library item: Section details out




**IPE 160 (Arcelor)**
Section depth, h, 160 mm
Section breadth, b, 82 mm
Mass of section, Mass, 15.8 kg/m
Flange thickness, $t_f$, 7.4 mm
Web thickness, $t_w$, 5 mm
Root radius, r, 9 mm
Area of section, A, 2009 mm$^2$
Radius of gyration about y-axis, $i_y$, 65.778 mm
Radius of gyration about z-axis, $i_z$, 18.44 mm
Elastic section modulus about y-axis, $W_{el,y}$, 108662 mm$^3$
Elastic section modulus about z-axis, $W_{el,z}$, 16662 mm$^3$
Plastic section modulus about y-axis, $W_{pl,y}$, 123860 mm$^3$
Plastic section modulus about z-axis, $W_{pl,z}$, 26100 mm$^3$
Second moment of area about y-axis, $I_y$, 8692929 mm$^4$
Second moment of area about z-axis, $I_z$, 683146 mm$^4$

**Lateral restraint**

Both flanges have lateral restraint at supports only

**Classification of cross sections - Section 5.5**

$\varepsilon = \sqrt{[235\ N/mm^2 / f_y]} =$ **1.00**

Library item: Class heading out

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

Width of section $c = d =$ **127.2** mm

$\alpha = \min([h / 2 + N_{Ed} / (2 \times t_w \times f_y) - (t_f + r)] / c, 1) =$ **0.539**

$c / t_w = 25.4 = 25.4 \times \varepsilon <= 396 \times \varepsilon / (13 \times \alpha - 1)$ Class 1

Library item: Int bend comp class out

**Outstand flanges - Table 5.2 (sheet 2 of 3)**

Width of section $c = (b - t_w - 2 \times r) / 2 =$ **29.5** mm

$c / t_f = 4 = 4 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Library item: Out flange class out

***Section is class 1***

**Check compression - Section 6.2.4**

Design compression force $N_{Ed} =$ **11.6** kN

Design resistance of section - eq 6.10 $N_{c,Rd} = N_{pl,Rd} = A \times f_y / \gamma_{M0} =$ **472.1** kN

$N_{Ed} / N_{c,Rd} =$ **0.025**

***PASS - Design compression resistance exceeds design compression***

Library item: Compression out

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

Critical buckling length $L_{cr,y} = L_{m3_s1}$ = **2880** mm

Critical buckling force $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2$ = **2172.2** kN

Slenderness ratio for buckling - eq 6.50 $\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})}$ = **0.466**

**Check y-y axis flexural buckling resistance - Section 6.3.1.1**

Buckling curve - Table 6.2 a

Imperfection factor - Table 6.1 $\alpha_y$ = **0.21**

Buckling reduction determination factor $\phi_y = 0.5 \times (1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2)$ = **0.637**

Buckling reduction factor - eq 6.49 $\chi_y = \min(1 / (\phi_y + \sqrt{(\phi_y^2 - \bar{\lambda}_y^2)}), 1)$ = **0.934**

Design buckling resistance - eq 6.47 $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1}$ = **441.2** kN

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

***PASS - Design buckling resistance exceeds design compression***

Library item: Compression flexural out

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

Critical buckling length $L_{cr,z} = L_{m3_s1_seg1}$ = **2880** mm

Critical buckling force $N_{cr,z} = \pi^2 \times E \times I_z / L_{cr,z}^2$ = **170.7** kN

Slenderness ratio for buckling - eq 6.50 $\bar{\lambda}_z = \sqrt{(A \times f_y / N_{cr,z})}$ = **1.663**

**Check z-z axis flexural buckling resistance - Section 6.3.1.1**

Buckling curve - Table 6.2 b

Imperfection factor - Table 6.1 $\alpha_z$ = **0.34**

Buckling reduction determination factor $\phi_z = 0.5 \times (1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2)$ = **2.132**

Buckling reduction factor - eq 6.49 $\chi_z = \min(1 / (\phi_z + \sqrt{(\phi_z^2 - \bar{\lambda}_z^2)}), 1)$ = **0.289**

Design buckling resistance - eq 6.47 $N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1}$ = **136.3** kN

$N_{Ed} / N_{b,z,Rd}$ = **0.085**

***PASS - Design buckling resistance exceeds design compression***

Library item: Compression flexural out

**Check torsional and torsional-flexural buckling - Section 6.3.1.4**

Torsional buckling length $L_{cr,T} = L_{m3_s1_seg1_R}$ = **2880** 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 $i_0 = \sqrt{(i_y^2 + i_z^2)}$ = **68.3** mm

Elastic critical torsional buckling force $N_{cr,T} = 1 / i_0^2 \times (G \times I_t + \pi^2 \times E \times I_w / L_{cr,T}^2)$ = **825.7** kN

