Hudds Design

Structural & Architectural Engineering
DESIGN CONSULTANTS

PLANNING AND BUILDING CONTROL; CALCULATIONS & DRAWINGS

Office:	103-105 Bradford Road Huddersfield, HD1 6DZ
Office:	01484 941051
Web:	www.huddsdesign.com
Email:	eng@huddsdesign.com

STRUCTURAL CALCULATIONS

FOR

SANTOKH SINGH
@
2 VERNON CLOSE, EDGERTON, HUDDERSFIELD, HD1 5QE

Design Summary: (See Sheet A1)

Notes:

- All dimensions and Beam Lengths to be confirmed and checked by builder on site prior to works commencing and materials being ordered
- All Beam Heights and Levels to be agreed with client on site.
- All steel to be minimum grade S275 (UNO)
- All Timber / Timber Packing to be minimum grade C24 (UNO)
- Minimum end bearing for steel to be 100mm (UNO)
- Minimum end bearing for timber to be 100mm
- Minimum end bearing for lintels to be 150mm
- All bolted connections to be Min. 4 No. M16 Grade 8.8 Bolts per 10mm end plates fully welded to flanges. Edge distance for bolts to be 30mm to bolting member (UNO)
- All welded connections for steel to be minimum 6mm fillet welds all round connecting member (UNO)
- All Beams to have minimum fire protection of 30 minutes or as required by building regulations
- All new masonry blockwork to be minimum 7.3 N/mm^2 strength
- All returns and loadbearing walls as shown on Architects drawings and mark ups
- All concrete to be minimum grade C30
- All steel to be high yield strength, minimum 500 N/mm^2
- All ground conditions to approval of local authority / building inspector
- All temporary support works to builder/contractor's specification
- All party wall notices and approvals are client/contractor's responsibility.
- Hudds Design has not been appointed as CDM coordinator and is the responsibility of the builder.

ISSUED FOR BUILDING CONTROL APPROVAL:	**27th January 2026**
HUDDS DESIGN	**Ref: HD-S26-0104**

SHEET A1
Refer to sketch mark ups for references and Refer to drawings for dimensions

DESIGN SUMMARY

'Rafters RR'	**Roof Rafters** 50x150mm Deep Timber Rafters @400mm Ctrs Min Grade C24 Note: Double Up Rafters all around Roof-lights (UNO)
'Floor FJ'	**Floor Joists** 75x200mm Deep Timber Joists @400mm Ctrs Min Grade C24 Double all joists below stud partitions **OR** Easi Joists WS200 Easi Joist @ 400mm Ctrs, 195mm Depth 122x35 Chords Double all joists below stud partitions
'Trimmer T1/T2'	**Floor Trimmers** 2 No. 75x200mm Deep Timber Joists Min Grade C24
'Purlin P1'	**203x203x46 UC** **RA:** Bolted to Lintel L1 using 4 No. M16 bolts with full length 10mm angle cleats / 10mm end plates fully welded Min 6mm Fillet welds. Min 30mm edge distance for bolts **RB:** Bolted to Purlin P2 using 4 No. M16 bolts with full length 10mm angle cleats / 10mm end plates fully welded Min 6mm Fillet welds. Min 30mm edge distance for bolts **RC:** 300x100x215mm Deep Concrete Padstone
'Hip H1'	**152x152x23 UC** **RA:** Bolted to Masonry with 400mm End Stub/plates using 4 No. M12 Masonry Bolts 400x100x215mm / 400x140x215mm Deep Concrete Padstones **RB:** Bolted to Purlin P2 using 4 No. M16 bolts with full length 10mm angle cleats / 10mm end plates fully welded Min 6mm Fillet welds. Min 30mm edge distance for bolts
'Purlin P2'	**203x203x60 UC** **RA:** 400x100x215mm Deep Concrete Padstone **RB:** 300x140x215mm Deep Concrete Padstone **Full End Bearing 140mm over solid wall for P2**
'Purlin P2A'	**203x203x46 UC** Bolted to Purlin P2 using 4 No. M16 bolts with full length 10mm angle cleats / 10mm end plates fully welded Min 6mm Fillet welds. Min 30mm edge distance for bolts
'Purlin P3'	**152x152x23 UC** 300x140x215mm Deep Concrete Padstone OR **100x300mm Deep Timber Purlins - Min Grade C24**

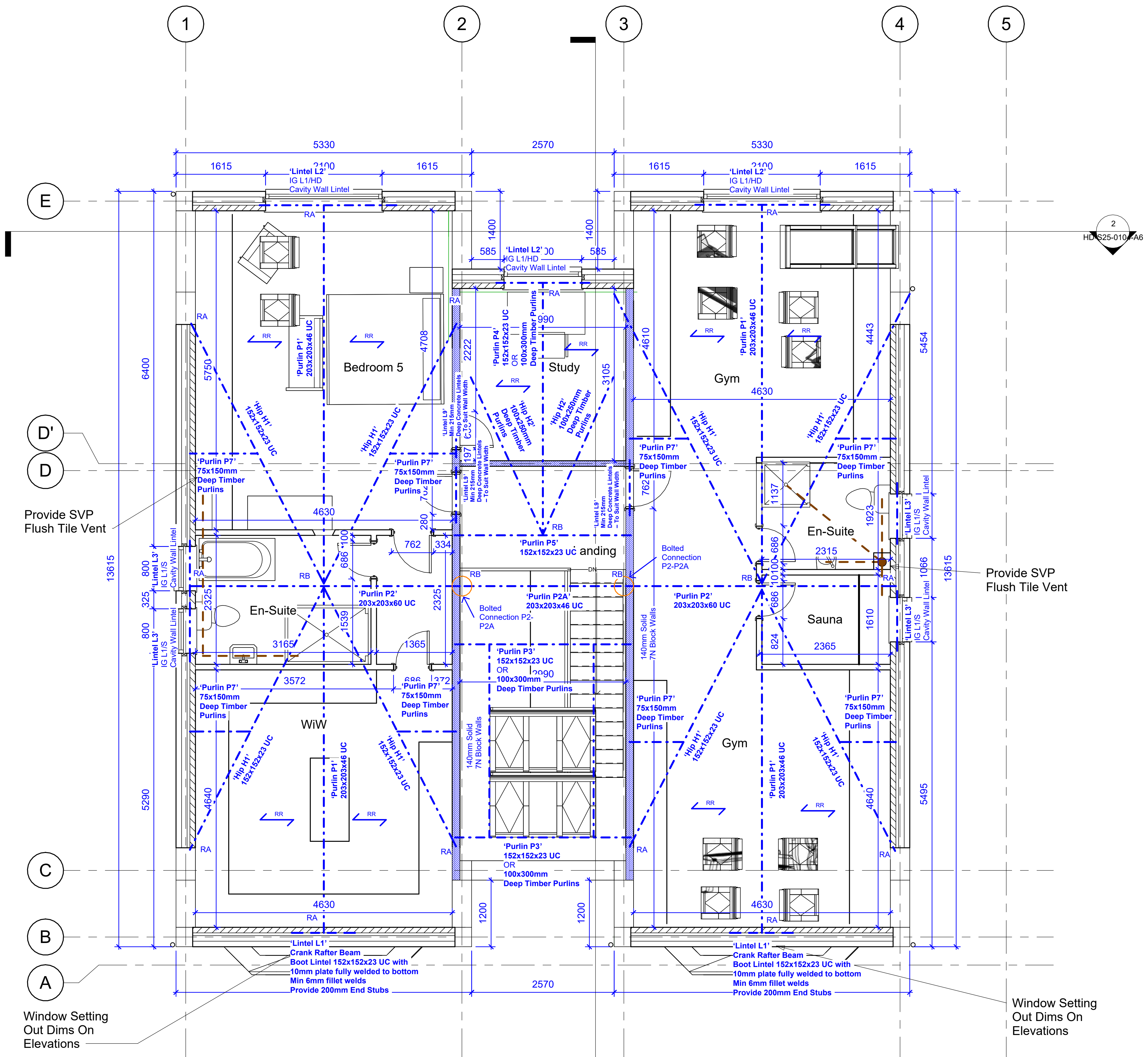
'Purlin P4'	**152x152x23 UC** 300x140x215mm Deep Concrete Padstone <u>OR</u> **100x300mm Deep Timber Purlins - Min Grade C24**
'Hip H2'	**100x250mm Deep Timber Purlins - Min Grade C24**
'Purlin P5'	**152x152x23 UC** 300x140x215mm Deep Concrete Padstone
'Purlin P6'	**Crank Rafter Purlin – 152x152x37 UC** **Fully Welded at joint – Min 6mm Filler Welds** 400x100x215mm Deep Concrete Padstone Bolted to Lintel Hip H3 using 4 No. M16 bolts with full length 10mm angle cleats / 10mm end plates fully welded Min 6mm Fillet welds. Min 30mm edge distance for bolts Bolted to masonry using 400mm end stubs/10mm plates fully welded using 4 No. M12 Anchor Bolts
'Hip H3'	**152x152x23 UC** 400x100x215mm Deep Concrete Padstone Bolted to Lintel L16 using 4 No. M16 bolts with full length 10mm angle cleats / 10mm end plates fully welded Min 6mm Fillet welds. Min 30mm edge distance for bolts Bolted to masonry using 400mm end stubs/10mm plates fully welded using 4 No. M12 Anchor Bolts
'Purlin P7'	**75x150mm Deep Timber Purlins - Min Grade C24**
'Beam B1'	**2 No. 152x152x23 UC's** 440x100x215mm Deep Concrete Padstone Bolted to Lintel L10 using 4 No. M16 bolts with full length 10mm angle cleats / 10mm end plates fully welded Min 6mm Fillet welds. Min 30mm edge distance for bolts
'Beam B2'	**152x152x23 UC** 440x100x215mm Deep Concrete Padstone Bolted to Lintel L10 using 4 No. M16 bolts with full length 10mm angle cleats / 10mm end plates fully welded Min 6mm Fillet welds. Min 30mm edge distance for bolts
'Beam B3'	**2 No. 254x146x43 UB's – Grade S355** **RA**: 500x100x215mm Deep Concrete LINTEL Padstones **RB:** Bolted to Beam B7 using 6 No. M20 bolts per beam with full length 15mm angle cleats / 15mm end plates fully welded Min 6mm Fillet welds. Min 30mm edge distance for bolts
'Beam B4'	**152x152x23 UC** Bolted to Beam B5/B8/B7 using 4 No. M16 bolts per beam with full length 10mm angle cleats / 10mm end plates fully welded Min 6mm Fillet welds. Min 30mm edge distance for bolts

'Beam B5'	**2 No. 152x152x23 UC's** Bolted to Beam B6/B9 using 4 No. M16 bolts with full length 10mm angle cleats / 10mm end plates fully welded Min 6mm Fillet welds. Min 30mm edge distance for bolts
'Beam B6'	**203x203x86 UC** **RA:** 600x200x215mm Deep Concrete LINTEL Padstones **RB:** Bolted to Beam B7/B8 using 4 No. M20 bolts with full length 15mm angle cleats / 15mm end plates fully welded Min 6mm Fillet welds. Min 30mm edge distance for bolts
'Beam B7'	**356x368x202 UC** <u>OR</u> **305x305x240 UC** **RA:** Bolted Moment Connection to Column C2. Bolted connection to be minimum 8 No. M24 Bolts with 10mm end plates fully welded to flanges. Min 30mm end distance for bolts **RB:** Bolted Moment Connection to Beam B9 using 8 No. M24 bolts with full length 15mm angle cleats / 15mm end plates fully welded Min 8mm Fillet welds. Min 30mm edge distance for bolts
'Beam B8'	**356x368x129 UC** <u>OR</u> **305x305x118 UC** **RA:** Bolted Moment Connection to Column C2. Bolted connection to be minimum 8 No. M24 Bolts with 10mm end plates fully welded to flanges. Min 30mm end distance for bolts **RB:** Bolted Moment Connection to Beam B9 using 8 No. M24 bolts with full length 15mm angle cleats / 15mm end plates fully welded Min 8mm Fillet welds. Min 30mm edge distance for bolts
'Beam B9'	**356x368x177 UC** <u>OR</u> **305x305x198 UC** **RA:** Bolted Moment Connection to Beam B10 using 6 No. M20 bolts with full length 15mm angle cleats / 15mm end plates fully welded Min 8mm Fillet welds. Min 30mm edge distance for bolts **RB:** Bolted Moment Connection to Column C1. Bolted connection to be minimum 8 No. M24 Bolts with 10mm end plates fully welded to flanges. Min 30mm end distance for bolts
'Beam B10'	**203x203x46 UC / 305x165x54 UB** 440x300x215mm Deep Concrete Padstone <u>Min 250mm End Bearing</u>
'Beam B11'	**203x102x23 UB** Provide Corrosion Protection
'Column C1/C2'	**203x203x86 UC Grade** **(Provide ties to existing masonry at 225mm ctrs)** With 400x400x15mm Thick **Baseplate** and 4 No. M20 Anchor bolts to Raft Slab with Edge Thickening
'Pier MP'	**Min 1000x300mm Solid 7N Block Pier** **(Provide ties to existing masonry at 225mm ctrs)** On to Raft Slab with edge thickening

'Lintel L1'	**Crank Rafter Beam** **Boot Lintel 152x152x23 UC with 10mm plate fully welded to bottom** **Min 6mm fillet welds** **Provide 200mm End Stubs** 300x100x215mm Deep Concrete Padstones <u>Min 200mm End bearing</u>
'Lintel L2/L8'	IG L1/HD Cavity Wall Lintel
'Lintel L3/L4/L5/L6'	IG L1/S Cavity Wall Lintel
'Lintel L7/L14/L15'	IG L1/S Cavity Wall Lintel
'Lintel L9/L11'	**Min 215mm Deep Concrete Lintels – To Suit Wall Width** <u>Min 150mm End bearing</u>
'Lintel L10'	**Arched Boot Lintel Beam** **Boot Lintel 152x152x23 UC with 10mm plate fully welded to bottom** **Min 6mm fillet welds** **Provide 250mm End Returns to maosnry** 440x100x215mm Deep Concrete Padstones <u>Min 250mm End bearing</u>
'Lintel L12'	**Boot Lintel 203x203x46 UC with 10mm plate fully welded to bottom** **Min 10mm fillet welds** 400x100x215mm Deep Concrete Padstones <u>Min 200mm End bearing</u>
'Lintel L13'	**Boot Lintel 152x152x23 UC with 10mm plate fully welded to bottom** **Min 6mm fillet welds** RA: 300x100x215mm Deep Concrete Padstones <u>Min 150mm End Bearing</u> **RB:** Bolted to Column C3
'Column C3'	**150x150x8 SHS** with 150x150x12 RSA fully welded to support Lintels L13 Provide 4 No. M16 bolt fixings to Lintels /Support Beams Provide 400x400x10mm Thick **Baseplate** with 4 No. M16 Anchor bolts to Footing
'Lintel L16'	**Boot Lintel 152x152x23 UC with 10mm plate fully welded to bottom** **Min 6mm fillet welds** 300x100x215mm Deep Concrete Padstones <u>Min 150mm End Bearing</u>
'Lintel L17'	**Boot Lintel 203x203x86 UC with 15mm plate fully welded to bottom** **Min 15mm fillet welds** 440x100x215mm Deep Concrete Padstones <u>Min 200mm End bearing</u>

'Windpost WP'	**100x100x8 SHS in cavity/inner skin of masonry** with fixings top and bottom Fixed top and Bottom using 10mm cleats fully welded with 2 No. M12 grade 8.8 bolts. Fixings to floor joists, steel beams as required. Provide Ties at 225mm Ctrs to masonry
'Raft Foundation RF'	**USE** 250mm Thick Reinforced Concrete Raft Slab With **2 Layers** A393 Mesh Top **and** Bottom (2 Top & 2 Bottom) Provide edge and internal thickenings as shown 450x450mm RC Beams with 3 No. T16 Bars top and bottom with T10 links at 200mm ctrs. Provide links at 100mm ctrs to all corners 1m away. Add diagonal T16 corner bars (top + bottom) at re-entrant corners and sharp external/internal corners. Full lap throughs for Bars on all corners. **Column C1/C2/MP** **Additional thickening below columns/MP as noted 2.4x2.4m square with T16 Bars 200mm Ctrs** Min Well Compacted Hardcore below to siltstone Min 50mm Cover to all reinforcement Min Grade C30 Concrete **Note:** Coal Depth is 17m at 1.2m Max. 10T (12m) Rule satisfied for Strip Footing. Raft provided for additional robustness. **Refer to Geo Tech Report G25226 Rev B, dated 8th Sept 2025.**

eng@huddsdesign.com
www.huddsdesign.com



Proposed Second Floor Plan

1 : 50

NOTE-
All Dimensions Are Approximate
And TBC On Site By Contractor

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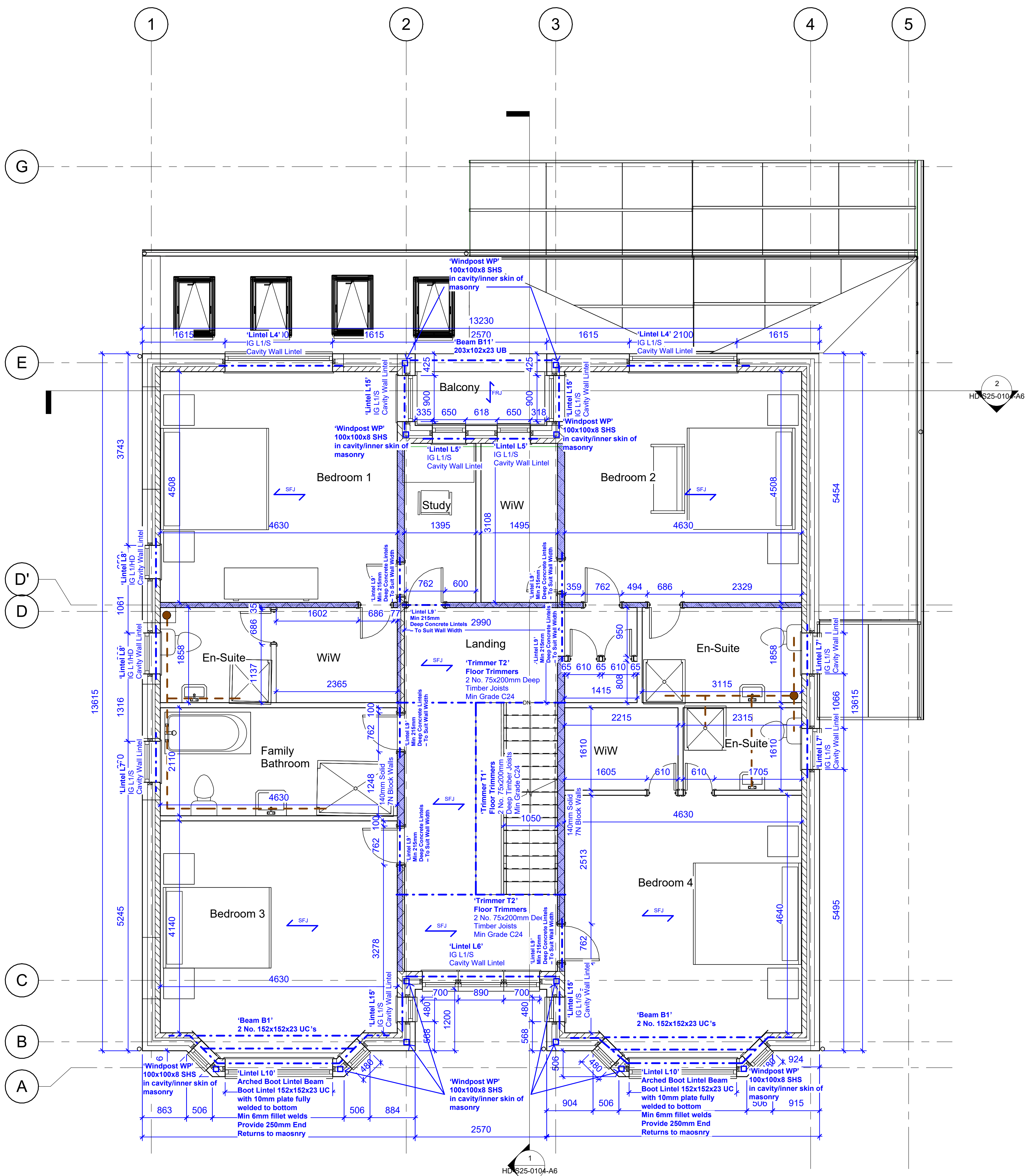
Structural Engineering Consultants
T: 01484 941051
E : eng@huddsdesign.com
www.huddsdesign.com

Project Name 2 Vernon Close, Edgerton, Huddersfield, HD1 5QE	
Drawing Title: Proposed Second Floor Plan	
Drawing Number: HD-S25-0104-A2	Scale: 1 : 50@A1
Date: 29/01/26	Revision:
Drawn By: AM	Checked by: JA
Reason For Issue: Building Regs	



1 **Proposed First Floor Plan**
1 : 50

NOTE-
All Dimensions Are Approximate
And TBC On Site By Contractor

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E : eng@huddsdesign.com
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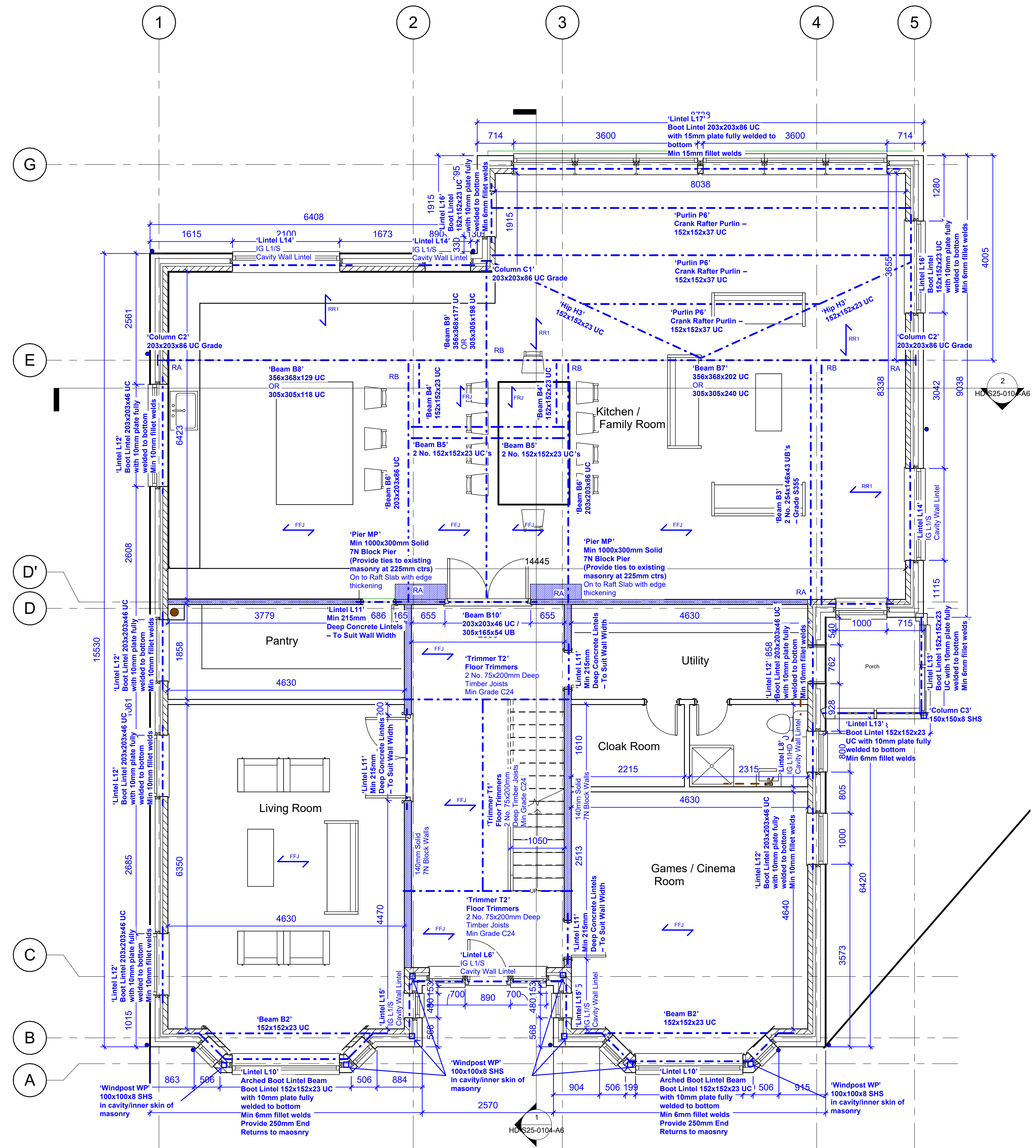
Project Name
2 Vernon Close, Edgerton, Huddersfield, HD1 5QE

Drawing Title:
Proposed First Floor Plan

Drawing Number: HD-S25-0104-A3	Scale: 1 : 50@A1
Date: 29/01/26	Revision: Approver
Drawn By: AM	Checked by: JA

Reason For Issue:
Building Regs



1 **Proposed Ground Floor Plan**
1 : 50

NOTE-
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And TBC On Site By Contractor

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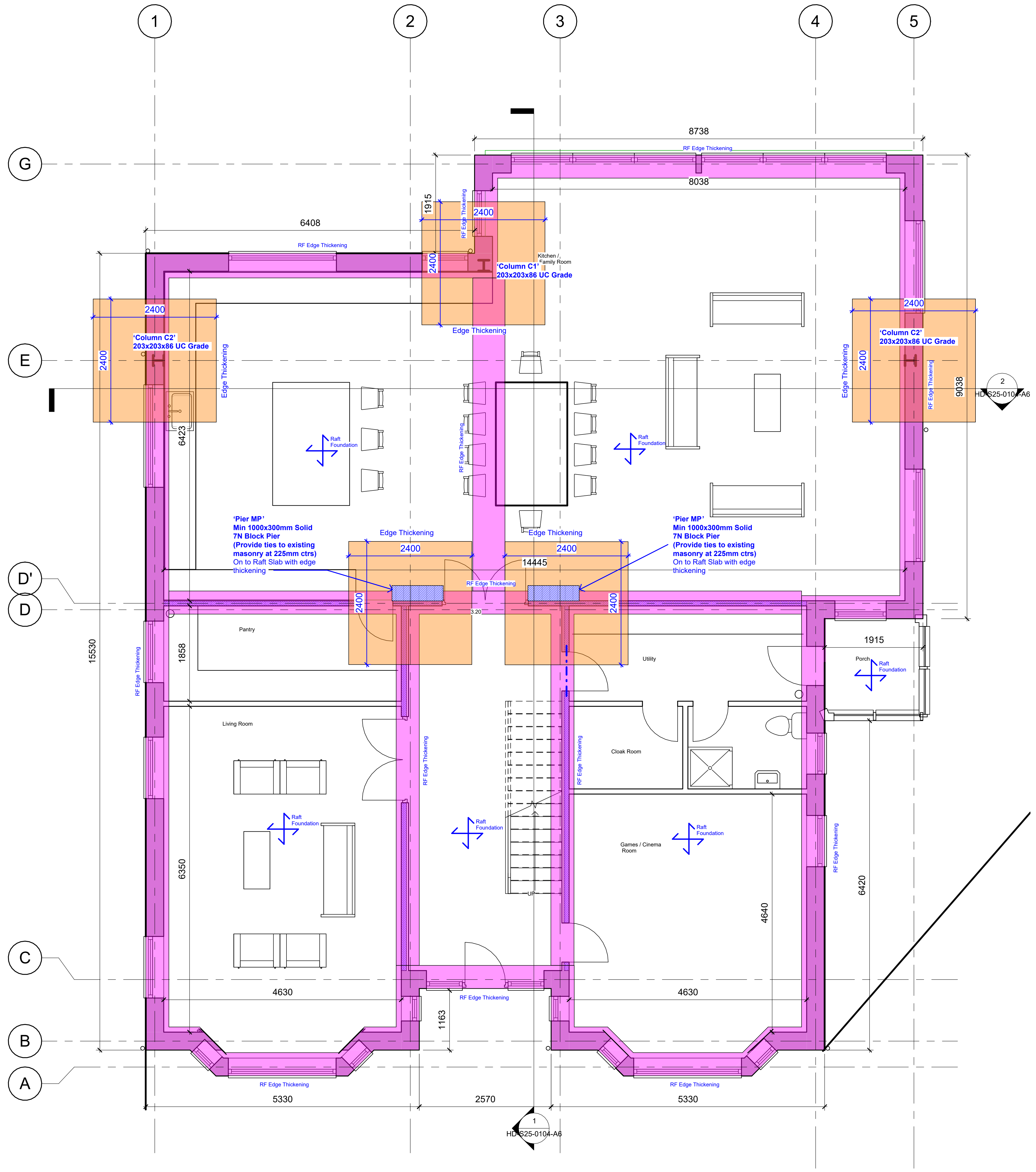
Project Name 2 Vernon Close, Edgerton, Huddersfield, HD1 5QE	
Drawing Title: Proposed Ground Floor Plan	
Drawing Number: HD-S25-0104-A4	Scale: 1 : 50@A1
Date: 29/01/26	Revision: Approver
Drawn By: AM	Checked by: JA
Reason For Issue: Building Regs	



Proposed Foundation Plan

1 : 50

NOTE-
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And TBC On Site By Contractor

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Structural Engineering Consultants
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Project Name
2 Vernon Close, Edgerton, Huddersfield, HD1 5QE

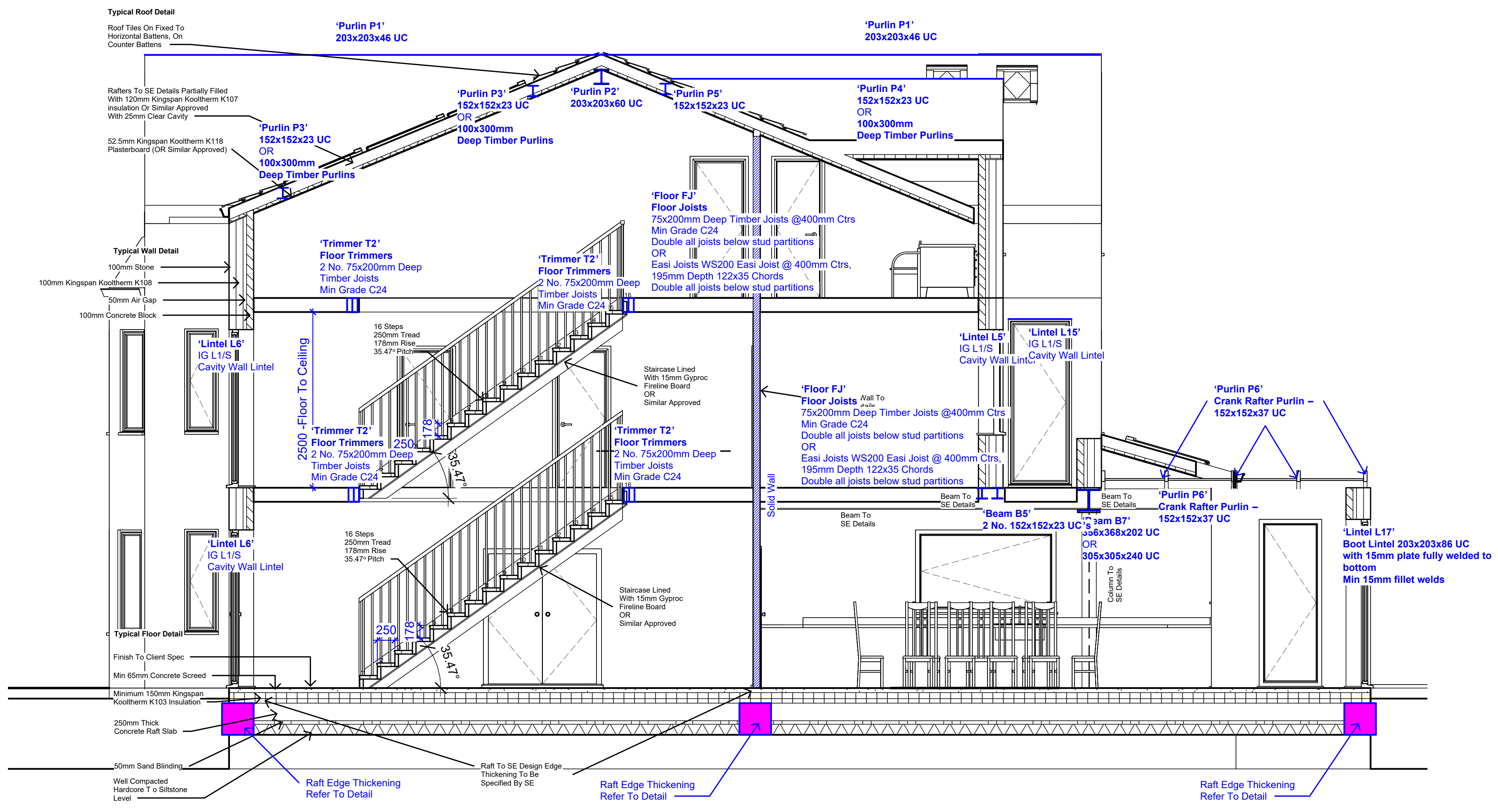
Drawing Title:
Proposed Foundation Plan

Drawing Number: HD-S25-0104-A5
Scale: 1 : 50@A1

Date: 29/01/26
Revision: Approver

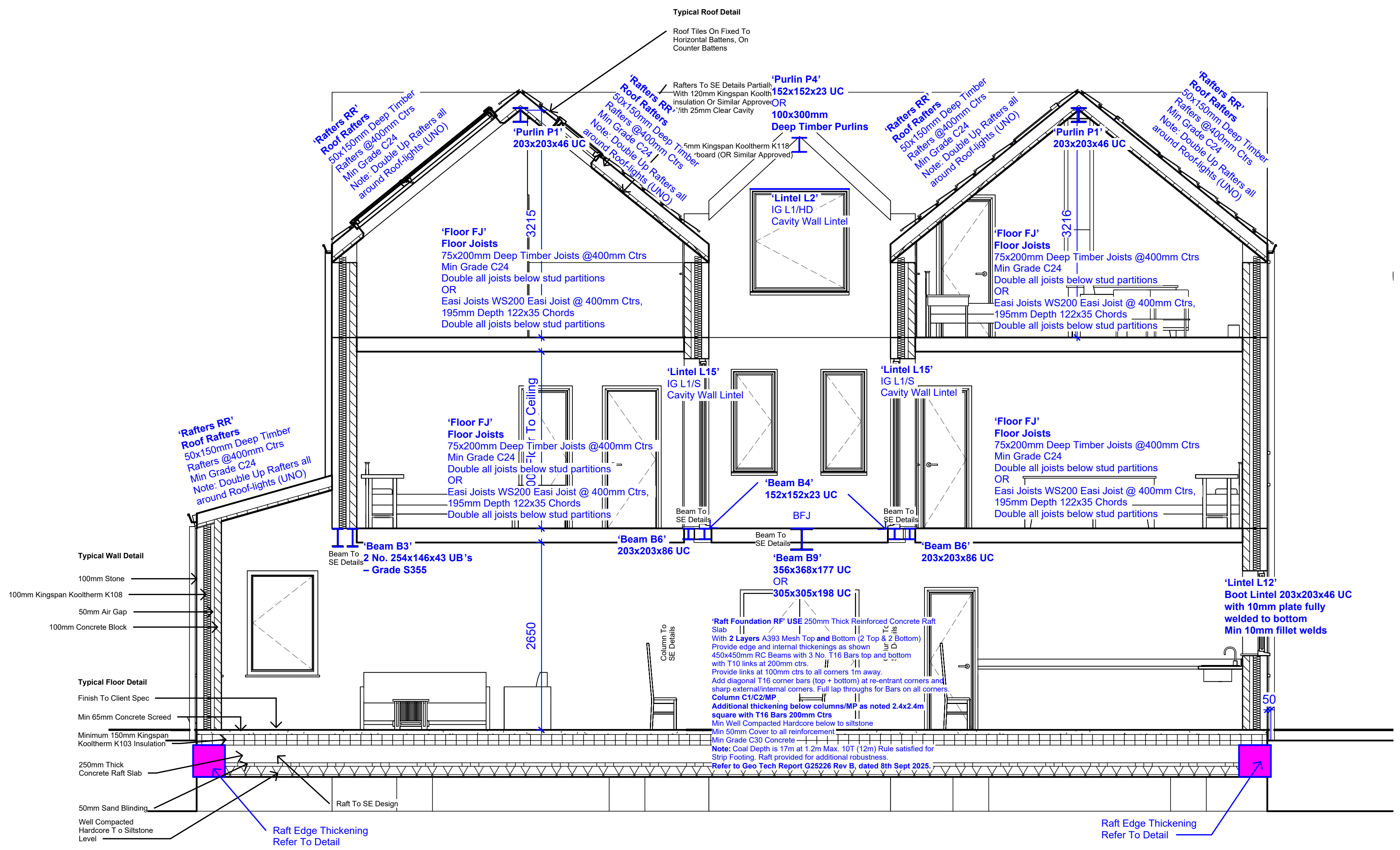
Drawn By: AM
Checked by: JA

Reason For Issue:
Building Regs



① Proposed Section A-A

1 : 50



② Proposed Section B-B

1 : 50

NOTE-
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Project Name
2 Vernon Close, Edgerton, Huddersfield, HD1 5QE

