# Structural Design Calculations



**Site Address**

**Redditch,**
**Worcestershire.**

**Prepared for**

**A+ Construction Ltd**
**Date: 30 August 2020**

**IKT Consulting Structural Engineers Limited**
*6 David Lane, Basford, Nottingham. NG6 0JU*

**Project No.: IKT - 0000**

## DESIGN CALCULATIONS

### Introduction

The following calculations have been produced for the proposed structural alteration referred to as ***** Close, Redditch.

The existing property appeared to be a typical cavity wall construction, with an inner blockwork leaf and outer brickwork leaf with a traditional truss roof.

The proposed building has been designed in accordance with Part A of the Building Regulations for disproportionate collapse (Class 1).

### Scope of Design /work

IKT Consulting Limited design was limited to a number of loose steel beams required to support the existing first floor and roof loadings.

### General Notes

The Engineer has carried out the design in accordance with the information and drawings provided to him by the client.

However, unless and until the structural fabric of the building is fully exposed, these should be treated as assumptions and not certainties and should be confirmed or otherwise by the contractor on site. Should the contractor's site discoveries indicate that these assumptions are incorrect he should advise the Engineer immediately and await the Engineers advice on how to proceed.

The fabricator/supplier will normally bear responsibility for the structural members up to the point where they are off-loaded onto the site: thereupon they become the responsibility of the contractor.

The delivery should be checked to ensure that it complies with the specification and that the quantities and dimensions are correct. Any discrepancies must immediately be brought to the attention of the supplier.

Contractor/builder appointed to carry out the construction work must carefully assess our proposed layouts, proposed structural specifications and existing site before undertaking construction work.

If the contractor is unsure about the length or size of any design structural element, he must contact the structural engineer for clarification before undertaking construction work.

The contractor must demonstrate a full understanding of the project before starting deconstruction/construction work, and if unclear about any part of the design the contractor must contact us before undertaking the work.

If needed client or contractor must get the local authority approval, i.e. submit the proposed design and layout for approval before undertaking any construction work.

The contractor must provide the client with details of the construction process and risks involve (i.e. damage to existing decoration, existing features and fixtures etc.) before carrying out the construction work.

**DESIGN CALCULATIONS**

Fire protection to be in accordance with relevant Building Regulations and Architect's details. New steel beams to be fire protected using 2 layers of plasterboard and skim to achieve 1-hour fire protection.

All dimensions are to be confirmed by the contractor on-site prior to construction.

All bolts to be minimum M16, Grade 8.8 u.n.o.

All internal steelwork to be shot blast to SA2.5 Standard and painted with 2 coats zinc phosphate min 120 microns except as noted on drawings.

All external steelwork to be galvanised to suit exposure condition.

All temporary works to the contractor's design and details.

**Amendments to the design**

Before placing an order or commencing work on site the contractor should be satisfied that the design brief is correct and that he has sufficient information to perform the works safely. IKT Consulting Ltd cannot be held responsible for any incorrect or incomplete design brief.

**Codes of Practice**

**This project was generally designed using the following standards:**

The Building Regs. – Approved Document A (2004)

BS EN 1990 - Basis of structural design

BS EN 1991 - Actions on structures

BS EN 1993 - Design of steel structures

BS EN 1996 – Design of Masonry Structures

**Calculation Method**

Tekla TEDDS  v3.0.14 design software will be used to assist with these calculations (print outs are included) to Eurocodes / British Standards.

**Structural Consideration**

The condition and adequacy of existing structures and foundation to support additional loads should be confirmed onsite before commencing construction works.

All ground floor internal walls are assumed to be solid brickwork wall and to be confirmed on site.

Section of the existing gable wall at first floor to the rear of the property to be removed, and retain 330mm minimum nib return for stability.

**DESIGN CALCULATIONS**

## Design Notes

1. This design/sketch to be read in conjunction with all specifications and all relevant architects, engineers, services and specialist drawings.
2. **All dimensions are to be confirmed by the contractor on-site prior to construction.**
3. Steelwork to be grade S355, execution class 2and CE marked, unless otherwise noted.
4. To minimise deflections of the existing structure, new beams must be pinned uptight to existing construction with slate or dry-pack mortar, and all mortar allowed to cure prior to de-propping.
5. All work to be undertaken in accordance with the current Building Regulations Part A, British Standards and good building practice.
6. Beams and lintels to have a minimum bearing length of 100mm when perpendicular to the wall, and 150mm when parallel to the wall unless noted otherwise.
7. Due to significant structural works, minor post-construction deflection of brittle finishes may be expected in the existing building.
8. Masonry to be clay or calcium silicate with a minimum compressive strength of 50N/mm2 or class A or B Engineering bricks in accordance with BS EN 771-1. Mortar to be M4(iii) unless noted otherwise. Masonry repairs to match the existing brick.
9. All load-bearing inner skin walls to be minimum 100mm thick medium density (7N) concrete blockwork wall unless noted otherwise.
10. All steels that support timber work are to have the flanges pre-drilled @ 500mm centres to accept timber plates.
11. Refer to Archerite's drawing for lintel schedule.
12. Drawings are not to scale.

## Design Summary - Member Sizes

| | |
|---|---|
| Beam 1: | 1No. 152 x 152 x 30UC, Grade S355, @ 2.66m (TBC) |
| Beam 2: | 2No. 152 x 152 x 23UC, S355 steel section, bolted together with M16, grade 8.8 @450 centres, @ 2.400m (TBC). OR CH90/100 Catnic Lintel or similar approved. Safe Working Load 50kN. |
| Beam 3: | 2No. 152 x 152 x 30UC, S355 steel section, bolted together with M16, grade 8.8 @450 centres, @ 3.400m (TBC) |
| Beam 4: | CH90/100 Catnic Lintel or similar approved. Safe Working Load 45kN. Or, 1No. Keystone XHD/K-90. Safe Working Load 50kN. |

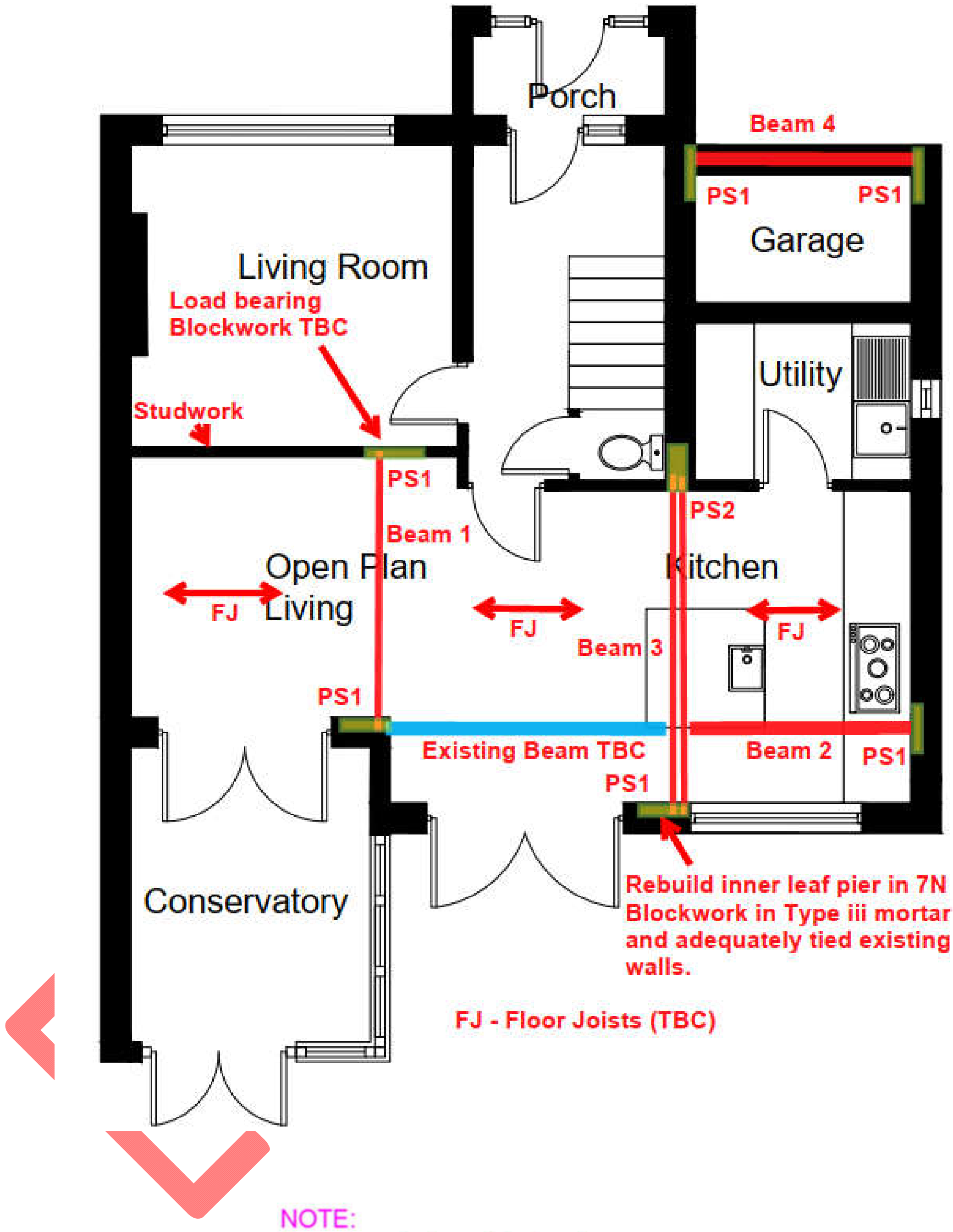
**Padstones**

| | | |
|---|---|---|
| PS 1: | 1No. | 440 x 100 x 215dp, C35 Mass Concrete padstones |
| PS 2: | 2No. | 440 x 100 x 215dp, C35 Mass Concrete padstones |

