RCA STRUCTURES

2nd Floor, Unit 1, Birchanger Industrial Estate,
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Structural Calculations

Project:- Lemsford Village Hall, Lemsford, Welwyn Garden City

Design:- Design of Foundation & Superstructure

Job No:- 20655 *Date:-* Apr-19

Prepared By:- C. Silva MEng

Basis Of Design:-

BS EN 1990	Basis of Structural Design
BS EN 1990 NA	UK National Annex for Basis of Structural Design
BS EN 1991	Actions on Structures
BS EN 1991 NA	UK National Annex for Actions on Structures
BS EN 1992	Design of Concrete Structures
BS EN 1992 NA	UK National Annex for Design of Concrete Structures
BS EN 1993	Design of Steel Structures
BS EN 1993 NA	UK National Annex for Design of Steel Structures
BS EN 1995	Design of Timber Structures
BS EN 1995 NA	UK National Annex for Design of Timber Structures
BS EN 1996	Design of Masonry Structures
BS EN 1996 NA	UK National Annex for Design of Masonry Structures

Notes:-

1. *Full building regulations and checking engineer approval must be obtained prior to installation or fabrication.*
2. *Installation to be in accordance with current codes and standards.*
3. *All lengths and dimensions in these calculations are for design purposes only and should not be used for setting out on site. Contractor/Builder must measure up lengths/heights for setting out before ordering of any materials.*
4. *All temporary works to be designed and undertaken by a suitably qualified contractor.*
5. *All loadings to existing structures have been calculated following a visual inspection on site and further investigative works may be required to verify the type of construction.*
6. *All construction work to comply with the Construction Design & Management (CDM) Regulations 2015.*
7. *All planning and other elements of Building Regulations by others.*

Revisions:-

Rev	Date	Revision
A	01/05/2019	Steel Beam C revised.

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DESIGN BRIEF/SPECIFICATION

Drawings used:-	Drawings provided by client.

Scope of required design works:-

1 *Strip Foundation*
2 *Superstructure Design*

Please also refer to RCA drawings and sketchs where applicable.

STEELWORK SPECIFICATION

The Steelwork in the following calculations including UB & UC Sections have been designed as S355 grade in accordance with BS EN 1993. (Unless noted otherwise).
Grade of fixing, end, toe and baseplates as noted in calculations.
If cold rolled grade hollow Sections2 (2) (2) have been specified, client to be aware of increased radius sizes.
Where steelwork is to be used internally grade JR to be used and where external, grade J0 to be used to BS EN 10025:1993.
All bolts in connections to be Grade 8.8 to BS EN 1993 unless specified otherwise.
All welding to be in accordance with BS EN 1011-2:2001.
Where fillet welds are specified, these are generally to be 6mm fillet weld unless noted otherwise.
Where partial penetration fillet welds are specified, these are generally to be installed to a depth of 2mm less than the thickness of the connected part. (Minimum throat size = 2 √t)
Partial penetration butt welds to be generally a minimum of 6mm thick.
All full and partial penetration butt welds should be made using matching electrodes with a specified minimum tensile strength, yield strength, elongation at failure and Charpy impact value equivalent or better than the parent material.
Should Hollo-bolts be used, installation and detailing in accordance with Table H61 of SCI "Joints in Steel construction:Simple Joints" guidelines.
Finishes to Steel work to Architects specification.
Finishes to Steel work to provide suitable fire resistance to building regulation requirements.
All steelwork to be installed strictly in accordance with BS EN 1993.
Where steel bears on masonry, solid mass concrete padstone to be placed below. Size to RCA calculations.

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DESIGN BRIEF/SPECIFICATION (CONT.)

STEELWORK - CPR & CE Marking Compliance.