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

Elastic critical torsional-flexural buckling force

$N_{cr,TF} = N_{cr,y} / (2 \times \beta_T) \times [1 + N_{cr,T} / N_{cr,y} - \sqrt{[(1 - N_{cr,T} / N_{cr,y})^2 + 4 \times (y_0 / i_0)^2 \times N_{cr,T} / N_{cr,y}]}$ = **825.7** kN

Elastic critical buckling force $N_{cr} = \min(N_{cr,T}, N_{cr,TF})$ = **825.7** kN

Slenderness ratio for torsional buckling - eq 6.52 $\bar{\lambda}_T = \sqrt{[A \times f_y / N_{cr}]}$ = **0.756**

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

Buckling curve - Table 6.2 b

Imperfection factor - Table 6.1 $\alpha_T$ = **0.34**

Buckling reduction determination factor $\phi_T = 0.5 \times (1 + \alpha_T \times (\bar{\lambda}_T - 0.2) + \bar{\lambda}_T^2)$ = **0.88**

Buckling reduction factor - eq 6.49 $\chi_T = \min(1 / (\phi_T + \sqrt{(\phi_T^2 - \bar{\lambda}_T^2)}), 1)$ = **0.751**

Design buckling resistance - eq 6.47 $N_{b,T,Rd} = \chi_T \times A \times f_y / \gamma_{M1}$ = **354.6** kN

$N_{Ed} / N_{b,T,Rd}$ = **0.033**

***PASS - Design buckling resistance exceeds design compression***

Library item: Compression torsional out

**Check design at start of span**

**Check shear - Section 6.2.6**

Height of web $h_w = h - 2 \times t_f$ = **145.2** mm $\eta$ = **1.000**

$h_w / t_w = 29 = 29 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

***Shear buckling resistance can be ignored***

Library item: Shear slenderness out

Design shear force $V_{y,Ed}$ = **4.3** kN

Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **966** mm$^2$

Design shear resistance - cl 6.2.6(2) $V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **131** kN

$V_{y,Ed} / V_{c,y,Rd}$ = **0.033**

***PASS - Design shear resistance exceeds design shear force***

Library item: Shear resistance out

**Check design at end of span**

**Check shear - Section 6.2.6**

Height of web $h_w = h - 2 \times t_f$ = **145.2** mm $\eta$ = **1.000**

$h_w / t_w = 29 = 29 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

***Shear buckling resistance can be ignored***

Library item: Shear slenderness out

Design shear force $V_{y,Ed}$ = **3.9** kN

Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **966** $mm^2$

Design shear resistance - cl 6.2.6(2) $V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **131** kN

$V_{y,Ed} / V_{c,y,Rd}$ = **0.03**

***PASS - Design shear resistance exceeds design shear force***

Library item: Shear resistance out

**Check bending moment - Section 6.2.5**

Design bending moment $M_{y,Ed}$ = **11.8** kNm

Design bending resistance moment - eq 6.13 $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **29.1** kNm

$M_{y,Ed} / M_{c,y,Rd}$ = **0.406**

***PASS - Design bending resistance moment exceeds design bending moment***

Library item: Bending resistance out

**Slenderness ratio for lateral torsional buckling**

Correction factor - For cantilever beams $k_c$ = **1**

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

Poissons ratio $\nu$ = **0.3**

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

Unrestrained effective length $L = 1.0 \times L_{m3_s1_seg1_T}$ = **2880** mm

Elastic critical buckling moment $M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **25.6** kNm

Slenderness ratio for lateral torsional buckling $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **1.065**

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

***$\bar{\lambda}_{LT} > \bar{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored***

Library item: Bending LTB slenderness out

**Check buckling resistance - Section 6.3.2.1**

Buckling curve - Table 6.5 b

Imperfection factor - Table 6.3 $\alpha_{LT}$ = **0.34**

Correction factor for rolled sections $\beta$ = **0.75**

LTB reduction determination factor $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **1.039**

LTB reduction factor - eq 6.57 $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.660**

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 $\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.660**

Design buckling resistance moment - eq 6.55 $M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1}$ = **19.2** kNm

$M_{y,Ed} / M_{b,y,Rd}$ = **0.615**

***PASS - Design buckling resistance moment exceeds design bending moment***

Library item: Bending buckling out

**Check bending and axial force - Section 6.2.9**

Bending and axial force check - eq.6.33 & eq.6.34 $N_{y,lim} = \min(0.25 \times N_{pl,Rd}, 0.5 \times h_w \times t_w \times f_y / \gamma_{M0})$ = **85.3** kN

$N_{Ed} / N_{y,lim}$ = **0.129**

***Allowance need not be made for the effect of the axial force on the plastic resistance moment about the y-y axis***

Library item: Combined axial check out

**Check combined bending and compression - Section 6.3.3**

Equivalent uniform moment factors - Table B.3 $\psi_y$ = 0 kNm / 11.806 kNm = **0.000**