DESIGN CALCULATIONS

# KEY PLAN

ALL TOP OF STEEL BEAMS TO BE CONFIRMED ON SITE

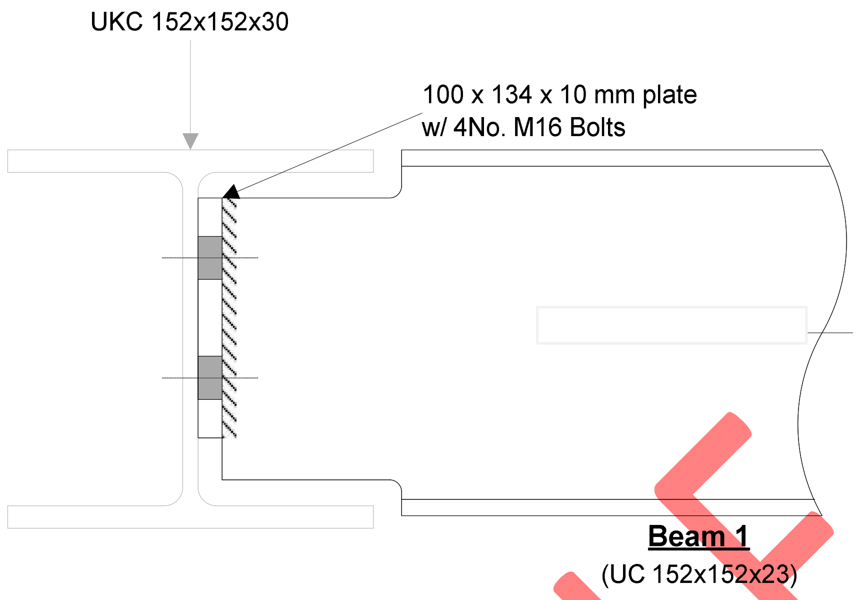



NOTE:
Do not scale from this drawing.
Existing structures must be adequately propped during beam installation.

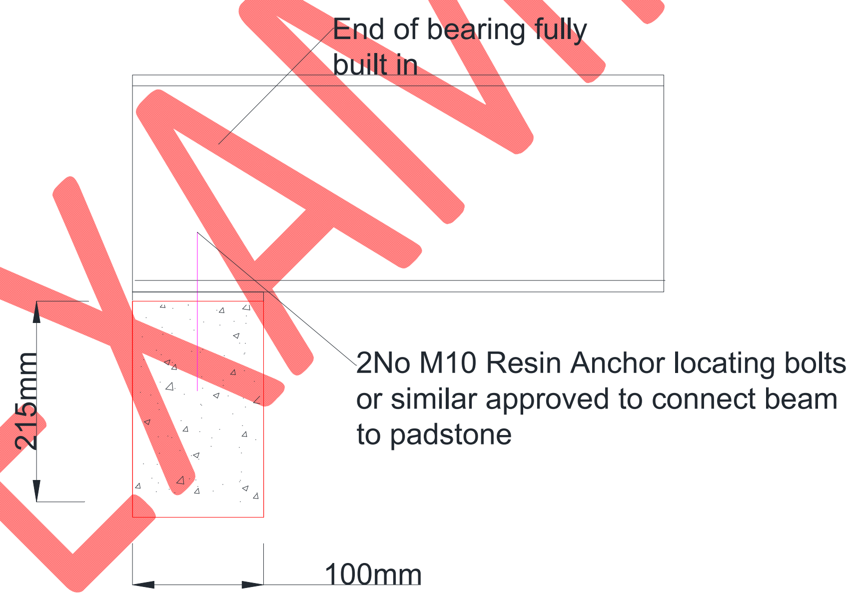
All site works shall be in accordance with the health & safety Act & associated regulations issued by the Health & Safety Executive & the Construction Regulations.

Materials in excess of 20kg must be 2 man lift or machine lift

DESIGN CALCULATIONS




## Detail 1 - Typical End Plate Connection




## Detail 2 - Typical Padstone

# LOADING SCHEDULE

## Pitched Truss Roof

Roof Pitch 40 degrees

| | | | | Service | | Factored |
|---|---|---|---|---|---|---|
| Dead | Tiles | 0.70 | /Cos Pitch | | | |
| | Felt/Battens | 0.05 | /Cos Pitch | | | |
| | Insulation | 0.05 | | | | |
| | Trusses | 0.20 | | | | |
| | Ceiling | 0.20 | | | | |
| | | 1.20 | /Cos Pitch | = 1.43 | x 1.35 | = 2.0 |
| Imposed | | | | | | |
| | Imposed | 0.60 | Use 0.6[(60-pitch)/30 | | | |
| | Attic Storage | 0.25 | | | | |
| | | 0.85 | | = 0.85 | x 1.5 | = 1.4 |
| | | | | $2.3 kN/m^2$ | | $3.4 kN/m^2$ |

## Brick Cavity Wall

| | | | | Service | | Factored |
|---|---|---|---|---|---|---|
| Dead | 100 Brick | 2.20 | | | | |
| | Insulation | 0.05 | | | | |
| | Block Inner | 1.45 | | | | |
| | Plaster | 0.20 | | | | |
| | | 3.90 | | = 3.90 | x 1.35 | = 5.5 |
| Imposed | | | | | | |
| | N/A | N/A | | | | |
| | | 0.00 | | = 0.00 | x 1.5 | = 0.0 |
| | | | | $3.9 kN/m^2$ | | $5.5 kN/m^2$ |

## Timber Floor

40

| | | | Service | | Factored |
|---|---|---|---|---|---|
| Dead | Finishes | 0.00 | | | |
| | Boarding | 0.20 | | | |
| | Joists | 0.15 | | | |
| | Ceiling | 0.20 | | | |
| | Services | 0.05 | | | |
| | | 0.60 | = 0.60 | x 1.35 | = 0.8 |
| Imposed | | | | | |
| | Imposed | 1.50 Use 0.6[(60-pitch)/30 | | | |
| | Partitions | 0.50 | | | |
| | | 2.00 | = 2.00 | x 1.5 | = 3.2 |
| | | | **2.6kN/m²** | | **4.0kN/m²** |

## Stud Wall

| | | | Service | | Factored |
|---|---|---|---|---|---|
| Dead | Plaster | 0.20 | | | |
| | Timber Studs | 0.25 | | | |
| | Plaster | 0.20 | | | |
| | | 0.65 | = 0.65 | x 1.35 | = 0.9 |
| Imposed | | | | | |
| | N/A | N/A | | | |
| | | 0.00 | = 0.00 | x 1.5 | = 0.0 |
| | | | **0.7kN/m²** | | **0.9kN/m²** |

DESIGN CALCULATIONS

## BEAM 01  - FLOOR

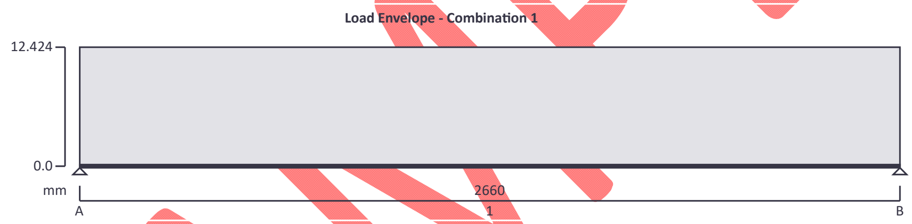
| Bearing – RHS (mm) | Opening (mm) | BEARING – LHS (mm) | Total Span (mm) |
|---|---|---|---|
| 100 | 2460 | 100 | 2660 |