References:

BS EN 1090	The harmonized standard (hEN) covering fabricated structural steelwork : Execution of steel structures and aluminium structures. (Part 1 & 2)
BS EN 10025-1	Steel Sections and Plates
Hollow Sections:	
BS EN 10210-1	Hot finished
BS EN 10219-1	Cold formed welded
Stainless Steels:	
BS EN 10088-4	Sheet, Plate & Strip
BS EN 10088-5	Bars, Rods, Wire, Sections and Bright Products
BS EN 15088-1	Aluminium
Structural bolts:	
BS EN 15048-1	Non-preloaded structural bolting assemblies
BS EN 14399-1	High strength structural bolting assemblies for preloading
Welding:	
EN ISO 3834-2:2005	Quality requirements for fusion welding of metallic materials – Part 2: Comprehensive quality requirements.
BS EN 13479	Welding Consumables

Execution Class to BS EN 1090-2

Consequence Class from Table B1 - BS EN 1990 =

CC1 *Low Consequence for loss of human life*

Building use from BS EN 1991-1-7 :-

Agricultural Buildings, Houses less than 4 storeys.

Service Category to BS EN 1090-2 =

SC1 *Quasi static actions, low seismic activity, fatigue from crane actions.*

Production Category to BS EN 1090-2 =

PC2 *Welded components grade S355 and above,*

Execution Class from Table B.3 of BS EN 1090-2 = EXC2

Therefore, fabricator to note execution class EXC2 to BS EN 1090-2

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DESIGN BRIEF/SPECIFICATION (CONT.)

TIMBER SPECIFICATION

The Timber in the following calculations has generally been designed as C16 grade in accordance with BS EN 1995. (Unless noted otherwise).
Timber has been designed as generally covered by suitable finishes such as plasterboard to give suitable fire protection. Client to advise if charring rates are to be assessed for unprotected timbers.
All timber connections to be made in accordance with BS EN 1995 / NHBC Standards.
Solid blocking to be used where joists are notched into steel beams.
Lateral Restraint straps required to walls parallel to joists and timber roof spans.
Straps to be at a maximum 1.2 c/c apart and fixed to a minimum of 3 No. Joists
All straps installed to BS EN 1995 and BS8103.
Herringbone strutting or solid blocking required perpendicular to joist spans.
2.5m - 4.5m spans, 1 row required. Over 4.5m spans, 2 rows required equally spaced.

LINTEL SPECIFICATION

All Lintels to be installed in accordance with BS5977.
Individual manufacturers literature must be referred to for installation procedure.
All Lintels to bear 150mm where possible. No steel beams must bear directly on lintels.

CONCRETE SPECIFICATION

The Concrete in the following calculations has generally been designed as C28/35 grade in accordance with BS EN 1992 & BS8500. (Unless noted otherwise).
(Or equivalent RC35 designated mix to NHBC standards)
Concrete may be site mixed if in accordance with NHBC 2.1 tables 1 & 2.
All reinforcement bars are taken as "H" type bars with a yield strength of 500N/mm^2.
All concrete to be installed in accordance with BS EN 1992.

MASONRY SPECIFICATION

All masonry in the following calculations has been designed in accordance with BS EN 1996.
All blockwork below DPC level to be constructed in mortar designation (i) in accordance with BS EN 1996 part 1, Table 1. (Cement : Sand 1:3)
All load bearing masonry to be minimum 100mm wide.
Movement joints placed in accordance with BS EN 1996.
Joints every 12m (6m from corners) in brickwork and every 6m (3m from corners) in blockwork.
Wall ties to be installed and specified to DD140 & NHBC Standards.
No individual block to weigh more than 20Kg for Health & Safety purposes.
Blockwork designed to have a maximum density of 1400Kg/m^3. RCA to be advised if different.

PERMANENT & VARIABLE ACTIONS TO BS EN 1991

	Permanent G_k (kN/m^2)		Variable Q_k (kN/m^2)	
Roof Actions				
Pitched Roof				
Self weight of rafters, say	0.10			
Felt and Battens	0.12			
Roof Tiles	0.65			
Variable - Cat. H			0.52	
	0.87	(on slope)	0.52	(on slope)
12 mm Plasterboard / skim	0.20			
	1.07			
Flat Roof				
Joists	0.15			
Decking/Finishes	0.30			
12 mm Plasterboard / skim	0.20			
Variable Action - Category H			0.60	
	0.65		0.60	
Loft				
Ceiling Joists / Insulation / Felt	0.15			
12 mm Plasterboard / skim	0.20			
Variable Action			0.35	
	0.35		0.35	
Wall Actions				
External Cavity Wall (Brick outer skin)				
102 mm Brickwork	2.04			
100 mm Blockwork (up to 10.5N/mm^2)	1.40			
12 mm Plasterboard / skim	0.20			
	3.64			