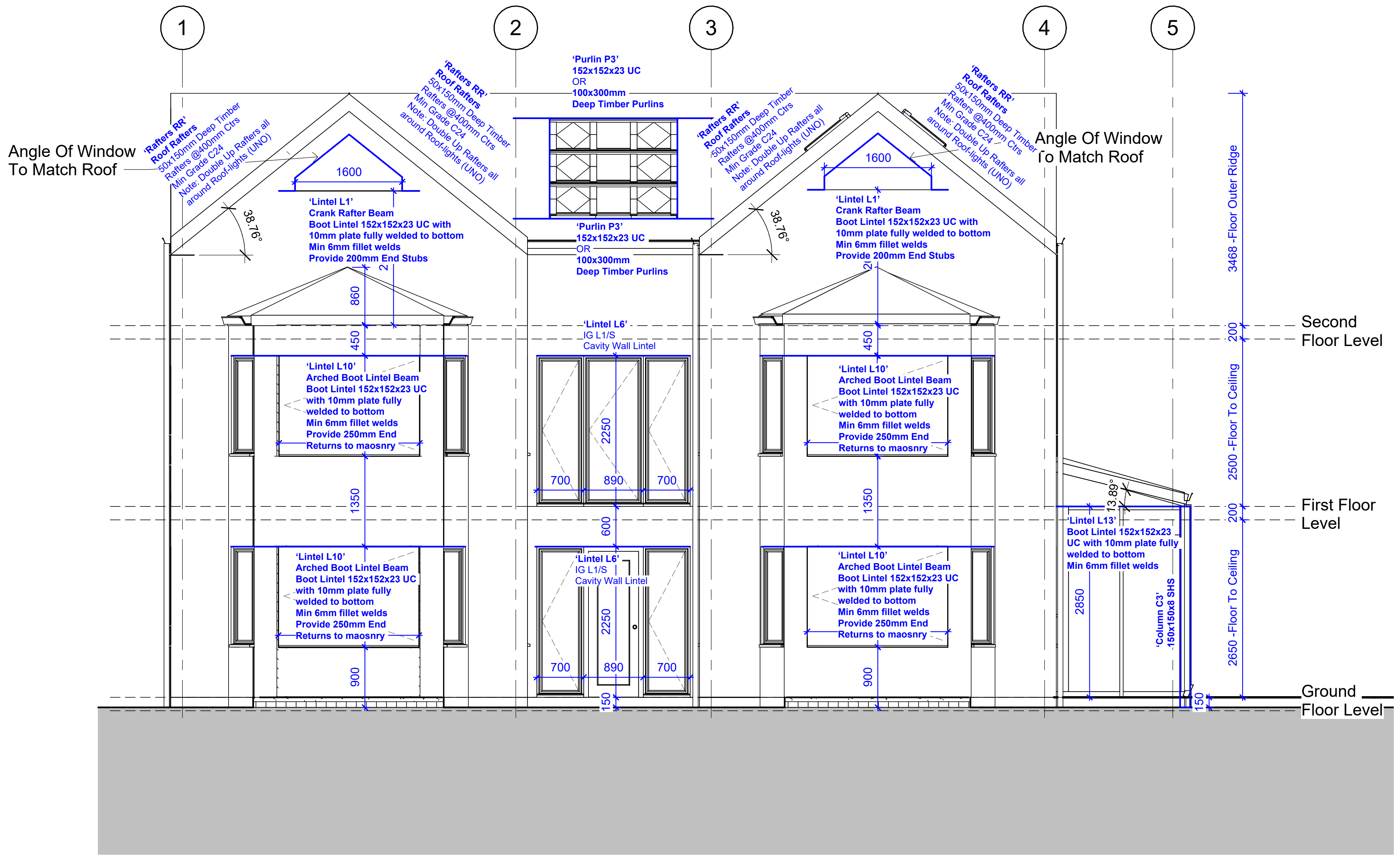
Drawing Title:
Proposed Sections

Drawing Number: HD-S25-0104-A6 | Scale: 1 : 50@A1

Date: 29/01/26 | Revision: Approver

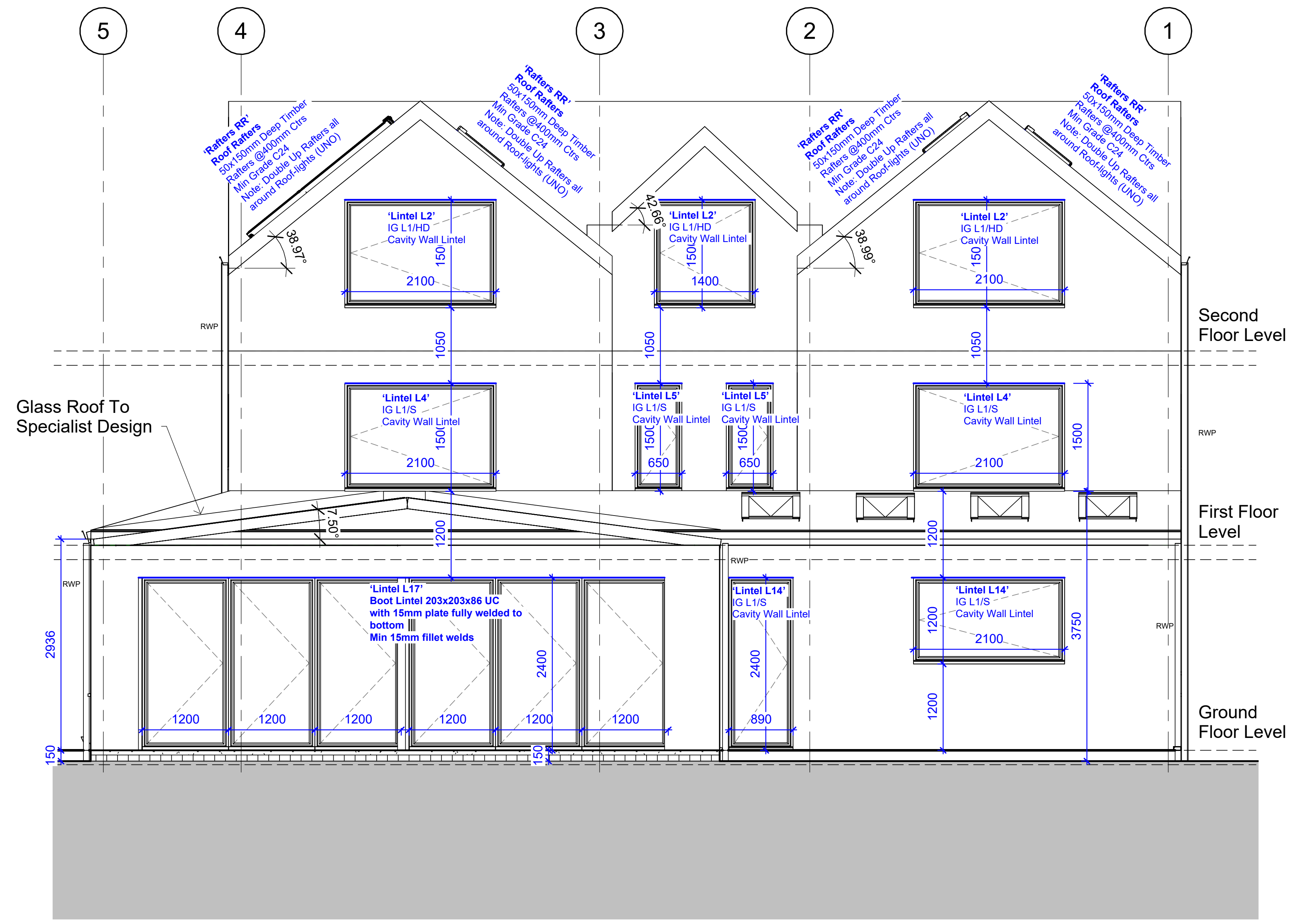
Drawn By: AM | Checked by: JA

Reason For Issue:
Building Regs



2 Proposed South Elevation (Front)

1 : 50



1 Proposed North Elevation (Rear)

1 : 50

NOTE-
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E : eng@huddsdesign.com
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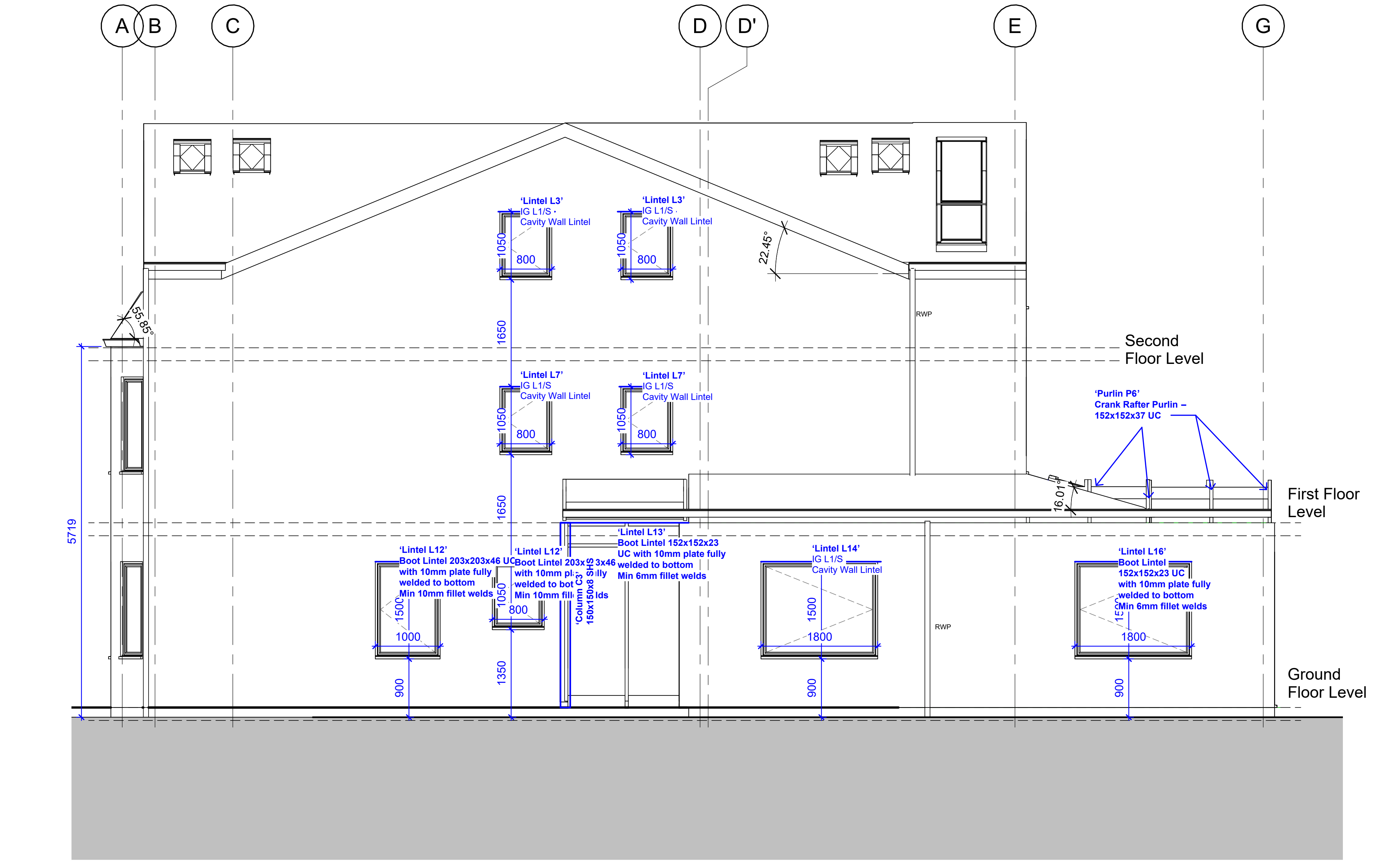
Project Name
2 Vernon Close, Edgerton, Huddersfield, HD1 5QE

Drawing Title:
Proposed Elevations (Front & Rear)

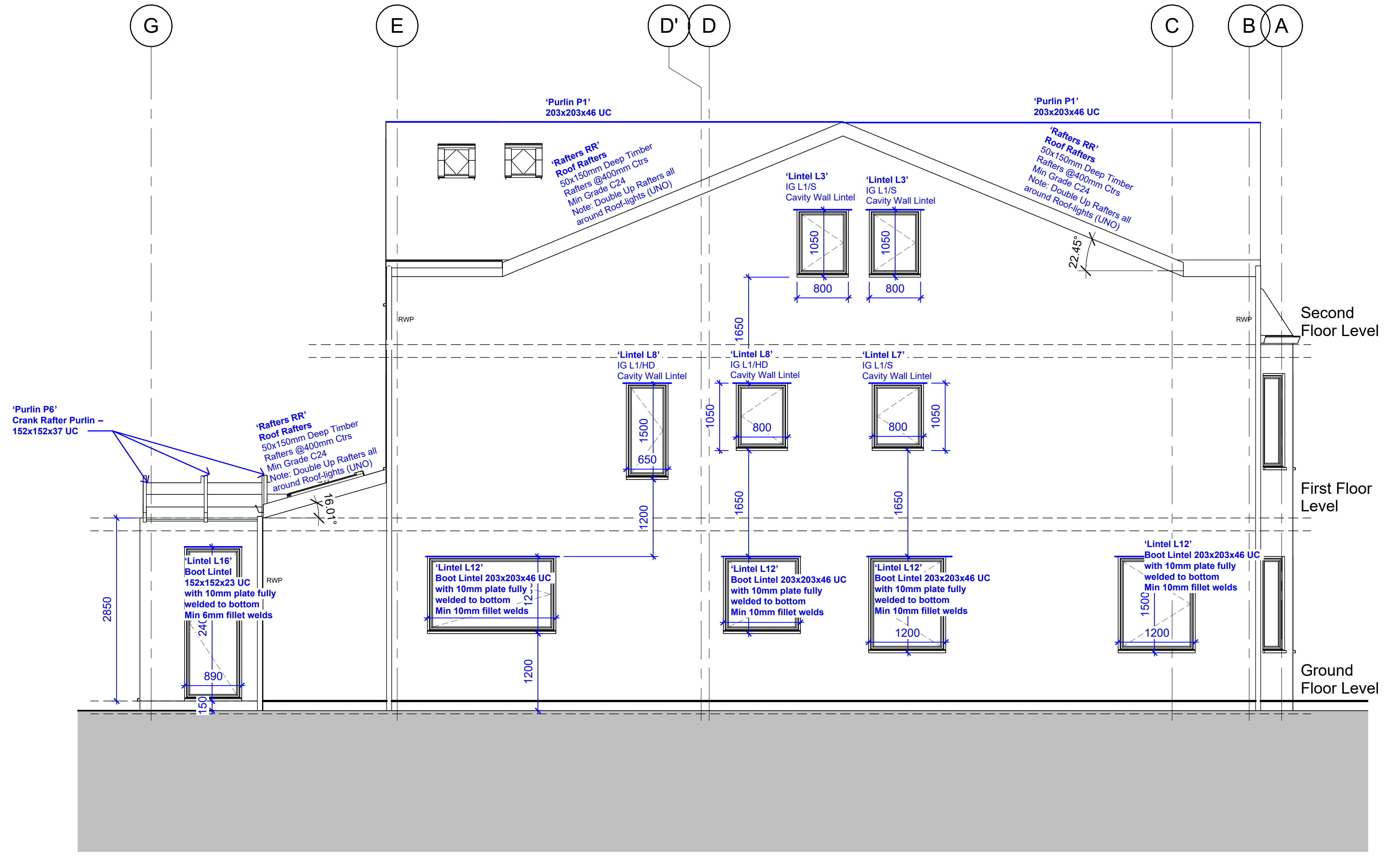
Drawing Number: HD-S25-0104-A7	Scale: 1 : 50@A1
Date: 29/01/26	Revision: Approver
Drawn By: AM	Checked by: JA

Reason For Issue:
Building Regs



1 **Proposed East Elevation**

1 : 50



2 **Proposed West Elevation**

1 : 50

NOTE-
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And TBC On Site By Contractor

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E : eng@huddsdesign.com
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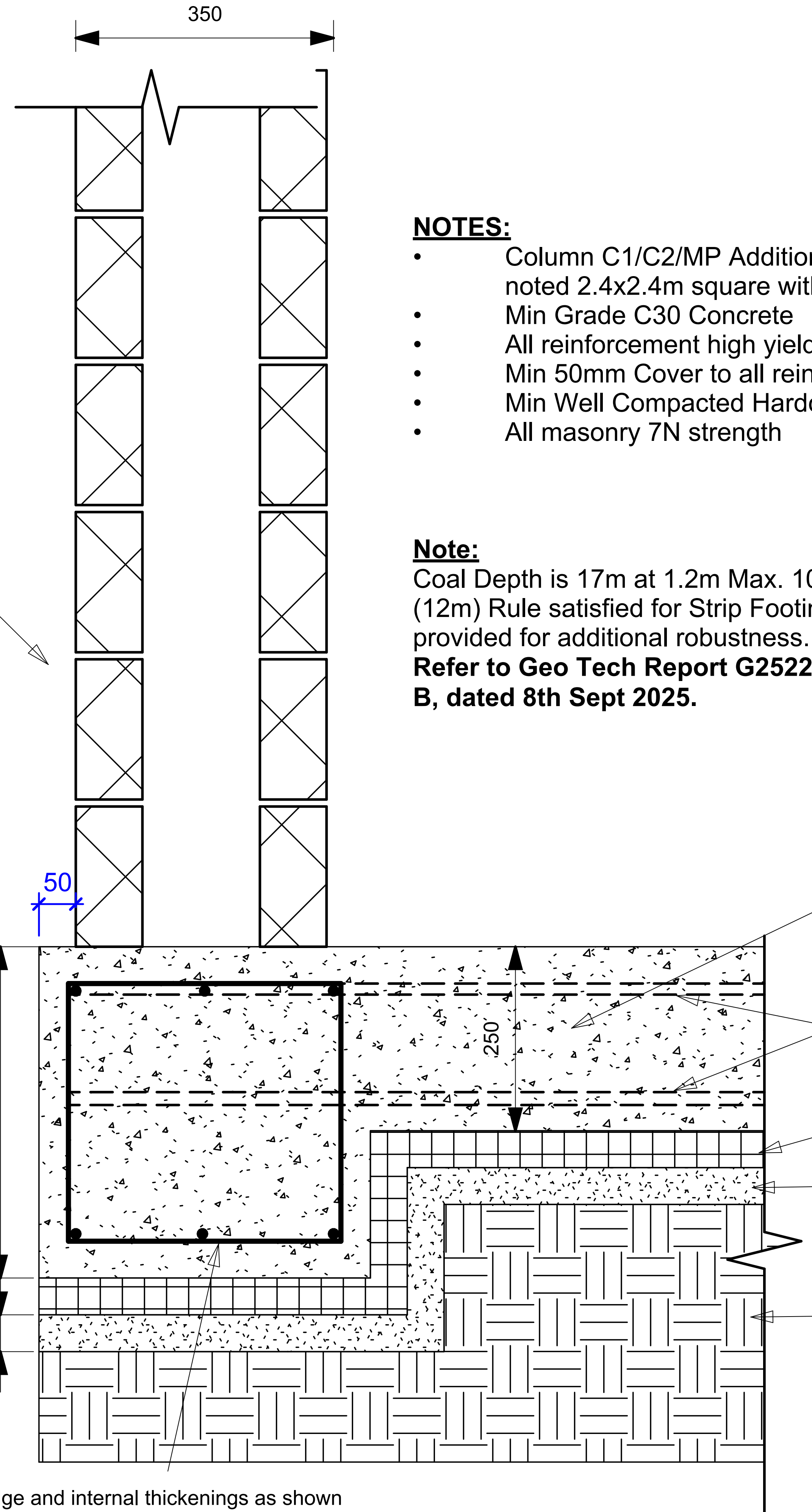
Project Name
2 Vernon Close, Edgerton, Huddersfield, HD1 5QE

Drawing Title:
Proposed Elevations (Side)

Drawing Number: HD-S25-0104-A8	Scale: 1 : 50@A1
Date: 29/01/26	Revision: Approver
Drawn By: AM	Checked by: JA

Reason For Issue:
Building Regs



NOTES:

- Column C1/C2/MP Additional thickening below columns/MP as noted 2.4x2.4m square with T16 Bars 200mm Ctrs
- Min Grade C30 Concrete
- All reinforcement high yield $500N/mm^2$
- Min 50mm Cover to all reinforcement
- Min Well Compacted Hardcore below to siltstone
- All masonry 7N strength

Note:
Coal Depth is 17m at 1.2m Max. 10T (12m) Rule satisfied for Strip Footing. Raft provided for additional robustness.
Refer to Geo Tech Report G25226 Rev B, dated 8th Sept 2025.

RF Detail
1 : 10

NOTE-
All Dimensions Are Approximate
And TBC On Site By Contractor

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Project Name	
2 Vernon Close, Edgerton, Huddersfield, HD1 5QE	
Drawing Title:	
Proposed Raft Foundation Detail	
Drawing Number:	Scale:
HD-S25-0104-A9	1 : 10@A3
Date: 29/01/26	Revision: Approver
Drawn By: AM	Checked by: JA
Reason For Issue:	
Building Regs	

STRUCTURAL CALCULATIONS

Address: **2 Vernon Close, Edgerton, Huddersfield, HD1 5QE**

Calculations for:

New Build with raft foundation Design

DESIGN LOADINGS

External Walls

Inner Skin → 0.1m x 20 kN/m^3 = 2.0 kN/m^2 – (**Block**)

Inner Skin → 0.15m x 20 kN/m^3 = 3.0 kN/m^2 – (**Block 140mm**)

External Skin → 0.1m x 24 kN/m^3 = 2.4 kN/m^2 – (**Stone**)

Roof – (Tiles)

Dead Loads	= 1.1 kN/m^2 – (1.3 to Allow for Ceiling Joists Loads)
Imposed Loads	= 0.6 kN/m^2
Total	**= 1.9 kN/m^2**

First/Second Floor – (Timber)

Dead Loads	= 1.0 kN/m^2
Imposed Loads	= 1.5 kN/m^2
Total	**= 2.1 kN/m^2**

eng@huddsdesign.com
www.huddsdesign.com

Hudds Design Structural Engineering Consultants Hudds Design 103-105 Bradford Rd Huddersfield HD1 6DZ	Project 2 Vernon Close, Edgerton, Huddersfield, HD1 5QE				Job no. HD-S26-0104	
	Calcs for Floor Joists FJ				Start page no./Revision FJ- 1	
	Calcs by AM	Calcs date 28/01/2026	Checked by JA	Checked date 28/01/2026	Approved by	Approved date

TIMBER JOIST DESIGN (BS5268-2:2002)

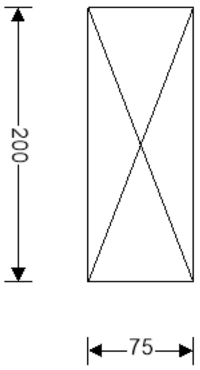
Tedds calculation version 1.1.04

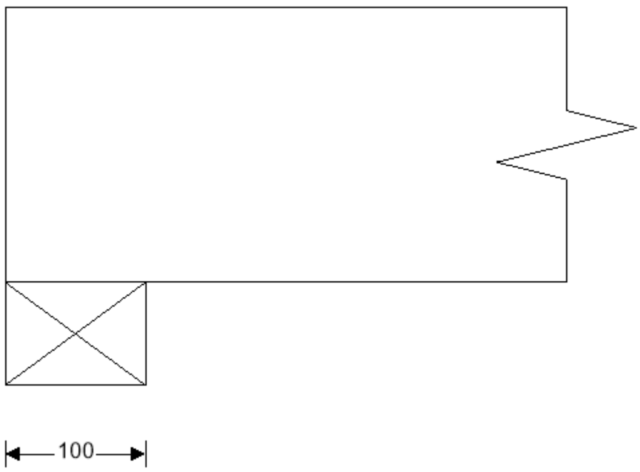
Joist details

Joist breadth	b = **75** mm	Joist depth	h = **200** mm
Joist spacing	s = **400** mm	Service class of timber	**1**
Timber strength class	**C24**		

Span details

Number of spans	N_{span} = **1**	Length of bearing	L_b = **100** mm
Clear length of span	L_{s1} = **4650** mm		





Section properties

Second moment of area	I = **50000000** mm^4	Section modulus	Z = **500000** mm^3

Loading details

Joist self weight	F_{swt} = **0.05** kN/m	Dead load	F_{d_udl} = **1.00** kN/m^2
Imposed UDL(Long term)	F_{i_udl} = **1.50** kN/m^2		
Imposed point load (Medium)	F_{i_pt} = **1.40** kN		

Consider long term loads

Design bending moment	M = **2.842** kNm	Design shear force	V = **2.445** kN
Design support reaction	R = **2.445** kN	Design deflection	δ = **12.191** mm

Check bending stress

Permissible bending stress	σ_{m_adm} = **8.626** N/mm^2	Applied bending stress	σ_{m_max} = **5.684** N/mm^2

PASS - Applied bending stress within permissible limits

Check shear stress

Permissible shear stress τ_{adm} = **0.781** N/mm^2 Applied shear stress τ_{max} = **0.244** N/mm^2

PASS - Applied shear stress within permissible limits

Check bearing stress

Permissible bearing stress σ_{c_adm} = **2.640** N/mm^2 Applied bearing stress σ_{c_max} = **0.326** N/mm^2

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection δ_{adm} = **13.950** mm Actual deflection δ = **12.191** mm

PASS - Actual deflection within permissible limits

Consider medium term loads

Design bending moment M = **2.848** kNm Design shear force V = **2.450** kN

Design support reaction R = **2.450** kN Design deflection δ = **10.858** mm

Check bending stress

Permissible bending stress σ_{m_adm} = **10.783** N/mm^2 Applied bending stress σ_{m_max} = **5.696** N/mm^2

PASS - Applied bending stress within permissible limits

Check shear stress

Permissible shear stress τ_{adm} = **0.976** N/mm^2 Applied shear stress τ_{max} = **0.245** N/mm^2

PASS - Applied shear stress within permissible limits

Check bearing stress

Permissible bearing stress σ_{c_adm} = **3.300** N/mm^2 Applied bearing stress σ_{c_max} = **0.327** N/mm^2

PASS - Applied bearing stress within permissible limits

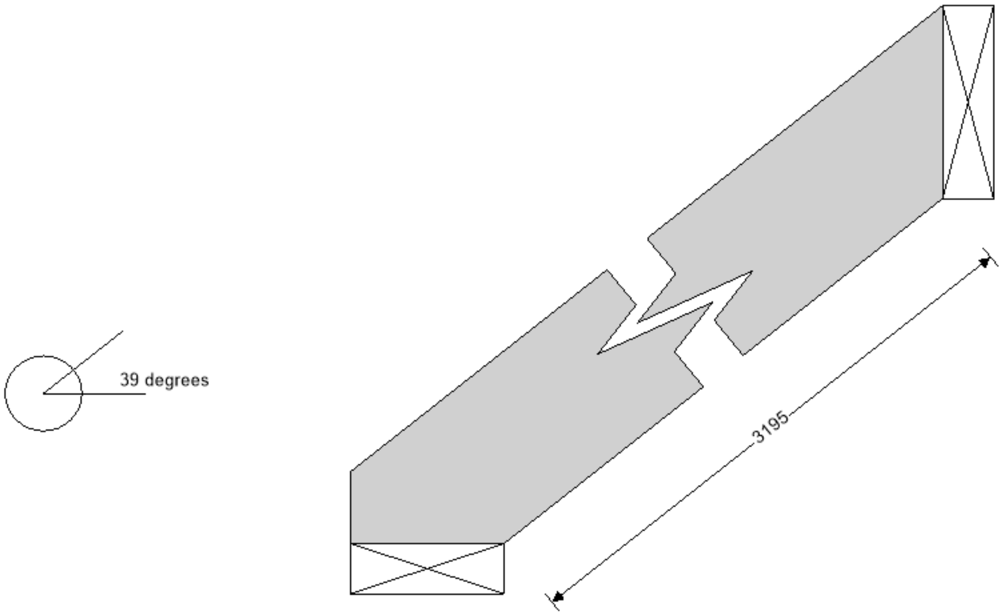
Check deflection

Permissible deflection δ_{adm} = **13.950** mm Actual deflection δ = **10.858** mm

PASS - Actual deflection within permissible limits

TIMBER RAFTER DESIGN (BS5268-2:2002)

TEDDS calculation version 1.0.03



Rafter details

Breadth of timber sections b = **50** mm
Depth of timber sections h = **150** mm
Rafter spacing s = **400** mm
Rafter span **Single span**
Clear length of span on slope L_{cl} = **3195** mm
Rafter slope α = **38.8** deg
Timber strength class **C24**

Section properties

Cross sectional area of rafter A = **7500** mm^2
Section modulus Z = **187500** mm^3
Radius of gyration r = **43** mm
Second moment of area I = **14062500** mm^4

Loading details

Rafter self weight F_j = **0.03** kN/m
Dead load on slope F_d = **1.00** kN/m^2
Imposed snow load on plan F_u = **0.60** kN/m^2
Imposed point load F_p = **0.90** kN

Modification factors

Section depth factor K_7 = **1.08**
Load sharing factor K_8 = **1.10**

Consider long term load condition

Load duration factor K_3 = **1.00**
Total UDL perp. to rafter F = **0.332** kN/m
Notional bearing length L_b = **4** mm
Effective span L_{eff} = **3199** mm

Check bending stress

Permissible bending stress σ_{m_adm} = **8.904** N/mm^2
Applied bending stress σ_{m_max} = **2.264** N/mm^2

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain

Permissible comp. stress σ_{c_adm} = **4.928** N/mm^2
Applied compressive stress σ_{c_max} = **0.259** N/mm^2

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain

Combined loading check **0.311 < 1**

PASS - Combined compressive and bending stresses are within permissible limits

Check shear stress

Permissible shear stress τ_{adm} = **0.781** N/mm^2 Applied shear stress τ_{max} = **0.106** N/mm^2

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection δ_{adm} = **9.597** mm Total deflection δ_{max} = **3.080** mm

PASS - Total deflection within permissible limits

Consider medium term load condition

Load duration factor K_3 = **1.25** Total UDL perp. to rafter F = **0.478** kN/m

Notional bearing length L_b = **6** mm Effective span L_{eff} = **3201** mm

Check bending stress

Permissible bending stress σ_{m_adm} = **11.130** N/mm^2 Applied bending stress σ_{m_max} = **3.262** N/mm^2

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain

Permissible comp. stress σ_{c_adm} = **5.677** N/mm^2 Applied compressive stress σ_{c_max} = **0.373** N/mm^2