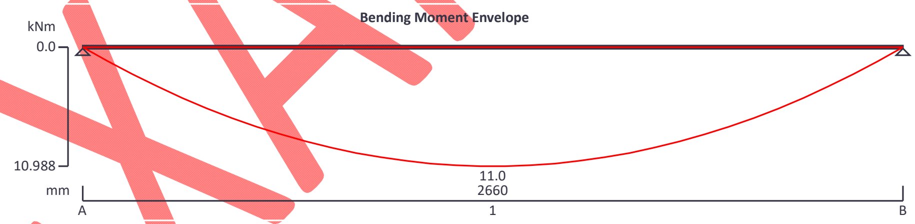
| LOADS FROM | WIDTH SUPPORTED (m) | LIVE LOADS (kN/m) | DEAD LOADS (kN/m) |
|---|---|---|---|
| Roof | 0 | 0 | 0 |
| Stud Wall | 2 | | 1.6 |
| 1st Floor | 2.6 | 5.2 | 1.6 |

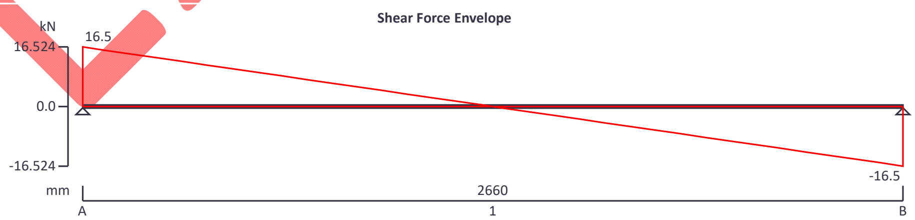
### STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

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

TEDDS calculation version 3.0.14




**Support conditions**

| | |
|---|---|
| Support A | Vertically restrained |
| | Rotationally free |
| Support B | Vertically restrained |
| | Rotationally free |

## DESIGN CALCULATIONS



### Applied loading

| | |
|---|---|
| Beam loads | Stud wall - Permanent full UDL 1.6 kN/m |
| | 1st FL - Variable full UDL 5.2 kN/m |
| | 1st FL - Permanent full UDL 1.6 kN/m |
| | Roof - Variable full UDL 0 kN/m |
| | Roof - Permanent full UDL 0 kN/m |
| | Beam - Variable full UDL 0 kN/m |
| | Beam - Permanent full UDL 0 kN/m |
| | Permanent self weight of beam * 1 |

### Load combinations

| | | |
|---|---|---|
| Load combination 1 | Support A | Permanent * 1.35 |
| | | Variable * 1.50 |
| | | Permanent * 1.35 |
| | | Variable * 1.50 |
| | Support B | Permanent * 1.35 |
| | | Variable * 1.50 |

### Analysis results

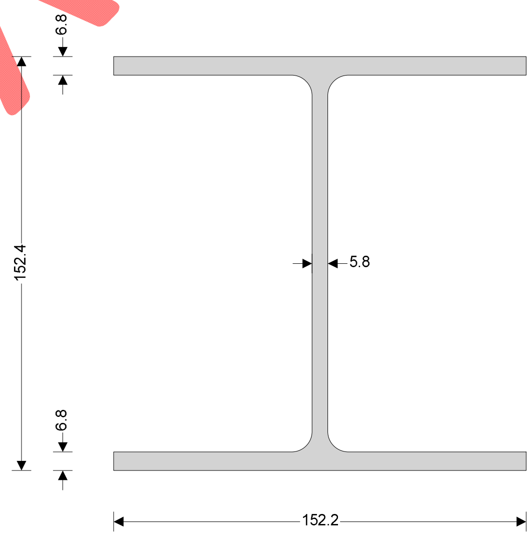
| | | |
|---|---|---|
| Maximum moment; | $M_{max}$ = **11** kNm; | $M_{min}$ = **0** kNm |
| Maximum shear; | $V_{max}$ = **16.5** kN; | $V_{min}$ = **-16.5** kN |
| Deflection; | $\delta_{max}$ = **1.3** mm; | $\delta_{min}$ = **0** mm |
| Maximum reaction at support A; | $R_{A\_max}$ = **16.5** kN; | $R_{A\_min}$ = **16.5** kN |
| Unfactored permanent load reaction at support A; | $R_{A\_Permanent}$ = **4.6** kN | |
| Unfactored variable load reaction at support A; | $R_{A\_Variable}$ = **6.9** kN | |
| Maximum reaction at support B; | $R_{B\_max}$ = **16.5** kN; | $R_{B\_min}$ = **16.5** kN |
| Unfactored permanent load reaction at support B; | $R_{B\_Permanent}$ = **4.6** kN | |
| Unfactored variable load reaction at support B; | $R_{B\_Variable}$ = **6.9** kN | |

### Section details

| | |
|---|---|
| Section type; | **UC 152x152x23 (BS4-1)** |
| Steel grade; | **S355** |

**EN 10025-2:2004 - Hot rolled products of structural steels**

| | |
|---|---|
| Nominal thickness of element; | $t = \max(t_f, t_w)$ = **6.8** mm |
| Nominal yield strength; | $f_y$ = **355** N/mm$^2$ |
| Nominal ultimate tensile strength; | $f_u$ = **470** N/mm$^2$ |
| Modulus of elasticity; | E = **210000** N/mm$^2$ |



## DESIGN CALCULATIONS

**Partial factors - Section 6.1**

Resistance of cross-sections; $\gamma_{M0}$ = **1.00**

Resistance of members to instability; $\gamma_{M1}$ = **1.00**

Resistance of tensile members to fracture; $\gamma_{M2}$ = **1.10**

**Lateral restraint**

Span 1 has lateral restraint at supports only

**Effective length factors**

Effective length factor in major axis; $K_y$ = **1.000**

Effective length factor in minor axis; $K_z$ = **1.000**

Effective length factor for torsion; $K_{LT.A}$ = **1.000**; + 2 * h

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

**Classification of cross sections - Section 5.5**

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

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

Width of section; c = d = **123.6** mm

$c / t_w = 26.2 * \varepsilon <= 72 * \varepsilon$; Class 1

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

Width of section; $c = (b - t_w - 2 * r) / 2$ = **65.6** mm

$c / t_f = 11.9 * \varepsilon <= 14 * \varepsilon$; Class 3

***Section is class 3***

**Check shear - Section 6.2.6**

Height of web; $h_w = h - 2 * t_f$ = **138.8** mm

Shear area factor; $\eta$ = **1.000**

$h_w / t_w < 72 * \varepsilon / \eta$

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

Design shear force; $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **16.5** kN

Shear area - cl 6.2.6(3); $A_v = \max(A - 2 * b * t_f + (t_w + 2 * r) * t_f, \eta * h_w * t_w)$ = **997** mm$^2$

Design shear resistance - cl 6.2.6(2); $V_{pl,Rd} = A_v * (f_y / \sqrt{[3]}) / \gamma_{M0}$ = **204.4** kN

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

**Check bending moment major (y-y) axis - Section 6.2.5**

Design bending moment; $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **11** kNm

Design bending resistance moment - eq 6.14; $M_{c,Rd} = M_{el,Rd} = W_{el.y} * f_y / \gamma_{M0}$ = **58.2** kNm

**Slenderness ratio for lateral torsional buckling**

Correction factor - Table 6.6; $k_c$ = **0.94**

$C_1 = 1 / k_c^2$ = **1.132**

Curvature factor; $g = \sqrt{[1 - (I_z / I_y)]}$ = **0.825**

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

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

Unrestrained length; $L = (1.0 * L_{s1} + 2 * h + 1.0 * L_{s1}) / 2$ = **2812** mm

Elastic critical buckling moment; $M_{cr} = C_1 * \pi^2 * E * I_z / (L^2 * g) * \sqrt{[I_w / I_z + L^2 * G * I_t / (\pi^2 * E * I_z)]}$ = **135.5** kNm

Slenderness ratio for lateral torsional buckling; $\bar{\lambda}_{LT} = \sqrt{(W_{el.y} * f_y / M_{cr})}$ = **0.656**

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

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

**Design resistance for buckling - Section 6.3.2.1**

Buckling curve - Table 6.5; b

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

**DESIGN CALCULATIONS**

| | |
|---|---|
| Correction factor for rolled sections; | $\beta$ = **0.75** |
| LTB reduction determination factor; | $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **0.705** |
| LTB reduction factor - eq 6.57; | $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.891** |
| Modification factor; | $f = \min(1 - 0.5 * (1 - k_c) * [1 - 2 * (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.971** |
| Modified LTB reduction factor - eq 6.58; | $\chi_{LT,mod} = \min(\chi_{LT} / f, 1)$ = **0.918** |
| Design buckling resistance moment - eq 6.55; | $M_{b,Rd} = \chi_{LT,mod} * W_{el.y} * f_y / \gamma_{M1}$ = **53.4** kNm |