$\alpha_y$ = 6.043 kNm / 11.806 kNm = **0.512**

$C_{my} = \max(0.2 + 0.8 \times \alpha_y, 0.4)$ = **0.609**

Library item: Comb check factor calc out

$\psi_{LT}$ = 0 kNm / 11.806 kNm = **0.000**

$\alpha_{LT}$ = 6.043 kNm / 11.806 kNm = **0.512**

$C_{mLT} = \max(0.2 + 0.8 \times \alpha_{LT}, 0.4)$ = **0.609**

Library item: Comb check factor calc out

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

Characteristic moment resistance $M_{y,Rk} = W_{pl.y} \times f_y$ = **29.1** kNm

Characteristic moment resistance $M_{z,Rk} = W_{pl.z} \times f_y$ = **6.1** kNm

Characteristic resistance to normal force $N_{Rk} = A \times f_y$ = **472.1** kN

Library item: Combined forces out

Interaction factors $k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}))$ = **0.614**

$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1})$ = **0.978**

Library item: Combined factors out

Interaction formulae - eq 6.61 & eq 6.62

$N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.402**

$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.682**

***PASS - Combined bending and compression checks are satisfied***

Library item: Combined check out

**Consider Combination 6 - 1.0G + 1.0$\psi_0$Q + 0.5S + 1.0W (Service)**

**Check design at end of span**

**Check y-y axis deflection - Section 7.2.1**

Maximum deflection $\delta_y$ = **15.7** mm

Allowable deflection $\delta_{y,Allowable} = L_{m3_s1} / 180$ = **16** mm

$\delta_y / \delta_{y,Allowable}$ = **0.983**

***PASS - Allowable deflection exceeds design deflection***

Library item: Deflection out

**Portal Frame Rafter 1 design**

**Section details**

Section type IPE 160 (Arcelor)

Steel grade - EN 10025-2:2004 S235

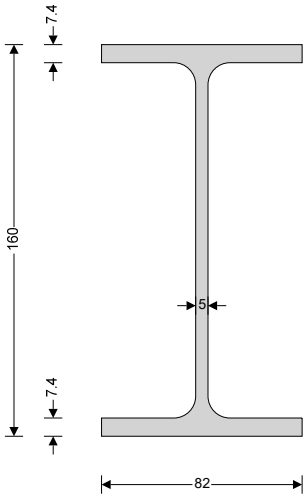
Nominal thickness of element $t_{nom} = \max(t_f, t_w)$ = **7.4** mm

Nominal yield strength $f_y$ = **235** N/mm$^2$

Nominal ultimate tensile strength $f_u$ = **360** N/mm$^2$

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

Library item: Section details out




**IPE 160 (Arcelor)**
Section depth, h, 160 mm
Section breadth, b, 82 mm
Mass of section, Mass, 15.8 kg/m
Flange thickness, $t_f$, 7.4 mm
Web thickness, $t_w$, 5 mm
Root radius, r, 9 mm
Area of section, A, 2009 mm$^2$
Radius of gyration about y-axis, $i_y$, 65.778 mm
Radius of gyration about z-axis, $i_z$, 18.44 mm
Elastic section modulus about y-axis, $W_{el,y}$, 108662 mm$^3$
Elastic section modulus about z-axis, $W_{el,z}$, 16662 mm$^3$
Plastic section modulus about y-axis, $W_{pl,y}$, 123860 mm$^3$
Plastic section modulus about z-axis, $W_{pl,z}$, 26100 mm$^3$
Second moment of area about y-axis, $I_y$, 8692929 mm$^4$
Second moment of area about z-axis, $I_z$, 683146 mm$^4$

**Lateral restraint**

Both flanges have lateral restraint at supports only

**Classification of cross sections - Section 5.5**

$\varepsilon = \sqrt{[235\ N/mm^2 / f_y]} =$ **1.00**

Library item: Class heading out

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

Width of section $c = d =$ **127.2** mm

$\alpha = \min([h / 2 + N_{Ed} / (2 \times t_w \times f_y) - (t_f + r)] / c, 1) =$ **0.515**

$c / t_w = 25.4 = 25.4 \times \varepsilon <= 396 \times \varepsilon / (13 \times \alpha - 1)$ Class 1

Library item: Int bend comp class out

**Outstand flanges - Table 5.2 (sheet 2 of 3)**

Width of section $c = (b - t_w - 2 \times r) / 2 =$ **29.5** mm

$c / t_f = 4 = 4 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Library item: Out flange class out

***Section is class 1***

**Check compression - Section 6.2.4**

Design compression force $N_{Ed} =$ **4.4** kN

Design resistance of section - eq 6.10 $N_{c,Rd} = N_{pl,Rd} = A \times f_y / \gamma_{M0} =$ **472.1** kN

$N_{Ed} / N_{c,Rd} =$ **0.009**

***PASS - Design compression resistance exceeds design compression***

Library item: Compression out

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

Critical buckling length $L_{cr,y} = L_{m4_s1} =$ **2422** mm

Critical buckling force $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2 =$ **3071.5** kN