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PERMANENT & VARIABLE ACTIONS TO BS EN 1991

	Permanent G_k (kN/m^2)		Variable Q_k (kN/m^2)
Wall Actions			
External Wall Below DPC			
102 mm Brickwork	2.04		
102 mm Brickwork	2.04		
100 Cavity Fill	2.50		
	6.58		
Internal Timber Stud Partitions			
Studwork	0.15		
24 mm Plasterboard / skim	0.40		
	0.55		
Floor Actions			
First Floor - Timber Floor (Jsts @ 400 c/c)			
195 x 50 Joists (estimated)	0.11		
12.5 mm Boarding	0.09		
12 mm Plasterboard / skim	0.20		
Variable Action - Category A (Domestic)		Category A1	1.50
	0.41	Private	1.50
Timber Stud Partitions (General)	0.35		
	0.76		
Bi-Fold Doors (Self Weight)			
16 mm Glazing (2No 8mm Panels worst c	0.30		
Framework	0.15		
	0.45		

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Title of Scheme Lemsford Village Hall, Lemsford, Welwyn Garden City | Job No. 20655

TIMBER BEAM DESIGN to BS EN 1995-1-1

Beam Reference = Flat Roof Joists

Actions

	Permanent Action, G_k (kN/m^2)	Leading Variable Action, $Q_{k\,1}$ (kN/m^2)	Variable Action, $Q_{k\,i}$ (kN/m^2)	Distance(m)		Permanent Action, G_k (kN/m)	Leading Variable Action, $Q_{k\,1}$ (kN/m)	Variable Action, $Q_{k\,i}$ (kN/m)
Flat Roof	0.65	0.60		c/c's	0.40	0.26	0.24	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
					TOTALS:	*0.26*	*0.24*	*0.00*

Clear Span = 5400 mm. Bearings = 100 mm minimum each side

Design span = **5500 mm** ***(NB - Design span only - not for setting out on site)***

Conservatively, use equation 6.10 = $1.35\ G_k + 1.5\ Q_{k\,1} + \Sigma\ \psi_{0\,i}\ 1.5\ Q_{k\,i}$

Conservatively, take $\psi_{0\,i}$ = 1.00

Design UDL= 0.35 + 0.36 + 0 = 0.71 kN/m

Unfactored UDL = 0.50 kN/m

Design Point Load 1= 0.00 kN

Distance from A to point load 1= 0 mm

Distance from B to point load 1= 5500 mm

Design Point Load 2= 0.00 kN

Distance from A to point load 2= 0 mm

Distance from B to point load 2= 5500 mm

Design Point Load 3= 0.00 kN

Distance from A to point load 3= 0 mm

Distance from B to point load 3= 5500 mm

Beam analysed with simple supports:-

Design Moment = 2.69 + 0.00 + 0.00 + 0.00

Design Shear Force At A = 1.95 + 0.00 + 0.00 + 0.00

Design Shear Force At B = 1.95 + 0.00 + 0.00 + 0.00

Design Moment =	2.69 kNm.
Design. Shear A =	1.95 kN
Design. Shear B =	1.95 kN

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TIMBER BEAM DESIGN to BS EN 1995-1-1

Beam Reference = Flat Roof Joists

From previous calculation :-

Max. Shear = 1.95 kN Maximum Moment = 2.69 kNm

Clear Span = 5400 mm. Bearings = 100 mm minimum each side
Design span = 5.50 m = 5500 mm

Exposure condition = Service Class 1
(Intermediate Floors, Warm Roofs, internal and party timber frame walls)
Load Duration = Short Term *(Snow, Maintenance on roofs.)*