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

**Check vertical deflection - Section 7.2.1**

Consider deflection due to variable loads

| | |
|---|---|
| Limiting deflection; | $\delta_{lim} = L_{s1} / 360$ = **7.4** mm |
| Maximum deflection span 1; | $\delta = \max(abs(\delta_{max}), abs(\delta_{min}))$ = **1.292** mm |

***PASS - Maximum deflection does not exceed deflection limit***

**PROVIDE: 1NO. UC 152 X 152 X 23, S355**

## BEAM 02 - FLOOR

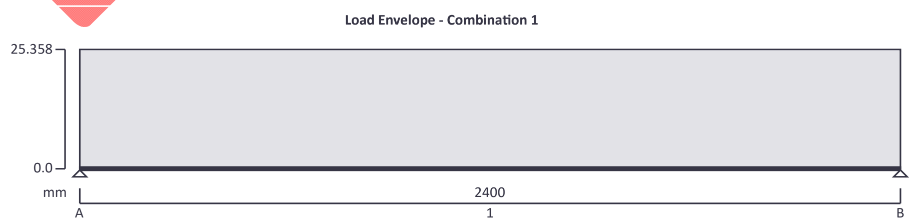
| Bearing – RHS (mm) | Opening (mm) | BEARING – LHS (mm) | Total Span (mm) |
|---|---|---|---|
| 100 | 2200 | 100 | 2400 |

| LOADS FROM | WIDTH SUPPORTED (m) | LIVE LOADS (kN/m) | DEAD LOADS (kN/m) |
|---|---|---|---|
| Roof | 3.5 | 3.0 | 5.0 |
| External Wall | 2.5 | | 9.8 |
| Floor | 0 | 0.0 | 0.0 |

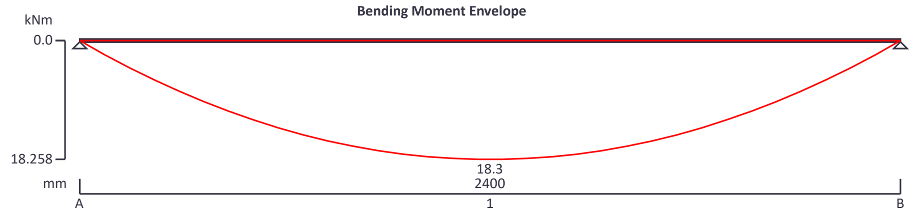
**STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)**

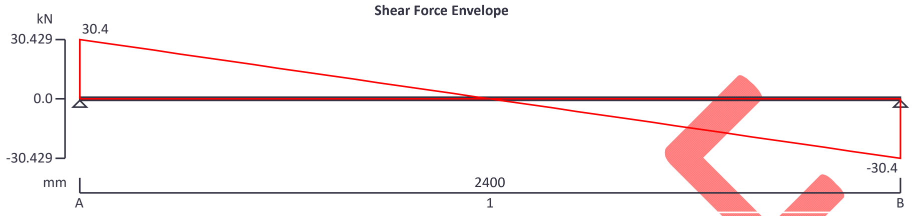
**In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex**

TEDDS calculation version 3.0.14



## DESIGN CALCULATIONS






**Support conditions**

| | |
|---|---|
| Support A | Vertically restrained |
| | Rotationally free |
| Support B | Vertically restrained |
| | Rotationally free |

**Applied loading**

| | |
|---|---|
| Beam loads | Ext wall - Permanent full UDL 10 kN/m |
| | 1st FL - Variable full UDL 0 kN/m |
| | 1st FL - Permanent full UDL 0 kN/m |
| | Roof - Variable full UDL 3 kN/m |
| | Roof - Permanent full UDL 5 kN/m |
| | Beam - Variable full UDL 0 kN/m |
| | Beam - Permanent full UDL 0 kN/m |
| | Permanent self weight of beam * 1 |

**Load combinations**

| | | |
|---|---|---|
| Load combination 1 | Support A | Permanent * 1.35 |
| | | Variable * 1.50 |
| | | Permanent * 1.35 |
| | | Variable * 1.50 |
| | Support B | Permanent * 1.35 |
| | | Variable * 1.50 |

**Analysis results**

| | | |
|---|---|---|
| Maximum moment; | $M_{max}$ = **18.3** kNm; | $M_{min}$ = **0** kNm |
| Maximum shear; | $V_{max}$ = **30.4** kN; | $V_{min}$ = **-30.4** kN |
| Deflection; | $\delta_{max}$ = **1.5** mm; | $\delta_{min}$ = **0** mm |
| Maximum reaction at support A; | $R_{A\_max}$ = **30.4** kN; | $R_{A\_min}$ = **30.4** kN |
| Unfactored permanent load reaction at support A; | $R_{A\_Permanent}$ = **18.5** kN | |
| Unfactored variable load reaction at support A; | $R_{A\_Variable}$ = **3.6** kN | |
| Maximum reaction at support B; | $R_{B\_max}$ = **30.4** kN; | $R_{B\_min}$ = **30.4** kN |
| Unfactored permanent load reaction at support B; | $R_{B\_Permanent}$ = **18.5** kN | |
| Unfactored variable load reaction at support B; | $R_{B\_Variable}$ = **3.6** kN | |

**Section details**

| | |
|---|---|
| Section type; | **2 x UC 152x152x23 (BS4-1)** |

## DESIGN CALCULATIONS

Steel grade; **S355**

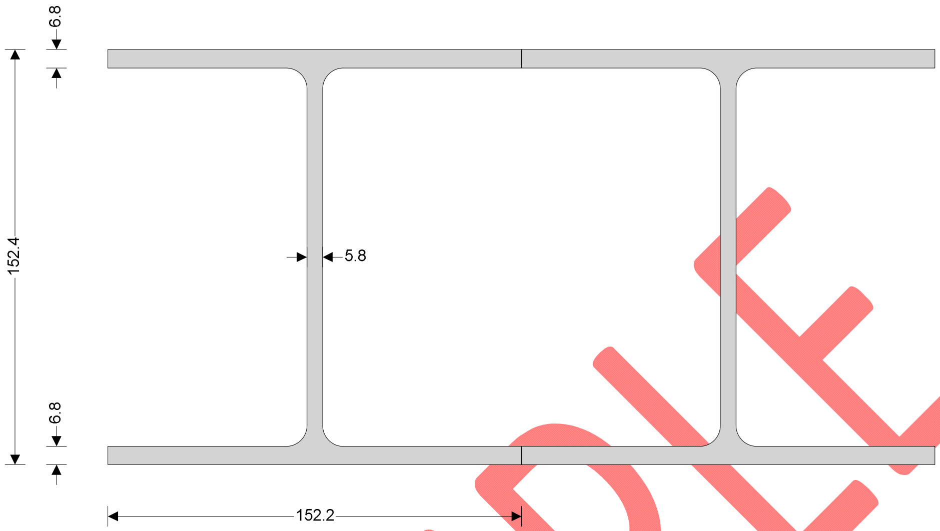
**EN 10025-2:2004 - Hot rolled products of structural steels**

Nominal thickness of element; $t = max(t_f, t_w) = \mathbf{6.8}$ mm

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

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

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




**Partial factors - Section 6.1**

Resistance of cross-sections; $\gamma_{M0} = \mathbf{1.00}$

Resistance of members to instability; $\gamma_{M1} = \mathbf{1.00}$

Resistance of tensile members to fracture; $\gamma_{M2} = \mathbf{1.10}$

**Lateral restraint**

Span 1 has lateral restraint at supports only

**Effective length factors**

Effective length factor in major axis; $K_y = \mathbf{1.000}$

Effective length factor in minor axis; $K_z = \mathbf{1.000}$

Effective length factor for torsion; $K_{LT.A} = \mathbf{1.000}$; + 2 * h

$K_{LT.B} = \mathbf{1.000}$;

**Classification of cross sections - Section 5.5**

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

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

Width of section; $c = d = \mathbf{123.6}$ mm

$c / t_w = 26.2 * \varepsilon <= 72 * \varepsilon$; Class 1

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

Width of section; $c = (b - t_w - 2 * r) / 2 = \mathbf{65.6}$ mm

$c / t_f = 11.9 * \varepsilon <= 14 * \varepsilon$; Class 3

***Section is class 3***

**Check shear - Section 6.2.6**

Height of web; $h_w = h - 2 * t_f = \mathbf{138.8}$ mm

Shear area factor; $\eta = \mathbf{1.000}$

$h_w / t_w < 72 * \varepsilon / \eta$

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

Design shear force; $V_{Ed} = max(abs(V_{max}), abs(V_{min})) = \mathbf{30.4}$ kN