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain

Combined loading check **0.365 < 1**

PASS - Combined compressive and bending stresses are within permissible limits

Check shear stress

Permissible shear stress τ_{adm} = **0.976** N/mm^2 Applied shear stress τ_{max} = **0.153** N/mm^2

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection δ_{adm} = **9.602** mm Total deflection δ_{max} = **4.443** mm

PASS - Total deflection within permissible limits

Consider short term load condition

Load duration factor K_3 = **1.50** Total UDL perp. to rafter F = **0.332** kN/m

Notional bearing length L_b = **7** mm Effective span L_{eff} = **3202** mm

Check bending stress

Permissible bending stress σ_{m_adm} = **13.355** N/mm^2 Applied bending stress σ_{m_max} = **5.262** N/mm^2

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain

Permissible comp. stress σ_{c_adm} = **6.268** N/mm^2 Applied compressive stress σ_{c_max} = **0.334** N/mm^2

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain

Combined loading check **0.455 < 1**

PASS - Combined compressive and bending stresses are within permissible limits

Check shear stress

Permissible shear stress τ_{adm} = **1.172** N/mm^2 Applied shear stress τ_{max} = **0.247** N/mm^2

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection δ_{adm} = **9.605** mm Total deflection δ_{max} = **6.381** mm

PASS - Total deflection within permissible limits

Purlin P1

Max Clear Span = 6700mm

Dead Load

Roof DL	=	3.2m x 1.1 kN/m^2	= 3.5 kN/m
		TOTAL DL	**= 3.5 kN/m**

Live Load

Roof LL	=	3.2m x 0.6 kN/m^2	= 1.9 kN/m
		TOTAL LL	**= 1.9 kN/m**

USE 203x203x46 UC
(SEE CALC P1)

Padstones

R = 29 kN

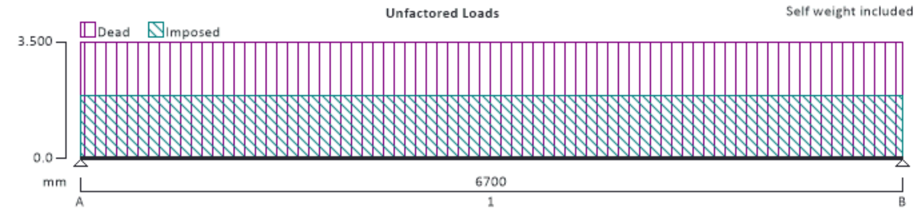
USE 300 x 100 x 215mm DEEP CONCRETE PADSTONES

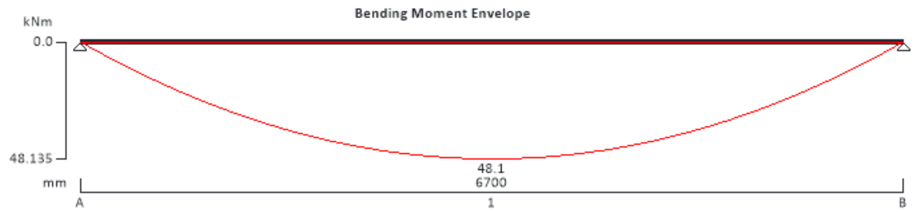
Stress Check = 29 x 1000 / 300 x 100 = 1.0 N/mm^2 < 2.4 N/mm^2
Satisfactory

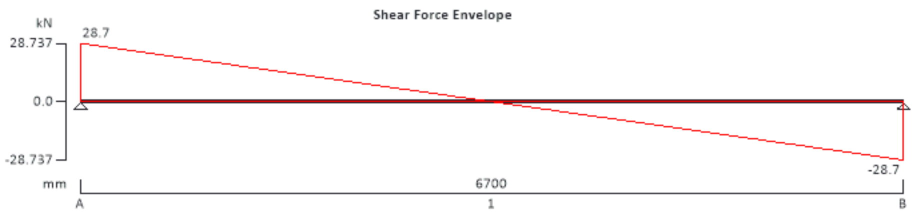
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.07







Support conditions

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

Applied loading

Beam loads	Dead self weight of beam × 1 Dead full UDL 3.5 kN/m Imposed full UDL 1.9 kN/m

Load combinations

Load combination 1	Support A	Dead × 1.40 Imposed × 1.60
		Dead × 1.40 Imposed × 1.60
	Support B	Dead × 1.40 Imposed × 1.60

Analysis results

Maximum moment	M_{max} = **48.1** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **28.7** kN	V_{min} = **-28.7** kN
Deflection	δ_{max} = **5.3** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **28.7** kN	R_{A_min} = **28.7** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **13.3** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **6.4** kN	
Maximum reaction at support B	R_{B_max} = **28.7** kN	R_{B_min} = **28.7** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **13.3** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **6.4** kN	

Section details

Section type **UC 203x203x46 (British Steel Section Range 2022 (BS4-1))** Steel grade **S275**

Classification of cross sections - Section 3.5

Tensile strain coefficient ε = **1.00** Section classification **Compact**

Shear capacity - Section 4.2.3

Design shear force F_v = **28.7** kN Design shear resistance P_v = **241.4** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment M = **48.1** kNm Moment capacity low shear M_c = **138** kNm

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment M_b = **70.8** kNm M_b / m_{LT} = **76.5** kNm

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection δ_{lim} = **18.611** mm Maximum deflection δ = **5.281** mm

PASS - Maximum deflection does not exceed deflection limit

Hip H1

Max Clear Span = 5100mm

Dead Load

Roof DL	=	2.0m x 1.1 kN/m^2	= 2.2 kN/m
		TOTAL DL	**= 2.2 kN/m**

Live Load

Roof LL	=	2.0m x 0.6 kN/m^2	= 1.2 kN/m
		TOTAL LL	**= 1.2 kN/m**

USE 152x152x23 UC
(SEE CALC H1)
Provide 400mm End Stubs/Plates to bolt to masonry

Padstones

R = 14 kN

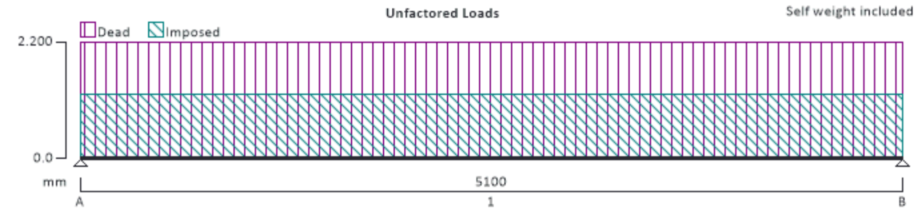
USE 400 x 100 x 215mm DEEP CONCRETE PADSTONES

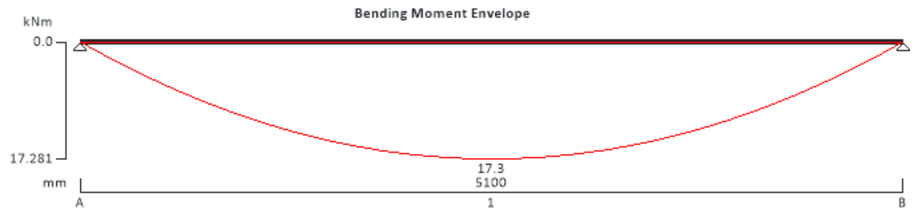
Stress Check = 14 x 1000 / 400 x 100 = 0.5 N/mm^2 < 2.4 N/mm^2
Satisfactory

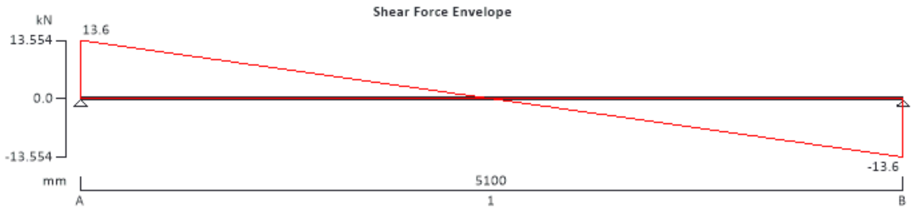
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.07







Support conditions

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

Applied loading

Beam loads	Dead self weight of beam × 1
	Dead full UDL 2.2 kN/m
	Imposed full UDL 1.2 kN/m

Load combinations

Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60
		Dead × 1.40
		Imposed × 1.60
	Support B	Dead × 1.40
		Imposed × 1.60

Analysis results

Maximum moment M_{max} = **17.3** kNm M_{min} = **0** kNm

Maximum shear V_{max} = **13.6** kN V_{min} = **-13.6** kN

Deflection δ_{max} = **4.1** mm δ_{min} = **0** mm

Maximum reaction at support A R_{A_max} = **13.6** kN R_{A_min} = **13.6** kN

Unfactored dead load reaction at support A R_{A_Dead} = **6.2** kN

Unfactored imposed load reaction at support A $R_{A\_Imposed}$ = **3.1** kN

Maximum reaction at support B R_{B_max} = **13.6** kN R_{B_min} = **13.6** kN

Unfactored dead load reaction at support B R_{B_Dead} = **6.2** kN

Unfactored imposed load reaction at support B $R_{B\_Imposed}$ = **3.1** kN

Section details

Section type **UC 152x152x23 (British Steel Section Range 2022 (BS4-1))** Steel grade **S275**

Classification of cross sections - Section 3.5

Tensile strain coefficient ε = **1.00** Section classification **Semi-compact**

Shear capacity - Section 4.2.3

Design shear force F_v = **13.6** kN Design shear resistance P_v = **145.8** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment M = **17.3** kNm Moment capacity low shear M_c = **48.5** kNm

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment M_b = **22.5** kNm M_b / m_{LT} = **24.3** kNm

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection δ_{lim} = **14.167** mm Maximum deflection δ = **4.126** mm

PASS - Maximum deflection does not exceed deflection limit

Purlin P2

Max Clear Span = 4650mm

Dead Load

Roof DL	=	2.0m x 1.1 kN/m^2	= 2.2 kN/m
		TOTAL DL	**= 2.2 kN/m**

Live Load

Roof LL	=	2.0m x 0.6 kN/m^2	= 1.2 kN/m
		TOTAL LL	**= 1.2 kN/m**

DL P1	=	13.3 kN x 2 No.	= 26.6 kN
DL H1	=	6.2 kN x 4 No.	= 24.8 kN
		TOTAL DL	**= 51.4 kN @ Mid Span**

LL P1	=	6.4 kN x 2 No.	= 12.8 kN
LL H1	=	3.1 kN x 4 No.	= 12.4 kN
		TOTAL LL	**= 25.2 kN @ Mid Span**

USE 203x203x60 UC
(SEE CALC P2)

Padstones

R = 70 kN

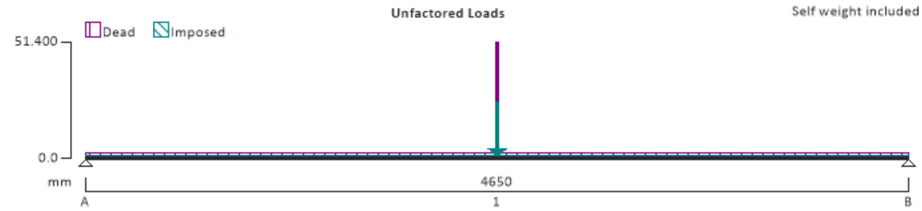
USE 400 x 100 x 215mm DEEP CONCRETE PADSTONES

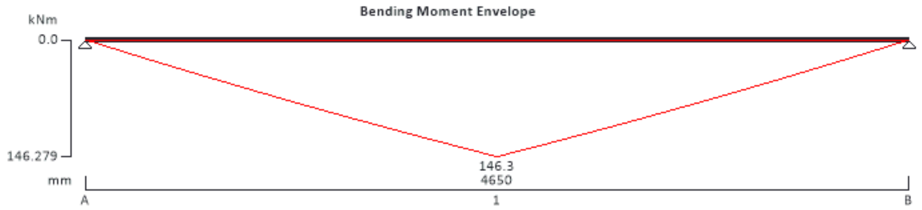
Stress Check = 70 x 1000 / 400 x 100 = 1.8 N/mm^2 < 2.4 N/mm^2
Satisfactory

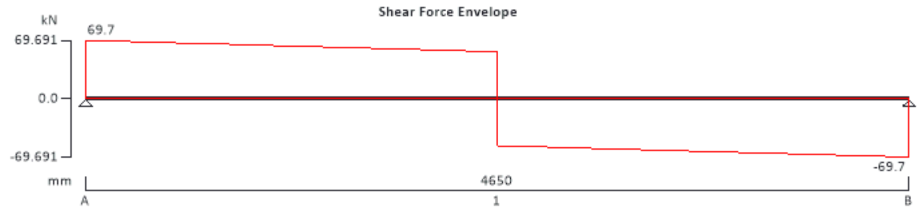
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.07







Support conditions

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

Applied loading

Beam loads	Dead self weight of beam × 1
	Dead full UDL 2.2 kN/m
	Imposed full UDL 1.2 kN/m
	Dead point load 51.4 kN at 2325 mm
	Imposed point load 25.2 kN at 2325 mm

Load combinations

Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60
		Dead × 1.40
		Imposed × 1.60
	Support B	Dead × 1.40
		Imposed × 1.60

Analysis results

Maximum moment	M_{max} = **146.3** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **69.7** kN	V_{min} = **-69.7** kN
Deflection	δ_{max} = **4.8** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **69.7** kN	R_{A_min} = **69.7** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **32.2** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **15.4** kN	
Maximum reaction at support B	R_{B_max} = **69.7** kN	R_{B_min} = **69.7** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **32.2** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **15.4** kN	

Section details

Section type **UC 203x203x60 (British Steel Section Range 2022 (BS4-1))** Steel grade **S275**

Classification of cross sections - Section 3.5

Tensile strain coefficient ε = **1.00** Section classification **Plastic**

Shear capacity - Section 4.2.3

Design shear force F_v = **69.7** kN Design shear resistance P_v = **325.1** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment M = **146.3** kNm Moment capacity low shear M_c = **181.6** kNm

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment M_b = **128.8** kNm M_b / m_{LT} = **150.1** kNm

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection δ_{lim} = **12.917** mm Maximum deflection δ = **4.757** mm

PASS - Maximum deflection does not exceed deflection limit

Purlin P3

Max Clear Span = 3100mm

Dead Load

Roof DL = 2.4m x 1.1 kN/m^2 = 2.7 kN/m

TOTAL DL = 2.7 kN/m

Live Load

Roof LL = 2.4m x 0.6 kN/m^2 = 1.5 kN/m

TOTAL LL = 1.5 kN/m

USE
152x152x23 UC
OR
100x250mm Deep Timber – Min Grade C24
(SEE CALC P3/P3T)

Padstones

R = 12 kN

USE 200 x 140 x 215mm DEEP CONCRETE PADSTONES

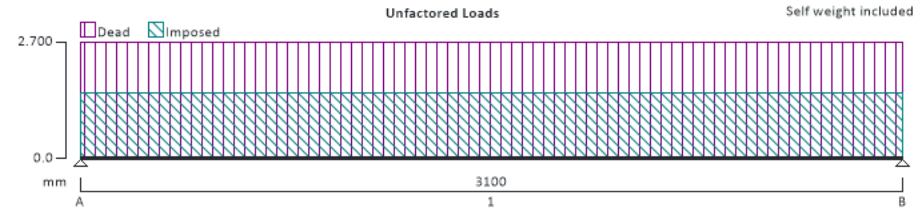
Stress Check = 12 x 1000 / 200 x 100 = 0.6 N/mm^2 < 2.4 N/mm^2

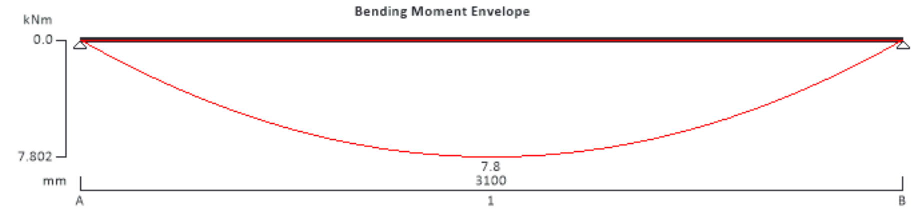
Satisfactory

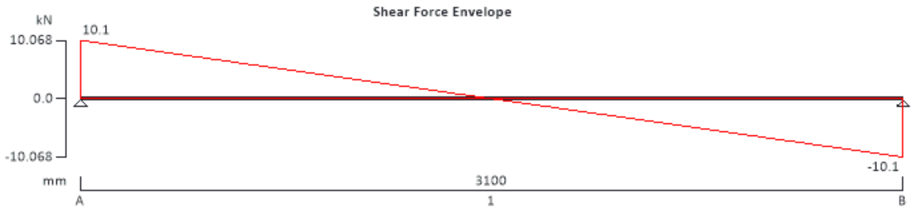
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.07







Support conditions

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

Applied loading

Beam loads	Dead self weight of beam × 1
	Dead full UDL 2.7 kN/m
	Imposed full UDL 1.5 kN/m

Load combinations

Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60
		Dead × 1.40
		Imposed × 1.60
	Support B	Dead × 1.40
		Imposed × 1.60

Analysis results

Maximum moment	M_{max} = **7.8** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **10.1** kN	V_{min} = **-10.1** kN
Deflection	δ_{max} = **0.7** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **10.1** kN	R_{A_min} = **10.1** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **4.5** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **2.3** kN	
Maximum reaction at support B	R_{B_max} = **10.1** kN	R_{B_min} = **10.1** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **4.5** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **2.3** kN	

Section details

Section type **UC 152x152x23 (British Steel Section Range 2022 (BS4-1))** Steel grade **S275**

Classification of cross sections - Section 3.5

Tensile strain coefficient ε = **1.00** Section classification **Semi-compact**

Shear capacity - Section 4.2.3

Design shear force F_v = **10.1** kN Design shear resistance P_v = **145.8** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment M = **7.8** kNm Moment capacity low shear M_c = **48.5** kNm

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment M_b = **32.2** kNm M_b / m_{LT} = **34.8** kNm

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

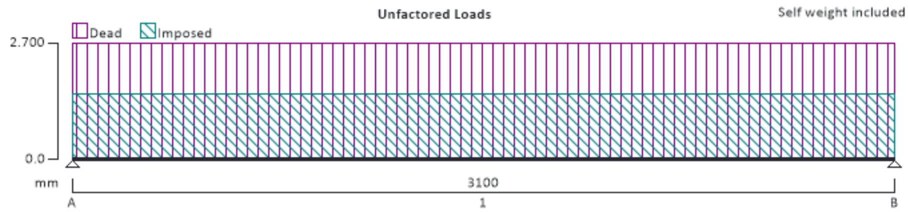
Consider deflection due to imposed loads

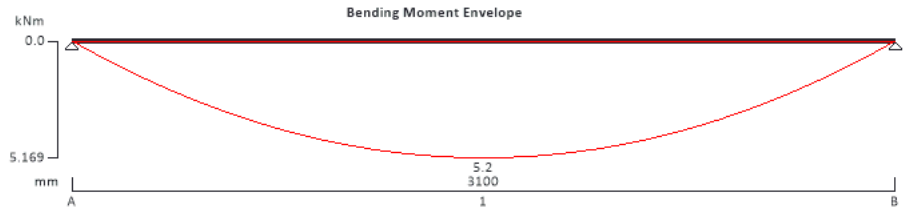
Limiting deflection δ_{lim} = **8.611** mm Maximum deflection δ = **0.704** mm

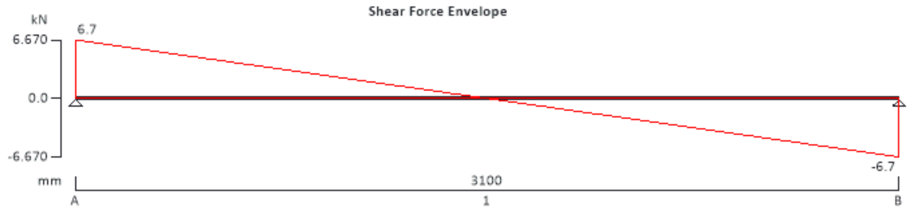
PASS - Maximum deflection does not exceed deflection limit

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.03







Applied loading

Beam loads

Dead self weight of beam × 1
Dead full UDL 2.700 kN/m
Imposed full UDL 1.500 kN/m

Load combinations

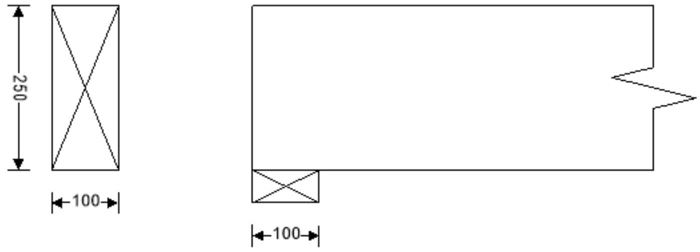
Load combination 1	Support A	Dead × 1.00
		Imposed × 1.00
	Span 1	Dead × 1.00
		Imposed × 1.00
	Support B	Dead × 1.00
		Imposed × 1.00

Analysis results

Design moment	M = **5.169** kNm	Design shear	F = **6.670** kN
Total load on beam	W_{tot} = **13.339** kN		
Reactions at support A	R_{A_max} = **6.670** kN	R_{A_min} = **6.670** kN	
Unfactored dead load reaction at support A	R_{A_Dead} = **4.345** kN		
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **2.325** kN		
Reactions at support B	R_{B_max} = **6.670** kN	R_{B_min} = **6.670** kN	

Unfactored dead load reaction at support B R_{B_Dead} = **4.345** kN

Unfactored imposed load reaction at support B $R_{B\_Imposed}$ = **2.325** kN



Timber section details

Breadth of section b = **100** mm

Depth of section h = **250** mm

Number of sections N = **1**

Breadth of beam b_b = **100** mm

Timber strength class **C24**

Member details

Service class of timber **1**

Load duration **Medium term**

Length of span L_{s1} = **3100** mm

Length of bearing L_b = **100** mm

Lateral support - cl.2.10.8

Permiss.depth-to-breadth ratio **4.00**

Actual depth-to-breadth ratio **2.50**

PASS - Lateral support is adequate

Check bearing stress

Permissible bearing stress σ_{c_adm} = **3.000** N/mm^2

Applied bearing stress σ_{c_a} = **0.667** N/mm^2

PASS - Applied compressive stress is less than permissible compressive stress at bearing

Bending parallel to grain

Permissible bending stress σ_{m_adm} = **9.565** N/mm^2

Applied bending stress σ_{m_a} = **4.962** N/mm^2

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

Permissible shear stress τ_{adm} = **0.888** N/mm^2

Applied shear stress τ_a = **0.400** N/mm^2

PASS - Applied shear stress is less than permissible shear stress

Deflection

Permissible deflection δ_{adm} = **9.300** mm

Total deflection δ_a = **6.071** mm

PASS - Total deflection is less than permissible deflection

Purlin P4

Max Clear Span = 4400mm

Dead Load

Roof DL	=	1.8m x 1.1 kN/m^2	= 2.0 kN/m
		TOTAL DL	**= 2.0 kN/m**

Live Load

Roof LL	=	1.8m x 0.6 kN/m^2	= 1.1 kN/m
		TOTAL LL	**= 1.1 kN/m**

USE
152x152x23 UC
OR
100x300mm Deep Timber – Min Grade C24
(SEE CALC P4/P4T)

Padstones

R = 12 kN

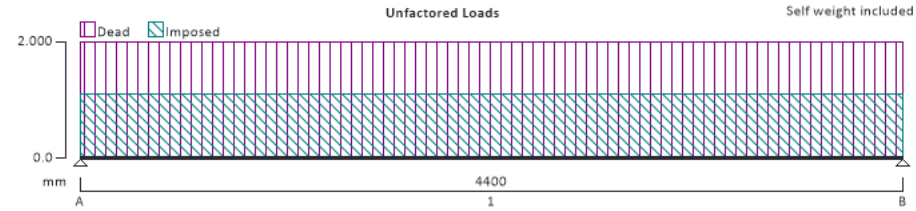
USE 200 x 140 x 215mm DEEP CONCRETE PADSTONES

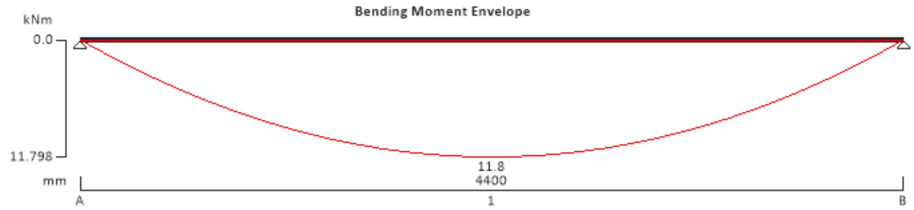
Stress Check = 12 x 1000 / 200 x 140 = 0.6 N/mm^2 < 2.4 N/mm^2
Satisfactory

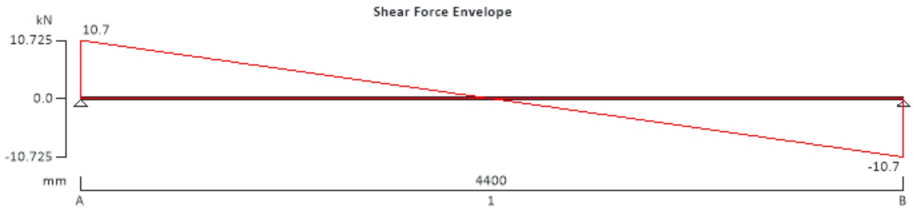
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.07







Support conditions

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

Applied loading

Beam loads	Dead self weight of beam × 1
	Dead full UDL 2 kN/m
	Imposed full UDL 1.1 kN/m

Load combinations

Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60
		Dead × 1.40
		Imposed × 1.60
	Support B	Dead × 1.40
		Imposed × 1.60

Analysis results

Maximum moment	M_{max} = **11.8** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **10.7** kN	V_{min} = **-10.7** kN
Deflection	δ_{max} = **2.1** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **10.7** kN	R_{A_min} = **10.7** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **4.9** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **2.4** kN	
Maximum reaction at support B	R_{B_max} = **10.7** kN	R_{B_min} = **10.7** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **4.9** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **2.4** kN	

Section details

Section type **UC 152x152x23 (British Steel Section Range 2022 (BS4-1))** Steel grade **S275**

Classification of cross sections - Section 3.5

Tensile strain coefficient ε = **1.00** Section classification **Semi-compact**

Shear capacity - Section 4.2.3

Design shear force F_v = **10.7** kN Design shear resistance P_v = **145.8** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment M = **11.8** kNm Moment capacity low shear M_c = **48.5** kNm

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment M_b = **25.2** kNm M_b / m_{LT} = **27.3** kNm

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

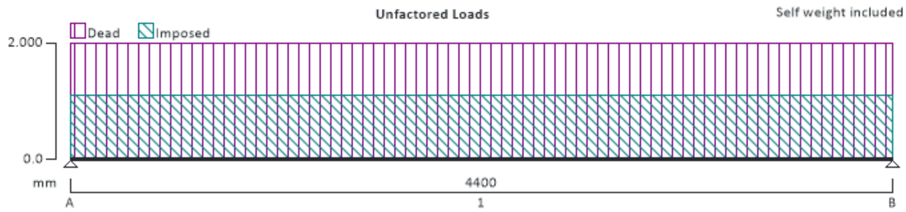
Consider deflection due to imposed loads

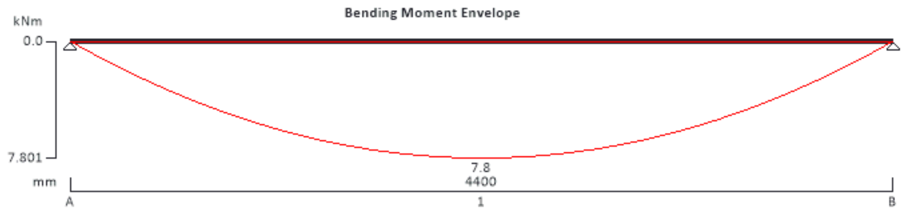
Limiting deflection δ_{lim} = **12.222** mm Maximum deflection δ = **2.095** mm

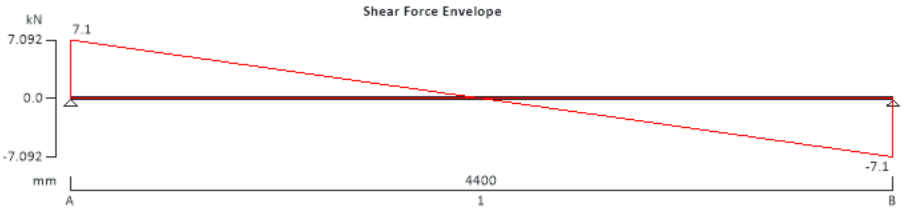
PASS - Maximum deflection does not exceed deflection limit

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.03







Applied loading

Beam loads

Dead self weight of beam × 1

Dead full UDL 2.000 kN/m

Imposed full UDL 1.100 kN/m

Load combinations

Load combination 1	Support A	Dead × 1.00
		Imposed × 1.00
	Span 1	Dead × 1.00
		Imposed × 1.00
	Support B	Dead × 1.00
		Imposed × 1.00

Analysis results

Design moment	M = **7.801** kNm	Design shear	F = **7.092** kN
Total load on beam	W_{tot} = **14.184** kN		
Reactions at support A	R_{A_max} = **7.092** kN	R_{A_min} = **7.092** kN	

Unfactored dead load reaction at support A R_{A_Dead} = **4.672** kN

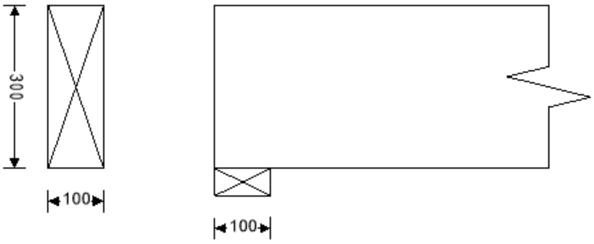
Unfactored imposed load reaction at support A $R_{A\_Imposed}$ = **2.420** kN

Reactions at support B	R_{B_max} = **7.092** kN	R_{B_min} = **7.092** kN

Hudds Design Structural Engineering Consultants Hudds Design 103 - 105 Bradford Rd Huddersfield HD1 6DZ	Project 2 Vernon Close, Edgerton, Huddersfield, HD1 5QE				Job no. HD-S26-0128	
	Calcs for Purlin P4 - Timber				Start page no./Revision P4T- 3	
	Calcs by AM	Calcs date 28/01/2026	Checked by JA	Checked date 28/01/2026	Approved by	Approved date

Unfactored dead load reaction at support B R_{B_Dead} = **4.672** kN

Unfactored imposed load reaction at support B $R_{B\_Imposed}$ = **2.420** kN



Timber section details

Breadth of section b = **100** mm

Depth of section h = **300** mm

Number of sections N = **1**

Breadth of beam b_b = **100** mm

Timber strength class **C24**

Member details

Service class of timber **1**

Load duration **Medium term**

Length of span L_{s1} = **4400** mm

Length of bearing L_b = **100** mm

Lateral support - cl.2.10.8

Permiss.depth-to-breadth ratio **4.00**

Actual depth-to-breadth ratio **3.00**

PASS - Lateral support is adequate

Check bearing stress

Permissible bearing stress σ_{c_adm} = **3.000** N/mm^2

Applied bearing stress σ_{c_a} = **0.709** N/mm^2

PASS - Applied compressive stress is less than permissible compressive stress at bearing

Bending parallel to grain

Permissible bending stress σ_{m_adm} = **9.375** N/mm^2

Applied bending stress σ_{m_a} = **5.201** N/mm^2

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

Permissible shear stress τ_{adm} = **0.888** N/mm^2

Applied shear stress τ_a = **0.355** N/mm^2

PASS - Applied shear stress is less than permissible shear stress

Deflection

Permissible deflection δ_{adm} = **13.200** mm

Total deflection δ_a = **10.405** mm

PASS - Total deflection is less than permissible deflection

Hip H2

Max Clear Span = 3500mm

Dead Load

Roof DL	=	2.0m x 1.1 kN/m^2	= 2.2 kN/m
		TOTAL DL	**= 2.2 kN/m**

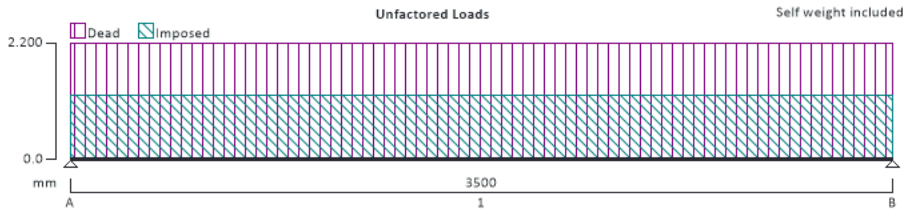
Live Load

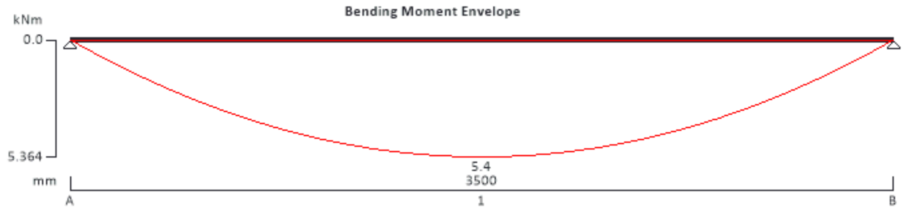
Roof LL	=	2.0m x 0.6 kN/m^2	= 1.2 kN/m
		TOTAL LL	**= 1.2 kN/m**