Design Timber Grade = C24 Material = Solid Timber. Treated or untreated
Bending Strength, $f_{m,k}$ = 24.0 N/mm^2
Shear Strength, $f_{v,k}$ = 4.00 N/mm^2

Try 1 No. 220 deep x 50 wide member
Type of fire exposure = Not Exposed
Charring rate = General Rate = 0.60 mm/min
Required fire rating = 30 minutes
Effective size for design = 220 mm deep x 50 mm wide member
Elastic Modulus , W_{yy} = 4.0E+05 mm^3. Partial Safety factor, γ_m = 1.3
K_h Factor = 1.00 Type of Beam= Load Sharing k_{sys} = 1.1
k_{mod} = 0.9 k_{crit} = 1 (No LTB due to boarding)

Bending
Design bending strength $f_{m,d}$ = $k_{mod} \cdot k_h \cdot k_{crit} \cdot k_{sys} \cdot f_{m,k} / \gamma_m$ = 18.28 N/mm^2

$M_{ult} = f_{m,d} \cdot W_{yy}$ = 7.4 kNm > 2.69 kNm O.K.

Shear k_{cr} = 0.67

Applied shear stress = $\dfrac{3V_d}{2bh\,k_{cr}}$ = 0.40 N/mm^2

Design shear strength $f_{v,d}$ = $k_{mod} \cdot k_{sys} \cdot f_{v,k} / \gamma_m$ = 3.05 N/mm^2

As 3.0 N/mm^2 > 0.40 N/mm^2 O.K.

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TIMBER BEAM DESIGN to BS EN 1995-1-1

Beam Reference = Flat Roof Joists

Deflection A = 11000 mm^2 $E_{0,mean}$ = 11.00 kN/mm^2

I_{yy} = 4.4E+07 mm^4 k_{def} = 0.6 ψ_2 = 0.0

Building use = Roofs Assume point loads approx 75 % imposed

Equivalent unfactored UDL Perm. = 0.26 kN/m M(Perm)= 0.98 kNm
Equivalent unfactored PL Perm. = 0.00 kN/m

$$w_{fin,G} = \frac{5wL^4}{384EI} + \frac{19.2\,M}{EA} \times (1 + k_{def})$$

$$w_{fin,G} = 6.34 + 0.16 \times 1.6 = 10.39 \text{ mm}$$

Equivalent unfactored UDL Variable = 0.24 kN/m M(Variable)= 0.91 kNm
Equivalent unfactored PL Variable = 0.00 kN/m

$$w_{fin,Q} = \frac{5wL^4}{384EI} + \frac{19.2\,M}{EA} \times (1 + \psi_2\, k_{def})$$

$$w_{fin,Q} = 5.86 + 0.14 \times 1 = 6.00 \text{ mm}$$

Final Deflection (including creep and shear) = 16.40 mm

Type of finish:- Brittle Finish Limiting value = L / 250

Allowable deflection = 21.6 mm

NB Deflection not checked in a fire situation as serviceability only.

As 21.6 mm > 16.40 mm O.K.

Therefore, use:-

1 No. 220 x 50 Joist/s, C24 Grade Timber

Centres = 400 mm

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TIMBER BEAM DESIGN to BS EN 1995-1-1

Beam Reference = Trimmer 1

Actions

	Permanent Action, G_k (kN/m^2)	Leading Variable Action, $Q_{k\,1}$ (kN/m^2)	Variable Action, $Q_{k\,i}$ (kN/m^2)	Distance(m)		Permanent Action, G_k (kN/m)	Leading Variable Action, $Q_{k\,1}$ (kN/m)	Variable Action, $Q_{k\,i}$ (kN/m)
Flat Roof	0.65	0.60		3.4/2=	1.70	1.10	1.02	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
					TOTALS:	*1.10*	*1.02*	*0.00*

Clear Span = 1200 mm. Bearings = 100 mm minimum each side

Design span = **1300 mm** ***(NB - Design span only - not for setting out on site)***

Conservatively, use equation 6.10 = $1.35\ G_k + 1.5\ Q_{k\,1} + \Sigma\ \psi_{0\,i}\ 1.5\ Q_{k\,i}$