Shear area - cl 6.2.6(3); $A_v = max(A - 2 * b * t_f + (t_w + 2 * r) * t_f, \eta * h_w * t_w) = \mathbf{997}$ mm$^2$

## DESIGN CALCULATIONS

Design shear resistance - cl 6.2.6(2); $V_{pl,Rd} = N * A_v * (f_y / \sqrt{[3]}) / \gamma_{M0}$ = **408.9** kN

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

**Check bending moment major (y-y) axis - Section 6.2.5**

Design bending moment; $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **18.3** kNm

Design bending resistance moment - eq 6.14; $M_{c,Rd} = M_{el,Rd} = N * W_{el.y} * f_y / \gamma_{M0}$ = **116.5** kNm

**Slenderness ratio for lateral torsional buckling**

Correction factor - Table 6.6; $k_c$ = **0.94**

$C_1 = 1 / k_c^2$ = **1.132**

Curvature factor; $g = \sqrt{[1 - (I_z / I_y)]}$ = **0.825**

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

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

Unrestrained length; $L = (1.0 * L_{s1} + 2 * h + 1.0 * L_{s1}) / 2$ = **2552** mm

Elastic critical buckling moment; $M_{cr} = C_1 * \pi^2 * E * I_z / (L^2 * g) * \sqrt{[I_w / I_z + L^2 * G * I_t / (\pi^2 * E * I_z)]}$ = **158.5** kNm

Slenderness ratio for lateral torsional buckling; $\bar{\lambda}_{LT} = \sqrt{(W_{el.y} * f_y / M_{cr})}$ = **0.606**

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

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

**Design resistance for buckling - 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; $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **0.673**

LTB reduction factor - eq 6.57; $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.914**

Modification factor; $f = \min(1 - 0.5 * (1 - k_c)* [1 - 2 * (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.972**

Modified LTB reduction factor - eq 6.58; $\chi_{LT,mod} = \min(\chi_{LT} / f, 1)$ = **0.940**

Design buckling resistance moment - eq 6.55; $M_{b,Rd} = \chi_{LT,mod} * N * W_{el.y} * f_y / \gamma_{M1}$ = **109.5** kNm

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

**Check vertical deflection - Section 7.2.1**

Consider deflection due to permanent and variable loads

Limiting deflection; $\delta_{lim} = L_{s1} / 250$ = **9.6** mm

Maximum deflection span 1; $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min}))$ = **1.518** mm

***PASS - Maximum deflection does not exceed deflection limit***

**PROVIDE: 2NO. UC152 X 152 X 23, S355**

DESIGN CALCULATIONS

**TYPICAL PADSTONE – PS1**

Inner skin wall – 3.6N/mm² Blockwork in M4, fk = 3.5N/mm²

| **Padstone on the internal wall** | | | | | | |
|---|---|---|---|---|---|---|
| Consider Bearings-assume wall in (fk) | | | = | 3.5 | $N/mm^2$ | |
| Max. Load | | | = | 35 | kN | |
| | | | | Bearing Type 2 | | |
| $\gamma_m$ | | | = | 3.5 | | |
| Wall Thickness | | | = | 100 | mm | |
| Required Bearing length | | | = | 233 | mm | |
| | | | | | | |
| **Provide 1No** | **440 x 140 x 215** | | | C35 Padstone at each end. | | |
| | | | | | | |

**Provide 1No. 440 x 100 x 215mm dp, C35 Mass concrete Padstone.**

## BEAM 03 - FLOOR

| | | | | |
|---|---|---|---|---|
| **Bearing – RHS (mm)** | **Opening (mm)** | **BEARING – LHS (mm)** | **Total Span (mm)** |
| 100 | 3100 | 150 | 3350 |

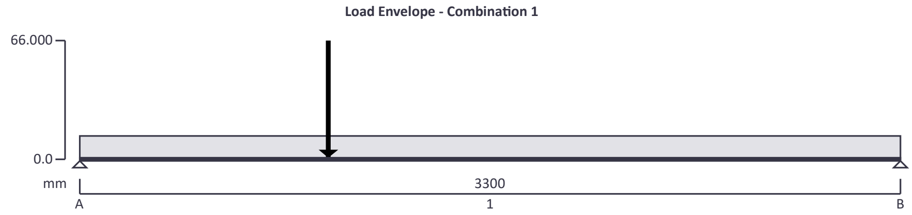
| **LOADS FROM** | **WIDTH SUPPORTED (m)** | **LIVE LOADS (kN/m)** | **DEAD LOADS (kN/m)** |
|---|---|---|---|
| Roof | 0 | 0 | 0 |
| Stud Wall | 3 | | 2.0 |
| Floor | 2.4 | 4.8 | 1.4 |
| Existing Beam A | | 4 | 20 |
| Existing Beam B | | 4 | 20 |

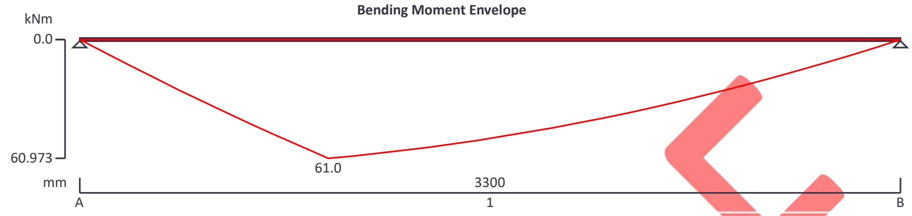
**STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)**

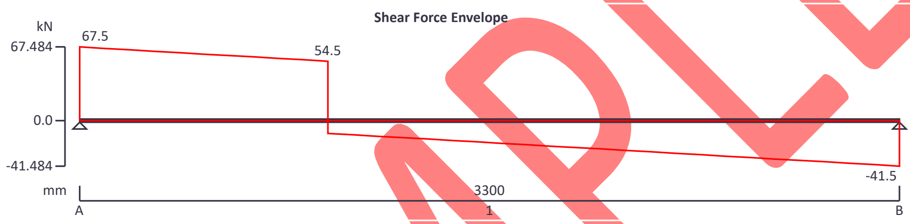
**In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex**

TEDDS calculation version 3.0.14

## DESIGN CALCULATIONS






**Support conditions**

| | |
|---|---|
| Support A | Vertically restrained |
| | Rotationally free |
| Support B | Vertically restrained |
| | Rotationally free |

**Applied loading**

| | |
|---|---|
| Beam loads | Ext wall - Permanent full UDL 2 kN/m |
| | 1st FL - Variable full UDL 5 kN/m |
| | 1st FL - Permanent full UDL 1.5 kN/m |
| | Roof - Variable full UDL 0 kN/m |
| | Roof - Permanent full UDL 0 kN/m |
| | Beam - Variable point load 8 kN at 1000 mm |
| | Beam - Permanent point load 40 kN at 1000 mm |
| | Permanent self weight of beam * 1 |

**Load combinations**

| | | |
|---|---|---|
| Load combination 1 | Support A | Permanent * 1.35 |
| | | Variable * 1.50 |
| | | Permanent * 1.35 |
| | | Variable * 1.50 |
| | Support B | Permanent * 1.35 |
| | | Variable * 1.50 |

**Analysis results**

| | | |
|---|---|---|
| Maximum moment; | $M_{max}$ = **61** kNm; | $M_{min}$ = **0** kNm |
| Maximum shear; | $V_{max}$ = **67.5** kN; | $V_{min}$ = **-41.5** kN |
| Deflection; | $\delta_{max}$ = **5.9** mm; | $\delta_{min}$ = **0** mm |

## DESIGN CALCULATIONS

| | | |
|---|---|---|
| Maximum reaction at support A; | $R_{A\_max}$ = **67.5** kN; | $R_{A\_min}$ = **67.5** kN |
| Unfactored permanent load reaction at support A; | $R_{A\_Permanent}$ = **34.6** kN | |
| Unfactored variable load reaction at support A; | $R_{A\_Variable}$ = **13.8** kN | |
| Maximum reaction at support B; | $R_{B\_max}$ = **41.5** kN; | $R_{B\_min}$ = **41.5** kN |
| Unfactored permanent load reaction at support B; | $R_{B\_Permanent}$ = **18.9** kN | |
| Unfactored variable load reaction at support B; | $R_{B\_Variable}$ = **10.7** kN | |