Slenderness ratio for buckling - eq 6.50 $\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})} =$ **0.392**

**Check y-y axis flexural buckling resistance - Section 6.3.1.1**

Buckling curve - Table 6.2 a

Imperfection factor - Table 6.1 $\alpha_y =$ **0.21**

Buckling reduction determination factor $\phi_y = 0.5 \times (1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2) =$ **0.597**

Buckling reduction factor - eq 6.49 $\chi_y = \min(1 / (\phi_y + \sqrt{(\phi_y^2 - \bar{\lambda}_y^2)}), 1) =$ **0.955**

Design buckling resistance - eq 6.47 $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1} =$ **450.8** kN

$N_{Ed} / N_{b,y,Rd} =$ **0.01**

***PASS - Design buckling resistance exceeds design compression***

Library item: Compression flexural out

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

Critical buckling length $L_{cr,z} = L_{m4_s1_seg1}$ = **2422** mm

Critical buckling force $N_{cr,z} = \pi^2 \times E \times I_z / L_{cr,z}^2$ = **241.4** kN

Slenderness ratio for buckling - eq 6.50 $\bar{\lambda}_z = \sqrt{(A \times f_y / N_{cr,z})}$ = **1.399**

**Check z-z axis flexural buckling resistance - Section 6.3.1.1**

Buckling curve - Table 6.2 b

Imperfection factor - Table 6.1 $\alpha_z$ = **0.34**

Buckling reduction determination factor $\phi_z = 0.5 \times (1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2)$ = **1.682**

Buckling reduction factor - eq 6.49 $\chi_z = \min(1 / (\phi_z + \sqrt{(\phi_z^2 - \bar{\lambda}_z^2)}), 1)$ = **0.382**

Design buckling resistance - eq 6.47 $N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1}$ = **180.5** kN

$N_{Ed} / N_{b,z,Rd}$ = **0.024**

***PASS - Design buckling resistance exceeds design compression***

Library item: Compression flexural out

**Check torsional and torsional-flexural buckling - Section 6.3.1.4**

Torsional buckling length $L_{cr,T} = L_{m4_s1_seg1_R}$ = **2422** 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 $i_0 = \sqrt{(i_y^2 + i_z^2)}$ = **68.3** mm

Elastic critical torsional buckling force $N_{cr,T} = 1 / i_0^2 \times (G \times I_t + \pi^2 \times E \times I_w / L_{cr,T}^2)$ = **913.9** kN

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

Elastic critical torsional-flexural buckling force

$N_{cr,TF} = N_{cr,y} / (2 \times \beta_T) \times [1 + N_{cr,T} / N_{cr,y} - \sqrt{[(1 - N_{cr,T} / N_{cr,y})^2 + 4 \times (y_0 / i_0)^2 \times N_{cr,T} / N_{cr,y}]}]$ = **913.9** kN

Elastic critical buckling force $N_{cr} = \min(N_{cr,T}, N_{cr,TF})$ = **913.9** kN

Slenderness ratio for torsional buckling - eq 6.52 $\bar{\lambda}_T = \sqrt{[A \times f_y / N_{cr}]}$ = **0.719**

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

Buckling curve - Table 6.2 b

Imperfection factor - Table 6.1 $\alpha_T$ = **0.34**

Buckling reduction determination factor $\phi_T = 0.5 \times (1 + \alpha_T \times (\bar{\lambda}_T - 0.2) + \bar{\lambda}_T^2)$ = **0.847**

Buckling reduction factor - eq 6.49 $\chi_T = \min(1 / (\phi_T + \sqrt{(\phi_T^2 - \bar{\lambda}_T^2)}), 1)$ = **0.773**

Design buckling resistance - eq 6.47 $N_{b,T,Rd} = \chi_T \times A \times f_y / \gamma_{M1}$ = **365** kN

$N_{Ed} / N_{b,T,Rd}$ = **0.012**

***PASS - Design buckling resistance exceeds design compression***

Library item: Compression torsional out

**Check design 1285 mm along span**

**Check bending moment - Section 6.2.5**

Design bending moment $M_{y,Ed}$ = **6.7** kNm

Design bending resistance moment - eq 6.13 $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0}$ = **29.1** kNm

$M_{y,Ed} / M_{c,y,Rd}$ = **0.23**

***PASS - Design bending resistance moment exceeds design bending moment***

Library item: Bending resistance out

**Slenderness ratio for lateral torsional buckling**

Correction factor - Table 6.6 $k_c$ = **0.97**

$C_1 = 1 / k_c^2$ = **1.063**

Poissons ratio $\nu$ = **0.3**

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

Unrestrained effective length $L = 1.0 \times L_{m4_s1_seg1_T}$ = **2422** mm

Elastic critical buckling moment $M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **34.1** kNm

Slenderness ratio for lateral torsional buckling $\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})}$ = **0.924**

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

$\bar{\lambda}_{LT} > \bar{\lambda}_{LT,0}$ -
***Lateral torsional buckling cannot be ignored***