Conservatively, take $\psi_{0\,i}$ = 1.00

Design UDL= 1.49 + 1.53 + 0 = 3.02 kN/m

Unfactored UDL = 2.12 kN/m

Design Point Load 1= 0.00 kN

Distance from A to point load 1= 0 mm

Distance from B to point load 1= 1300 mm

Design Point Load 2= 0.00 kN

Distance from A to point load 2= 0 mm

Distance from B to point load 2= 1300 mm

Design Point Load 3= 0.00 kN

Distance from A to point load 3= 0 mm

Distance from B to point load 3= 1300 mm

Beam analysed with simple supports:-

Design Moment = 0.64 + 0.00 + 0.00 + 0.00

Design Shear Force At A = 1.96 + 0.00 + 0.00 + 0.00

Design Shear Force At B = 1.96 + 0.00 + 0.00 + 0.00

Design Moment =	0.64 kNm.
Design. Shear A =	1.96 kN
Design. Shear B =	1.96 kN

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TIMBER BEAM DESIGN to BS EN 1995-1-1

Beam Reference = Trimmer 1

From previous calculation :-

Max. Shear = 1.96 kN Maximum Moment = 0.64 kNm

Clear Span = 1200 mm. Bearings = 100 mm minimum each side
Design span = 1.30 m = 1300 mm

Exposure condition = Service Class 1
(Intermediate Floors, Warm Roofs, internal and party timber frame walls)
Load Duration = Short Term *(Snow, Maintenance on roofs.)*

Design Timber Grade = C24 Material = Solid Timber. Treated or untreated

Bending Strength, $f_{m,k}$ = 24.0 N/mm^2

Shear Strength, $f_{v,k}$ = 4.00 N/mm^2

Try 2 No. 220 deep x 50 wide member
Type of fire exposure = Not Exposed
Charring rate = General Rate = 0.60 mm/min
Required fire rating = 30 minutes
Effective size for design = 220 mm deep x 100 mm wide member
Elastic Modulus , W_{yy} = 8.1E+05 mm^3. Partial Safety factor, γ_m = 1.3
K_h Factor = 1.00 Type of Beam= Load Sharing k_{sys} = 1.1
k_{mod} = 0.9 k_{crit} = 1 (No LTB due to boarding)

Bending

Design bending strength $f_{m,d}$ = $k_{mod} \cdot k_h \cdot k_{crit} \cdot k_{sys} \cdot f_{m,k} / \gamma_m$ = 18.28 N/mm^2

$M_{ult} = f_{m,d} \cdot W_{yy}$ = 14.7 kNm > 0.64 kNm **O.K.**

Shear k_{cr} = 0.67

Applied shear stress = $\dfrac{3V_d}{2bh\,k_{cr}}$ = 0.20 N/mm^2

Design shear strength $f_{v,d}$ = $k_{mod} \cdot k_{sys} \cdot f_{v,k} / \gamma_m$ = 3.05 N/mm^2

As 3.0 N/mm^2 > 0.20 N/mm^2 **O.K.**

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TIMBER BEAM DESIGN to BS EN 1995-1-1

Beam Reference = Trimmer 1

Deflection $A =$ 22000 mm^2 $E_{0,mean} =$ 11.00 kN/mm^2

$I_{yy} =$ 8.9E+07 mm^4 $k_{def} =$ 0.6 $\psi_2 =$ 0.0

Building use = Roofs Assume point loads approx 75 % imposed

Equivalent unfactored UDL Perm. = 1.10 kN/m M(Perm)= 0.23 kNm

Equivalent unfactored PL Perm. = 0.00 kN/m

$$w_{fin,G} = \frac{5wL^4}{384EI} + \frac{19.2\ M}{EA} \times (1 + k_{def})$$

$$w_{fin,G} = 0.04 + 0.02 \times 1.6 = 0.10\ mm$$

Equivalent unfactored UDL Variable = 1.02 kN/m M(Variable)= 0.22 kNm

Equivalent unfactored PL Variable = 0.00 kN/m

$$w_{fin,Q} = \frac{5wL^4}{384EI} + \frac{19.2\ M}{EA} \times (1 + \psi_2\ k_{def})$$

$$w_{fin,Q} = 0.04 + 0.02 \times 1 = 0.06\ mm$$

Final Deflection (including creep and shear) = 0.15 mm

Type of finish:- Brittle Finish Limiting value = L / 250

Allowable deflection = 4.8 mm

NB Deflection not checked in a fire situation as serviceability only.