**Section details**

Section type; **2 x UC 152x152x30 (BS4-1)**

Steel grade; **S355**

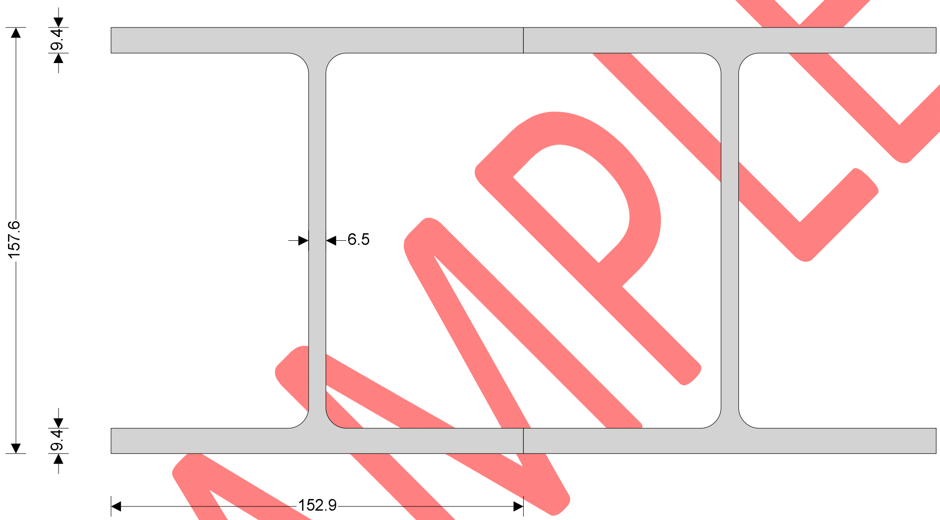
**EN 10025-2:2004 - Hot rolled products of structural steels**

Nominal thickness of element; $t = \max(t_f, t_w)$ = **9.4** mm

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

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

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






**Partial factors - Section 6.1**

Resistance of cross-sections; $\gamma_{M0}$ = **1.00**

Resistance of members to instability; $\gamma_{M1}$ = **1.00**

Resistance of tensile members to fracture; $\gamma_{M2}$ = **1.10**

**Lateral restraint**

Span 1 has lateral restraint at supports only

**Effective length factors**

Effective length factor in major axis; $K_y$ = **1.000**

Effective length factor in minor axis; $K_z$ = **1.000**

Effective length factor for torsion; $K_{LT.A}$ = **1.000**; + 2 * h

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

**Classification of cross sections - Section 5.5**

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

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

Width of section; c = d = **123.6** mm

$c / t_w = 23.4 * \varepsilon <= 72 * \varepsilon$; Class 1

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

Width of section; $c = (b - t_w - 2 * r) / 2$ = **65.6** mm

$c / t_f = 8.6 * \varepsilon <= 9 * \varepsilon$; Class 1

## DESIGN CALCULATIONS

*Section is class 1*

**Check shear - Section 6.2.6**

Height of web; $h_w = h - 2 * t_f =$ **138.8** mm

Shear area factor; $\eta =$ **1.000**

$h_w / t_w < 72 * \varepsilon / \eta$

*Shear buckling resistance can be ignored*

Design shear force; $V_{Ed} = \max(abs(V_{max}), abs(V_{min})) =$ **67.5** kN

Shear area - cl 6.2.6(3); $A_v = \max(A - 2 * b * t_f + (t_w + 2 * r) * t_f, \eta * h_w * t_w) =$ **1156**

$mm^2$

Design shear resistance - cl 6.2.6(2); $V_{c,Rd} = V_{pl,Rd} = N * A_v * (f_y / \sqrt{[3]}) / \gamma_{M0} =$ **473.8** kN

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

**Check bending moment major (y-y) axis - Section 6.2.5**

Design bending moment; $M_{Ed} = \max(abs(M_{s1\_max}), abs(M_{s1\_min})) =$ **61** kNm

Design bending resistance moment - eq 6.13; $M_{c,Rd} = M_{pl,Rd} = N * W_{pl.y} * f_y / \gamma_{M0} =$ **175.8** kNm

**Slenderness ratio for lateral torsional buckling**

Correction factor - Table 6.6; $k_c =$ **0.94**

$C_1 = 1 / k_c^2 =$ **1.132**

Curvature factor; $g = \sqrt{[1 - (I_z / I_y)]} =$ **0.824**

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

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

Unrestrained length; $L = (1.0 * L_{s1} + 2 * h + 1.0 * L_{s1}) / 2 =$ **3458** mm

Elastic critical buckling moment; $M_{cr} = C_1 * \pi^2 * E * I_z / (L^2 * g) * \sqrt{[I_w / I_z + L^2 * G * I_t / (\pi^2 * E * I_z)]} =$ **159.2** kNm

Slenderness ratio for lateral torsional buckling; $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} * f_y / M_{cr})} =$ **0.743**

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

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

**Design resistance for buckling - 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; $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] =$ **0.765**

LTB reduction factor - eq 6.57; $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) =$ **0.848**

Modification factor; $f = \min(1 - 0.5 * (1 - k_c)* [1 - 2 * (\bar{\lambda}_{LT} - 0.8)^2], 1) =$ **0.970**

Modified LTB reduction factor - eq 6.58; $\chi_{LT,mod} = \min(\chi_{LT} / f, 1) =$ **0.874**

Design buckling resistance moment - eq 6.55; $M_{b,Rd} = \chi_{LT,mod} * N * W_{pl.y} * f_y / \gamma_{M1} =$ **153.6** kNm

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

**Check vertical deflection - Section 7.2.1**

Consider deflection due to permanent and variable loads

Limiting deflection; $\delta_{lim} = L_{s1} / 250 =$ **13.2** mm

Maximum deflection span 1; $\delta = \max(abs(\delta_{max}), abs(\delta_{min})) =$ **5.851** mm

***PASS - Maximum deflection does not exceed deflection limit***

**PROVIDE: 2NO. UC152 X 152 X 30, S355**

DESIGN CALCULATIONS

## TYPICAL PADSTONE – PS2

Inner skin wall – 7.3N/mm$^2$ Blockwork in M4, fk = 6.4N/mm$^2$

| Padstone on the internal wall | | | | | | |
|---|---|---|---|---|---|---|
| Consider Bearings-assume wall in (fk) | | = | 6.4 | N/mm$^2$ | |
| Max. Load | | | = | 70 | kN | |
| | | | Bearing Type 2 | | | |
| $\gamma_m$ | | | = | 3.5 | | |
| Wall Thickness | | | = | 100 | mm | |
| Required Bearing length | | | = | 255 | mm | |
| | | | | | | |
| **Provide 1No** | **440 x 100 x 215 dp** | | | 35 Padstone at each end. | | |
| | | | | | | |

**Provide 1No. 440 x 100 x 215mm dp, C35 Mass concrete Padstone.**

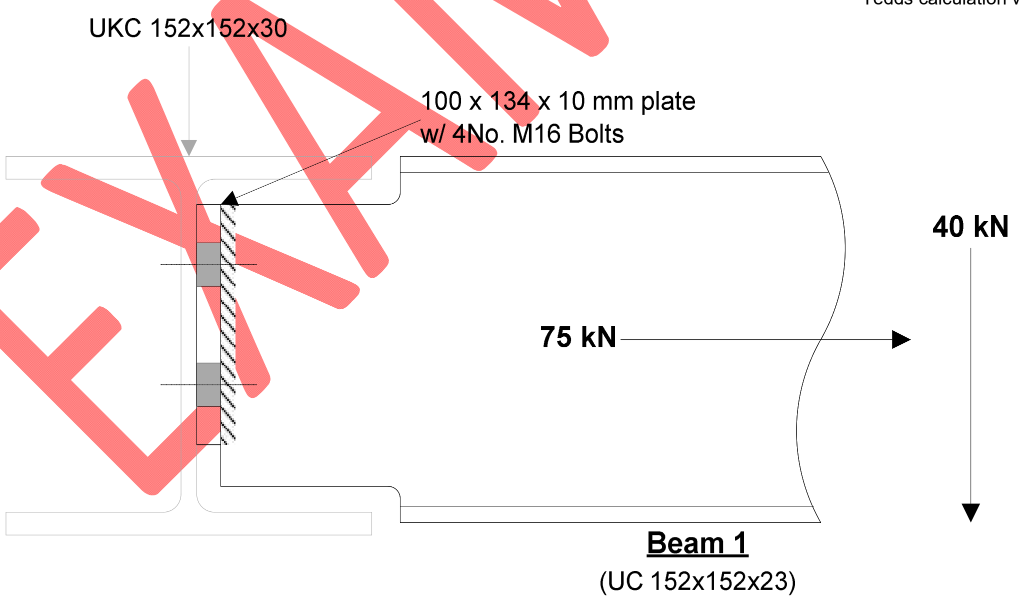


## TYPICAL CONNECTION

### STEEL CONNECTION DESIGN

**In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009, and EN1993-1-8:2005 incorporating Corrigenda December 2005, September 2006 and July 2009, and the UK National Annex.**