Library item: Bending LTB slenderness out

**Check buckling resistance - Section 6.3.2.1**

Buckling curve - Table 6.5 b

Imperfection factor - Table 6.3 $\alpha_{LT}$ = **0.34**

Correction factor for rolled sections $\beta$ = **0.75**

LTB reduction determination factor $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **0.909**

LTB reduction factor - eq 6.57 $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.746**

Modification factor $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.985**

Modified LTB reduction factor - eq 6.58 $\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.757**

Design buckling resistance moment - eq 6.55 $M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1}$ = **22** kNm

$M_{y,Ed} / M_{b,y,Rd}$ = **0.304**

***PASS - Design buckling resistance moment exceeds design bending moment***

Library item: Bending buckling out

**Check bending and axial force - Section 6.2.9**

Bending and axial force check - eq.6.33 & eq.6.34 $N_{y,lim} = \min(0.25 \times N_{pl,Rd}, 0.5 \times h_w \times t_w \times f_y / \gamma_{M0})$ = **85.3** kN

$N_{Ed} / N_{y,lim}$ = **0.04**

***Allowance need not be made for the effect of the axial force on the plastic resistance moment about the y-y axis***

Library item: Combined axial check out

**Check combined bending and compression - Section 6.3.3**

Equivalent uniform moment factors - Table B.3 $\psi_y$ = 3.079 kNm / 3.869 kNm = **0.796**

$\alpha_y$ = 3.869 kNm / 6.691 kNm = **0.578**

$C_{my} = 0.95 + 0.05 \times \alpha_y =$ **0.979**

Library item: Comb check factor calc out

$\psi_{LT}$ = 3.079 kNm / 3.869 kNm = **0.796**

$\alpha_{LT}$ = 3.869 kNm / 6.691 kNm = **0.578**

$C_{mLT} = 0.95 + 0.05 \times \alpha_{LT} =$ **0.979**

Library item: Comb check factor calc out

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

Characteristic moment resistance $M_{y,Rk} = W_{pl.y} \times f_y =$ **29.1** kNm

Characteristic moment resistance $M_{z,Rk} = W_{pl.z} \times f_y =$ **6.1** kNm

Characteristic resistance to normal force $N_{Rk} = A \times f_y =$ **472.1** kN

Library item: Combined forces out

Interaction factors $k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1})) =$ **0.980**

$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1}) =$ **0.997**

Library item: Combined factors out

Interaction formulae - eq 6.61 & eq 6.62 $N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) =$ **0.31**

$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) =$ **0.327**

***PASS - Combined bending and compression checks are satisfied***

Library item: Combined check out

**Consider Combination 6 - 1.0G + 1.0$\psi_0$Q + 0.5S + 1.0W (Service)**

**Check design 1775 mm along span**

**Check y-y axis deflection - Section 7.2.1**

Maximum deflection $\delta_y =$ **7.3** mm

Allowable deflection $\delta_{y,Allowable} = L_{m4_s1} / 300 =$ **8.1** mm

$\delta_y / \delta_{y,Allowable} =$ **0.907**

***PASS - Allowable deflection exceeds design deflection***

Library item: Deflection out

## Portal Fraem Rafter 2 design

### Section details

Section type IPE 160 (Arcelor)

Steel grade - EN 10025-2:2004 S235

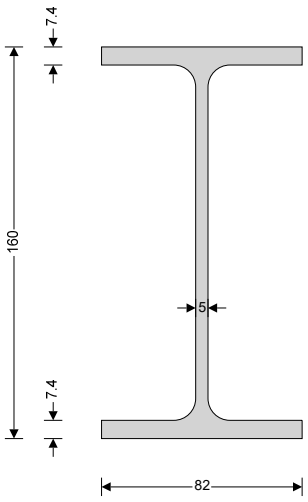
Nominal thickness of element $t_{nom} = \max(t_f, t_w) =$ **7.4** mm

Nominal yield strength $f_y =$ **235** N/mm$^2$

Nominal ultimate tensile strength $f_u =$ **360** N/mm$^2$

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

Library item: Section details out




**IPE 160 (Arcelor)**
Section depth, h, 160 mm
Section breadth, b, 82 mm
Mass of section, Mass, 15.8 kg/m
Flange thickness, $t_f$, 7.4 mm
Web thickness, $t_w$, 5 mm
Root radius, r, 9 mm
Area of section, A, 2009 mm$^2$
Radius of gyration about y-axis, $i_y$, 65.778 mm
Radius of gyration about z-axis, $i_z$, 18.44 mm
Elastic section modulus about y-axis, $W_{el,y}$, 108662 mm$^3$
Elastic section modulus about z-axis, $W_{el,z}$, 16662 mm$^3$
Plastic section modulus about y-axis, $W_{pl,y}$, 123860 mm$^3$
Plastic section modulus about z-axis, $W_{pl,z}$, 26100 mm$^3$
Second moment of area about y-axis, $I_y$, 8692929 mm$^4$
Second moment of area about z-axis, $I_z$, 683146 mm$^4$