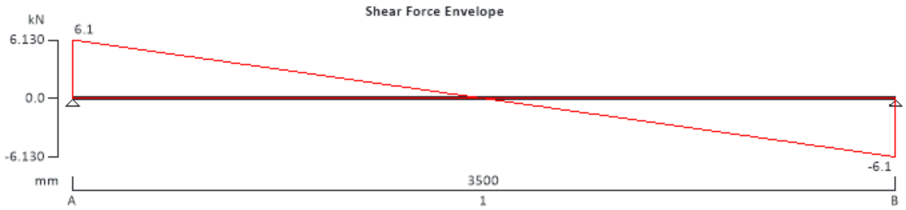
USE
152x152x23 UC
OR
100x250mm Deep Timber – Min Grade C24
(SEE CALC H2T)

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.03







Applied loading

Beam loads

Dead self weight of beam × 1

Dead full UDL 2.200 kN/m

Imposed full UDL 1.200 kN/m

Load combinations

Load combination 1	Support A	Dead × 1.00
		Imposed × 1.00
	Span 1	Dead × 1.00
		Imposed × 1.00
	Support B	Dead × 1.00
		Imposed × 1.00

Analysis results

Design moment M = **5.364** kNm Design shear F = **6.130** kN

Total load on beam W_{tot} = **12.260** kN

Reactions at support A R_{A_max} = **6.130** kN R_{A_min} = **6.130** kN

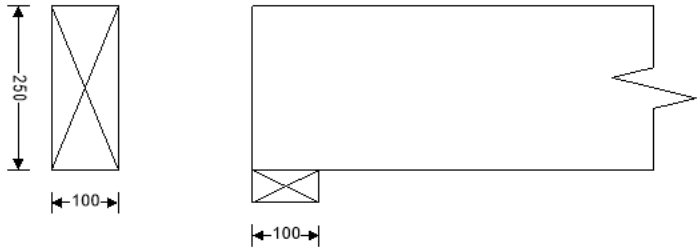
Unfactored dead load reaction at support A R_{A_Dead} = **4.030** kN

Unfactored imposed load reaction at support A $R_{A\_Imposed}$ = **2.100** kN

Reactions at support B R_{B_max} = **6.130** kN R_{B_min} = **6.130** kN

Unfactored dead load reaction at support B R_{B_Dead} = **4.030** kN

Unfactored imposed load reaction at support B $R_{B\_Imposed}$ = **2.100** kN



Timber section details

Breadth of section b = **100** mm Depth of section h = **250** mm

Number of sections N = **1** Breadth of beam b_b = **100** mm

Timber strength class **C24**

Member details

Service class of timber **1** Load duration **Medium term**

Length of span L_{s1} = **3500** mm

Length of bearing L_b = **100** mm

Lateral support - cl.2.10.8

Permiss.depth-to-breadth ratio **4.00** Actual depth-to-breadth ratio **2.50**

PASS - Lateral support is adequate

Check bearing stress

Permissible bearing stress σ_{c_adm} = **3.000** N/mm² Applied bearing stress σ_{c_a} = **0.613** N/mm²

PASS - Applied compressive stress is less than permissible compressive stress at bearing

Bending parallel to grain

Permissible bending stress σ_{m_adm} = **9.565** N/mm² Applied bending stress σ_{m_a} = **5.149** N/mm²

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

Permissible shear stress τ_{adm} = **0.888** N/mm² Applied shear stress τ_a = **0.368** N/mm²

PASS - Applied shear stress is less than permissible shear stress

Deflection

Permissible deflection δ_{adm} = **10.500** mm Total deflection δ_a = **7.873** mm

PASS - Total deflection is less than permissible deflection

Purlin P5

Max Clear Span **= 3100mm**

Dead Load

Roof DL	=	2.0m x 1.1 kN/m^2	= 2.2 kN/m
		TOTAL DL	**= 2.2 kN/m**

Live Load

Roof LL	=	2.0m x 0.6 kN/m^2	= 1.2 kN/m
		TOTAL LL	**= 1.2 kN/m**

DL P4	=	4.9 kN x 1 No.	= 4.9 kN
DL H2	=	4.0 kN x 2 No.	= 8.0 kN
		TOTAL DL	**= 12.9 kN @ Mid Span**

LL P4	=	2.4 kN x 1 No.	= 2.4 kN
LL H2	=	2.1 kN x 2 No.	= 4.2 kN
		TOTAL LL	**= 6.6 kN @ Mid Span**

USE
152x152x23 UC
(SEE CALC P5)

Padstones

R = 23 kN

USE 200 x 140 x 215mm DEEP CONCRETE PADSTONES

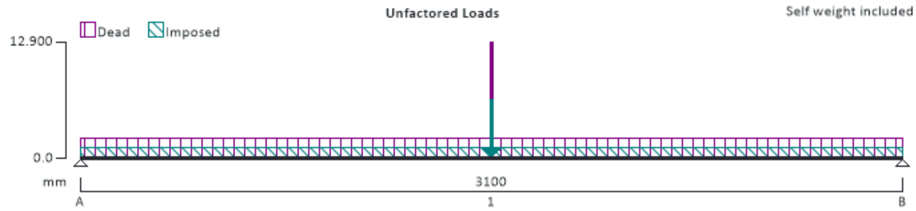
Stress Check = 23 x 1000 / 200 x 140 = 0.8 N/mm^2 < 2.4 N/mm^2
Satisfactory

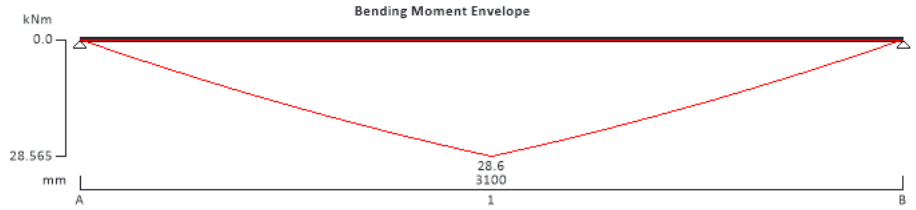
eng@huddsdesign.com
www.huddsdesign.com

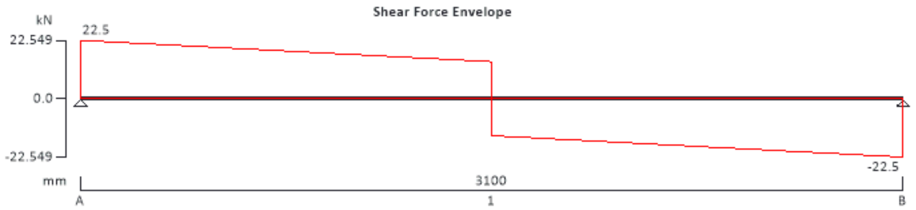
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.07







Support conditions

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

Applied loading

Beam loads	Dead self weight of beam × 1
	Dead full UDL 2.2 kN/m
	Imposed full UDL 1.2 kN/m
	Dead point load 12.9 kN at 1550 mm
	Imposed point load 6.6 kN at 1550 mm

Load combinations

Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60
		Dead × 1.40
		Imposed × 1.60
	Support B	Dead × 1.40
		Imposed × 1.60

Analysis results

Maximum moment	M_{max} = **28.6** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **22.5** kN	V_{min} = **-22.5** kN
Deflection	δ_{max} = **2.2** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **22.5** kN	R_{A_min} = **22.5** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **10.2** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **5.2** kN	
Maximum reaction at support B	R_{B_max} = **22.5** kN	R_{B_min} = **22.5** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **10.2** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **5.2** kN	

Section details

Section type **UC 152x152x23 (British Steel Section Range 2022 (BS4-1))** Steel grade **S275**

Classification of cross sections - Section 3.5

Tensile strain coefficient ε = **1.00** Section classification **Semi-compact**

Shear capacity - Section 4.2.3

Design shear force F_v = **22.5** kN Design shear resistance P_v = **145.8** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment M = **28.6** kNm Moment capacity low shear M_c = **48.5** kNm

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment M_b = **32.2** kNm M_b / m_{LT} = **37.1** kNm

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection δ_{lim} = **8.611** mm Maximum deflection δ = **2.162** mm

PASS - Maximum deflection does not exceed deflection limit

Trimmer T1

Max Clear Span **= 3750mm**

Dead Load

Floor DL	=	0.7m x 0.6 kN/m^2	= 0.42 kN/m
		TOTAL DL	**= 0.42 kN/m**

Live Load

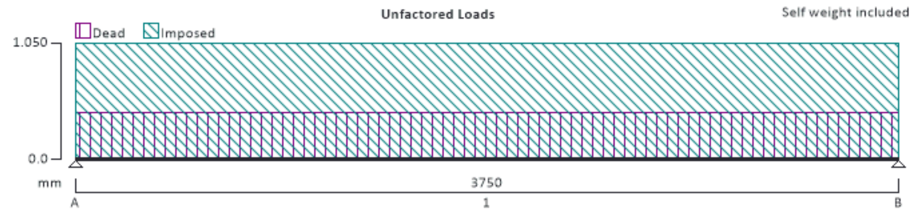
Floor LL	=	0.7m x 1.5 kN/m^2	= 1.05 kN/m
		TOTAL LL	**= 1.05 kN/m**

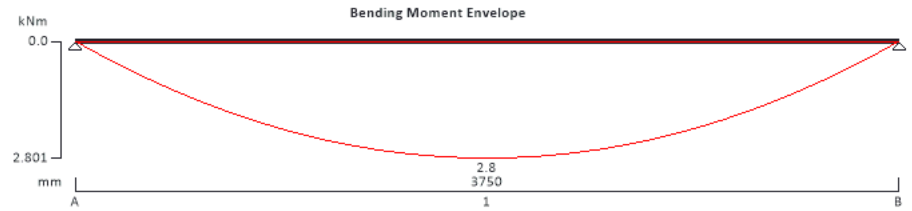
USE
2 No. 75x200mm Deep Timbers – Min Grade C24
(SEE CALC T1)

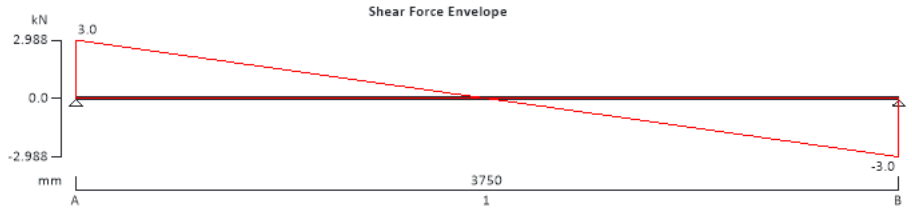
Hudds Design Structural Engineering Consultants Hudds Design 103 - 105 Bradford Rd Huddersfield HD1 6DZ	Project 2 Vernon Close, Edgerton, Huddersfield, HD1 5QE				Job no. HD-S26-0128	
	Calcs for Trimmer T1				Start page no./Revision T1- 2	
	Calcs by AM	Calcs date 28/01/2026	Checked by JA	Checked date 28/01/2026	Approved by	Approved date

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.03







Applied loading

Beam loads

Dead self weight of beam × 1

Dead full UDL 0.420 kN/m

Imposed full UDL 1.050 kN/m

Load combinations

Load combination 1	Support A	Dead × 1.00
		Imposed × 1.00
	Span 1	Dead × 1.00
		Imposed × 1.00
	Support B	Dead × 1.00
		Imposed × 1.00

Analysis results

Design moment M = **2.801** kNm   Design shear F = **2.988** kN

Total load on beam W_{tot} = **5.976** kN

Reactions at support A R_{A_max} = **2.988** kN R_{A_min} = **2.988** kN

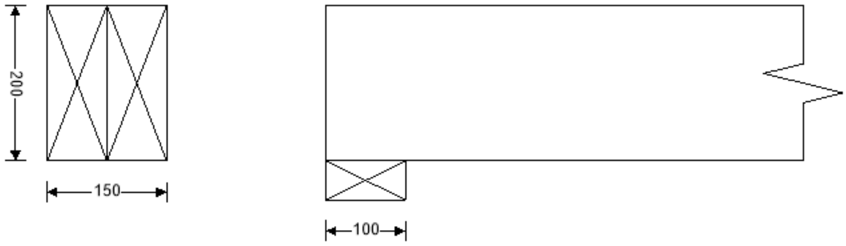
Unfactored dead load reaction at support A R_{A_Dead} = **1.019** kN

Unfactored imposed load reaction at support A $R_{A\_Imposed}$ = **1.969** kN

Reactions at support B R_{B_max} = **2.988** kN R_{B_min} = **2.988** kN

Unfactored dead load reaction at support B R_{B_Dead} = **1.019** kN

Unfactored imposed load reaction at support B $R_{B\_Imposed}$ = **1.969** kN



Timber section details

Breadth of section b = **75** mm Depth of section h = **200** mm

Number of sections N = **2** Breadth of beam b_b = **150** mm

Timber strength class **C24**

Member details

Service class of timber **1** Load duration **Long term**

Length of span L_{s1} = **3750** mm

Length of bearing L_b = **100** mm

Lateral support - cl.2.10.8

Permiss.depth-to-breadth ratio **4.00** Actual depth-to-breadth ratio **1.33**

PASS - Lateral support is adequate

Check bearing stress

Permissible bearing stress σ_{c_adm} = **2.400** N/mm² Applied bearing stress σ_{c_a} = **0.199** N/mm²

PASS - Applied compressive stress is less than permissible compressive stress at bearing

Bending parallel to grain

Permissible bending stress σ_{m_adm} = **7.842** N/mm² Applied bending stress σ_{m_a} = **2.801** N/mm²

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

Permissible shear stress τ_{adm} = **0.710** N/mm² Applied shear stress τ_a = **0.149** N/mm²

PASS - Applied shear stress is less than permissible shear stress

Deflection

Permissible deflection δ_{adm} = **11.250** mm Total deflection δ_a = **5.948** mm

PASS - Total deflection is less than permissible deflection

Trimmer T2

Max Clear Span = 3100mm

Dead Load

Floor DL	=	0.4m x 0.6 kN/m^2	= 0.24 kN/m
		TOTAL DL	**= 0.24 kN/m**

Live Load

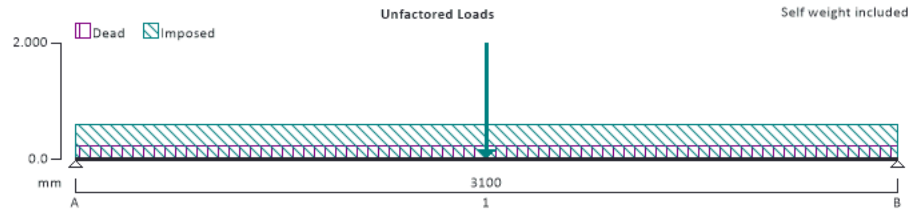
Floor LL	=	0.4m x 1.5 kN/m^2	= 0.6 kN/m
		TOTAL LL	**= 0.6 kN/m**

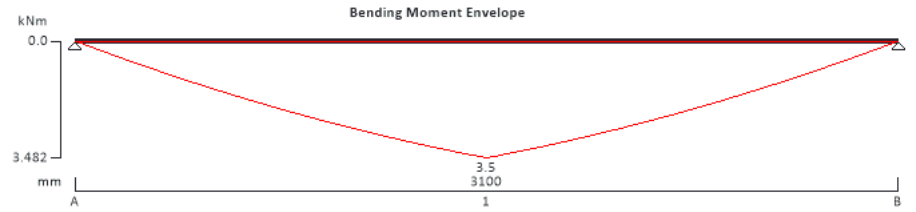
DL T1	=	1.0 kN @ mid Span
LL T1	=	2.0 kN @ mid Span

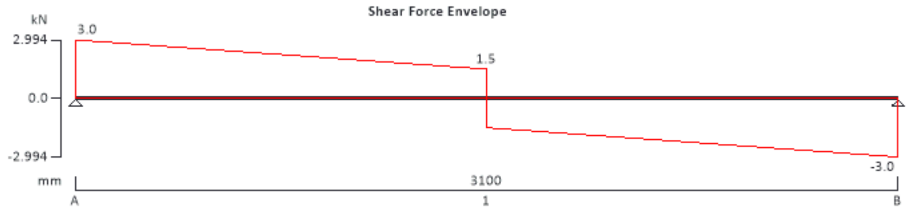
USE
2 No. 75x200mm Deep Timbers – Min Grade C24
(SEE CALC T2)

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.03







Applied loading

Beam loads

Dead self weight of beam × 1
Dead full UDL 0.240 kN/m
Imposed full UDL 0.600 kN/m
Dead point load 1.000 kN at 1550 mm
Imposed point load 2.000 kN at 1550 mm

Load combinations

Load combination 1	Support A	Dead × 1.00
		Imposed × 1.00
	Span 1	Dead × 1.00
		Imposed × 1.00
	Support B	Dead × 1.00
		Imposed × 1.00

Analysis results

Design moment	M = **3.483** kNm	Design shear	F = **2.994** kN
Total load on beam	W_{tot} = **5.987** kN		
Reactions at support A	R_{A_max} = **2.994** kN	R_{A_min} = **2.994** kN	

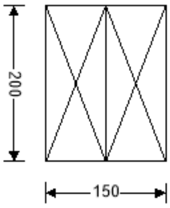
Unfactored dead load reaction at support A R_{A_Dead} = **1.064** kN

Unfactored imposed load reaction at support A $R_{A\_Imposed}$ = **1.930** kN

Reactions at support B R_{B_max} = **2.994** kN R_{B_min} = **2.994** kN

Unfactored dead load reaction at support B R_{B_Dead} = **1.064** kN

Unfactored imposed load reaction at support B $R_{B\_Imposed}$ = **1.930** kN



Timber section details

Breadth of section b = **75** mm Depth of section h = **200** mm

Number of sections N = **2** Breadth of beam b_b = **150** mm

Timber strength class **C24**

Member details

Service class of timber **1** Load duration **Long term**

Length of span L_{s1} = **3100** mm

Length of bearing L_b = **100** mm

Lateral support - cl.2.10.8

Permiss.depth-to-breadth ratio **4.00** Actual depth-to-breadth ratio **1.33**

PASS - Lateral support is adequate

Check bearing stress

Permissible bearing stress σ_{c_adm} = **2.400** N/mm² Applied bearing stress σ_{c_a} = **0.200** N/mm²

PASS - Applied compressive stress is less than permissible compressive stress at bearing

Bending parallel to grain

Permissible bending stress σ_{m_adm} = **7.842** N/mm² Applied bending stress σ_{m_a} = **3.483** N/mm²

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

Permissible shear stress τ_{adm} = **0.710** N/mm² Applied shear stress τ_a = **0.150** N/mm²

PASS - Applied shear stress is less than permissible shear stress

Deflection

Permissible deflection δ_{adm} = **9.300** mm Total deflection δ_a = **4.505** mm

PASS - Total deflection is less than permissible deflection

Beam B1 - 2 No. Beams

Max Clear Span = 3300mm

Internal Beam

Dead Load

Wall DL Gable	=	2.0m x 2.0 kN/m^2	= 4.0 kN/m (Trap Load)
Wall DL	=	1.2m x 2.0 kN/m^2	= 2.4 kN/m
Floor DL	=	0.4m x 1.0 kN/m^2	= 0.4 kN/m
Roof DL	=	3.1m x 1.3 kN/m^2	= 4.1 kN/m
		TOTAL DL	**= 6.9 kN/m**

Live Load

Floor LL	=	0.4m x 1.5 kN/m^2	= 0.6 kN/m
Roof LL	=	3.1m x 0.6 kN/m^2	= 4.1 kN/m
		TOTAL LL	**= 4.7 kN/m**

External Beam

Dead Load

Wall DL Gable	=	2.0m x 2.4 kN/m^2	= 4.8 kN/m (Trap Load)
Wall DL	=	1.2m x 2.4 kN/m^2	= 2.9 kN/m
Roof DL	=	0.5m x 1.3 kN/m^2	= 0.7 kN/m
		TOTAL DL	**= 3.6 kN/m**

Live Load

Roof LL	=	0.5m x 0.6 kN/m^2	= 0.3 kN/m
		TOTAL LL	**= 0.3 kN/m**

USE 2 No. 152x152x23 UC's
(SEE CALC B1)
Beams to be bolted to L1
Use same for Beam B2 as single beam

Padstones

Reaction Maximum = 49 kN

USE 400 x 100 x 215mm DEEP CONCRETE PADSTONES

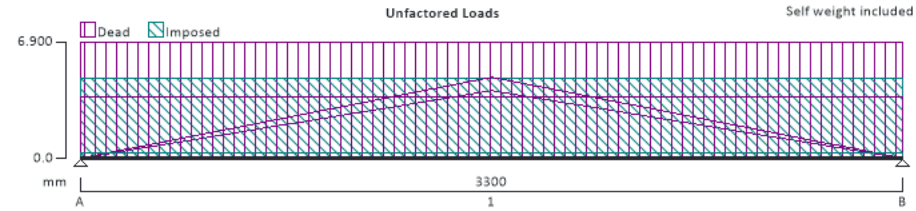
Stress Check = 49 x 1000 / 400 x 100 = 1.3 N/mm^2 < 2.4 N/mm^2
Satisfactory

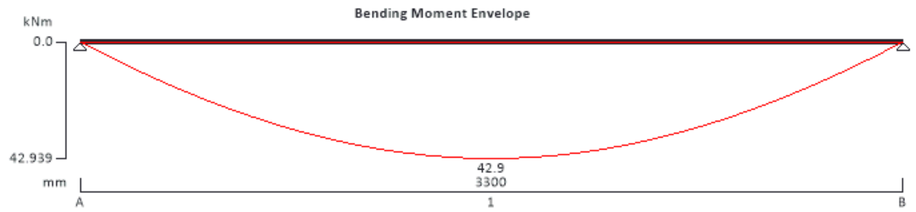
eng@huddsdesign.com
www.huddsdesign.com

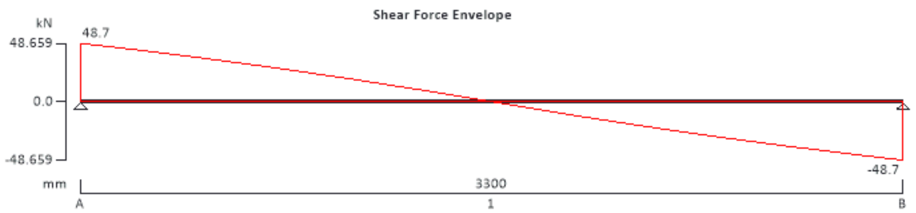
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.07







Support conditions

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

Applied loading

Beam loads	Dead self weight of beam × 1
	Dead trapezoidal load 4 kN/m from 1650 mm to 1650 mm
	Dead full UDL 6.9 kN/m
	Imposed full UDL 4.7 kN/m
	Dead trapezoidal load 4.8 kN/m from 1650 mm to 1650 mm
	Dead full UDL 3.6 kN/m
	Imposed full UDL 0.3 kN/m

Load combinations

Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60
		Dead × 1.40
		Imposed × 1.60

Support B — Dead × 1.40
Imposed × 1.60

Analysis results

Maximum moment	M_{max} = **42.9** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **48.7** kN	V_{min} = **-48.7** kN
Deflection	δ_{max} = **1.5** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **48.7** kN	R_{A_min} = **48.7** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **25.3** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **8.3** kN	
Maximum reaction at support B	R_{B_max} = **48.7** kN	R_{B_min} = **48.7** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **25.3** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **8.3** kN	

Section details

Section type **2 x UC 152x152x23 (British Steel Section Range 2022 (BS4-1))** Steel grade **S275**

Classification of cross sections - Section 3.5

Tensile strain coefficient ε = **1.00** Section classification **Semi-compact**

Shear capacity - Section 4.2.3

Design shear force F_v = **48.7** kN Design shear resistance P_v = **291.7** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment M = **42.9** kNm Moment capacity low shear M_c = **96.9** kNm

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment M_b = **61.9** kNm M_b / m_{LT} = **67.2** kNm

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection δ_{lim} = **9.167** mm Maximum deflection δ = **1.507** mm

PASS - Maximum deflection does not exceed deflection limit

Beam B3 - 2 No. Beams

Max Clear Span = 4550mm

Internal Beam

Dead Load

Wall DL	=	4.0m x 2.0 kN/m^2	= 8.0 kN/m
Floor DL	=	4.65m x 1.0 kN/m^2	= 4.7 kN/m
Roof DL	=	2.0m x 1.3 kN/m^2	= 2.6 kN/m
		TOTAL DL	**= 15.3 kN/m**

Live Load

Floor LL	=	4.65m x 1.5 kN/m^2	= 7.0 kN/m
Roof LL	=	2.0m x 0.6 kN/m^2	= 1.2 kN/m
		TOTAL LL	**= 8.2 kN/m**

External Beam

Dead Load

Wall DL	=	4.0m x 2.4 kN/m^2	= 9.6 kN/m
Roof DL	=	0.8m x 1.3 kN/m^2	= 1.0 kN/m
		TOTAL DL	**= 10.6 kN/m**

Live Load

Roof LL	=	0.8m x 0.6 kN/m^2	= 0.5 kN/m
		TOTAL LL	**= 0.5 kN/m**

USE 2 No. 254x146x43 UB's – Grade S355
Bolted together at 400mm ctrs with M20 bolts with tubular spacers
(SEE CALC B3)

Padstones

Reaction Maximum = 117 kN

USE 500 x 100 x 215mm DEEP CONCRETE LINTEL PADSTONES

Stress Check = 117 x 1000 / 500 x 100 = 2.35 N/mm^2 < 2.4 N/mm^2

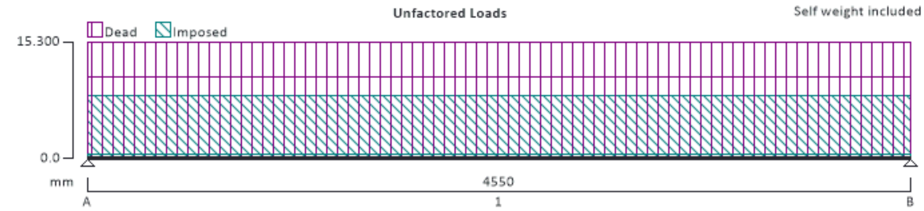
Satisfactory

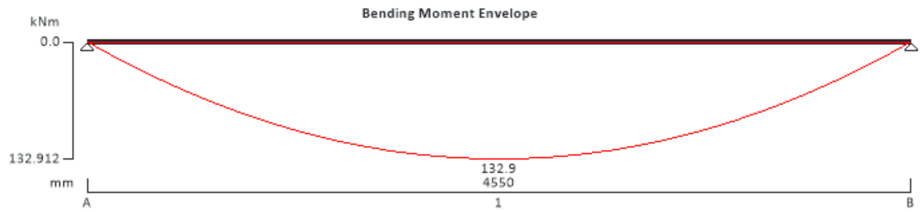
Hudds Design Structural Engineering Consultants Hudds Design 103 - 105 Bradford Rd Huddersfield HD1 6DZ	Project 2 Vernon Close, Edgerton, Huddersfield, HD1 5QE				Job no. HD-S26-0104	
	Calcs for Beam B3				Start page no./Revision B3 - 2	
	Calcs by AM	Calcs date 28/01/2026	Checked by JA	Checked date 28/01/2026	Approved by	Approved date

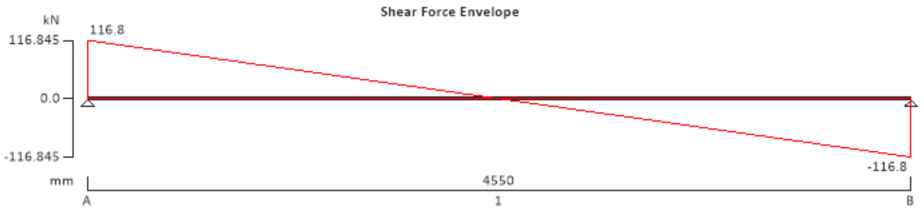
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.07







Support conditions

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

Applied loading

Beam loads	Dead self weight of beam × 1
	Dead full UDL 15.3 kN/m
	Imposed full UDL 8.2 kN/m
	Dead full UDL 10.6 kN/m
	Imposed full UDL 0.5 kN/m

Load combinations

Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60
		Dead × 1.40
		Imposed × 1.60
	Support B	Dead × 1.40
		Imposed × 1.60

Analysis results

Maximum moment M_{max} = **132.9** kNm M_{min} = **0** kNm

Maximum shear V_{max} = **116.8** kN V_{min} = **-116.8** kN

Deflection δ_{max} = **1.8** mm δ_{min} = **0** mm

Maximum reaction at support A R_{A_max} = **116.8** kN R_{A_min} = **116.8** kN

Unfactored dead load reaction at support A R_{A_Dead} = **60.8** kN

Unfactored imposed load reaction at support A $R_{A\_Imposed}$ = **19.8** kN

Maximum reaction at support B R_{B_max} = **116.8** kN R_{B_min} = **116.8** kN

Unfactored dead load reaction at support B R_{B_Dead} = **60.8** kN

Unfactored imposed load reaction at support B $R_{B\_Imposed}$ = **19.8** kN

Section details

Section type **2 x UB 254x146x43 (British Steel Section Range 2022 (BS4-1))** Steel grade **S355**

Classification of cross sections - Section 3.5

Tensile strain coefficient ε = **0.88** Section classification **Plastic**

Shear capacity - Section 4.2.3

Design shear force F_v = **116.8** kN Design shear resistance P_v = **796.2** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment M = **132.9** kNm Moment capacity low shear M_c = **402.1** kNm

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment M_b = **144.6** kNm M_b / m_{LT} = **156.3** kNm

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection δ_{lim} = **12.639** mm Maximum deflection δ = **1.81** mm

PASS - Maximum deflection does not exceed deflection limit

Beam B4

Max Clear Span = 1200mm

Internal Beam
Dead Load

Wall DL	=	4.0m x 2.4 kN/m^2	= 9.6 kN/m
Floor DL	=	1.4m x 1.0 kN/m^2	= 1.4 kN/m
Roof DL	=	1.0m x 1.3 kN/m^2	= 1.3 kN/m
		TOTAL DL	**= 12.3 kN/m**

Live Load

Floor LL	=	1.4m x 1.5 kN/m^2	= 2.1 kN/m
Roof LL	=	1.0m x 0.6 kN/m^2	= 0.6 kN/m
		TOTAL LL	**= 2.7 kN/m**

USE 152x152x23 UC
(SEE CALC B4)
Bolted to B5/B7/B8

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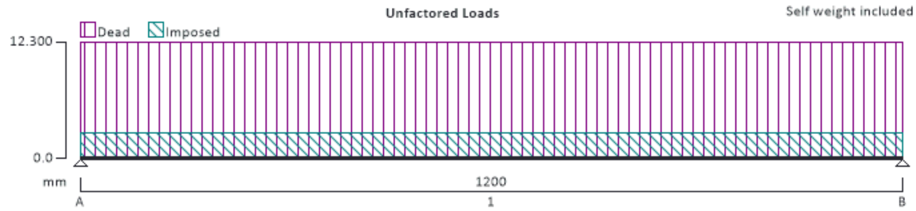
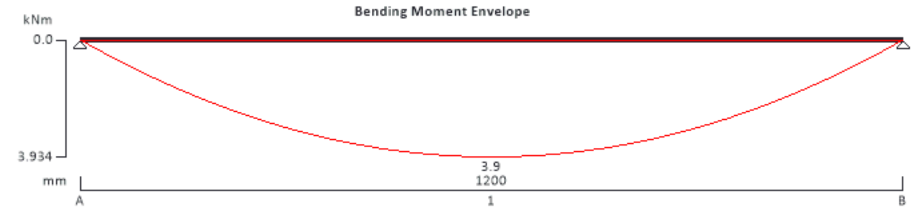
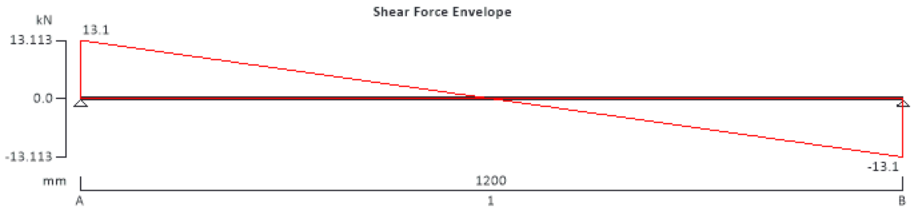
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.07







Support conditions

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

Applied loading

Beam loads	Dead self weight of beam × 1
	Dead full UDL 12.3 kN/m
	Imposed full UDL 2.7 kN/m

Load combinations

Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60
		Dead × 1.40
		Imposed × 1.60
	Support B	Dead × 1.40
		Imposed × 1.60

Analysis results

Maximum moment	M_{max} = **3.9** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **13.1** kN	V_{min} = **-13.1** kN
Deflection	δ_{max} = **0** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **13.1** kN	R_{A_min} = **13.1** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **7.5** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **1.6** kN	
Maximum reaction at support B	R_{B_max} = **13.1** kN	R_{B_min} = **13.1** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **7.5** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **1.6** kN	

Section details

Section type **UC 152x152x23 (British Steel Section Range 2022 (BS4-1))** Steel grade **S275**

Classification of cross sections - Section 3.5

Tensile strain coefficient ε = **1.00** Section classification **Semi-compact**

Shear capacity - Section 4.2.3

Design shear force F_v = **13.1** kN Design shear resistance P_v = **145.8** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment M = **3.9** kNm Moment capacity low shear M_c = **48.5** kNm

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment M_b = **47.5** kNm M_b / m_{LT} = **51.3** kNm

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection δ_{lim} = **3.333** mm Maximum deflection δ = **0.028** mm

PASS - Maximum deflection does not exceed deflection limit

Beam B5 - 2 No. Beams

Max Clear Span = 1400mm

Internal Beam

Dead Load

Wall DL Gable	=	1.2m x 2.0 kN/m^2	= 2.4 kN/m
Wall DL	=	4.7m x 2.0 kN/m^2	= 9.4 kN/m
Floor DL	=	0.4m x 1.0 kN/m^2	= 0.4 kN/m
Roof DL	=	2.2m x 1.3 kN/m^2	= 2.9 kN/m
		TOTAL DL	**= 12.7 kN/m**