Tedds calculation version 1.1.01




**Connection details**

| | |
|---|---|
| Connection type; | **Partial depth end plate** |
| Number of supported beams; | **1 supported beam** |

**Partial factors**

| | |
|---|---|
| Resistance of cross-section; | $\gamma_{M0}$ = **1.00** |
| Resistance of members to instability; | $\gamma_{M1}$ = **1.00** |

## DESIGN CALCULATIONS

Cross-sections in tension to fracture; $\gamma_{M2,c}$ = **1.10**

Resistance of bolts; $\gamma_{M2,b}$ = **1.25**

Structural integrity; $\gamma_{M,u}$ = **1.10**

**Supporting beam details**

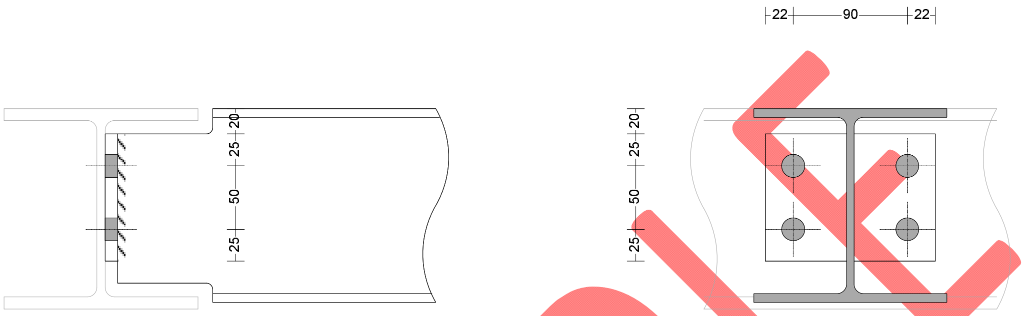
Section name; **UKC 152x152x30**

Steel grade; **S355**

Yield strength; $f_y$ = **355** N/mm$^2$

Ultimate strength; $f_u$ =**470** N/mm$^2$

**Beam 1**




**Summary Table**

| Check | Description | Units | Design Force | Design Resistance | Utilisation | |
|---|---|---|---|---|---|---|
| 1 | Recommended detailing practices | | | | | PASS |
| 2 | Supported beam - Welds | kN | 42.5 | 106.3 | 0.400 | PASS |
| 4 | Supported beam - Web in shear | kN | 40 | 107 | 0.374 | PASS |
| 5 | Supported beam - Resistance at notch | kNm | 3.4 | 4.7 | 0.719 | PASS |
| 6 | Supported beam - Local stability notch | | | | | PASS |
| 8 | Connection - Bolt group | kN | 40 | 167.4 | 0.239 | PASS |
| 9 | Connection - End plate in shear | kN | 40 | 200.9 | 0.199 | PASS |
| 10 | Supporting beam - Shear | kN | 20 | 166.8 | 0.120 | PASS |
| 11 | Tying resistance - Plate and bolts | kN | 75 | 120.8 | 0.621 | PASS |
| 12 | Tying resistance - Supported beam web | kN | 75 | 247.8 | 0.303 | PASS |

**Design forces**

Design shear; $V_{Ed1}$ = **40** kN

Design tying force; $F_{Ed1}$ = **75** kN

**Supported beam details**

Section name; **UC 152x152x23**

Steel grade; **S355**

Yield strength; $f_{y,b}$ = **355** N/mm$^2$

Ultimate strength; $f_{u,b}$ = **470** N/mm$^2$

Correlation factor; $\beta_{w,b}$ = **0.9**

**End plate details**

Plate height; $h_p$ = **100** mm

Plate width; $b_p$ = **134** mm

## DESIGN CALCULATIONS

| | |
|---|---|
| Plate thickness; | $t_p$ = **10** mm |
| Plate grade; | **S275** |
| Yield strength; | $f_{y,p}$ = **275** N/mm$^2$ |
| Ultimate strength; | $f_{u,p}$ = **410** N/mm$^2$ |
| Correlation factor; | $\beta_{w,p}$ = **0.85** |
| **Bolt details** | |
| Number of bolt rows; | $n_{1,1}$ = **2** |
| Total number of bolts; | $n_b$ = **4** |
| End distance; | $e_1$ = **25** mm |
| Edge distance; | $e_2$ = **22** mm |
| Pitch; | $p_1$ = **50** mm |
| Gauge; | $p_3$ = **90** mm |
| Bolt hole; | $d_0$ = **18** mm |
| Bolt size; | **M16** |
| Bolt grade; | **8.8** |
| Yield strength; | $f_{y,bolt}$ = **640** N/mm$^2$ |
| Ultimate strength; | $f_{u,bolt}$ = **800** N/mm$^2$ |
| **Check 1: Recommended detailing practice** | |
| Minimum plate height; | $0.6 * h_b$ = **91.4 mm** |
| Actual plate height; | $h_p$ = **100** mm |
| Maximum depth to plate; | **50 mm** |
| Actual depth to plate; | $d_p$ = **20** mm |
| Maximum plate thickness; | **10 mm** |
| Actual plate thickness; | $t_p$ = **10** mm |
| Minimum bolt gauge; | **90 mm** |
| Actual bolt gauge; | $p_3$ = **90** mm |
| **Top Notch** | |
| Depth of notch; | $d_{nt}$ = **20** mm |
| Length of notch; | $l_n$ = **75** mm |
| Minimum vertical clearance; | $\mathrm{Max}(t_{f,b} + r_b,\ t_f + r)$ = **17** mm |
| Actual vertical clearance; | $d_{nt}$ = **20** mm |
| Minimum horizontal clearance; | **10 mm** |
| Actual horizontal clearance; | $l_n - (b - t_w) / 2 + t_p$ = **11.8** mm |
| **Bottom Notch** | |
| Depth of notch; | $d_{nb}$ = **15** mm |
| Length of notch; | $l_n$ = **75** mm |
| Minimum vertical clearance; | $t_{f,b,A\_c1} + r_b$ = **14.4** mm |
| Actual vertical clearance; | $d_{nb}$ = **15** mm |

***PASS - Recommended detailing practices are met***

| | |
|---|---|
| **Check 2: Supported beam - Welds** | |
| Weld leg size; | $s_w$ = **6.0** mm |
| Minimum weld throat thickness; | $0.48 * t_{w,b}$ = **2.8** mm |
| Effective weld throat thickness; | $a_w = 0.7 * s_w$ = **4.2** mm |
| Correlation factor; | $\beta_w = \mathrm{Min}(\beta_{w,b},\ \beta_{w,p})$ = **0.85** |
| Design shear strength; | $f_{vw,d} = \mathrm{Min}(f_{u,b},f_{u,p}) / \sqrt{(3)} / (\beta_w * \gamma_{M2,c})$ = **253.17** N/mm$^2$ |
| Design resistance; | $F_{w,Rd} = f_{vw,d} * a_w * h_p$ = **106.33** kN |
| Design weld force; | $F_{w,Ed} = \sqrt{(V_{Ed1}^2 + F_{Ed1}^2)} / 2$ = **42.5** kN |
| Utilisation; | $F_{w,Ed} / F_{w,Rd}$ = **0.400** |

***PASS - Weld throat thickness greater than required***

## DESIGN CALCULATIONS



### Check 4: Supported beam - Web in shear

| | |
|---|---|
| Shear area; | $A_v = 0.9 * h_p * t_{w,b} =$ **522** $mm^2$ |
| Plastic shear resistance of beam web; | $V_{pl,Rd} = A_v * (f_{y,b} / \sqrt{(3)}) / \gamma_{M0} =$ **106.99** kN |
| Design shear resistance; | $V_{c,Rd} = V_{pl,Rd} =$ **106.99** kN |
| Utilisation; | $V_{Ed1} / V_{c,Rd} =$ **0.374** |
| | ***PASS - Web shear resistance greater than design shear*** |

### Check 5: Supported beam - Resistance at notch

**Double Notch (low shear, $V_{Ed} <= 0.5V_{pl,DN,Rd}$)**

| | |
|---|---|
| Shear area at notch; | $A_{v,DN} = 0.9 * (h_b - d_{nt} - d_{nb}) * t_{w,b} =$ **613** $mm^2$ |
| Shear resistance at notch; | $V_{pl,DN,Rd} = (A_{v,DN} * f_{y,b}) / (\sqrt{(3)} * \gamma_{M0}) =$ **125.6** kN |
| Moment resistance at notch; | $M_{v,DN,Rd} = f_{y,b} * t_{w,b} / (6 * \gamma_{M0}) * (h_b - d_{nt} - d_{nb})^2 =$ **4.73** kNm |
| Design moment at notch; | $M_{v,Ed} = V_{Ed1} * (t_p + l_n) =$ **3.4** kNm |
| Utlilisation; | $M_{v,Ed} / M_{v,N,Rd} = 0.719$ |
| | ***PASS - Notch resistance is greater than design force*** |