### Lateral restraint

Both flanges have lateral restraint at supports only

### Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]} =$ **1.00**

Library item: Class heading out

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

Width of section c = d = **127.2** mm

$\alpha = \min([h / 2 + N_{Ed} / (2 \times t_w \times f_y) - (t_f + r)] / c, 1) =$ **0.522**

$c / t_w = 25.4 = 25.4 \times \varepsilon <= 396 \times \varepsilon / (13 \times \alpha - 1)$ Class 1

Library item: Int bend comp class out

### Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section $c = (b - t_w - 2 \times r) / 2 =$ **29.5** mm

$c / t_f = 4 = 4 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Library item: Out flange class out

***Section is class 1***

**Check compression - Section 6.2.4**

Design compression force $N_{Ed}$ = **4.6** kN

Design resistance of section - eq 6.10 $N_{c,Rd} = N_{pl,Rd} = A \times f_y / \gamma_{M0}$ = **472.1** kN

$N_{Ed} / N_{c,Rd}$ = **0.01**

***PASS - Design compression resistance exceeds design compression***

Library item: Compression out

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

Critical buckling length $L_{cr,y} = L_{m5_s1}$ = **2422** mm

Critical buckling force $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2$ = **3071.5** kN

Slenderness ratio for buckling - eq 6.50 $\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})}$ = **0.392**

**Check y-y axis flexural buckling resistance - Section 6.3.1.1**

Buckling curve - Table 6.2 a

Imperfection factor - Table 6.1 $\alpha_y$ = **0.21**

Buckling reduction determination factor $\phi_y = 0.5 \times (1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2)$ = **0.597**

Buckling reduction factor - eq 6.49 $\chi_y = \min(1 / (\phi_y + \sqrt{(\phi_y^2 - \bar{\lambda}_y^2)}), 1)$ = **0.955**

Design buckling resistance - eq 6.47 $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1}$ = **450.8** kN

$N_{Ed} / N_{b,y,Rd}$ = **0.01**

***PASS - Design buckling resistance exceeds design compression***

Library item: Compression flexural out

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

Critical buckling length $L_{cr,z} = L_{m5_s1_seg1}$ = **2422** mm

Critical buckling force $N_{cr,z} = \pi^2 \times E \times I_z / L_{cr,z}^2$ = **241.4** kN

Slenderness ratio for buckling - eq 6.50 $\bar{\lambda}_z = \sqrt{(A \times f_y / N_{cr,z})}$ = **1.399**

**Check z-z axis flexural buckling resistance - Section 6.3.1.1**

Buckling curve - Table 6.2 b

Imperfection factor - Table 6.1 $\alpha_z$ = **0.34**

Buckling reduction determination factor $\phi_z = 0.5 \times (1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2)$ = **1.682**

Buckling reduction factor - eq 6.49 $\chi_z = \min(1 / (\phi_z + \sqrt{(\phi_z^2 - \bar{\lambda}_z^2)}), 1)$ = **0.382**

Design buckling resistance - eq 6.47 $N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1}$ = **180.5** kN

$N_{Ed} / N_{b,z,Rd}$ = **0.025**

***PASS - Design buckling resistance exceeds design compression***

Library item: Compression flexural out

**Check torsional and torsional-flexural buckling - Section 6.3.1.4**

Torsional buckling length $L_{cr,T} = L_{m5_s1_seg1_R}$ = **2422** 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 $i_0 = \sqrt{(i_y^2 + i_z^2)}$ = **68.3** mm

Elastic critical torsional buckling force $N_{cr,T} = 1 / i_0^2 \times (G \times I_t + \pi^2 \times E \times I_w / L_{cr,T}^2)$ = **913.9** kN

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

Elastic critical torsional-flexural buckling force

$N_{cr,TF} = N_{cr,y} / (2 \times \beta_T) \times [1 + N_{cr,T} / N_{cr,y} - \sqrt{[(1 - N_{cr,T} / N_{cr,y})^2 + 4 \times (y_0 / i_0)^2 \times N_{cr,T} / N_{cr,y}]}]$ = **913.9** kN

Elastic critical buckling force $N_{cr} = \min(N_{cr,T}, N_{cr,TF})$ = **913.9** kN

Slenderness ratio for torsional buckling - eq 6.52 $\bar{\lambda}_T = \sqrt{[A \times f_y / N_{cr}]}$ = **0.719**

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

Buckling curve - Table 6.2 b

Imperfection factor - Table 6.1 $\alpha_T$ = **0.34**

Buckling reduction determination factor $\phi_T = 0.5 \times (1 + \alpha_T \times (\bar{\lambda}_T - 0.2) + \bar{\lambda}_T^2)$ = **0.847**

Buckling reduction factor - eq 6.49 $\chi_T = \min(1 / (\phi_T + \sqrt{(\phi_T^2 - \bar{\lambda}_T^2)}), 1)$ = **0.773**