Live Load

Floor LL	=	0.4m x 1.5 kN/m^2	= 0.6 kN/m
Roof LL	=	2.2m x 0.6 kN/m^2	= 1.3 kN/m
		TOTAL LL	**= 2.0 kN/m**

External Beam

Dead Load

Wall DL Gable	=	1.2m x 2.4 kN/m^2	= 2.9 kN/m
Wall DL	=	4.7m x 2.4 kN/m^2	= 11.3 kN/m
Floor DL	=	0.5m x 1.5 kN/m^2	= 0.7 kN/m
		TOTAL DL	**= 12.0 kN/m**

Live Load

Floor LL	=	0.5m x 1.5 kN/m^2	= 0.8 kN/m
		TOTAL LL	**= 0.8 kN/m**

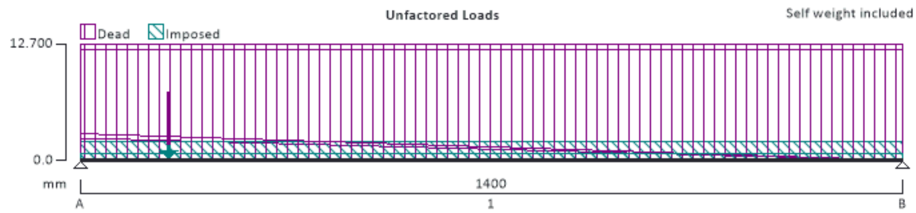
DL B4	=	7.5 kN @ 150mm
LL B4	=	1.6 kN @ 150mm

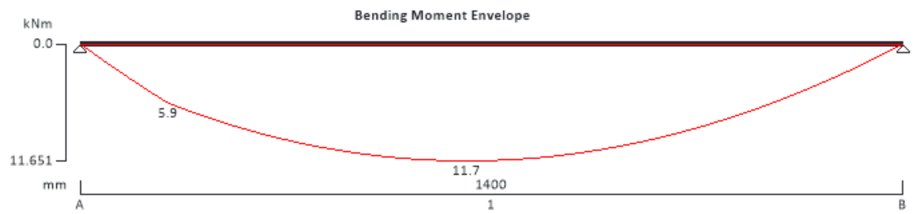
USE 2 No. 152x152x23 UC's
(SEE CALC B5)
Bolted to B6/B9

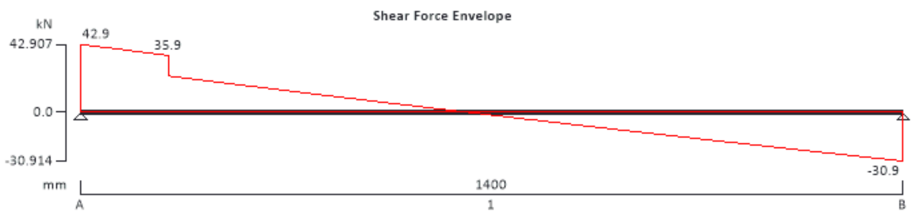
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.07







Support conditions

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

Applied loading

Beam loads	Dead self weight of beam × 1
	Dead trapezoidal load 2.4 kN/m from 0 mm to 0 mm
	Dead full UDL 12.7 kN/m
	Imposed full UDL 2 kN/m
	Dead trapezoidal load 2.9 kN/m from 0 mm to 0 mm
	Dead full UDL 12 kN/m
	Imposed full UDL 0.8 kN/m
	Dead point load 7.5 kN at 150 mm
	Imposed point load 1.6 kN at 150 mm

Load combinations

Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60

		Dead × 1.40
		Imposed × 1.60
	Support B	Dead × 1.40
		Imposed × 1.60

Analysis results

Maximum moment	M_{max} = **11.7** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **42.9** kN	V_{min} = **-30.9** kN
Deflection	δ_{max} = **0** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **42.9** kN	R_{A_min} = **42.9** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **26.8** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **3.4** kN	
Maximum reaction at support B	R_{B_max} = **30.9** kN	R_{B_min} = **30.9** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **19.6** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **2.1** kN	

Section details

Section type **2 x UC 152x152x23 (British Steel Section Range 2022 (BS4-1))** Steel grade **S275**

Classification of cross sections - Section 3.5

Tensile strain coefficient ε = **1.00** Section classification **Semi-compact**

Shear capacity - Section 4.2.3

Design shear force F_v = **42.9** kN Design shear resistance P_v = **291.7** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment M = **11.7** kNm Moment capacity low shear M_c = **96.9** kNm

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment M_b = **91.3** kNm M_b / m_{LT} = **98.3** kNm

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection δ_{lim} = **3.889** mm Maximum deflection δ = **0.033** mm

PASS - Maximum deflection does not exceed deflection limit

Beam B6

Max Clear Span = 4550mm

Dead Load

Wall DL	=	4.0m x 3.0 kN/m^2	= 12.0 kN/m
Floor DL	=	6.2m x 1.0 kN/m^2	= 6.2 kN/m
Roof DL	=	2.5m x 1.3 kN/m^2	= 3.3 kN/m
		TOTAL DL	**= 21.5 kN/m**

Live Load

Floor LL	=	6.2m x 1.5 kN/m^2	= 9.3 kN/m
Roof LL	=	2.5m x 0.6 kN/m^2	= 1.5 kN/m
		TOTAL LL	**= 10.8 kN/m**

DL B5 = 26.8 kN @ 3200mm
LL B5 = 3.4 kN @ 3200mm

USE 203x203x86 UC
(SEE CALC B6)
Bolted to B8

Padstones

Reaction Maximum = 123.2 kN

USE 550 x 100 x 215mm DEEP CONCRETE LINTEL PADSTONES

Stress Check = 123 x 1000 / 550 x 100 = 2.25 N/mm^2 < 2.4 N/mm^2
Satisfactory

Hudds Design Structural Engineering Consultants Hudds Design 103 - 105 Bradford Rd Huddersfield HD1 6DZ	Project 2 Vernon Close, Edgerton, Huddersfield, HD1 5QE				Job no. HD-S26-0104	
	Calcs for Beam B6				Start page no./Revision B6 - 2	
	Calcs by AM	Calcs date 28/01/2026	Checked by JA	Checked date 28/01/2026	Approved by	Approved date

STEEL BEAM ANALYSIS & DESIGN (BS5950)

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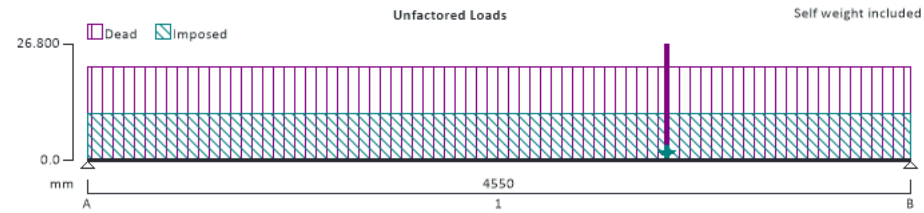
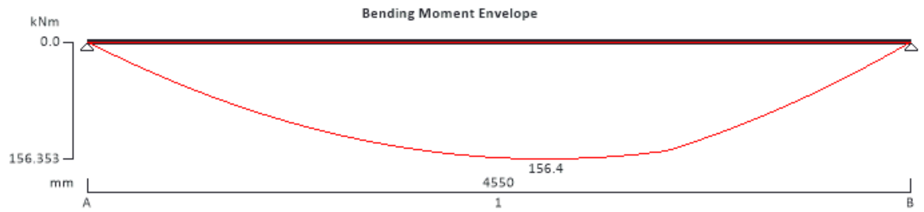
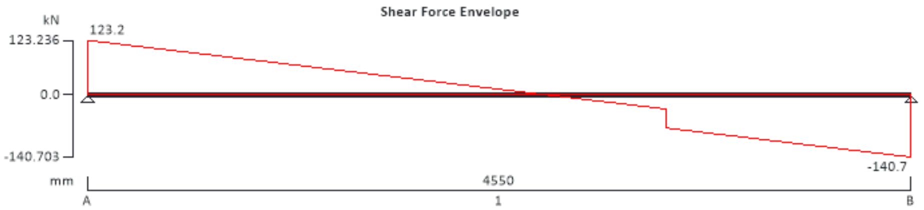
TEDDS calculation version 3.0.07







Support conditions

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

Applied loading

Beam loads	Dead self weight of beam × 1 Dead full UDL 21.5 kN/m Imposed full UDL 10.8 kN/m Dead point load 26.8 kN at 3200 mm Imposed point load 3.4 kN at 3200 mm

Load combinations

Load combination 1	Support A	Dead × 1.40 Imposed × 1.60
		Dead × 1.40 Imposed × 1.60
	Support B	Dead × 1.40 Imposed × 1.60

Analysis results

Maximum moment	M_{max} = **156.4** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **123.2** kN	V_{min} = **-140.7** kN
Deflection	δ_{max} = **3.4** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **123.2** kN	R_{A_min} = **123.2** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **58.8** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **25.6** kN	
Maximum reaction at support B	R_{B_max} = **140.7** kN	R_{B_min} = **140.7** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **69.7** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **27** kN	

Section details

Section type **UC 203x203x86 (British Steel Section Range 2022 (BS4-1))** Steel grade **S275**

Classification of cross sections - Section 3.5

Tensile strain coefficient ε = **1.02** Section classification **Plastic**

Shear capacity - Section 4.2.3

Design shear force F_v = **140.7** kN Design shear resistance P_v = **448.7** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment M = **156.4** kNm Moment capacity low shear M_c = **260** kNm

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment M_b = **208.3** kNm M_b / m_{LT} = **225.8** kNm

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection δ_{lim} = **12.639** mm Maximum deflection δ = **3.369** mm

PASS - Maximum deflection does not exceed deflection limit

Beam B7

Max Clear Span = 8200mm

Internal Beam

Dead Load

Wall DL Gable	=	1.0m x 2.0 kN/m^2	= 2.0 kN/m
Wall DL	=	4.0m x 2.0 kN/m^2	= 8.0 kN/m
Floor DL	=	0.4m x 1.0 kN/m^2	= 0.4 kN/m
Roof DL	=	2.0m x 1.3 kN/m^2	= 2.6 kN/m
		TOTAL DL	**= 13.0 kN/m (Partial 1300-6500)**

Live Load

Floor LL	=	0.4m x 1.5 kN/m^2	= 0.6 kN/m
Roof LL	=	2.0m x 0.6 kN/m^2	= 1.2 kN/m
		TOTAL LL	**= 1.8 kN/m (Partial 1300-6500)**

DL B3	=	60.8 kN @ 6500mm
LL B3	=	19.8 kN @ 6500mm
DL B6/B4	=	77.2 kN @ 1300mm
LL B6/B4	=	28.6 kN @ 1300mm

External Beam

Dead Load

Wall DL Gable	=	1.0m x 2.4 kN/m^2	= 2.4 kN/m
Wall DL	=	4.0m x 2.4 kN/m^2	= 9.6 kN/m
Roof DL	=	1.8m x 1.0 kN/m^2	= 1.8 kN/m
		TOTAL DL	**= 13.8 kN/m (Partial 1300-6500)**

Live Load

Roof LL	=	1.8m x 0.6 kN/m^2	= 1.1 kN/m
		TOTAL LL	**= 1.1 kN/m (Partial 1300-6500)**

USE 356x368x202 UC

OR

305x305x240 UC with 10mm top plate to suit wall width

(SEE CALC B7)

Bolted to B9

STEEL BEAM ANALYSIS & DESIGN (BS5950)

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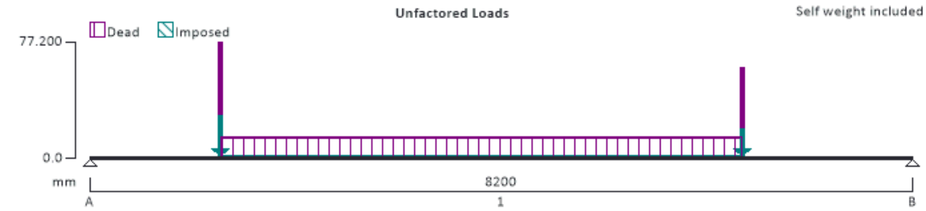
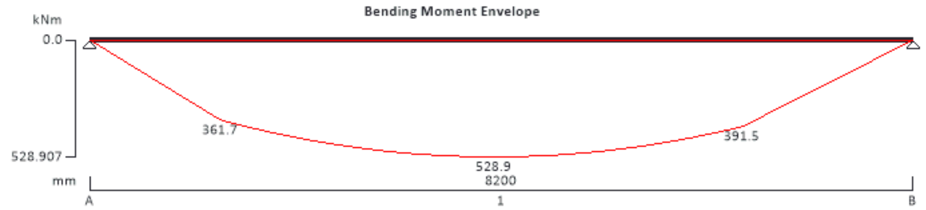
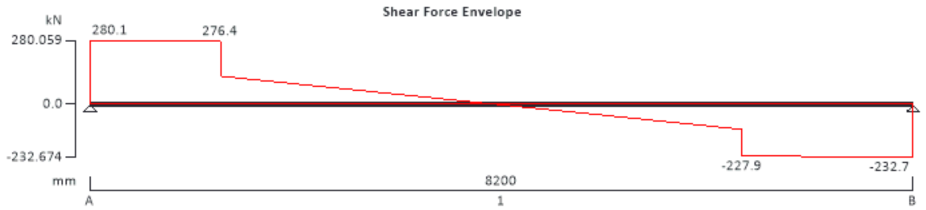
TEDDS calculation version 3.0.07







Support conditions

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

Applied loading

Beam loads	Dead self weight of beam × 1
	Dead partial UDL 13 kN/m from 1300 mm to 6500 mm
	Imposed partial UDL 1.8 kN/m from 1300 mm to 6500 mm
	Dead point load 77.2 kN at 1300 mm
	Imposed point load 28.6 kN at 1300 mm
	Dead point load 60.8 kN at 6500 mm
	Imposed point load 19.8 kN at 6500 mm
	Dead partial UDL 13.8 kN/m from 1300 mm to 6500 mm
	Imposed partial UDL 1.1 kN/m from 1300 mm to 6500 mm

Load combinations

Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60

		Dead × 1.40
		Imposed × 1.60
	Support B	Dead × 1.40
		Imposed × 1.60

Analysis results

Maximum moment	M_{max} = **528.9** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **280.1** kN	V_{min} = **-232.7** kN
Deflection	δ_{max} = **3.1** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **280.1** kN	R_{A_min} = **280.1** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **158.8** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **36.1** kN	
Maximum reaction at support B	R_{B_max} = **232.7** kN	R_{B_min} = **232.7** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **134.9** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **27.4** kN	

Section details

Section type **UC 356x368x202 (British Steel Section Range 2022 (BS4-1))** Steel grade **S275**

Classification of cross sections - Section 3.5

Tensile strain coefficient ε = **1.02** Section classification **Plastic**

Shear capacity - Section 4.2.3

Design shear force F_v = **280.1** kN Design shear resistance P_v = **982.8** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment M = **528.9** kNm Moment capacity low shear M_c = **1058.4** kNm

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment M_b = **789.5** kNm M_b / m_{LT} = **834.4** kNm

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection δ_{lim} = **22.778** mm Maximum deflection δ = **3.134** mm

PASS - Maximum deflection does not exceed deflection limit

Beam B8

Max Clear Span = 6200mm

Internal Beam
Dead Load

Wall DL Gable	=	1.0m x 2.0 kN/m^2	= 2.0 kN/m
Wall DL	=	4.0m x 2.0 kN/m^2	= 8.0 kN/m
Floor DL	=	0.4m x 1.0 kN/m^2	= 0.4 kN/m
Roof DL	=	2.0m x 1.3 kN/m^2	= 2.6 kN/m
		TOTAL DL	**= 13.0 kN/m (Partial 0-5000)**

Live Load

Floor LL	=	0.4m x 1.5 kN/m^2	= 0.6 kN/m
Roof LL	=	2.0m x 0.6 kN/m^2	= 1.2 kN/m
		TOTAL LL	**= 1.8 kN/m (Partial 0-5000)**

DL B6/B4	=	77.2 kN @ 4650mm
LL B6/B4	=	28.6 kN @ 4650mm

External Beam
Dead Load

Wall DL Gable	=	1.0m x 2.4 kN/m^2	= 2.4 kN/m
Wall DL	=	4.0m x 2.4 kN/m^2	= 9.6 kN/m
Roof DL	=	1.0m x 1.3 kN/m^2	= 1.3 kN/m
		TOTAL DL	**= 13.3 kN/m (Partial 0-5000)**

Live Load

Roof LL	=	1.0m x 0.6 kN/m^2	= 0.6 kN/m
		TOTAL LL	**= 0.6 kN/m (Partial 0-5000)**

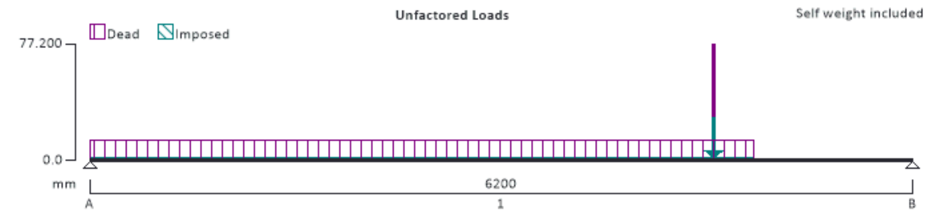
USE 356x368x129 UC
OR
305x305x118 UC with 10mm top plate to suit wall width
(SEE CALC B8)
Bolted to B9

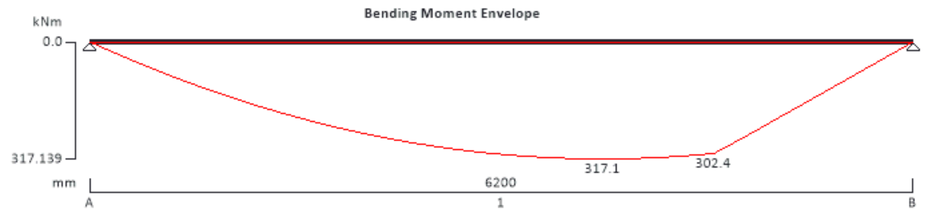
eng@huddsdesign.com
www.huddsdesign.com

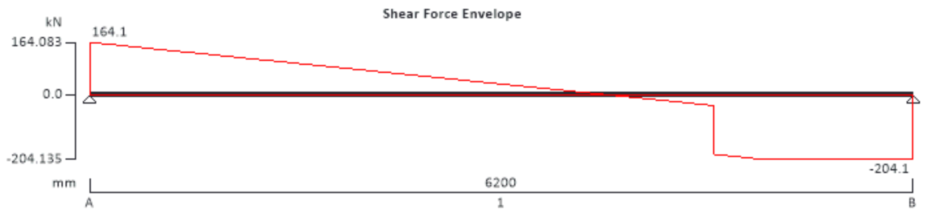
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.07







Support conditions

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

Applied loading

Beam loads	Dead self weight of beam × 1
	Dead partial UDL 13 kN/m from 0 mm to 5000 mm
	Imposed partial UDL 1.8 kN/m from 0 mm to 5000 mm
	Dead point load 77.2 kN at 4700 mm
	Imposed point load 28.6 kN at 4700 mm
	Dead partial UDL 13.3 kN/m from 0 mm to 5000 mm
	Imposed partial UDL 0.6 kN/m from 0 mm to 5000 mm

Load combinations

Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60
		Dead × 1.40
		Imposed × 1.60

Support B — Dead × 1.40

Imposed × 1.60

Analysis results

Maximum moment	M_{max} = **317.1** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **164.1** kN	V_{min} = **-204.1** kN
Deflection	δ_{max} = **10.6** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **164.1** kN	R_{A_min} = **164.1** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **101.1** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **14.1** kN	
Maximum reaction at support B	R_{B_max} = **204.1** kN	R_{B_min} = **204.1** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **115.5** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **26.5** kN	

Section details

Section type **UC 356x368x129 (British Steel Section Range 2022 (BS4-1))** Steel grade **S275**

Classification of cross sections - Section 3.5

Tensile strain coefficient ε = **1.02** Section classification **Semi-compact**

Shear capacity - Section 4.2.3

Design shear force F_v = **204.1** kN Design shear resistance P_v = **588** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment M = **317.1** kNm Moment capacity low shear M_c = **657.3** kNm

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment M_b = **505.9** kNm M_b / m_{LT} = **549.6** kNm

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection δ_{lim} = **24.8** mm Maximum deflection δ = **10.597** mm

PASS - Maximum deflection does not exceed deflection limit

Beam B9

Max Clear Span = 6450mm

DL B5	=	19.6 kN @ 3250mm
LL B5	=	2.1 kN @ 3250mm
DL B7	=	158.8 kN @ 4650mm
LL B7	=	36.1 kN @ 4650mm
DL B8	=	115.5 kN @ 4650mm
LL B8	=	26.5 kN @ 4650mm

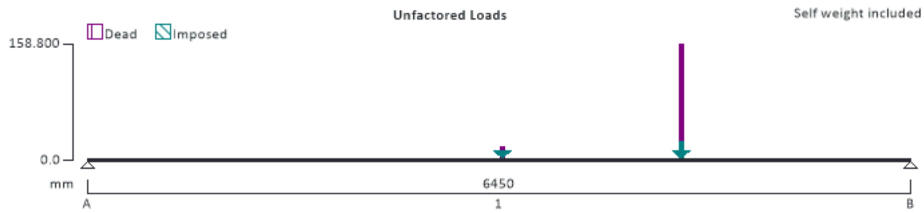
USE 356x368x177 UC
OR
305x305x198 UC
(SEE CALC B9)
Bolted to Columns / B10

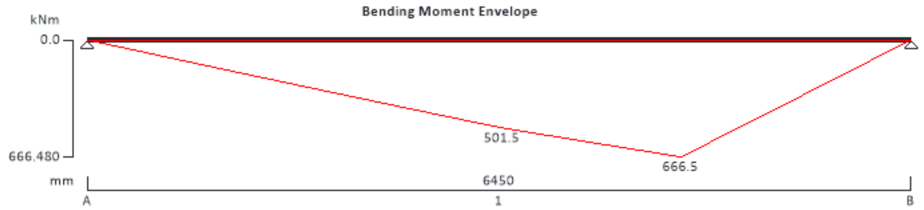
Hudds Design Structural Engineering Consultants Hudds Design 103 - 105 Bradford Rd Huddersfield HD1 6DZ	Project 2 Vernon Close, Edgerton, Huddersfield, HD1 5QE				Job no. HD-S26-0104	
	Calcs for Beam B9				Start page no./Revision B9 - 2	
	Calcs by AM	Calcs date 28/01/2026	Checked by JA	Checked date 28/01/2026	Approved by	Approved date

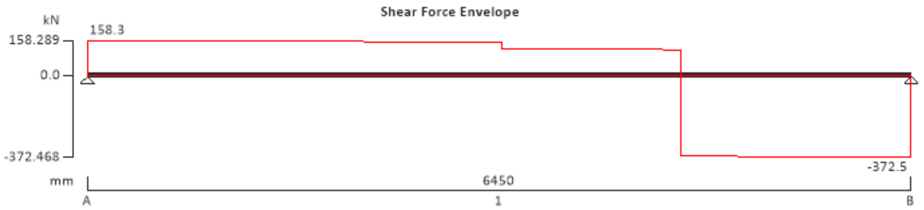
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.07







Support conditions

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

Applied loading

Beam loads	Dead self weight of beam × 1
	Dead point load 19.6 kN at 3250 mm
	Imposed point load 2.1 kN at 3250 mm
	Dead point load 158.8 kN at 4650 mm
	Imposed point load 36.1 kN at 4650 mm
	Dead point load 115.5 kN at 4650 mm
	Imposed point load 26.5 kN at 4650 mm

Load combinations

Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60
		Dead × 1.40
		Imposed × 1.60

	Support B	Dead × 1.40
		Imposed × 1.60

Analysis results

Maximum moment	M_{max} = **666.5** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **158.3** kN	V_{min} = **-372.5** kN
Deflection	δ_{max} = **2.4** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **158.3** kN	R_{A_min} = **158.3** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **91.9** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **18.5** kN	
Maximum reaction at support B	R_{B_max} = **372.5** kN	R_{B_min} = **372.5** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **213.3** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **46.2** kN	

Section details

Section type **UC 356x368x177 (British Steel Section Range 2022 (BS4-1))** Steel grade **S275**

Classification of cross sections - Section 3.5

Tensile strain coefficient ε = **1.02** Section classification **Plastic**

Shear capacity - Section 4.2.3

Design shear force F_v = **372.5** kN Design shear resistance P_v = **843** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment M = **666.5** kNm Moment capacity low shear M_c = **921.6** kNm

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment M_b = **737.1** kNm M_b / m_{LT} = **964** kNm

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection δ_{lim} = **17.917** mm Maximum deflection δ = **2.359** mm

PASS - Maximum deflection does not exceed deflection limit

Beam B10

Max Clear Span = 1700mm

Dead Load

Wall DL	=	4.0m x 2.0 kN/m^2	= 8.0 kN/m
DL B9	=	92.0 kN @ 850mm	
LL B9	=	19.0 kN @ 850mm	

USE 203x203x46 UC
(SEE CALC B8)
Min 300mm End Bearing

Padstones

Reaction Maximum = 90.0 kN

USE 440 x 200 x 215mm DEEP CONCRETE PADSTONES

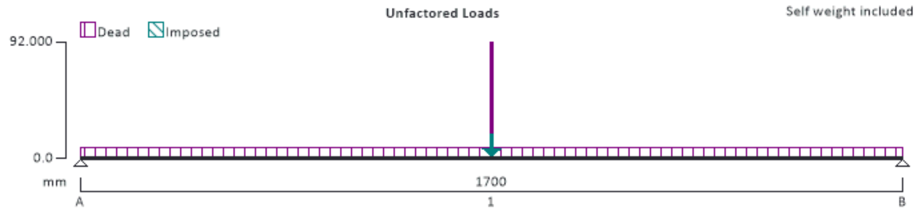
Stress Check = 90 x 1000 / 440 x 200 = 1.0 N/mm^2 < 2.4 N/mm^2
Satisfactory

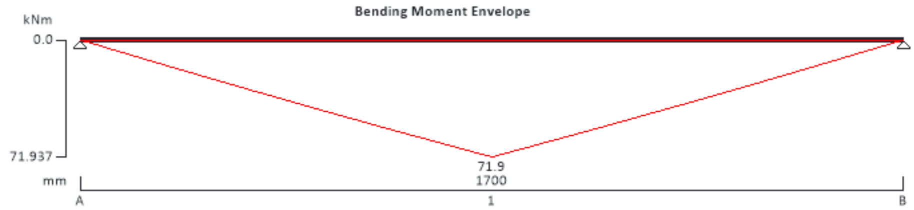
Hudds Design Structural Engineering Consultants Hudds Design 103 - 105 Bradford Rd Huddersfield HD1 6DZ	Project 2 Vernon Close, Edgerton, Huddersfield, HD1 5QE				Job no. HD-S26-0104	
	Calcs for Beam B10				Start page no./Revision B10 - 2	
	Calcs by AM	Calcs date 28/01/2026	Checked by JA	Checked date 28/01/2026	Approved by	Approved date

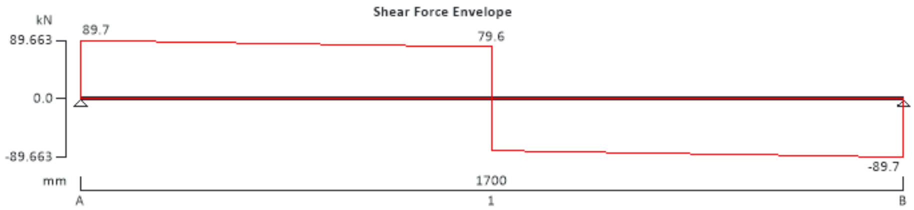
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.07







Support conditions

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

Applied loading

Beam loads	Dead self weight of beam × 1
	Dead full UDL 8 kN/m
	Dead point load 92 kN at 850 mm
	Imposed point load 19 kN at 850 mm

Load combinations

Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60
		Dead × 1.40
		Imposed × 1.60
	Support B	Dead × 1.40
		Imposed × 1.60

Analysis results

Maximum moment	M_{max} = **71.9** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **89.7** kN	V_{min} = **-89.7** kN
Deflection	δ_{max} = **0.2** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **89.7** kN	R_{A_min} = **89.7** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **53.2** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **9.5** kN	
Maximum reaction at support B	R_{B_max} = **89.7** kN	R_{B_min} = **89.7** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **53.2** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **9.5** kN	

Section details

Section type **UC 203x203x46 (British Steel Section Range 2022 (BS4-1))** Steel grade **S275**

Classification of cross sections - Section 3.5

Tensile strain coefficient ε = **1.00** Section classification **Compact**

Shear capacity - Section 4.2.3

Design shear force F_v = **89.7** kN Design shear resistance P_v = **241.4** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment M = **71.9** kNm Moment capacity low shear M_c = **138** kNm

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment M_b = **134.4** kNm M_b / m_{LT} = **157.3** kNm

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection δ_{lim} = **4.722** mm Maximum deflection δ = **0.206** mm

PASS - Maximum deflection does not exceed deflection limit

Column C1 /C2

C1 Loads
Reaction Maximum = 373 kN
Moment M = 50 kNm

C2 Loads
Reaction Maximum = 233 kN
Moment M = 100 kNm

USE 203x203x86 UC (Grade S275)
With 400x400x15mm Base Plate with 4 No. M20 Anchor bolts
To RF/PF1

Increase local RAFT slab thickening as below
2.4x2.4x0.25m Deep RC Footing with T16 Bars both directions top and Bottom
Min 50mm Cover
Min 150mm Hardcore Base

See Calcs C1 / C2 / PF1

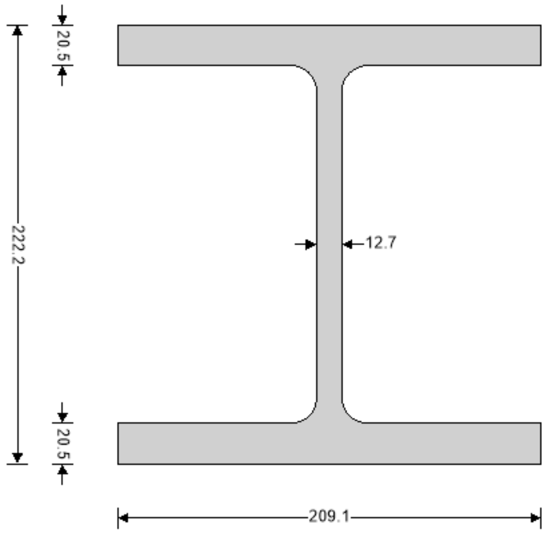
STEEL MEMBER DESIGN (BS5950)

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

TEDDS calculation version 3.0.07

Section details

Section type **UC 203x203x86 (British Steel Section Range 2022 (BS4-1))** Steel grade **S275**



Classification of cross sections - Section 3.5

Tensile strain coefficient ε = **1.02** Section classification **Plastic**

Shear capacity - Section 4.2.3

Shear capacity - Section 4.2.3

Design shear force F_v = **375** kN Design shear resistance $P_{x,v}$ = **1226.8** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment M_x = **50** kNm Moment capacity high shear M_{cx} = **241.2** kNm

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment M_b = **245.4** kNm M_b / m_{LT} = **245.4** kNm

PASS - Moment capacity exceeds design bending moment

Moment capacity - Section 4.2.5

Design bending moment M_y = **10** kNm Moment capacity low shear M_{cy} = **95.1** kNm

PASS - Moment capacity exceeds design bending moment

Compression members - Section 4.7

Design compression force F_c = **375** kN Compression resistance P_{cx} = **2654** kN

PASS - Compression resistance exceeds design compression force

Design compression force F_c = **375** kN Compression resistance P_{cy} = **1932.9** kN

PASS - Compression resistance exceeds design compression force

Compression members with moments - Section 4.8.3

Comp.and bending check $F_c / (A \times p_y) + M_x / M_{cx} + M_y / M_{cy}$ = **0.426**

PASS - Combined bending and compression check is satisfied

Member buckling resistance - cl.4.8.3.3.2

Buckling resistance checks $F_c / P_{cx} + m_x \times M_x / M_{cx} \times (1 + 0.5 \times F_c / P_{cx}) + 0.5 \times m_{yx} \times M_y / M_{cy}$ = **0.400**

$F_c / P_{cy} + m_{LT} \times M_{LT} / M_b + m_y \times M_y / M_{cy} \times (1 + F_c / P_{cy})$ = **0.523**

Interactive buckling

$m_x \times M_x \times (1 + 0.5 \times (F_c / P_{cx})) / (M_{cx} \times (1 - F_c / P_{cx})) + m_y \times M_y \times (1 + (F_c / P_{cy})) / (M_{cy} \times (1 - F_c / P_{cy}))$ = **0.396**

PASS - Member buckling resistance checks are satisfied

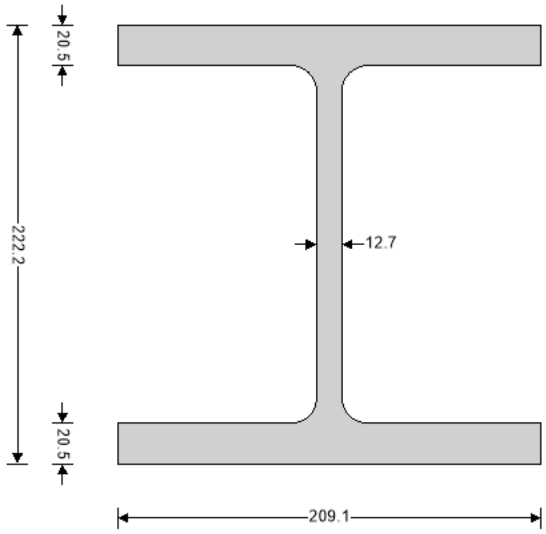
STEEL MEMBER DESIGN (BS5950)

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

TEDDS calculation version 3.0.07

Section details

Section type **UC 203x203x86 (British Steel Section Range 2022 (BS4-1))** Steel grade **S275**