### Check 6: Supported beam - Local stability of notched beam

**Double notch**

| | |
|---|---|
| Maximum notch depth; | $h_b / 5 =$ **30.5** mm |
| Actual notch depth; | $Max(d_{nt}, d_{nb}) =$ **20** mm |
| Maximum notch length ($h_b/t_{w,b} <= 48.0$); | $h_b =$ **152.4** mm |
| Actual notch length; | $l_n =$ **75** mm |
| | ***PASS - Local stability is accounted for*** |

### Check 8: Connection - Bolt group

| | |
|---|---|
| Bolt tensile stress area; | $A_s =$ **157** $mm^2$ |
| Bolt shear stress factor; | $\alpha_v =$ **0.6** |
| Bolt shear resistance; | $F_{v,Rd} = \alpha_v * f_{u,bolt} * A_s / \gamma_{M2,b} =$ **60.29** kN |
| For the end plate; | $\alpha_{b,p} = Min(e_1 / (3 * d_0), p_1 / (3 * d_0) - 1/4, f_{u,bolt} / f_{u,p}, 1) =$ **0.46** |
| | $k_{1,p} = Min(2.8 * e_2 / d_0 - 1.7, 1.4 * p_3 / d_0 - 1.7, 2.5) =$ **1.72** |
| For the supporting member; | $\alpha_{b,2} = Min(p_1 / (3 * d_0) - 1/4, f_{u,bolt} / f_u, 1) =$ **0.68** |
| | $k_{1,2} = Min(1.4 * p_3 / d_0 - 1.7, 2.5) =$ **2.5** |
| Bearing on the end plate; | $F_{b,Rd,p} = k_{1,p} * \alpha_{b,p} * f_{u,p} * d_b * t_p / \gamma_{M2,b} =$ **41.84** kN |
| Bearing on the supporting member; | $F_{b,Rd,2} = k_{1,2} * \alpha_{b,2} * f_u * d_b * t_w / \gamma_{M2,b} =$ **66.08** kN |
| Minimum bearing resistance; | $F_{b,Rd1} = Min(F_{b,Rd,p}, F_{b,Rd,2}) =$ **41.84** kN |
| Resistance of the bolt group; | $F_{Rd} = n_b * F_{b,Rd1} =$ **167.37** kN |
| Utilisation; | $V_{Ed1} / F_{Rd} =$ **0.239** |
| | ***PASS - Bolt group resistance is greater than design force*** |

### Check 9: Connection - End plate in shear

| | |
|---|---|
| Net shear area; | $A_{v,net} = t_p * (h_p - n_{1,1} * d_0) =$ **640** $mm^2$ |
| Edge shear area; | $A_{nt} = t_p * (e_2 - d_0 / 2) =$ **130** $mm^2$ |
| Shear area from end bolt; | $A_{nv} = t_p * (h_p - e_1 - (n_{1,1} - 0.5) * d_0) =$ **480** $mm^2$ |
| Gross section shear resistance; | $V_{Rd,g} = (2 * h_p * t_p) / 1.27 * f_{y,p} / (\sqrt{(3)} * \gamma_{M0}) =$ **250.03** kN |
| Net section shear resistance; | $V_{Rd,n} = 2 * A_{v,net,A\_c1} * f_{u,plate,A\_c1} / (\sqrt{(3)} * \gamma_{M2,c}) =$ **275.45** kN |
| Block tearing resistance; | $V_{Rd,b} = 2 * (0.5 * f_{u,p} * A_{nt} / \gamma_{M2,c} + f_{y,p} * A_{nv} / (\sqrt{(3)} * \gamma_{M0})) =$ **200.88** kN |
| End plate in-plane bending resistance; | $V_{Rd,ip} = 2 * t_{p,A\_c1} * h_{p,A\_c1}^2 * f_{y,plate,A\_c1} / (3 * (p_{3\_c1} - t_{w,b,A\_c1}) *$ |

$\gamma_{M0}) =$ **217.74** kN

| | |
|---|---|
| End plate shear resistance; | $V_{Rd,pl,min} = Min(V_{Rd,g}, V_{Rd,n}, V_{Rd,b}, V_{Rd,ip}) =$ **200.88** kN |
| Utilisation; | _PlateShearUtilisationA_c1 = **0.199** |
| | ***PASS - Shear resistance of end plate greater than design force*** |

**DESIGN CALCULATIONS**

**Check 10: Supporting beam - Shear**

Distance from top bolt to flange; $e_{1,t}$ = **45** mm

Distance from bottom bolt to flange; $e_{1,b}$ = **63** mm

Minimum top distance; $e_t = Min(e_{1,t}, 5 * d_b)$ = **45** mm

Minimum bottom distance; $e_b = Min(e_{1,b}, p_3 / 2, 5 * d_b)$ = **45** mm

Shear area of supporting member; $A_v = t_w * (e_t + (n_{1,1} - 1) * p_1 + e_b)$ = **910** mm$^2$

Net shear area of supporting member; $A_{v,net} = A_v - n_{1,1} * d_0 * t_w$ = **676** mm$^2$

Local shear resistance; $V_{Rd,min} = Min(A_v * f_y / (\sqrt{(3)} * \gamma_{M0}), A_{v,net} * f_u / (\sqrt{(3)} * \gamma_{M2,c}))$ = **166.76** kN

Utilisation; $V_{Ed1}/2 / V_{Rd,min}$ = **0.12**

***PASS - Beam shear resistance is greater than design force***

**Check 11: Tying resistance - Plate and bolts**

Effective end distance; $e_{1A} = Min(e_1, 0.5 * (p_3 - t_{w,b} - 2 * a_w * \sqrt{(2)}) + d_0/2)$ = **25** mm

Effective bolt pitch; $p_{1A} = Min(p_1, p_3 - t_{w,b} - 2 * a_w * \sqrt{(2)} + d_0)$ = **50** mm

Minimum end distance; $e_{min} = e_2$ = **22** mm

Bolt factor; $k_2$ = **0.9**

Distance from weld throat to bolt; $m_w = (p_3 - t_{w,b} - 2 * 0.8 * a_w * \sqrt{(2)}) / 2$ = **37.3** mm

$n_w = Min(e_{min}, 1.25 * m_w)$ = **22** mm

Width across bolt head points; $d_w$ = **26** mm

$e_w = d_w / 4$ = **6.5** mm

Effective length of equivalent T-stub; $\Sigma l_{eff} = 2 * e_{1A} + (n_{1,1} - 1) * p_{1A}$ = **100.0** mm

Moment resistance of plate; $M_{pl,1,Rd,u} = (0.25 * \Sigma l_{eff} * t_p^2 * f_{u,p}) / \gamma_{M,u}$ = **0.93** kNm

$M_{pl,2,Rd,u} = M_{pl,1,Rd,u}$ = **0.93** kNm

Mode 1 plate failure; $F_{Rd,u,1} = (8 * n_w - 2 * e_w) * M_{pl,1,Rd,u} / (2 * m_w * n_w - e_w * (m_w + n_w))$ = **120.78** kN

Individual bolt resistance; $F_{t,Rd,u} = k_2 * f_{u,bolt} * A_s / \gamma_{M,u}$ = **102.76** kN

Group bolt resistance; $\Sigma F_{t,Rd,u} = n_b * F_{t,Rd,u}$ = **411.05** kN

Mode 2 bolt and plate failure; $F_{Rd,u,2} = (2 * M_{pl,2,Rd,u} + n_w * \Sigma F_{t,Rd,u}) / (m_w + n_w)$ = **183.78** kN

Mode 3 bolt failure; $F_{Rd,u,3} = \Sigma F_{t,Rd,u}$ = **411.05** kN

Minimum resistance; $F_{Rd,u,min} = Min(F_{Rd,u,1}, F_{Rd,u,2}, F_{Rd,u,3})$ = **120.78** kN

Utilisation; $F_{Ed1} / F_{Rd,u,min}$ = **0.621**

***PASS - Tying resistance of plate and bolts is greater than design force***

**Check 12: Tying resistance - Supported beam web**

Web resistance; $F_{Rd,u} = (t_{w,b} * h_p * f_{u,b}) / \gamma_{M,u}$ = **247.82** kN

Utilisation; $F_{Ed1} / F_{Rd,u}$ = **0.303**

***PASS - Supported beam web tying resistance is greater than design force***

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| Revision | Date | Engineer | Checked By |
|---|---|---|---|
| - | August 2020 | D.I | M.F |