Design buckling resistance - eq 6.47 $N_{b,T,Rd} = \chi_T \times A \times f_y / \gamma_{M1}$ = **365** kN

$N_{Ed} / N_{b,T,Rd}$ = **0.013**

***PASS - Design buckling resistance exceeds design compression***

Library item: Compression torsional out

**Check design at start of span**

**Check shear - Section 6.2.6**

Height of web $h_w = h - 2 \times t_f$ = **145.2** mm $\eta$ = **1.000**

$h_w / t_w = 29 = 29 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

***Shear buckling resistance can be ignored***

Library item: Shear slenderness out

Design shear force $V_{y,Ed}$ = **3.2** kN

Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **966** mm$^2$

Design shear resistance - cl 6.2.6(2) $V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **131** kN

$V_{y,Ed} / V_{c,y,Rd}$ = **0.025**

***PASS - Design shear resistance exceeds design shear force***

Library item: Shear resistance out

**Check bending moment - Section 6.2.5**

Design bending moment $M_{y,Ed}$ = **3.9** kNm

Design bending resistance moment - eq 6.13 $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **29.1** kNm

$M_{y,Ed} / M_{c,y,Rd}$ = **0.133**

***PASS - Design bending resistance moment exceeds design bending moment***

Library item: Bending resistance out

**Slenderness ratio for lateral torsional buckling**

Correction factor - Table 6.6 $k_c$ = **0.602**

$C_1 = 1 / k_c^2$ = **2.755**

Poissons ratio $\nu$ = **0.3**

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

Unrestrained effective length $L = 1.0 \times L_{m5_s1_seg1_T}$ = **2422** mm

Elastic critical buckling moment $M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **88.4** kNm

Slenderness ratio for lateral torsional buckling $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})} =$ **0.574**

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

***$\bar{\lambda}_{LT} > \bar{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored***

Library item: Bending LTB slenderness out

**Check buckling resistance - Section 6.3.2.1**

Buckling curve - Table 6.5 b

Imperfection factor - Table 6.3 $\alpha_{LT} =$ **0.34**

Correction factor for rolled sections $\beta =$ **0.75**

LTB reduction determination factor $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] =$ **0.653**

LTB reduction factor - eq 6.57 $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) =$ **0.929**

Modification factor $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) =$ **0.822**

Modified LTB reduction factor - eq 6.58 $\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2) =$ **1.000**

Design buckling resistance moment - eq 6.55 $M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1} =$ **29.1** kNm

$M_{y,Ed} / M_{b,y,Rd} =$ **0.133**

***PASS - Design buckling resistance moment exceeds design bending moment***

Library item: Bending buckling out

**Check bending and axial force - Section 6.2.9**

Bending and axial force check - eq.6.33 & eq.6.34 $N_{y,lim} = \min(0.25 \times N_{pl,Rd}, 0.5 \times h_w \times t_w \times f_y / \gamma_{M0}) =$ **85.3** kN

$N_{Ed} / N_{y,lim} =$ **0.054**

***Allowance need not be made for the effect of the axial force on the plastic resistance moment about the y-y axis***

Library item: Combined axial check out

**Check combined bending and compression - Section 6.3.3**

Equivalent uniform moment factors - Table B.3 $\psi_y$ = 3.869 kNm / -11.806 kNm = **-0.328**

$\alpha_y$ = -1.994 kNm / -11.806 kNm = **0.169**

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

Library item: Comb check factor calc out

$\psi_{LT}$ = 3.869 kNm / -11.806 kNm = **-0.328**

$\alpha_{LT}$ = -1.994 kNm / -11.806 kNm = **0.169**

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

Library item: Comb check factor calc out

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

Characteristic moment resistance $M_{y,Rk} = W_{pl.y} \times f_y$ = **29.1** kNm

Characteristic moment resistance $M_{z,Rk} = W_{pl.z} \times f_y$ = **6.1** kNm

Characteristic resistance to normal force $N_{Rk} = A \times f_y$ = **472.1** kN

Library item: Combined forces out

Interaction factors

$k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}))$ = **0.401**

$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1})$ = **0.983**

Library item: Combined factors out

Interaction formulae - eq 6.61 & eq 6.62

$N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.068**

$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.166**

***PASS - Combined bending and compression checks are satisfied***

Library item: Combined check out

**Check design at end of span**

**Check shear - Section 6.2.6**

Height of web $h_w = h - 2 \times t_f$ = **145.2** mm $\eta$ = **1.000**

$h_w / t_w = 29 = 29 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

***Shear buckling resistance can be ignored***

Library item: Shear slenderness out

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

Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **966** mm$^2$

Design shear resistance - cl 6.2.6(2) $V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **131** kN

$V_{y,Ed} / V_{c,y,Rd}$ = **0.074**

***PASS - Design shear resistance exceeds design shear force***

Library item: Shear resistance out

**Check bending moment - Section 6.2.5**

Design bending moment $M_{y,Ed}$ = **11.8** kNm

Design bending resistance moment - eq 6.13 $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **29.1** kNm