Classification of cross sections - Section 3.5

Tensile strain coefficient ε = **1.02** Section classification **Plastic**

Shear capacity - Section 4.2.3

Shear capacity - Section 4.2.3

Design shear force F_v = **233** kN Design shear resistance $P_{x,v}$ = **1226.8** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment M_x = **100** kNm Moment capacity low shear M_{cx} = **260** kNm

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment M_b = **245.4** kNm M_b / m_{LT} = **245.4** kNm

PASS - Buckling resistance moment exceeds design bending moment

Moment capacity - Section 4.2.5

Design bending moment M_y = **10** kNm Moment capacity low shear M_{cy} = **95.1** kNm

PASS - Moment capacity exceeds design bending moment

Compression members - Section 4.7

Design compression force F_c = **233** kN Compression resistance P_{cx} = **2654** kN

PASS - Compression resistance exceeds design compression force

Design compression force F_c = **233** kN Compression resistance P_{cy} = **1932.9** kN

PASS - Compression resistance exceeds design compression force

Compression members with moments - Section 4.8.3

Comp.and bending check $F_c / (A \times p_y) + M_x / M_{cx} + M_y / M_{cy}$ = **0.570**

PASS - Combined bending and compression check is satisfied

Member buckling resistance - cl.4.8.3.3.2

Buckling resistance checks $F_c / P_{cx} + m_x \times M_x / M_{cx} \times (1 + 0.5 \times F_c / P_{cx}) + 0.5 \times m_{yx} \times M_y / M_{cy}$ = **0.542**

$F_c / P_{cy} + m_{LT} \times M_{LT} / M_b + m_y \times M_y / M_{cy} \times (1 + F_c / P_{cy}) =$ **0.646**

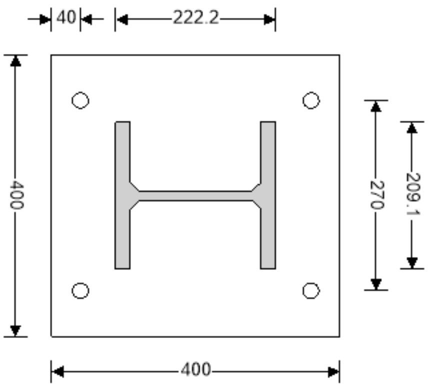
Interactive buckling

$m_x \times M_x \times (1 + 0.5 \times (F_c / P_{cx})) / (M_{cx} \times (1 - F_c / P_{cx})) + m_y \times M_y \times (1 + (F_c / P_{cy})) / (M_{cy} \times (1 - F_c / P_{cy})) =$ **0.574**

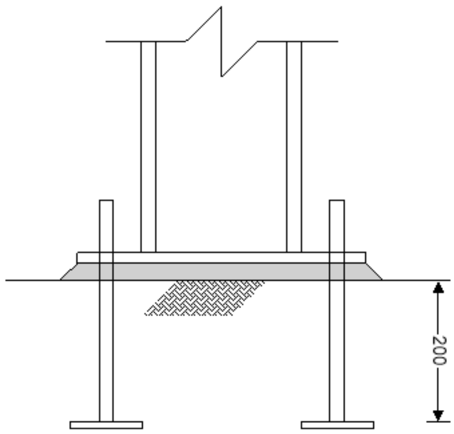
PASS - Member buckling resistance checks are satisfied

COLUMN BASE PLATE DESIGN (BS5950-1:2000)

TEDDS calculation version 1.0.09



15 mm thick base plate



Design forces and moments

Axial force	F_c = **375.0** kN (Compression)
Bending moment	M = **10.0** kNm
Shear force	F_v = **10.0** kN

Column details

Column section	**UC 203x203x86 (Grade S275)**
Depth	D = **222.2** mm
Breadth	B = **209.1** mm
Flange thickness	T = **20.5** mm
Web thickness	t = **12.7** mm
Design strength	p_{yc} = **265** N/mm^2
Column flange to base plate weld	**6 mm FW**
Column web to base plate weld	**6 mm FW**

Baseplate details

Steel grade	**S275**
Depth	D_p = **400** mm
Breadth	B_p = **400** mm
Thickness	t_p = **15** mm
Design strength	p_{yp} = **275** N/mm^2

Holding down bolt and anchor plate details

Total number of bolts	**4 No. M20 Grade 8.8**
Bolt spacing	s_{bolt} = **270** mm
Edge distance	e_1 = **40** mm
Anchor plate steel grade	**S275**
Anchor plate dimension (square)	b_{ap} = **100** mm
Anchor plate thickness	t_{ap} = **10** mm
Design strength	p_{yap} = **275** N/mm^2

Embeddment to top of anchor plate — $E = \mathbf{200}$ mm

Characteristic strength of concrete — $f_{cu} = \mathbf{25}$ N/mm^2

Concrete compression force and bolt tension force

Plate overhang beyond face of flange — $L_1 = (D_p - D)/2) = \mathbf{88.9}$ mm

Effective width of plate — $B_{pc} = \min(B_p, B + 2\times L_1) = \mathbf{386.9}$ mm

Distance from bolts to compression edge — $h = D_p - e_1 = \mathbf{360}$ mm

Assuming a rectangular compression block of width b_{pc}, length x and intensity $0.6f_{cu}$ then:-

From static equilibrium — $M = 0.6f_{cu}B_{pc}x(h-x/2) - F_c(h-D_p/2)$

Rearranging the quadratic equation — $0.3f_{cu}B_{pc}x^2 - 0.6f_{cu}B_{pc}hx + F_c(h-D_p/2) + M = 0$

Factor a — $a = 0.3 \times f_{cu} \times B_{pc} = \mathbf{2901.8}$ N/mm

Factor b — $b = -0.6 \times f_{cu} \times B_{pc} \times h = \mathbf{-2089260.0}$ N

Constant c — $c = F_c \times (h-D_p/2) + M = \mathbf{70000000.0}$ Nmm

Depth of compression block — $x = [-1.0\times b - \sqrt{(b^2 - 4\times a\times c)}]/(2\times a) = \mathbf{35.2}$ mm

Compression force in concrete — $C_f = 0.6 \times f_{cu} \times B_{pc} \times x = \mathbf{204.4}$ kN

Tension force in bolts — $T_f = C_f - F_c = \mathbf{-170.6}$ kN

Therefore the bolts are not in tension - Recalculate depth of compression block

Actual compression force in concrete — $C_f = F_c = \mathbf{375}$ kN

Actual force in bolts — $T_f = 0$ kN

Eccentricity of compression force — $e = M/F_c = \mathbf{26.7}$ mm

Depth of compression block — $x = 2 \times (D_p/2 - e) = \mathbf{346.7}$ mm

Stress in concrete — $f_c = F_c / (B_{pc} \times x) = \mathbf{2.8}$ N/mm^2

The depth of the compression block exceeds $D_p - D + T$

Therefore the compression block will be 'T' shaped

Let — $m = 2\times L_1 + T = \mathbf{198.3}$ mm

and let — $n = \min(2\times L_1 + t, B_{pc}) = \mathbf{190.5}$ mm

Then equating eccentricity of applied loading to centroid of compression 'T' and rearranging the equation (Note x is length of stem of 'T')

Quadratic equation — $0 = nx^2/2 + n(m-(D_p/2-M/F_c))x+ B_{pc}m(m-(D_p-2M/F_c))/2$

Factor a — $a = n/2 = \mathbf{95.3}$ mm

Factor b — $b = n\times(m-(D_p/2-M/F_c)) = \mathbf{4756}$ mm^2

Constant c — $c = B_{pc}\times m\times(m-(D_p-2\times M/F_c))/2 = \mathbf{-5691514}$ mm^3

Length of stem of 'T' — $x = [-1.0\times b + \sqrt{(b^2 - 4\times a\times c)}]/(2\times a) = \mathbf{220.8}$ mm

Length of stem of 'T' exceeds $D-2\times(L_1+T)$ - Recalculate with 'I' shaped compression block

Now additionally let — $l = D - 2\times(L_1+T) = \mathbf{3.4}$ mm

Equating eccentricity of applied loading to centroid of compression 'I' and rearranging the equation (Note x is depth of flange of 'I')

Quadratic equation — $0 = B_{pc}x^2/2 +$

$B_{pc}((m+l)-(D_p/2-M/F_c))x+B_{pc}m^2/2+nl(m+l/2)-(B_{pc}m+nl)\times(D_p/2-M/F_c)$

Factor a — $a = B_{pc}/2 = \mathbf{193.5}$ mm

Factor b — $b = B_{pc}\times((m+l)-(D_p/2-M/F_c)) = \mathbf{10975}$ mm^2

Constant c — $c = B_{pc}\times m^2/2+n\times l\times(m+l/2)-(B_{pc}\times m+n\times l)\times(D_p/2-M/F_c) = \mathbf{-5674242}$ mm^3

Depth of flange of 'I' — $x = [-1.0\times b + \sqrt{(b^2 - 4\times a\times c)}]/(2\times a) = \mathbf{145.2}$ mm

Area of compression 'I' — $A_{comp} = (B_{pc}\times m) + (l\times n) + (B_{pc}\times x) = \mathbf{133560}$ mm^2

Stress in concrete — $f_c = F_c / A_{comp} = \mathbf{2.8}$ N/mm^2

Compression side bending

Moment in plate — $m_c = f_c \times (L_1 - 0.8\times s_{wf})^2/2) = \mathbf{9929}$ **Nmm/mm**

Plate thickness required $t_{pc} = \sqrt{(4 \times m_c/p_{yp})}$ = **12.0** mm

Plate thickness

Plate thickness required $t_{p\_req} = t_{pc}$ = **12.0 mm**

Plate thickness provided t_p = **15** mm

PASS - Plate thickness provided is adequate (0.801)

Flange weld

Tension capacity of flange $P_{tf} = B \times T \times p_{yc}$ = **1135.9** kN

Force in tension flange $F_{tf} = M/(D - T) - F_c \times (B \times T)/A$ = **-96.4** kN

Flange weld design force $F_f = \min(P_{tf}, \max(F_{tf}, 0\text{ kN}))$ = **0.0** kN

Weld force per mm $f_{wf} = F_f/(2 \times B - t)$ = **0.000** kN/mm

Transverse capacity of 6 mm fillet weld p_{wf} = **1.155** kN/mm (Cl. 6.8.7.3)

PASS - Flange weld capacity is adequate (0.000)

Longitudinal capacity of web weld

Weld force per mm $f_{wwl} = F_v/(2 \times (D - 2 \times T))$ = **0.028** kN/mm

Longitudinal capacity of 6 mm fillet weld p_{wwl} = **0.924** kN/mm (Cl. 6.8.7.3)

PASS - Longitudinal capacity of web weld is adequate (0.030)

Shear transfer to concrete

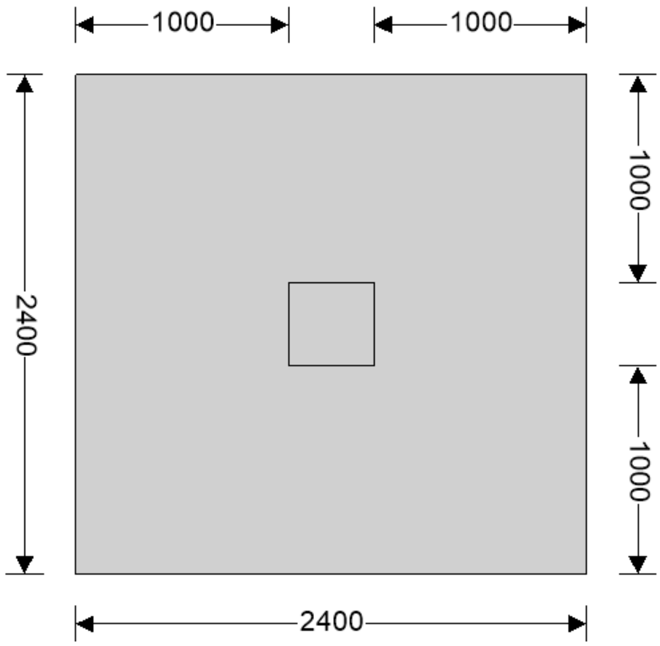
Assumed coefficient of friction μ = 0.30

Available shear resistance $P_v = C_f \times \mu$ = **113** kN

PASS - Frictional shear capacity is adequate (0.089)

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

Tedds calculation version 2.0.07



Pad footing details

Length of pad footing $L = \mathbf{2400}$ mm Width of pad footing $B = \mathbf{2400}$ mm

Depth of pad footing $h = \mathbf{250}$ mm Depth of soil over pad footing $h_{soil} = \mathbf{1500}$ mm

Density of concrete $\rho_{conc} = \mathbf{23.6}$ kN/m^3

Column details

Column base length $l_A = \mathbf{400}$ mm Column base width $b_A = \mathbf{400}$ mm

Column eccentricity in x $e_{PxA} = \mathbf{0}$ mm Column eccentricity in y $e_{PyA} = \mathbf{0}$ mm

Soil details

Dense, moderately to well graded, sub-angular, gravel

Mobilisation factor $m = \mathbf{1.5}$

Depth of soil over pad footing $h_{soil} = \mathbf{1500}$ mm Density of soil $\rho_{soil} = \mathbf{20.0}$ kN/m^3

Allowable bearing pressure $P_{bearing} = \mathbf{110}$ kN/m^2

Axial loading on column

Dead axial load $P_{GA} = \mathbf{215.0}$ kN Imposed axial load $P_{QA} = \mathbf{47.0}$ kN

Wind axial load $P_{WA} = \mathbf{0.0}$ kN Total axial load $P_A = \mathbf{262.0}$ kN

Foundation loads

Dead surcharge load $F_{Gsur} = \mathbf{0.000}$ kN/m^2 Imposed surcharge load $F_{Qsur} = \mathbf{1.500}$ kN/m^2

Pad footing self weight $F_{swt} = \mathbf{5.900}$ kN/m^2

Soil self weight $F_{soil} = \mathbf{30.000}$ kN/m^2 Total foundation load $F = \mathbf{215.4}$ kN

Calculate pad base reaction

Total base reaction $T = \mathbf{477.4}$ kN

Base reaction eccentricity in x $e_{Tx} = \mathbf{0}$ mm Base reaction eccentricity in y $e_{Ty} = \mathbf{0}$ mm

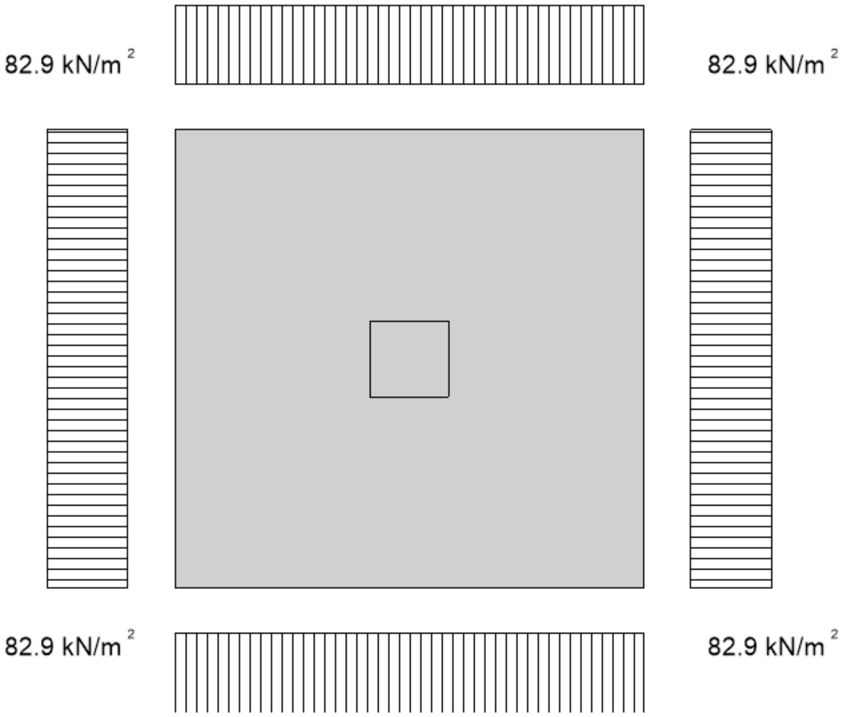
Base reaction acts within middle third of base

Calculate pad base pressures

$q_1 = \mathbf{82.886}$ kN/m^2 $q_2 = \mathbf{82.886}$ kN/m^2 $q_3 = \mathbf{82.886}$ kN/m^2 $q_4 = \mathbf{82.886}$ kN/m^2

Minimum base pressure q_{min} = **82.886** kN/m² Maximum base pressure q_{max} = **82.886** kN/m²

PASS - Maximum base pressure is less than allowable bearing pressure



Partial safety factors for loads

Dead loads γ_{fG} = **1.40** Imposed loads γ_{fQ} = **1.60**

Wind loads γ_{fW} = **0.00**

Ultimate axial loading on column

Ultimate axial load on column P_{uA} = **376.2** kN

Ultimate foundation loads

Ultimate foundation load F_u = **303.3** kN

Ultimate horizontal loading on column

Ult.horizontal load in x dir H_{xuA} = **0.0** kN Ult. horizontal load in y dir H_{yuA} = **0.0** kN

Ultimate moment on column

Ult.moment on column in x dir M_{xuA} = **0.000** kNm Ult.moment on column in y dir M_{yuA} = **0.000** kNm

Ultimate pad base reaction

Ultimate base reaction T_u = **679.5** kN

Ecc.of ult.base reaction in x e_{Txu} = **0** mm Ecc.of ult.base reaction in y e_{Tyu} = **0** mm

Calculate ultimate pad base pressures

q_{1u} = **117.973** kN/m² q_{2u} = **117.973** kN/m² q_{3u} = **117.973** kN/m² q_{4u} = **117.973** kN/m²

Minimum ult.base pressure q_{minu} = **117.973** kN/m² Maximum ult.base pressure q_{maxu} = **117.973** kN/m²

Library item: Ultimate pressures summary **Ultimate moments**

Ultimate moment in x dir M_x = **112.860** kNm Ultimate moment in y dir M_y = **112.860** kNm

Material details

Char.strength of concrete f_{cu} = **30** N/mm² Char.strength of reinft f_y = **500** N/mm²

Char.strength of shear reinft f_{yv} = **500** N/mm² Nom.cover to reinforcement c_{nom} = **50** mm

Moment design in x direction

Tens.reinforcement diameter ϕ_{xB} = **16** mm
Tens.reinforcement depth d_x = **192** mm

Design formula for rectangular beams (cl 3.4.4.4)
K_x = **0.043**
K_x' = **0.156**

$K_x < K_x'$ compression reinforcement is not required

Tens.reinforcement required $A_{s\_x\_req}$ = **1422** mm^2
Minimum tens.reinforcement $A_{s\_x\_min}$ = **780** mm^2
Tens.reinforcement provided **12 No. 16 dia. bars btm**
$A_{s\_xB\_prov}$ = **2413** mm^2

PASS - Tension reinforcement provided exceeds tension reinforcement required

Moment design in y direction

Tens.reinforcement diameter ϕ_{yB} = **16** mm
Tens.reinforcement depth d_y = **176** mm

Design formula for rectangular beams (cl 3.4.4.4)
K_y = **0.051**
K_y' = **0.156**

$K_y < K_y'$ compression reinforcement is not required

Tens.reinforcement required $A_{s\_y\_req}$ = **1568** mm^2
Minimum tens.reinforcement $A_{s\_y\_min}$ = **780** mm^2
Tens.reinforcement provided **12 No. 16 dia. bars btm**
$A_{s\_yB\_prov}$ = **2413** mm^2

PASS - Tension reinforcement provided exceeds tension reinforcement required

Calculate ultimate shear force at d from top face of column

Ult.pressure for shear q_{su} = **117.973** kN/m^2
Area loaded for shear A_s = **1.978** m^2
Ult.shear force V_{su} = **129.162** kN

Shear stresses at d from top face of column (cl 3.5.5.2)

Design shear stress v_{su} = **0.306** N/mm^2
Design concrete shear stress v_c = **0.667** N/mm^2
Allowable design shear stress v_{max} = **4.382** N/mm^2

PASS - $v_{su} < v_c$ - No shear reinforcement required

Calculate ultimate punching shear force at face of column

Ult.press.for punching shear q_{puA} = **117.973** kN/m^2
Avg.effective reinf.depth d = **184** mm
Area loaded A_{pA} = **0.160** m^2
Length of shear perimeter u_{pA} = **1600** mm
Ult.punching shear force V_{puA} = **365.750** kN
Eff.punching shear force V_{puAeff} = **365.750** kN

Punching shear stresses at face of column (cl 3.7.7.2)

Design shear stress v_{puA} = **1.242** N/mm^2

PASS - Design shear stress is less than allowable design shear stress

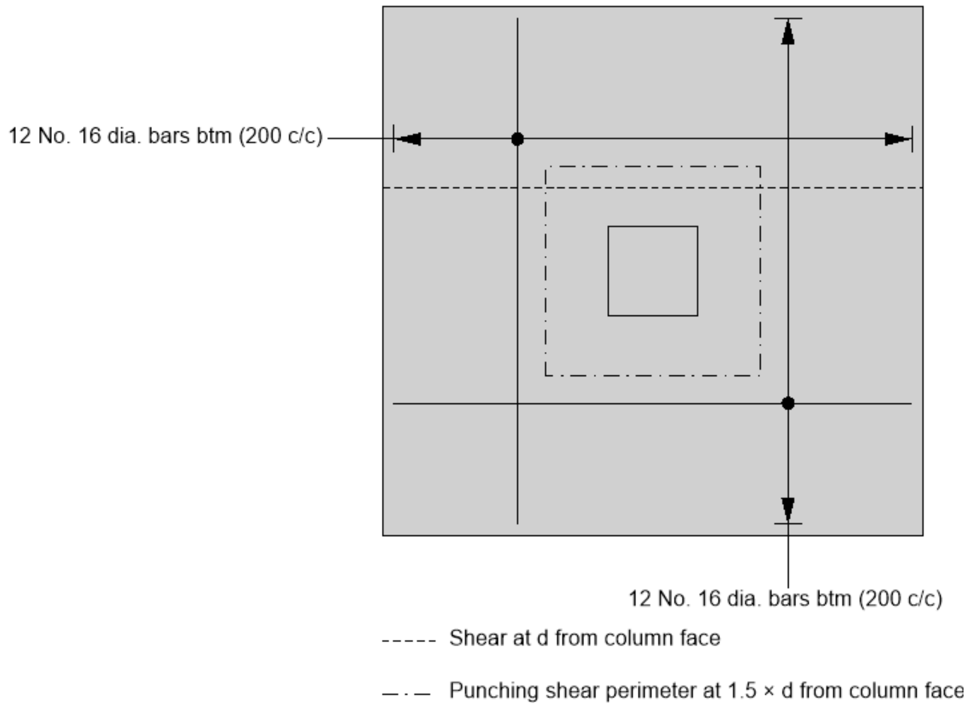
Calculate ultimate punching shear force at perimeter of 1.5 d from face of column

Ult.press.for punching shear $q_{puA1.5d}$ = **117.973** kN/m^2
Avg.effective reinf.depth d = **184** mm
Area loaded $A_{pA1.5d}$ = **0.906** m^2
Length of shear perimeter $u_{pA1.5d}$ = **3808** mm
Ult.punching shear force $V_{puA1.5d}$ = **317.007** kN
Eff.punching shear force $V_{puA1.5deff}$ = **317.007** kN

Punching shear stresses at perimeter of 1.5 d from face of column (cl 3.7.7.2)

Design shear stress $v_{puA1.5d}$ = **0.452** N/mm^2

PASS - $v_{puA1.5d} < v_c$ - No shear reinforcement required



12 No. 16 dia. bars btm (200 c/c)
12 No. 16 dia. bars btm (200 c/c)
Shear at d from column face
Punching shear perimeter at 1.5 × d from column face

Raft Slab - RF
Max Clear Span = 8000mm

Grid Line 1/4

Dead Load

Wall DL	=	7.0m x 4.4 kN/m^2	= 30.8 kN/m
Floor DL	=	4.7m x 1.0 kN/m^2	= 4.7 kN/m
Roof DL	=	2.0m x 1.3 kN/m^2	= 2.6 kN/m
		TOTAL DL	**= 38.1 kN/m**

Live Load

Floor LL	=	4.7m x 1.5 kN/m^2	= 7.1 kN/m
Roof LL	=	2.0m x 0.6 kN/m^2	= 1.2 kN/m
		TOTAL LL	**= 8.3 kN/m**

Grid Line 2/3

Dead Load

Wall DL	=	9.0m x 3.0 kN/m^2	= 27.0 kN/m
Floor DL	=	8.0m x 1.0 kN/m^2	= 8.0 kN/m
Roof DL	=	2.0m x 1.3 kN/m^2	= 2.6 kN/m
		TOTAL DL	**= 37.6 kN/m**

Live Load

Floor LL	=	8.0m x 1.5 kN/m^2	= 12.0 kN/m
Roof LL	=	2.0m x 0.6 kN/m^2	= 1.2 kN/m
		TOTAL LL	**= 13.2 kN/m**

Floor – (Raft Slab)

Dead Loads = 1.8 kN/m^2 (Additional Self-Weight Included in TEDDS Calc)

Imposed Loads = 1.5 kN/m^2

'Raft Foundation RF' – See calc RF)
USE 250mm Thick Reinforced Concrete Raft Slab
With **2 Layers** A393 Mesh Top **and** Bottom (2 Top & 2 Bottom)

Provide edge and internal thickenings as shown
450x450mm RC Beams with 3 No. T16 Bars top and bottom
with T10 links at 200mm ctrs.
Provide links at 100mm ctrs to all corners 1m away.
Add diagonal T16 corner bars (top + bottom) at re-entrant corners and sharp external/internal corners. Full lap throughs for Bars on all corners.

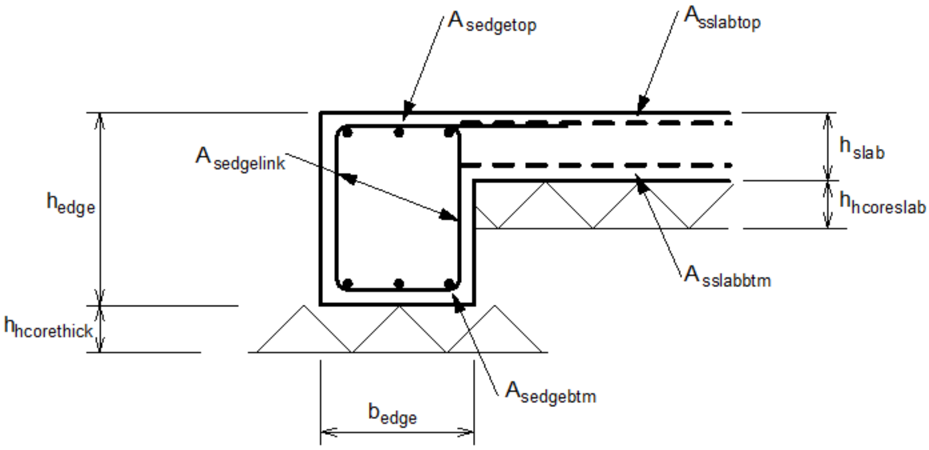
Additional thickening below columns as noted 2.4m square

Min Well Compacted Hardcore below to siltstone
Min 50mm Cover to all reinforcement
Min Grade C30 Concrete

Note: Coal Depth is 17m at 1.2m Max. 10T (12m) Rule satisfied for Strip Footing. Raft provided for additional robustness.
Refer to Geo Tech Report G25226 Rev B, dated 8th Sept 2025.

RAFT FOUNDATION DESIGN (BS8110 : PART 1 : 1997)

Tedds calculation version 1.0.13



Soil and raft definition

Soil definition

Allowable bearing pressure q_{allow} = **50.0** kN/m^2

Number of types of soil forming sub-soil **One type only**

Soil density **Firm**

Depth of hardcore beneath slab $h_{hcoreslab}$ = **150** mm (Dispersal allowed for bearing pressure check)

Depth of hardcore beneath thickenings $h_{hcorethick}$ = **150** mm (Dispersal allowed for bearing pressure check)

Density of hardcore γ_{hcore} = **20.0** kN/m^3

Basic assumed diameter of local depression $\phi_{depbasic}$ = **1500**mm

Diameter under slab modified for hardcore $\phi_{depslab} = \phi_{depbasic} - h_{hcoreslab}$ = **1350** mm

Diameter under thickenings modified for hardcore $\phi_{depthick} = \phi_{depbasic} - h_{hcorethick}$ = **1350** mm

Raft slab definition

Max dimension/max dimension between joints l_{max} = **8.000** m

Slab thickness h_{slab} = **250** mm

Concrete strength f_{cu} = **25** N/mm^2

Poissons ratio of concrete ν = **0.2**

Slab mesh reinforcement strength f_{yslab} = **500** N/mm^2

Partial safety factor for steel reinforcement γ_s = **1.15**

From C&CA document ‘Concrete ground floors’ Table 5

Minimum mesh required in top for shrinkage **A142**

Actual mesh provided in top **2 x A393 ($A_{sslabtop}$ = 786 mm^2/m)**

Mesh provided in bottom **2 x A393 ($A_{sslabbtm}$ = 786 mm^2/m)**

Top mesh bar diameter $\phi_{slabtop}$ = **10** mm

Bottom mesh bar diameter $\phi_{slabbtm}$ = **10** mm

Cover to top reinforcement c_{top} = **50** mm

Cover to bottom reinforcement c_{btm} = **50** mm

Average effective depth of top reinforcement $d_{tslabav} = h_{slab} - c_{top} - \phi_{slabtop}$ = **190** mm

Average effective depth of bottom reinforcement $d_{bslabav} = h_{slab} - c_{btm} - \phi_{slabbtm}$ = **190** mm

Overall average effective depth $d_{slabav} = (d_{tslabav} + d_{bslabav})/2$ = **190** mm

Minimum effective depth of top reinforcement $d_{tslabmin} = d_{tslabav} - \phi_{slabtop}/2 =$ **185** mm

Minimum effective depth of bottom reinforcement $d_{bslabmin} = d_{bslabav} - \phi_{slabbtm}/2 =$ **185** mm

Edge beam definition

Overall depth $h_{edge} =$ **450** mm

Width $b_{edge} =$ **450** mm

Strength of main bar reinforcement $f_y =$ **500** N/mm^2

Strength of link reinforcement $f_{ys} =$ **500** N/mm^2

Reinforcement provided in top **3 H16 bars ($A_{sedgetop}$ = 603 mm^2)**

Reinforcement provided in bottom **3 H16 bars ($A_{sedgebtm}$ = 603 mm^2)**

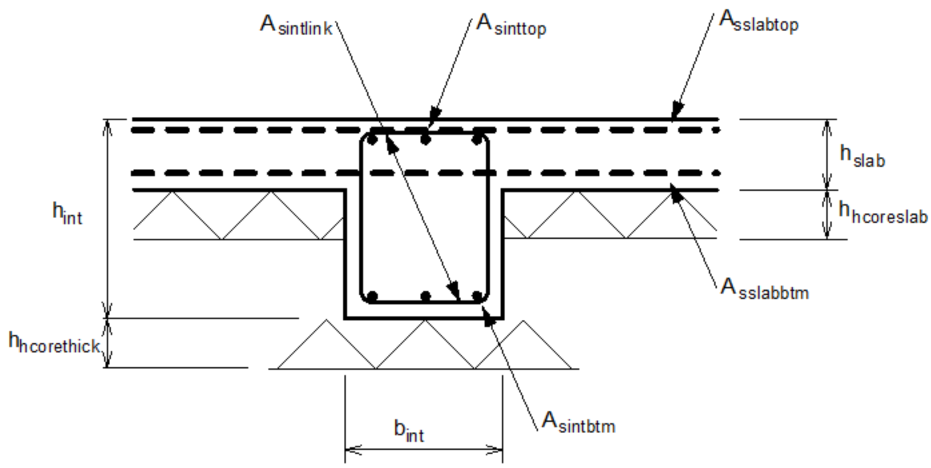
Link reinforcement provided **2 H10 legs at 200 ctrs (A_{sv}/s_v = 0.785 mm)**

Bottom cover to links $c_{beam} =$ **50** mm

Effective depth of top reinforcement $d_{edgetop} = h_{edge} - c_{top} - \phi_{slabtop} - \phi_{edgelink} - \phi_{edgetop}/2 =$ **372** mm

Effective depth of bottom reinforcement $d_{edgebtm} = h_{edge} - c_{beam} - \phi_{edgelink} - \phi_{edgebtm}/2 =$ **382** mm

Internal beam definition



Overall depth $h_{int} =$ **450** mm

Width $b_{int} =$ **450** mm

Strength of main bar reinforcement $f_y =$ **500** N/mm^2

Strength of link reinforcement $f_{ys} =$ **500** N/mm^2

Reinforcement provided in top **3 H16 bars ($A_{sinttop}$ = 603 mm^2)**

Reinforcement provided in bottom **3 H16 bars ($A_{sintbtm}$ = 603 mm^2)**

Link reinforcement provided **2 H10 legs at 200 ctrs (A_{sv}/s_v = 0.785 mm)**

Effective depth of top reinforcement $d_{inttop} = h_{int} - c_{top} - 2 \times \phi_{slabtop} - \phi_{inttop}/2 =$ **372** mm

Effective depth of bottom reinforcement $d_{intbtm} = h_{int} - c_{beam} - \phi_{intlink} - \phi_{intbtm}/2 =$ **382** mm

Internal slab design checks

Basic loading

Slab self weight $w_{slab} = 24$ kN/m$^3 \times h_{slab} =$ **6.0** kN/m^2

Hardcore $w_{hcoreslab} = \gamma_{hcore} \times h_{hcoreslab} =$ **3.0** kN/m^2

Applied loading

Uniformly distributed dead load $w_{Dudl} =$ **1.8** kN/m^2

Uniformly distributed live load $w_{Ludl} =$ **1.5** kN/m^2

Internal slab bearing pressure check

Total uniform load at formation level $w_{udl} = w_{slab} + w_{hcoreslab} + w_{Dudl} + w_{Ludl} = \mathbf{12.3}$ kN/m²