As 4.8 mm > 0.15 mm O.K.

Therefore, use:-

2 No. 220 x 50 Joist/s, C24 Grade Timber

Centres = 1700 mm

__TIMBER BEAM DESIGN to BS EN 1995-1-1__

Beam Reference = Trimmer 2

__Actions__	Permanent Action, G_k (kN/m^2)	Leading Variable Action, $Q_{k\,1}$ (kN/m^2)	Variable Action, $Q_{k\,i}$ (kN/m^2)	Distance(m)		Permanent Action, G_k (kN/m)	Leading Variable Action, $Q_{k\,1}$ (kN/m)	Variable Action, $Q_{k\,i}$ (kN/m)
Flat Roof	0.65	0.60		nominal	0.20	0.13	0.12	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
					TOTALS:	*0.13*	*0.12*	*0.00*

Clear Span = 5375 mm. Bearings = 100 mm minimum each side

Design span = **5475 mm** ***(NB - Design span only - not for setting out on site)***

Conservatively, use equation 6.10 = $1.35\ G_k + 1.5\ Q_{k\,1} + \Sigma\ \psi_{0\,i}\ 1.5\ Q_{k\,i}$

Conservatively, take $\psi_{0\,i}$ = 1.00

Design UDL= 0.18 + 0.18 + 0 = 0.36 kN/m

Unfactored UDL = 0.25 kN/m

Design Point Load 1= 1.96 kN Trimmer 1

Distance from A to point load 1= 940 mm

Distance from B to point load 1= 4535 mm

Design Point Load 2= 1.96 kN Trimmer 1

Distance from A to point load 2= 1945 mm

Distance from B to point load 2= 3530 mm

Design Point Load 3= 0.00 kN

Distance from A to point load 3= 0 mm

Distance from B to point load 3= 5475 mm

__Beam analysed with simple supports:-__

Design Moment = 1.33 + 1.53 + 2.46 + 0.00

Design Shear Force At A = 0.97 + 1.63 + 1.27 + 0.00

Design Shear Force At B = 0.97 + 0.34 + 0.70 + 0.00

Design Moment =	5.32 kNm.
Design. Shear A =	3.86 kN
Design. Shear B =	2.01 kN

TIMBER BEAM DESIGN to BS EN 1995-1-1

Beam Reference = Trimmer 2

From previous calculation :-

Max. Shear = 3.86 kN Maximum Moment = 5.32 kNm

Clear Span = 5375 mm. Bearings = 100 mm minimum each side
Design span = 5.48 m = 5475 mm

Exposure condition = Service Class 1
(Intermediate Floors, Warm Roofs, internal and party timber frame walls)
Load Duration = Short Term *(Snow, Maintenance on roofs.)*

Design Timber Grade = C24 Material = Solid Timber. Treated or untreated
Bending Strength, $f_{m,k}$ = 24.0 N/mm^2
Shear Strength, $f_{v,k}$ = 4.00 N/mm^2

Try 2 No. 220 deep x 50 wide member
Type of fire exposure = Not Exposed
Charring rate = General Rate = 0.60 mm/min
Required fire rating = 30 minutes
Effective size for design = 220 mm deep x 100 mm wide member
Elastic Modulus , W_{yy} = 8.1E+05 mm^3. Partial Safety factor, γ_m = 1.3
K_h Factor = 1.00 Type of Beam= Load Sharing k_{sys} = 1.1
k_{mod} = 0.9 k_{crit} = 1 (No LTB due to boarding)

Bending
Design bending strength $f_{m,d}$ = $k_{mod} \cdot k_h \cdot k_{crit} \cdot k_{sys} \cdot f_{m,k} / \gamma_m$ = 18.28 N/mm^2

$M_{ult} = f_{m,d} \cdot W_{yy}$ = 14.7 kNm > 5.32 kNm O.K.