$M_{y,Ed} / M_{c,y,Rd}$ = **0.406**

***PASS - Design bending resistance moment exceeds design bending moment***

Library item: Bending resistance out

**Slenderness ratio for lateral torsional buckling**

Correction factor - Table 6.6 $k_c$ = **0.602**

$C_1 = 1 / k_c^2$ = **2.755**

Poissons ratio $\nu$ = **0.3**

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

Unrestrained effective length $L = 1.0 \times L_{m5_s1_seg1_B}$ = **2422** mm

Elastic critical buckling moment $M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **88.4** kNm

Slenderness ratio for lateral torsional buckling $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **0.574**

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

***$\bar{\lambda}_{LT} > \bar{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored***

Library item: Bending LTB slenderness out

**Check buckling resistance - Section 6.3.2.1**

Buckling curve - Table 6.5 b

Imperfection factor - Table 6.3 $\alpha_{LT}$ = **0.34**

Correction factor for rolled sections $\beta$ = **0.75**

LTB reduction determination factor $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **0.653**

LTB reduction factor - eq 6.57 $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.929**

Modification factor $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.822**

Modified LTB reduction factor - eq 6.58 $\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2)$ = **1.000**

Design buckling resistance moment - eq 6.55 $M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1}$ = **29.1** kNm

$M_{y,Ed} / M_{b,y,Rd}$ = **0.406**

***PASS - Design buckling resistance moment exceeds design bending moment***

Library item: Bending buckling out

**Check combined bending and compression - Section 6.3.3**

Equivalent uniform moment factors - Table B.3 $\psi_y$ = 3.869 kNm / -11.806 kNm = **-0.328**

$\alpha_y$ = -1.994 kNm / -11.806 kNm = **0.169**

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

Library item: Comb check factor calc out

$\psi_{LT}$ = 3.869 kNm / -11.806 kNm = **-0.328**

$\alpha_{LT}$ = -1.994 kNm / -11.806 kNm = **0.169**

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

Library item: Comb check factor calc out

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

Characteristic moment resistance $M_{y,Rk} = W_{pl.y} \times f_y$ = **29.1** kNm

Characteristic moment resistance $M_{z,Rk} = W_{pl.z} \times f_y$ = **6.1** kNm

Characteristic resistance to normal force $N_{Rk} = A \times f_y$ = **472.1** kN

Library item: Combined forces out

Interaction factors

$k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}))$ = **0.401**

$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1})$ = **0.976**

Library item: Combined factors out

Interaction formulae - eq 6.61 & eq 6.62

$N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.189**

$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.462**

***PASS - Combined bending and compression checks are satisfied***

Library item: Combined check out

**Consider Combination 6 - 1.0G + 1.0$\psi_0$Q + 0.5S + 1.0W (Service)**

**Check design 2188 mm along span**

**Check y-y axis deflection - Section 7.2.1**

Maximum deflection $\delta_y$ = **3.9** mm

Allowable deflection $\delta_{y,Allowable}$ = $L_{m5_s1}$ / 300 = **8.1** mm

$\delta_y$ / $\delta_{y,Allowable}$ = **0.478**

***PASS - Allowable deflection exceeds design deflection***

Library item: Deflection out

| Project | Sheet no./rev. |
|---|---|
| AGRICULTURAL BARN, CHAPEL MEAD, FOXHAM | CML 69 |
| ANGLE RAFTER DESIGN CHECK | By/Date MM 4/9/23 |

FROM FRAME ANALYSIS, MAXIMUM FORCES IN ANGLE RAFTER ARE AS FOLLOWS

BENDING. 2.9 kNm
SHEAR 5.6 kN
} ULTIMATE

IF RAFTER IS 90 × 90 × 8 RSA

$$f_{bc} = \frac{2.9 \times 10^6}{16.1 \times 10^3} = 180 \text{ N/mm}^2$$

$$f_v = \frac{5.6 \times 10^3}{90 \times 8} = 8 \text{ N/mm}^2$$

NOTE: ANGLE IS RESTRAINED BY PURLINS

$\therefore L/r_y = 900/27.4 = 32.$

$\therefore$ OKAY BY INSPECTION

| Project | Sheet no./rev. |
| --- | --- |
| AGRICULTURAL BARN, CHAPEL MEAD, FOXHAM | CHL 70 |
| EXISTING FOOTING LOAD CAPACITY | By/Date MH 4/9/23 |

BASED ON ANALYSIS, MAX REACTION FROM FRAME ONTO INTERNAL POST

= 11.4 kN (SERVICE LOAD)

∴ IF FOOTING IS 700x700 INTERNAL PAD

GROUND BEARING PRESSURE

$$= \frac{11.4}{0.7^2} = 23.3 \text{ kN/m}^2$$

OKAY BY INSPECTION