PASS - $w_{udl} <= q_{allow}$ - Applied bearing pressure is less than allowable

Internal slab bending and shear check

Applied bending moments

Span of slab $l_{slab} = \phi_{depslab} + d_{tslabav} = \mathbf{1540}$ mm

Ultimate self weight udl $w_{swult} = 1.4 \times w_{slab} = \mathbf{8.4}$ kN/m²

Self weight moment at centre $M_{csw} = w_{swult} \times l_{slab}^2 \times (1 + \nu) / 64 = \mathbf{0.4}$ kNm/m

Self weight moment at edge $M_{esw} = w_{swult} \times l_{slab}^2 / 32 = \mathbf{0.6}$ kNm/m

Self weight shear force at edge $V_{sw} = w_{swult} \times l_{slab} / 4 = \mathbf{3.2}$ kN/m

Moments due to applied uniformly distributed loads

Ultimate applied udl $w_{udlult} = 1.4 \times w_{Dudl} + 1.6 \times w_{Ludl} = \mathbf{4.9}$ kN/m²

Moment at centre $M_{cudl} = w_{udlult} \times l_{slab}^2 \times (1 + \nu) / 64 = \mathbf{0.2}$ kNm/m

Moment at edge $M_{eudl} = w_{udlult} \times l_{slab}^2 / 32 = \mathbf{0.4}$ kNm/m

Shear force at edge $V_{udl} = w_{udlult} \times l_{slab} / 4 = \mathbf{1.9}$ kN/m

Resultant moments and shears

Total moment at edge $M_{\Sigma e} = \mathbf{1.0}$ kNm/m

Total moment at centre $M_{\Sigma c} = \mathbf{0.6}$ kNm/m

Total shear force $V_{\Sigma} = \mathbf{5.1}$ kN/m

Reinforcement required in top

K factor $K_{slabtop} = M_{\Sigma e}/(f_{cu} \times d_{tslabav}^2) = \mathbf{0.001}$

Lever arm $z_{slabtop} = d_{tslabav} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{slabtop}/0.9)}) = \mathbf{180.5}$ mm

Area of steel required for bending $A_{sslabtopbend} = M_{\Sigma e}/((1.0/\gamma_s) \times f_{yslab} \times z_{slabtop}) = \mathbf{13}$ mm²/m

Minimum area of steel required $A_{sslabmin} = 0.0013 \times h_{slab} = \mathbf{325}$ mm²/m

Area of steel required $A_{sslabtopreq} = \max(A_{sslabtopbend}, A_{sslabmin}) = \mathbf{325}$ mm²/m

PASS - $A_{sslabtopreq} <= A_{sslabtop}$ - Area of reinforcement provided in top to span local depressions is adequate

Reinforcement required in bottom

K factor $K_{slabbtm} = M_{\Sigma c}/(f_{cu} \times d_{bslabav}^2) = \mathbf{0.001}$

Lever arm $z_{slabbtm} = d_{bslabav} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{slabbtm}/0.9)}) = \mathbf{180.5}$ mm

Area of steel required for bending $A_{sslabbtmbend} = M_{\Sigma c}/((1.0/\gamma_s) \times f_{yslab} \times z_{slabbtm}) = \mathbf{8}$ mm²/m

Area of steel required $A_{sslabbtmreq} = \max(A_{sslabbtmbend}, A_{sslabmin}) = \mathbf{325}$ mm²/m

PASS - $A_{sslabbtmreq} <= A_{sslabbtm}$ - Area of reinforcement provided in bottom to span local depressions is adequate

Shear check

Applied shear stress $v = V_{\Sigma}/d_{tslabmin} = \mathbf{0.028}$ N/mm²

Tension steel ratio $\rho = 100 \times A_{sslabtop}/d_{tslabmin} = \mathbf{0.425}$

From BS8110-1:1997 - Table 3.8

Design concrete shear strength $v_c = \mathbf{0.576}$ N/mm²

PASS - $v <= v_c$ - Shear capacity of the slab is adequate

Internal slab deflection check

Basic allowable span to depth ratio $Ratio_{basic} = \mathbf{26.0}$

Moment factor $M_{factor} = M_{\Sigma c}/d_{bslabav}^2 = \mathbf{0.016}$ N/mm²

Steel service stress $f_s = 2/3 \times f_{yslab} \times A_{sslabbtmbend}/A_{sslabbtm} = \mathbf{3.201}$ N/mm²

Modification factor $MF_{slab} = \min(2.0, 0.55 + [(477\text{N/mm}^2 - f_s)/(120 \times (0.9\text{N/mm}^2 + M_{factor}))])$

$MF_{slab} = \mathbf{2.000}$

Modified allowable span to depth ratio $Ratio_{allow} = Ratio_{basic} \times MF_{slab} = \mathbf{52.000}$

Actual span to depth ratio $Ratio_{actual} = l_{slab}/ d_{bslabav} = \mathbf{8.105}$

PASS - $Ratio_{actual} <= Ratio_{allow}$ - Slab span to depth ratio is adequate

Edge beam design checks

Basic loading

Hardcore $w_{hcorethick} = \gamma_{hcore} \times h_{hcorethick} = \mathbf{3.0}$ kN/m^2

Edge beam self weight $w_{edge} = 24$ kN/m$^3 \times h_{edge} \times b_{edge} = \mathbf{4.9}$ kN/m

Edge load number 1

Load type **Longitudinal line load**

Dead load $w_{Dedge1} = \mathbf{38.1}$ kN/m

Live load $w_{Ledge1} = \mathbf{8.3}$ kN/m

Ultimate load $w_{ultedge1} = 1.4 \times w_{Dedge1} + 1.6 \times w_{Ledge1} = \mathbf{66.6}$ kN/m

Longitudinal line load width $b_{edge1} = \mathbf{350}$ mm

Centroid of load from outside face of raft $x_{edge1} = \mathbf{200}$ mm

Edge beam bearing pressure check

Effective bearing width of edge beam $b_{bearing} = b_{edge} = \mathbf{450}$ mm

Total uniform load at formation level $w_{udledge} = w_{Dudl}+w_{Ludl}+w_{edge}/b_{bearing}+w_{hcorethick} = \mathbf{17.1}$ kN/m^2

Centroid of longitudinal and equivalent line loads from outside face of raft

Load x distance for edge load 1 $Moment_1 = w_{ultedge1} \times x_{edge1} = \mathbf{13.3}$ kN

Sum of ultimate longitud'l and equivalent line loads $\Sigma UDL = \mathbf{66.6}$ kN/m

Sum of load x distances $\Sigma Moment = \mathbf{13.3}$ kN

Centroid of loads $x_{bar} = \Sigma Moment/\Sigma UDL = \mathbf{200}$ mm

Initially assume no moment transferred into slab due to load/reaction eccentricity

Sum of unfactored longitud'l and eff'tive line loads $\Sigma UDLsls = \mathbf{46.4}$ kN/m

Allowable bearing width $b_{allow} = 2 \times x_{bar} + 2 \times h_{hcoreslab} \times \tan(30) = \mathbf{573}$ mm

Bearing pressure due to line/point loads $q_{linepoint} = \Sigma UDLsls/ b_{allow} = \mathbf{80.9}$ kN/m^2

Total applied bearing pressure $q_{edge} = q_{linepoint} + w_{udledge} = \mathbf{98.0}$ kN/m^2

$q_{edge} > q_{allow}$ - The slab is required to resist a moment due to eccentricity

Now assume moment due to load/reaction eccentricity is resisted by slab

Bearing width required $b_{req} = \Sigma UDLsls/(q_{allow} - w_{udledge}) = \mathbf{1410}$ mm

Effective bearing width at u/s of slab $b_{reqeff} = b_{req} - 2 \times h_{hcoreslab} \times \tan(30) = \mathbf{1237}$ mm

Load/reaction eccentricity $e = b_{reqeff}/2 - x_{bar} = \mathbf{419}$ mm

Ultimate moment to be resisted by slab $M_{ecc} = \Sigma UDL \times e = \mathbf{27.9}$ kNm/m

From slab bending check

Moment due to depression under slab (hogging) $M_{\Sigma e} = \mathbf{1.0}$ kNm/m

Total moment to be resisted by slab top steel $M_{slabtop} = M_{ecc} + M_{\Sigma e} = \mathbf{28.9}$ kNm/m

K factor $K_{slab} = M_{slabtop}/(f_{cu} \times d_{tslabmin}^2) = \mathbf{0.034}$

Lever arm $z_{slab} = d_{tslabmin} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{slab}/0.9)}) = \mathbf{176}$ mm

Area of steel required $A_{sslabreq} = M_{slabtop}/((1.0/\gamma_s) \times f_y \times z_{slab}) = \mathbf{378}$ mm^2/m

PASS - $A_{sslabreq} <= A_{sslabtop}$ - Area of reinforcement provided to transfer moment into slab is adequate

The allowable bearing pressure under the edge beam will not be exceeded

Library item - B pressure with moment transfer

Edge beam bending check

Divider for moments due to udl's $\beta_{udl} = \mathbf{10.0}$

Applied bending moments

Span of edge beam	$l_{edge} = \phi_{depthick} + d_{edgetop}$ = **1722** mm
Ultimate self weight udl	$w_{edgeult} = 1.4 \times w_{edge}$ = **6.8** kN/m
Ultimate slab udl (approx)	$w_{edgeslab} = \max(0\text{ kN/m}, 1.4 \times w_{slab} \times ((\phi_{depthick}/2 \times 3/4) - b_{edge}))$ = **0.5** kN/m
Self weight and slab bending moment	$M_{edgesw} = (w_{edgeult} + w_{edgeslab}) \times l_{edge}^2/\beta_{udl}$ = **2.2** kNm
Self weight shear force	$V_{edgesw} = (w_{edgeult} + w_{edgeslab}) \times l_{edge}/2$ = **6.3** kN

Moments due to applied uniformly distributed loads

Ultimate udl (approx)	$w_{edgeudl} = w_{udlult} \times \phi_{depthick}/2 \times 3/4$ = **2.5** kN/m
Bending moment	$M_{edgeudl} = w_{edgeudl} \times l_{edge}^2/\beta_{udl}$ = **0.7** kNm
Shear force	$V_{edgeudl} = w_{edgeudl} \times l_{edge}/2$ = **2.1** kN

Moment and shear due to load number 1

Bending moment	$M_{edge1} = W_{ultedge1} \times l_{edge}^2/\beta_{udl}$ = **19.8** kNm
Shear force	$V_{edge1} = W_{ultedge1} \times l_{edge}/2$ = **57.4** kN

Resultant moments and shears

Total moment (hogging and sagging)	$M_{\Sigma edge}$ = **22.7** kNm
Maximum shear force	$V_{\Sigma edge}$ = **65.8** kN

Reinforcement required in top

Width of section in compression zone	$b_{edgetop} = b_{edge}$ = **450** mm
Average web width	$b_w = b_{edge}$ = **450** mm
K factor	$K_{edgetop} = M_{\Sigma edge}/(f_{cu} \times b_{edgetop} \times d_{edgetop}^2)$ = **0.015**
Lever arm	$z_{edgetop} = d_{edgetop} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{edgetop}/0.9)})$ = **353** mm
Area of steel required for bending	$A_{sedgetopbend} = M_{\Sigma edge}/((1.0/\gamma_s) \times f_y \times z_{edgetop})$ = **147** mm²
Minimum area of steel required	$A_{sedgetopmin} = 0.0013 \times 1.0 \times b_w \times h_{edge}$ = **263** mm²
Area of steel required	$A_{sedgetopreq} = \max(A_{sedgetopbend}, A_{sedgetopmin})$ = **263** mm²

PASS - $A_{sedgetopreq} <= A_{sedgetop}$ - Area of reinforcement provided in top of edge beams is adequate

Reinforcement required in bottom

Width of section in compression zone	$b_{edgebtm} = b_{edge} + 0.1 \times l_{edge}$ = **622** mm
K factor	$K_{edgebtm} = M_{\Sigma edge}/(f_{cu} \times b_{edgebtm} \times d_{edgebtm}^2)$ = **0.010**
Lever arm	$z_{edgebtm} = d_{edgebtm} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{edgebtm}/0.9)})$ = **363** mm
Area of steel required for bending	$A_{sedgebtmbend} = M_{\Sigma edge}/((1.0/\gamma_s) \times f_y \times z_{edgebtm})$ = **144** mm²
Minimum area of steel required	$A_{sedgebtmmin} = 0.0013 \times 1.0 \times b_w \times h_{edge}$ = **263** mm²
Area of steel required	$A_{sedgebtmreq} = \max(A_{sedgebtmbend}, A_{sedgebtmmin})$ = **263** mm²

PASS - $A_{sedgebtmreq} <= A_{sedgebtm}$ - Area of reinforcement provided in bottom of edge beams is adequate

Edge beam shear check

Applied shear stress	$v_{edge} = V_{\Sigma edge}/(b_w \times d_{edgetop})$ = **0.393** N/mm²
Tension steel ratio	$\rho_{edge} = 100 \times A_{sedgetop}/(b_w \times d_{edgetop})$ = **0.360**
From BS8110-1:1997 - Table 3.8	
Design concrete shear strength	v_{cedge} = **0.458** N/mm²

$v_{edge} <= v_{cedge}$ + 0.4N/mm² - Therefore minimum links required

Link area to spacing ratio required	$A_{sv\_upon\_s_{vreqedge}} = 0.4\text{N/mm}^2 \times b_w/((1.0/\gamma_s) \times f_{ys})$ = **0.414** mm
Link area to spacing ratio provided	$A_{sv\_upon\_s_{vprovedge}} = N_{edgelink} \times \pi \times \phi_{edgelink}^2/(4 \times s_{vedge})$ = **0.785** mm

PASS - $A_{sv\_upon\_s_{vreqedge}} <= A_{sv\_upon\_s_{vprovedge}}$ - Shear reinforcement provided in edge beams is adequate

Corner design checks

Basic loading

Corner bearing pressure check

Total uniform load at formation level $w_{udlcorner} = w_{Dudl}+w_{Ludl}+w_{edge}/b_{bearing}+w_{hcorethick}$ = **17.1** kN/m^2

PASS - $w_{udlcorner}$ <= q_{allow} - Applied bearing pressure is less than allowable

Corner beam bending check

Cantilever span of edge beam $l_{corner} = \phi_{depthick}/\sqrt{(2)} + d_{edgetop}/2$ = **1141** mm

Moment and shear due to self weight

Ultimate self weight udl $w_{edgeult} = 1.4 \times w_{edge}$ = **6.8** kN/m

Average ultimate slab udl (approx) $w_{cornerslab} = \max(0\ kN/m, 1.4\times w_{slab}\times(\phi_{depthick}/(\sqrt{(2)}\times2)-b_{edge}))$ = **0.2** kN/m

Self weight and slab bending moment $M_{cornersw} = (w_{edgeult} + w_{cornerslab}) \times l_{corner}^2/2$ = **4.6** kNm

Self weight and slab shear force $V_{cornersw} = (w_{edgeult} + w_{cornerslab}) \times l_{corner}$ = **8.0** kN

Moment and shear due to udls

Maximum ultimate udl $w_{cornerudl} = ((1.4\times w_{Dudl})+(1.6\times w_{Ludl})) \times \phi_{depthick}/\sqrt{(2)}$ = **4.7** kN/m

Bending moment $M_{cornerudl} = w_{cornerudl} \times l_{corner}^2/6$ = **1.0** kNm

Shear force $V_{cornerudl} = w_{cornerudl} \times l_{corner}/2$ = **2.7** kN

Resultant moments and shears

Total design moment $M_{\Sigma corner} = M_{cornersw}+ M_{cornerudl}$ = **5.6** kNm

Total design shear force $V_{\Sigma corner} = V_{cornersw}+ V_{cornerudl}$ = **10.7** kN

Reinforcement required in top of edge beam

K factor $K_{corner} = M_{\Sigma corner}/(f_{cu} \times b_{edgetop} \times d_{edgetop}^2)$ = **0.004**

Lever arm $z_{corner} = d_{edgetop} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{corner}/0.9)})$ = **353** mm

Area of steel required for bending $A_{scornerbend} = M_{\Sigma corner}/((1.0/\gamma_s) \times f_y \times z_{corner})$ = **36** mm^2

Minimum area of steel required $A_{scornermin} = A_{sedgetopmin}$ = **263** mm^2

Area of steel required $A_{scorner} = \max(A_{scornerbend}, A_{scornermin})$ = **263** mm^2

PASS - $A_{scorner}$ <= $A_{sedgetop}$ - Area of reinforcement provided in top of edge beams at corners is adequate

Corner beam shear check

Average web width $b_w = b_{edge}$ = **450** mm

Applied shear stress $v_{corner} = V_{\Sigma corner}/(b_w \times d_{edgetop})$ = **0.064** N/mm^2

Tension steel ratio $\rho_{corner} = 100 \times A_{sedgetop}/(b_w \times d_{edgetop})$ = **0.360**

From BS8110-1:1997 - Table 3.8

Design concrete shear strength $v_{ccorner}$ = **0.450** N/mm^2

v_{corner} <= $v_{ccorner}$ + 0.4N/mm^2 - Therefore minimum links required

Link area to spacing ratio required $A_{sv}\_upon\_s_{vreqcorner} = 0.4N/mm^2 \times b_w/((1.0/\gamma_s) \times f_{ys})$ = **0.414** mm

Link area to spacing ratio provided $A_{sv}\_upon\_s_{vprovedge} = N_{edgelink}\times\pi\times\phi_{edgelink}^2/(4\times s_{vedge})$ = **0.785** mm

PASS - $A_{sv}\_upon\_s_{vreqcorner}$ <= $A_{sv}\_upon\_s_{vprovedge}$ - Shear reinforcement provided in edge beams at corners is adequate

Corner beam deflection check

Basic allowable span to depth ratio $Ratio_{basiccorner}$ = **7.0**

Moment factor $M_{factorcorner} = M_{\Sigma corner}/(b_{edgetop} \times d_{edgetop}^2)$ = **0.090** N/mm^2

Steel service stress $f_{scorner} = 2/3 \times f_y \times A_{scornerbend}/A_{sedgetop}$ = **20.117** N/mm^2

Modification factor $MF_{corner}=\min(2.0,0.55+[(477N/mm^2-f_{scorner})/(120\times(0.9N/mm^2+M_{factorcorner}))])$

MF_{corner} = **2.000**

Modified allowable span to depth ratio $Ratio_{allowcorner} = Ratio_{basiccorner} \times MF_{corner}$ = **14.000**

Actual span to depth ratio $Ratio_{actualcorner} = l_{corner}/ d_{edgetop} = \mathbf{3.066}$

PASS - $Ratio_{actualcorner} <= Ratio_{allowcorner}$ - Edge beam span to depth ratio is adequate

<u>Internal beam design checks</u>

Basic loading

Hardcore $w_{hcorethick} = \gamma_{hcore} \times h_{hcorethick} = \mathbf{3.0}$ kN/m^2

Internal beam self weight $w_{int} = 24$ kN/m$^3 \times h_{int} \times b_{int} = \mathbf{4.9}$ kN/m

Internal beam load number 1

Load type **Longitudinal line load**

Dead load $w_{Dint1} = \mathbf{37.6}$ kN/m

Live load $w_{Lint1} = \mathbf{13.2}$ kN/m

Ultimate load $w_{ultint1} = 1.4 \times w_{Dint1} + 1.6 \times w_{Lint1} = \mathbf{73.8}$ kN/m

Longitudinal line load width $b_{int1} = \mathbf{140}$ mm

Centroid of load from centreline of beam $x_{int1} = \mathbf{70}$ mm

Internal beam bearing pressure check

Total uniform load at formation level $w_{udlint} = w_{Dudl}+w_{Ludl}+w_{hcorethick}+24\text{kN/m}^3\times h_{int} = \mathbf{17.1}$ kN/m^2

Sum of factored longitud'l and eff'tive line loads $\Sigma UDL_{int} = \mathbf{73.8}$ kN/m

Sum of unfactored longitud'l and eff'tive line loads $\Sigma UDLsls_{int} = \mathbf{50.8}$ kN/m

Centroid of loads from centreline of internal beam

Load x distance for internal load 1 $Moment_{int1} = w_{ultint1} \times x_{int1} = \mathbf{5.2}$ kN

Sum of load x distances $\Sigma Moment_{int} = \mathbf{5.2}$ kN

Centroid of loads $x_{barint} = \Sigma Moment_{int}/\Sigma UDL_{int} = \mathbf{70.0}$ mm

Moment due to eccentricity to be resisted by slab $M_{eccint} = \Sigma UDL_{int} \times abs(x_{barint}) = \mathbf{5.2}$ kNm/m

Assume moment due to eccentricity is resisted equally by top steel of slab on one side and bottom steel of slab on other

From slab bending check

Moment due to depression under slab (hogging) $M_{\Sigma e} = \mathbf{1.0}$ kNm/m

Total moment to be resisted by slab top steel $M_{slabtopint} = M_{eccint}/2 + M_{\Sigma e} = \mathbf{3.6}$ kNm/m

K factor $K_{slabtopint} = M_{slabtopint}/(f_{cu} \times d_{tslabmin}^2) = \mathbf{0.004}$

Lever arm $z_{slabtopint} = d_{tslabmin} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{slabtopint}/0.9)}) = \mathbf{176}$ mm

Area of steel required $A_{sslabtopintreq} = M_{slabtopint}/((1.0/\gamma_s) \times f_{yslab} \times z_{slabtopint}) = \mathbf{47}$ mm^2/m

PASS - $A_{sslabtopintreq} <= A_{sslabtop}$ - Area of reinforcement in top of slab is adequate to transfer moment into slab

Mt to be resisted by slab btm stl due to load ecc'ty $M_{slabbtmint} = M_{eccint}/2 = \mathbf{2.6}$ kNm/m

K factor $K_{slabbtmint} = M_{slabbtmint}/(f_{cu} \times d_{bslabmin}^2) = \mathbf{0.003}$

Lever arm $z_{slabbtmint} = d_{bslabmin} \times \min(0.95, 0.5 + \sqrt{(0.25-K_{slabbtmint}/0.9)}) = \mathbf{176}$ mm

Area of steel required in bottom $A_{sslabbtmintreq} = M_{slabbtmint}/((1.0/\gamma_s) \times f_{yslab} \times z_{slabbtmint}) = \mathbf{34}$ mm^2/m

PASS - $A_{sslabbtmintreq} <= A_{sslabbtm}$ - Area of reinforcement in bottom of slab is adequate to transfer moment into slab

Bearing pressure

Initially check bearing pressure based on beam soffit/chamfer width only

Allowable bearing width $b_{bearint} = b_{int} = \mathbf{450}$ mm

Bearing pressure due to line/point loads $q_{linepointint} = \Sigma UDLsls_{int}/b_{bearint} = \mathbf{112.9}$ kN/m^2

Total applied bearing pressure $q_{int} = q_{linepointint} + w_{udlint} = \mathbf{130.0}$ kN/m^2

$q_{int} > q_{allow}$ - The slab is required to resist some of the beam loading

Now consider slab to resist some of beam load

Bearing width required $b_{reqint} = \Sigma UDLsls_{int}/(q_{allow} - w_{udlint}) = \mathbf{1544}$ mm

Effective bearing width at u/s of beam/slab $b_{reqeffint} = b_{reqint} - 2 \times h_{hcorethick} \times \tan(30) = \mathbf{1371}$ mm

Resultant ult upward press acting on u/s of slab	$q_{result} = (q_{allow}-w_{udlint}) \times b_{reqint}/b_{reqeffint} \times \Sigma UDL_{int}/\Sigma UDLsls_{int}$ = **53.8** kN/m^2
Slab cantilever moment	$M_{cantint} = q_{result} \times [(b_{reqint} - b_{int})/2]^2/2$ = **8.1** kNm/m
Total moment to be resisted by slab	$M_{slabint} = M_{cantint} + M_{eccint}/2$ = **10.6** kNm/m
K factor	$K_{slabint} = M_{slabint}/(f_{cu} \times d_{bslabmin}^2)$ = **0.012**
Lever arm	$z_{slabint} = d_{bslabmin} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{slabint}/0.9)})$ = **176** mm
Area of steel required	$A_{sslabint} = M_{slabint}/((1.0/\gamma_s) \times f_{yslab} \times z_{slabint})$ = **139** mm^2/m

PASS - $A_{sslabint}$ <= $A_{sslabbtm}$ - Area of reinforcement provided to distribute the beam load is adequate

The allowable bearing pressure under the internal beam will not be exceeded

Internal beam bending check

Divider for moments due to udl's — β_{udl} = **10.0**

Applied bending moments

Span of internal beam	$l_{int} = \phi_{depthick} + d_{inttop}$ = **1722** mm
Ultimate self weight udl	$w_{intult} = 1.4 \times w_{int}$ = **6.8** kN/m
Ultimate slab udl (approx)	$w_{intslab} = \max(0 \text{ kN/m}, 1.4 \times w_{slab} \times ((\phi_{depthick} \times 3/4) - b_{int}))$ = **4.7** kN/m
Self weight and slab bending moment	$M_{intsw} = (w_{intult} + w_{intslab}) \times l_{int}^2/\beta_{udl}$ = **3.4** kNm
Self weight shear force	$V_{intsw} = (w_{intult} + w_{intslab}) \times l_{int}/2$ = **9.9** kN

Moments due to applied uniformly distributed loads

Ultimate udl (approx)	$w_{intudl} = w_{udlult} \times \phi_{depthick} \times 3/4$ = **5.0** kN/m
Bending moment	$M_{intudl} = w_{intudl} \times l_{int}^2/\beta_{udl}$ = **1.5** kNm
Shear force	$V_{intudl} = w_{intudl} \times l_{int}/2$ = **4.3** kN

Moment and shear due to load number 1

Bending moment	$M_{int1} = w_{ultint1} \times l_{int}^2/\beta_{udl}$ = **21.9** kNm
Shear force	$V_{int1} = w_{ultint1} \times l_{int}/2$ = **63.5** kN

Resultant moments and shears

Total moment (hogging and sagging)	$M_{\Sigma int}$ = **26.8** kNm
Maximum shear force	$V_{\Sigma int}$ = **77.7** kN

Reinforcement required in top

Width of section in compression zone	$b_{inttop} = b_{int}$ = **450** mm
Average web width	$b_{wint} = b_{int}$ = **450** mm
K factor	$K_{inttop} = M_{\Sigma int}/(f_{cu} \times b_{inttop} \times d_{inttop}^2)$ = **0.017**
Lever arm	$z_{inttop} = d_{inttop} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{inttop}/0.9)})$ = **353** mm
Area of steel required for bending	$A_{sinttopbend} = M_{\Sigma int}/((1.0/\gamma_s) \times f_y \times z_{inttop})$ = **174** mm^2
Minimum area of steel	$A_{sinttopmin} = 0.0013 \times b_{wint} \times h_{int}$ = **263** mm^2
Area of steel required	$A_{sinttopreq} = \max(A_{sinttopbend}, A_{sinttopmin})$ = **263** mm^2

PASS - $A_{sinttopreq}$ <= $A_{sinttop}$ - Area of reinforcement provided in top of internal beams is adequate

Reinforcement required in bottom

Width of section in compression zone	$b_{intbtm} = b_{int} + 0.2 \times l_{int}$ = **794** mm
K factor	$K_{intbtm} = M_{\Sigma int}/(f_{cu} \times b_{intbtm} \times d_{intbtm}^2)$ = **0.009**
Lever arm	$z_{intbtm} = d_{intbtm} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{intbtm}/0.9)})$ = **363** mm
Area of steel required for bending	$A_{sintbtmbend} = M_{\Sigma int}/((1.0/\gamma_s) \times f_y \times z_{intbtm})$ = **170** mm^2
Minimum area of steel required	$A_{sintbtmmin} = 0.0013 \times 1.0 \times b_{wint} \times h_{int}$ = **263** mm^2
Area of steel required	$A_{sintbtmreq} = \max(A_{sintbtmbend}, A_{sintbtmmin})$ = **263** mm^2

PASS - $A_{sintbtmreq}$ <= $A_{sintbtm}$ - Area of reinforcement provided in bottom of internal beams is adequate

Internal beam shear check

Applied shear stress $v_{int} = V_{\Sigma int}/(b_{wint} \times d_{inttop})$ = **0.464** N/mm^2

Tension steel ratio $\rho_{int} = 100 \times A_{sinttop}/(b_{wint} \times d_{inttop})$ = **0.360**

From BS8110-1:1997 - Table 3.8

Design concrete shear strength v_{cint} = **0.450** N/mm^2

v_{int} <= v_{cint} + 0.4N/mm^2 - Therefore minimum links required

Link area to spacing ratio required $A_{sv}\_upon\_s_{vreqint} = 0.4N/mm^2 \times b_{wint}/((1.0/\gamma_s) \times f_{ys})$ = **0.414** mm

Link area to spacing ratio provided $A_{sv}\_upon\_s_{vprovint} = N_{intlink} \times \pi \times \phi_{intlink}^2/(4 \times s_{vint})$ = **0.785** mm

PASS - $A_{sv}\_upon\_s_{vreqint}$ <= $A_{sv}\_upon\_s_{vprovint}$ - Shear reinforcement provided in internal beams is adequate

Lintel L2

Max Clear Span = 2100mm

Dead Load

Wall H	=	1.0m x 4.4 kN/m^2	= 4.4 kN/m
Roof DL	=	3.0m x 1.3 kN/m^2	= 3.9 kN/m
		TOTAL DL	**= 8.3 kN/m**

Live Load

Roof LL	=	3.0m x 0.6 kN/m^2	= 1.8 kN/m
		TOTAL LL	**= 1.8 kN/m**

TOTAL UDL = 10.1 kN/m
Load W = 21.1 kN

USE IG/HD Cavity Wall Lintel
Use same for L14

Le = 2.1m

SWL = 30 kN > 21 kN
Satisfactory

Lintel L3

Max Clear Span = 800mm

Dead Load

Wall H	=	1.0m x 4.4 kN/m^2	= 4.4 kN/m
Roof DL	=	2.4m x 1.3 kN/m^2	= 3.1 kN/m
		TOTAL DL	**= 7.5 kN/m**

Live Load

Roof LL	=	2.4m x 0.6 kN/m^2	= 1.5 kN/m
		TOTAL LL	**= 1.5 kN/m**

TOTAL UDL = 9.0 kN/m
Load W = 7.2 kN

USE IG/S Cavity Wall Lintel

Le = 0.8m **SWL = 16 kN > 7.2 kN**

Satisfactory

Lintel L4/L5

Max Clear Span = 2100/650mm

Dead Load
Wall H = 1.05m x 4.4 kN/m^2 = 4.5 kN/m

TOTAL UDL = 4.5 kN/m
Load W = 9.5 / 2.9 kN

USE IG/S Cavity Wall Lintel

Le = 2.1m **SWL = 24 kN > 9.5 kN**
Satisfactory

Le = 0.65m **SWL = 16 kN > 3.0 kN**
Satisfactory

Lintel L6

Max Clear Span **= 2200mm**

Dead Load

Wall H	=	1.7m x 4.4 kN/m^2	= 7.5 kN/m
Roof DL	=	0.3m x 1.3 kN/m^2	= 0.4 kN/m
		TOTAL DL	**= 7.9 kN/m**

Live Load

Roof LL	=	0.3m x 0.6 kN/m^2	= 0.2 kN/m
		TOTAL LL	**= 0.2 kN/m**

TOTAL UDL = 8.1 kN/m
Load W = 18.0 kN

USE IG/S Cavity Wall Lintel

Le = 2.2m **SWL = 24 kN > 18.0 kN**
Satisfactory

Lintel L7

Max Clear Span = 800mm

Dead Load

Wall H	=	1.65m x 4.4 kN/m^2	= 7.3 kN/m
Floor DL	=	2.4m x 1.0 kN/m^2	= 2.4 kN/m
		TOTAL DL	**= 9.7 kN/m**

Live Load

Floor LL	=	2.4m x 1.5 kN/m^2	= 3.6 kN/m
		TOTAL LL	**= 3.6 kN/m**
		TOTAL UDL	**= 13.3 kN/m**
		Load W	**= 10.7 kN**

USE IG/S Cavity Wall Lintel

Le = 0.8m **SWL = 16 kN > 11.0 kN**

Satisfactory

Lintel L8

Max Clear Span = 800mm

Dead Load

Wall H	=	3.3m x 4.4 kN/m^2	= 14.5 kN/m
Floor DL	=	2.4m x 1.0 kN/m^2	= 2.4 kN/m
		TOTAL DL	**= 16.9 kN/m**

Live Load

Floor LL	=	2.4m x 1.5 kN/m^2	= 3.6 kN/m
		TOTAL LL	**= 3.6 kN/m**

TOTAL UDL = 20.5 kN/m
Load W = 16.5 kN

USE IG/HD Cavity Wall Lintel

Le = 0.8m **SWL = 30 kN > 17.0 kN**

Satisfactory

eng@huddsdesign.com
www.huddsdesign.com

HEAVY DUTY LOAD

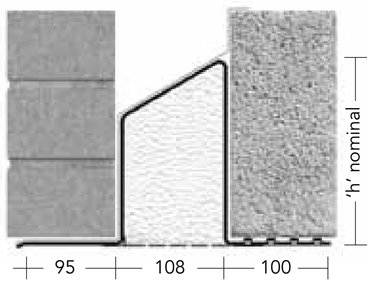
L1/HD

Blockwork built tight against inner face of the lintel. Place mortar bed on top of blockwork before floor units are laid to provide even distribution of load. Lintels may be propped to facilitate speed of construction. See Lintel Installation on page 12.

L1/HD 110 — For cavity widths 110-125mm

Manufactured length 150mm increments	600-1500	1650-2100	2250-3000	3150-3600	3750-4050
Height ‘h’	125	145	195	195	195
Thickness	2.9	2.9	2.9	3.2	3.2
Total UDL kN 3:1	30	30	35	32	30
Total UDL kN 19:1	20	22	30	28	26

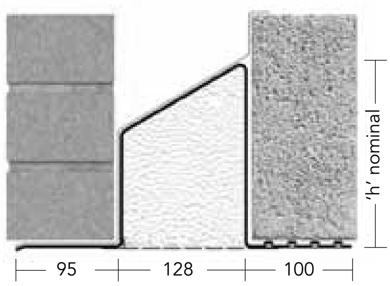
110-125mm cavity



L1/HD 130 — For cavity widths 130-145mm

Manufactured length 150mm increments	600-1500	1650-2100	2250-3000	3150-3600	3750-4050
Height ‘h’	115	155	190	190	212
Thickness	2.9	2.9	3.2	3.2	3.2
Total UDL kN 3:1	30	30	35	30	30
Total UDL kN 19:1	20	22	30	25	26

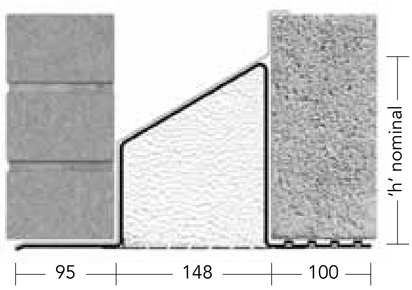
130-145mm cavity



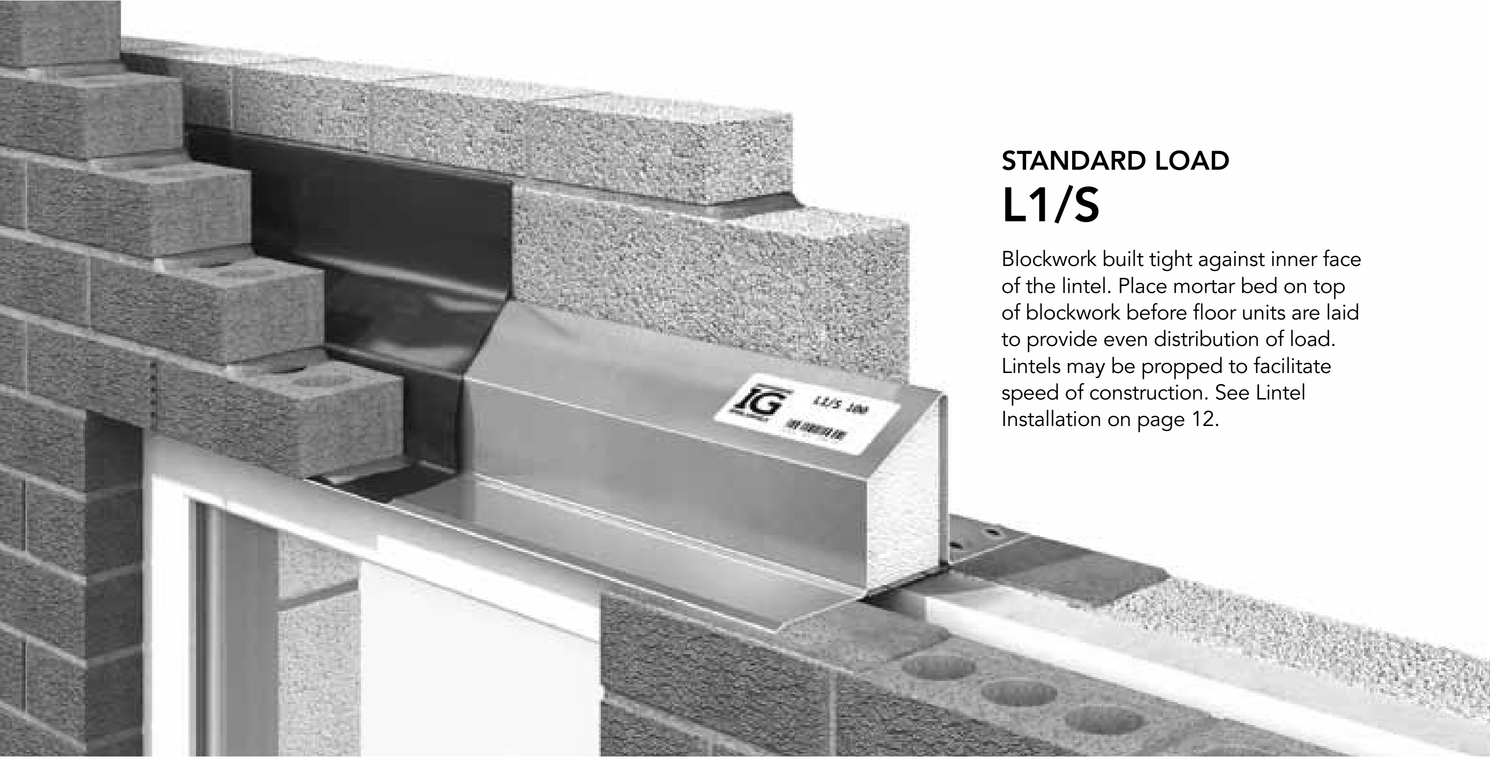
L1/HD 150 — For cavity widths 150-165mm

Manufactured length 150mm increments	600-1500	1650-2100	2250-3000	3150-3600	3750-4050
Height ‘h’	125	160	180	180	200
Thickness	2.9	2.9	3.2	3.2	3.2
Total UDL kN 3:1	30	30	35	30	30
Total UDL kN 19:1	20	22	30	25	26

150-165mm cavity



HEAVIER LOADINGS
Tables for heavier loads overleaf.



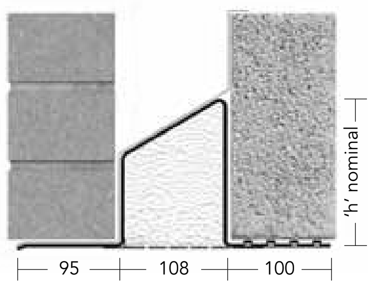
STANDARD LOAD

L1/S

Blockwork built tight against inner face of the lintel. Place mortar bed on top of blockwork before floor units are laid to provide even distribution of load. Lintels may be propped to facilitate speed of construction. See Lintel Installation on page 12.

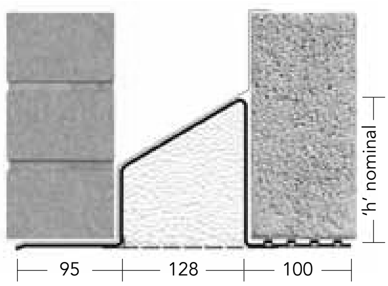
L1/S 110	For cavity widths **110-125mm**					
Manufactured length 150mm increments	600-1500	1650-1800	1950-2100	2250-3000	3150-4050	4200-4800
Height 'h'	95	121	145	195	195	195
Thickness	2.0	2.0	2.6	2.9	3.2	3.4
Total UDL kN 3:1	16	22	24	27	26	25
Total UDL kN 19:1	13	18	18	22	19	20

110-125mm cavity



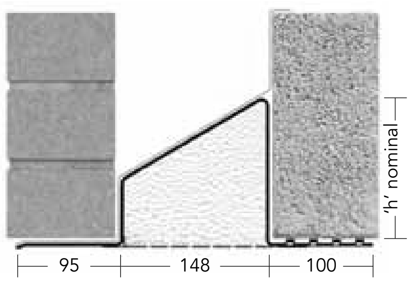
L1/S 130	For cavity widths **130-145mm**					
Manufactured length 150mm increments	600-1500	1650-1800	1950-2100	2250-3000	3150-4050	4200-4800
Height 'h'	103	128	190	190	190	190
Thickness	2.0	2.0	2.5	2.9	3.2	3.4
Total UDL kN 3:1	16	22	24	27	26	25
Total UDL kN 19:1	13	18	18	22	19	20

130-145mm cavity



L1/S 150	For cavity widths **150-165mm**					
Manufactured length 150mm increments	600-1500	1650-1800	1950-2100	2250-3000	3150-4050	4200-4800
Height 'h'	100	125	180	180	180	180
Thickness	2.0	2.0	2.6	2.9	3.2	3.4
Total UDL kN 3:1	16	22	24	27	26	25
Total UDL kN 19:1	13	18	18	22	19	20

150-165mm cavity



HEAVIER LOADINGS
Tables for heavier loads overleaf.

Lintel L12

Max Clear Span = 1200mm / 2000mm

Dead Load

Wall H	=	6.0m	
Roof DL	=	2.0m x 1.3 kN/m^2	= 2.6 kN/m
Floor DL	=	4.8m x 1.0 kN/m^2	= 4.8 kN/m
		TOTAL DL	**= 7.4 kN/m**

Live Load

Roof LL	=	2.0m x 0.6 kN/m^2	= 1.2 kN/m
Floor LL	=	4.8m x 1.5 kN/m^2	= 7.2 kN/m
		TOTAL LL	**= 8.4 kN/m**

USE Boot Lintel
203x203x46 UC with 10mm plate fully welded to bottom
Min 10mm Fillet Welds
(See Calc L12)
Min 200mm End Bearing

Padstones

Reaction Maximum = 62 kN

USE 400 x 100 x 2150mm DEEP CONCRETE PADSTONES

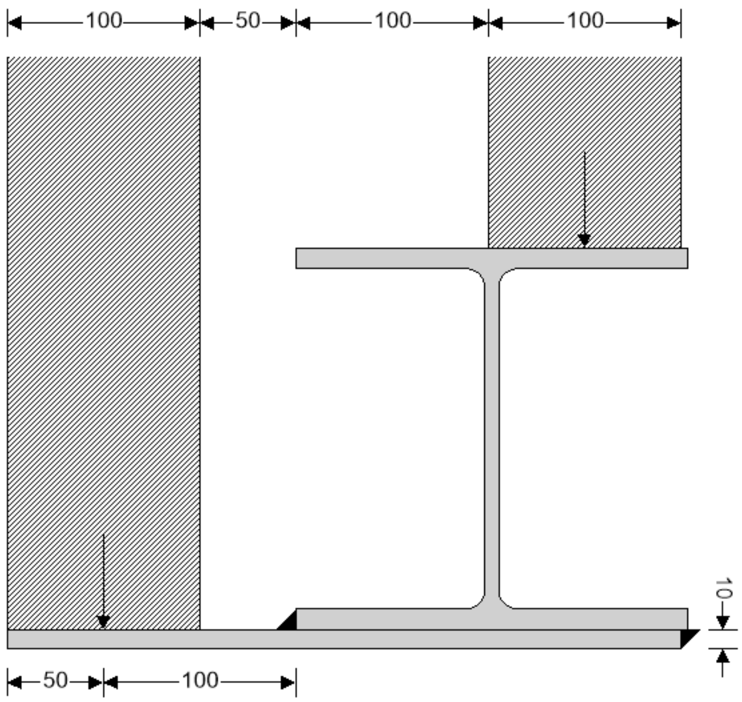
Stress Check = 62 x 1000 / 400 x 100 = 1.6 N/mm^2 < 2.4 N/mm^2 (New 7N Blocks)
Satisfactory

eng@huddsdesign.com
www.huddsdesign.com

STEEL MASONRY SUPPORT

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

Tedds calculation version 1.0.05



Steel member details

Torsion beam	UC 203x203x46	Masonry support angle	plate
Steel grade of support angle	User	Design strength support angle	p_{ysb} = **275** N/mm^2
Modulus of elasticity	E = 205000 N/mm^2	Constant	ε = **1.000**
Length of plate beyond beam	l_h = **150** mm	Total length of plate	l_{plate} = **350** mm
Thickness of plate	t_{sb} = **10** mm	Width of main beam	B_{mb} = **204** mm
Area of plate	A_{sbu} = **3500.0** mm^2	Dist weld position to CoG	c_{yysb} = **-25** mm

Supported materials detail

Density mas. main beam	$\rho_{m,mb}$ = **20.0** kN/m^3	Width masonry main beam	b_{mmb} = **100** mm
Height masonry main beam	h_{mmb} = **6000** mm		
Ecc. of main beam material	e_{mb} = **100** mm		
Add dead force main beam	P_{Gaddmb} = **7.4** kN/m	Add live force main beam	P_{Qaddmb} = **8.4** kN/m
Density mas. support beam	$\rho_{m,sb}$ = **24.0** kN/m^3	Width masonry support beam	b_{msb} = **100** mm
Height masonry support beam	h_{msb} = **6000** mm		
Add dead force support beam	P_{Gaddsb} = **0.0** kN/m	Add live force support beam	P_{Qaddsb} = **0.0** kN/m

Geometry

Cavity width	c = **150** mm	Supported width of masonry	d_m = **100** mm

Biaxial stress effects in the plate (SCI-P-110)

Max overall bending moment	M_x = **30.9** kNm	Dist to NA combined section	$y_{e,all}$ = **40** mm
Second moment of area	$I_{xx,all}$ = **7105** cm^4	Elastic section modulus	$Z_{xx,all}$ = **986.73** cm^3
Section modulus of plate	$Z_{xx,plate}$ = **16.67** cm^3/m	Eccentricity on support beam	e_1 = **100** mm
Force on support plate	P_1 = **20.2** kN/m	Bending at heel	$M_{x,plate}$ = **2.0** kNm/m
Moment capacity of plate	M_c = **5.5** kNm/m		

PASS - Design strength exceeds stress at heel

Long stress overall bending σ_1 = **31.3** N/mm^2 Von Mises curve constant c_{fp} = **547.3** N/mm^2

Trans bending stress ratio limit α_{ts} = **0.992** Trans bending stress ratio α_{ls} = **0.367**

PASS - Transverse bending stress ratio less than allowable limit

Deflection at toe

Unfact force on plate P_{1SLS} = **14.4** kN/m Distance from weld to load a_m = **100** mm

Load resultant to edge of plate b_m = **50** mm Weld to load pos as ratio a_l = **0.667**

Effect second mnt of inertia I_{eff_def} = **83333** mm^4/m Deflection at toe δ = **0.49** mm

Deflection limit δ_{lim} = **1.67** mm

PASS - Deflection is within specified criteria

Weld details - assume a full length weld and that the plate acts as a propped cantilever with the prop at the weld position and the fixed end at the centre of the torsion beam

Leg length of weld S_{weld} = **10** mm Throat size of weld a_{weld} = **7.1** mm

Shear force at weld position R_A = **49.9** kN/m Max possible force in plate R_p = **972.4** kN

Long shear beam/plate R_l = **972.4** kN/m Horizontal shear beam/plate R_h = **201.6** kN/m

Resultant weld force R_{weld} = **0.994** kN/mm Strength of weld (Table 37) p_{weld} = **220.0** N/mm^2

Capacity of full length weld $p_{c,weld}$ = **1.556** kN/mm

$1/\sqrt{(2)} \times S_{weld}$

Torsional loading ULS

Loading support beam w_{1ULS} = **20.16** kN/m Loading of main beam w_{2ULS} = **40.60** kN/m

Self weight of support beam w_{3ULS} = **0.38** kN/m

Torsional loading SLS

Loading support beam w_{1SLS} = **14.40** kN/m Loading of main beam w_{2SLS} = **27.80** kN/m

Self weight of support beam w_{3SLS} = **0.27** kN/m

Eccentricities

Distance of shear centre e_{0mb} = **0** mm Ecc of support beam masonry e_{1mb} = **202** mm

Ecc of main beam masonry e_{2mb} = **-48** mm Ecc of support beam e_{3mb} = **77** mm

Torsional effects

Applied torque T_{qULS} = **2.14** kNm/m Torsional moment (ULS) T_q = **4.28** kNm

Applied torque (SLS) T_{qSLS} = **1.59** kNm/m Torsional moment (SLS) T_{qu} = **3.17** kNm

STEEL BEAM TORSION DESIGN

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

Tedds calculation version 2.0.03

Section details

Section type UC 203x203x46 Steel grade S275

Design stength $p_{yw} = p_y$ = **275** N/mm^2 Constant ε = **1.000**

Geometry - Beam unrestrained against lateral-torsional buckling between supports.

Effective span L = **2000** mm

Length of segment LTB L_{LT} = **2000** mm Effective length for LTB L_{E_LT} = **2806** mm

Loading - Torsional loading comprises only full-length uniformly distributed load(s)

Internal forces & moments on member under factored loading for uls design

Applied shear force F_{vy} = **61.8** kN Maximum bending moment $M_{LT} = M_x$ = **30.90** kNm

Applied torque T_q = **4.28** kNm Minor axis bending moment M_y = 0 kNm

Compression force F_c = 0 kN

Equivalent uniform moment factors

EUM factor (Cl.4.3.6.6 & T18) m_{LT} = **1.000**

Torsional deflection parameters

Beam is torsion fixed and warping free at each end. (as defined in SCI-P-057 section 2.1.6) - Appendix B case 4

Dist for first deriv of twist z_1 = 0 mm

Dist for second deriv of twist z_2 = L / 2 = **1000** mm

First deriv of angle of twist ϕ'_1 = **1.95×10^{-2}** rads/m

Third deriv of angle of twist ϕ'''_1 = **-6.08×10^{-2}** rads/m^3

Angle of twist ϕ_2 = **0.012** rads

Second deriv of angle of twist ϕ''_2 = **-2.89×10^{-2}** rads/m^2

Design parameters

Total angle of twist ϕ = **0.012** rads

First derivative of ϕ ϕ' = **1.95×10^{-2}** rads/m

Second derivative of ϕ ϕ'' = **2.89×10^{-2}** rads/m^2

Third derivative of ϕ ϕ''' = **6.08×10^{-2}** rads/m^3

Section classification

b / T = **9.3**

d / t = **21.6**

r_{1s} = **0.000**

r_{2s} = **0.000**

Section classification is compact

Shear capacity (parallel to y-axis)

Design shear force F_{vy} = **61.8** kN

Design shear resist (cl. 4.2.3) P_{vy} = **241.4** kN

Pass - Shear

Moment capacity (x-axis)

Design bending moment M_x = **30.9** kNm

Mnt cap low shear (cl. 4.2.5.1) M_{cx} = **138.0** kNm

Pass - Moment capacity exceeds design bending moment

Lateral torsional buckling

Effective length for LTB L_{E_LT} = **2806** mm

Slenderness ratio λ = **55**

Buckling parameter u = **0.847**

Flange ratio η = 0.5

Torsional index x = **17.3**

Slenderness factor v = **0.90**

Ratio - cl 4.3.6.9 β_w = **1.000**

Equiv slenderness – cl 4.3.6.7 λ_{LT} = **42**

Limit slendernes – Ann B2.2 λ_{L0} = **34**

Euler stress p_E = **1146** N/mm^2

Perry factor η_{LT} = **0.054**

ϕ_{LT} = **741482313.522**

Bending strength p_b = **257** N/mm^2

Buckling resistance moment M_b = **129.0** kNm

Max mnt gov buckling resist M_{LT} = **30.9** kNm

Equiv uniform mnt factor LTB m_{LT} = **1.00**

M_b / m_{LT} = **129.0** kNm

Pass - lat. tors. buckling

Buckling under combined bending & torsion -SCI-P-057 section 2.3

For simplicity, a conservative check is applied using the maximum stresses due to each of the separate load effects, even though these do not necessarily all occur at the same section along the member.

Span factor L / a = **1.58**

Angle of twist ϕ = **0.012** rads

Second derivative of ϕ ϕ'' = **28.9×10^{-3}** rads/m^2

Induced minor axis moment M_{yt} = **0.37** kNm

Normal stress flange M_{yt} σ_{byt} = **2** N/mm^2

Normal stress flange warping σ_w = **58** N/mm^2

Interaction index i_b = **0.49**

Pass - Combined bending and torsion check satisfied

Local capacity under combined bending & torsion

For simplicity, a conservative check is applied using the maximum stresses due to each of the separate load effects, even though these do not necessarily all occur at the same section along the member.

Max. direct stress due to M_x σ_{bx} = M_x / Z_x = **68** N/mm^2

Combined stress - eqn 2.22 $\sigma_{bx} + \sigma_{byt} + \sigma_w$ = **129** N/mm^2

Design strength p_y = **275** N/mm^2

Pass - Local capacity

Combined shear stresses - SCI-P-057 section 2.3

For simplicity, a conservative check is applied using the maximum shear stresses due to each of the separate load effects, even though these do not necessarily all occur at the same section along the member.

Shear stress bending web τ_{bw} = **47** N/mm^2

Shear stress bending flange τ_{bf} = **13** N/mm^2

Shear stresses torsion web τ_{tw} = **11** N/mm^2

Shear stresses torsion flange τ_{tf} = **17** N/mm^2

Shear stresses warping flange τ_{wf} = **6** N/mm^2

Shear stress tors & warp web τ_{vtw} = **12** N/mm^2

Shear str tors & warp flange τ_{vtf} = **26** N/mm^2

Combined shear stresses due to bending, torsion & warping:

Comb shear stresses in web τ_w = **59** N/mm^2

Comb shear stresses in flange τ_f = **39** N/mm^2

Shear strength p_v = **165** N/mm^2

Pass - Combined shear stresses

Twist check

Total applied torque T_{qu} = **3.17** kNm

Max twist under sls loading ϕ_{sls} = **0.51** deg

Twist limit ϕ_{lim} = **2.00** deg

Pass - Twist

Deflection

Maximum y-axis deflection δ_{y_max} = **0.9** mm

Deflection limit - cl. 2.5.2 δ_{lim} = **5.6** mm

Pass - Deflection within specified limit

Lintel L17

Max Clear Span = 7300

Dead Load

Wall H	=	0.5m	
Roof DL	=	1.0m x 1.3 kN/m^2	= 1.3 kN/m
		TOTAL DL	**= 1.3 kN/m**

Live Load

Roof LL	=	1.0m x 0.6 kN/m^2	= 0.6 kN/m
		TOTAL LL	**= 0.6 kN/m**

USE Boot Lintel
203x203x86 UC with 15mm plate fully welded to bottom
Min 10mm Fillet Welds
(See Calc L17)

Padstones

Reaction Maximum = 30 kN

USE 300 x 100 x 2150mm DEEP CONCRETE PADSTONES

Stress Check = 30 x 1000 / 300 x 100 = 1.1 N/mm^2 < 2.4 N/mm^2 (New 7N Blocks)
Satisfactory

eng@huddsdesign.com
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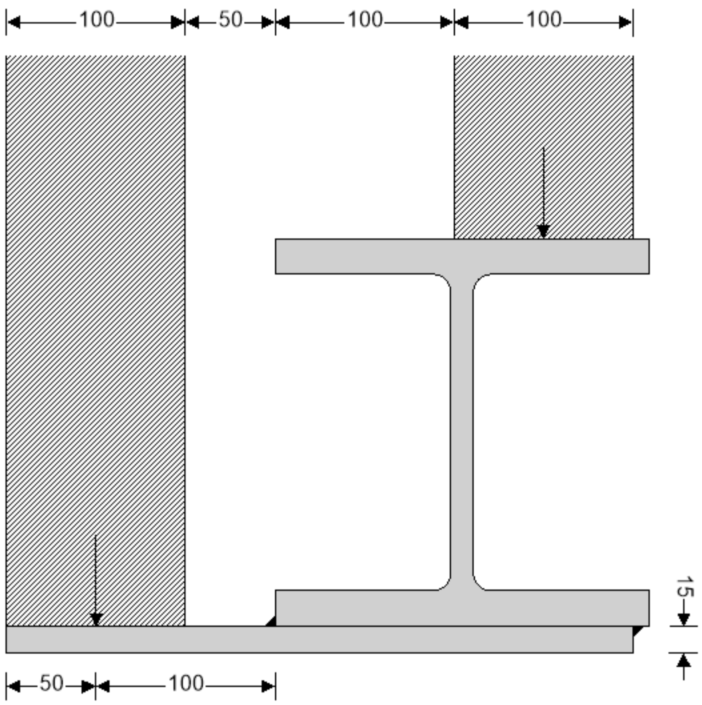
STEEL MASONRY SUPPORT

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

Tedds calculation version 1.0.05



Steel member details

Torsion beam UC 203x203x86

Masonry support angle plate

Steel grade of support angle User

Design strength support angle p_{ysb} = **275** N/mm^2

Modulus of elasticity E = 205000 N/mm^2

Constant ε = **1.000**

Length of plate beyond beam l_h = **150** mm

Total length of plate l_{plate} = **350** mm

Thickness of plate t_{sb} = **15** mm

Width of main beam B_{mb} = **209** mm

Area of plate A_{sbu} = **5250.0** mm^2

Dist weld position to CoG c_{yysb} = **-25** mm

Supported materials detail

Density mas. main beam $\rho_{m,mb}$ = **20.0** kN/m^3

Width masonry main beam b_{mmb} = **100** mm

Height masonry main beam h_{mmb} = **500** mm

Ecc. of main beam material e_{mb} = **100** mm

Add dead force main beam P_{Gaddmb} = **1.3** kN/m

Add live force main beam P_{Qaddmb} = **0.6** kN/m

Density mas. support beam $\rho_{m,sb}$ = **24.0** kN/m^3

Width masonry support beam b_{msb} = **100** mm

Height masonry support beam h_{msb} = **500** mm

Add dead force support beam P_{Gaddsb} = **0.0** kN/m

Add live force support beam P_{Qaddsb} = **0.0** kN/m

Geometry

Cavity width c = **150** mm

Supported width of masonry d_m = **100** mm

Biaxial stress effects in the plate (SCI-P-110)

Max overall bending moment M_x = **50.9** kNm

Dist to NA combined section $y_{e,all}$ = **38** mm

Second moment of area $I_{xx,all}$ = **14487** cm^4

Elastic section modulus $Z_{xx,all}$ = **1649.70** cm^3

Section modulus of plate $Z_{xx,plate}$ = **37.50** cm^3/m

Eccentricity on support beam e_1 = **100** mm

Force on support plate P_1 = **1.7** kN/m

Bending at heel $M_{x,plate}$ = **0.2** kNm/m

Moment capacity of plate M_c = **12.4** kNm/m

PASS - Design strength exceeds stress at heel

Long stress overall bending	σ_1 = **30.9** N/mm^2	Von Mises curve constant	c_{fp} = **547.4** N/mm^2
Trans bending stress ratio limit	α_{ts} = **0.992**	Trans bending stress ratio	α_{ls} = **0.014**

PASS - Transverse bending stress ratio less than allowable limit

Deflection at toe

Unfact force on plate	P_{1SLS} = **1.2** kN/m	Distance from weld to load	a_m = **100** mm
Load resultant to edge of plate	b_m = **50** mm	Weld to load pos as ratio	a_l = **0.667**
Effect second mnt of inertia	I_{eff_def} = **281250** mm^4/m	Deflection at toe	δ = **0.01** mm
Deflection limit	δ_{lim} = **1.67** mm		

PASS - Deflection is within specified criteria

Weld details - assume a full length weld and that the plate acts as a propped cantilever with the prop at the weld position and the fixed end at the centre of the torsion beam

Leg length of weld	s_{weld} = **6** mm	Throat size of weld	a_{weld} = **4.2** mm
Shear force at weld position	R_A = **4.1** kN/m	Max possible force in plate	R_p = **1481.3** kN
Long shear beam/plate	R_l = **405.8** kN/m	Horizontal shear beam/plate	R_h = **16.0** kN/m
Resultant weld force	R_{weld} = **0.406** kN/mm	Strength of weld (Table 37)	p_{weld} = **220.0** N/mm^2
Capacity of full length weld	$p_{c,weld}$ = **0.933** kN/mm		

$1/\sqrt{(2)} \times s_{weld}$

Torsional loading ULS

Loading support beam	w_{1ULS} = **1.68** kN/m	Loading of main beam	w_{2ULS} = **4.18** kN/m
Self weight of support beam	w_{3ULS} = **0.58** kN/m		

Torsional loading SLS

Loading support beam	w_{1SLS} = **1.20** kN/m	Loading of main beam	w_{2SLS} = **2.90** kN/m
Self weight of support beam	w_{3SLS} = **0.41** kN/m		

Eccentricities

Distance of shear centre	e_{0mb} = **0** mm	Ecc of support beam masonry	e_{1mb} = **205** mm
Ecc of main beam masonry	e_{2mb} = **-45** mm	Ecc of support beam	e_{3mb} = **80** mm

Torsional effects

Applied torque	T_{qULS} = **0.20** kNm/m	Torsional moment (ULS)	T_q = **1.46** kNm
Applied torque (SLS)	T_{qSLS} = **0.15** kNm/m	Torsional moment (SLS)	T_{qu} = **1.07** kNm

STEEL BEAM TORSION DESIGN

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

Tedds calculation version 2.0.03

Section details

Section type	UC 203x203x86	Steel grade	S275
Design stength	p_{yw} = p_y = **265** N/mm^2	Constant	ε = **1.019**

Geometry - Beam unrestrained against lateral-torsional buckling between supports.

Effective span	L = **7300** mm		
Length of segment LTB	L_{LT} = **7300** mm	Effective length for LTB	L_{E_LT} = **9204** mm

Loading - Torsional loading comprises only full-length uniformly distributed load(s)

Internal forces & moments on member under factored loading for uls design

Applied shear force	F_{vy} = **27.9** kN	Maximum bending moment	M_{LT} = M_x = **50.94** kNm
Applied torque	T_q = **1.46** kNm	Minor axis bending moment	M_y = 0 kNm
Compression force	F_c = 0 kN		

Equivalent uniform moment factors

EUM factor (Cl.4.3.6.6 & T18) m_{LT} = **1.000**

Torsional deflection parameters

Beam is torsion fixed and warping free at each end. (as defined in SCI-P-057 section 2.1.6) - Appendix B case 4

Dist for first deriv of twist	z_1 = 0 mm	Dist for second deriv of twist	z_2 = L / 2 = **3650** mm
First deriv of angle of twist	ϕ'_1 = **5.20×10^{-3}** rads/m	Third deriv of angle of twist	ϕ'''_1 = **-2.35×10^{-3}** rads/m^3
Angle of twist	ϕ_2 = **0.011** rads	Second deriv of angle of twist	ϕ''_2 = **-1.77×10^{-3}** rads/m^2

Design parameters

Total angle of twist	ϕ = **0.011** rads	First derivative of ϕ	ϕ' = **5.20×10^{-3}** rads/m
Second derivative of ϕ	ϕ'' = **1.77×10^{-3}** rads/m^2	Third derivative of ϕ	ϕ''' = **2.35×10^{-3}** rads/m^3

Section classification

	b / T = **5.1**		d / t = **12.3**
	r_{1s} = **0.000**		r_{2s} = **0.000**

Section classification is plastic

Shear capacity (parallel to y-axis)

Design shear force F_{vy} = **27.9** kN — Design shear resist (cl. 4.2.3) P_{vy} = **448.7** kN

Pass - Shear

Moment capacity (x-axis)

Design bending moment M_x = **50.9** kNm — Mnt cap low shear (cl. 4.2.5.1) M_{cx} = **260.0** kNm

Pass - Moment capacity exceeds design bending moment

Lateral torsional buckling

Effective length for LTB	L_{E_LT} = **9204** mm		
Slenderness ratio	λ = **173**	Buckling parameter	u = **0.850**
Flange ratio	η = 0.5	Torsional index	x = **10.1**
Slenderness factor	v = **0.50**	Ratio - cl 4.3.6.9	β_w = **1.000**
Equiv slenderness – cl 4.3.6.7	λ_{LT} = **74**	Limit slendernes – Ann B2.2	λ_{L0} = **35**
Euler stress	p_E = **370** N/mm^2	Perry factor	η_{LT} = **0.273**
	ϕ_{LT} = **368150516.877**	Bending strength	p_b = **175** N/mm^2
Buckling resistance moment	M_b = **171.4** kNm	Max mnt gov buckling resist	M_{LT} = **50.9** kNm
Equiv uniform mnt factor LTB	m_{LT} = **1.00**		M_b / m_{LT} = **171.4** kNm

Pass - lat. tors. buckling

Buckling under combined bending & torsion -SCI-P-057 section 2.3

For simplicity, a conservative check is applied using the maximum stresses due to each of the separate load effects, even though these do not necessarily all occur at the same section along the member.

Span factor	L / a = **9.51**	Angle of twist	ϕ = **0.011** rads
Second derivative of ϕ	ϕ'' = **1.77×10^{-3}** rads/m^2	Induced minor axis moment	M_{yt} = **0.56** kNm
Normal stress flange M_{yt}	σ_{byt} = **2** N/mm^2	Normal stress flange warping	σ_w = **4** N/mm^2
Interaction index	i_b = **0.32**		

Pass - Combined bending and torsion check satisfied

Local capacity under combined bending & torsion

For simplicity, a conservative check is applied using the maximum stresses due to each of the separate load effects, even though these do not necessarily all occur at the same section along the member.

Max. direct stress due to M_x	σ_{bx} = M_x / Z_x = **60** N/mm^2		
Combined stress - eqn 2.22	σ_{bx} + σ_{byt} + σ_w = **65** N/mm^2	Design strength	p_y = **265** N/mm^2

Pass - Local capacity

Combined shear stresses - SCI-P-057 section 2.3

For simplicity, a conservative check is applied using the maximum shear stresses due to each of the separate load effects, even though these do not necessarily all occur at the same section along the member.

Shear stress bending web τ_{bw} = **11** N/mm^2 Shear stress bending flange τ_{bf} = **3** N/mm^2

Shear stresses torsion web τ_{tw} = **5** N/mm^2 Shear stresses torsion flange τ_{tf} = **8** N/mm^2

Shear stresses warping flange τ_{wf} = **0** N/mm^2 Shear stress tors & warp web τ_{vtw} = **6** N/mm^2

Shear str tors & warp flange τ_{vtf} = **10** N/mm^2

Combined shear stresses due to bending, torsion & warping:

Comb shear stresses in web τ_w = **17** N/mm^2 Comb shear stresses in flange τ_f = **13** N/mm^2

Shear strength p_v = **159** N/mm^2

Pass - Combined shear stresses

Twist check

Total applied torque T_{qu} = **1.07** kNm

Max twist under sls loading ϕ_{sls} = **0.46** deg Twist limit ϕ_{lim} = **2.00** deg

Pass - Twist

Deflection

Maximum y-axis deflection δ_{y_max} = **10.2** mm Deflection limit - cl. 2.5.2 δ_{lim} = **11.0** mm

Pass - Deflection within specified limit