Shear k_{cr} = 0.67

Applied shear stress = $\dfrac{3V_d}{2bh\,k_{cr}}$ = 0.39 N/mm^2

Design shear strength $f_{v,d}$ = $k_{mod} \cdot k_{sys} \cdot f_{v,k} / \gamma_m$ = 3.05 N/mm^2

As 3.0 N/mm^2 > 0.39 N/mm^2 O.K.

TIMBER BEAM DESIGN to BS EN 1995-1-1

Beam Reference = Trimmer 2

Deflection $A =$ 22000 mm^2 $E_{0,mean} =$ 11.00 kN/mm^2

$I_{yy} =$ 8.9E+07 mm^4 $k_{def} =$ 0.6 $\psi_2 =$ 0.0

Building use = Roofs Assume point loads approx 75 % imposed

Equivalent unfactored UDL Perm. = 0.13 kN/m M(Perm)= 1.48 kNm

Equivalent unfactored PL Perm. = 0.27 kN/m

$$w_{fin,G} = \frac{5wL^4}{384EI} + \frac{19.2\,M}{EA} \times (1 + k_{def})$$

$$w_{fin,G} = 4.75 + 0.12 \times 1.6 = 7.78 \text{ mm}$$

Equivalent unfactored UDL Variable = 0.12 kN/m M(Variable)= 3.44 kNm

Equivalent unfactored PL Variable = 0.80 kN/m

$$w_{fin,Q} = \frac{5wL^4}{384EI} + \frac{19.2\,M}{EA} \times (1 + \psi_2\, k_{def})$$

$$w_{fin,Q} = 11.01 + 0.27 \times 1 = 11.28 \text{ mm}$$

Final Deflection (including creep and shear) = 19.07 mm

Type of finish:- Brittle Finish Limiting value = L / 250

Allowable deflection = 21.5 mm

NB Deflection not checked in a fire situation as serviceability only.

As 21.5 mm > 19.07 mm O.K.

Therefore, use:-

2 No. 220 x 50 Joist/s, C24 Grade Timber

Centres = 200 mm

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TIMBER BEAM DESIGN to BS EN 1995-1-1

Beam Reference = Rafters

Actions

	Permanent Action, G_k (kN/m^2)	Leading Variable Action, $Q_{k\,1}$ (kN/m^2)	Variable Action, $Q_{k\,i}$ (kN/m^2)	Distance(m)		Permanent Action, G_k (kN/m)	Leading Variable Action, $Q_{k\,1}$ (kN/m)	Variable Action, $Q_{k\,i}$ (kN/m)
Pitched Roof	1.07		0.52	c/c's	0.40	0.43	0.00	0.21
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
					TOTALS:	*0.43*	*0.00*	*0.21*

Clear Span = 2200 mm. Bearings = 100 mm minimum each side

Design span = **2300 mm** ***(NB - Design span only - not for setting out on site)***

Conservatively, use equation 6.10 = $1.35\,G_k + 1.5\,Q_{k\,1} + \Sigma\,\psi_{0\,i}\,1.5\,Q_{k\,i}$

Conservatively, take $\psi_{0\,i}$ = 1.00

Design UDL= 0.58 + 0.00 + 0.31177 = 0.89 kN/m

Unfactored UDL = 0.64 kN/m

Design Point Load 1= 0.00 kN

Distance from A to point load 1= 0 mm

Distance from B to point load 1= 2300 mm

Design Point Load 2= 0.00 kN

Distance from A to point load 2= 0 mm

Distance from B to point load 2= 2300 mm

Design Point Load 3= 0.00 kN

Distance from A to point load 3= 0 mm

Distance from B to point load 3= 2300 mm

Beam analysed with simple supports:-

Design Moment = 0.59 + 0.00 + 0.00 + 0.00

Design Shear Force At A = 1.02 + 0.00 + 0.00 + 0.00

Design Shear Force At B = 1.02 + 0.00 + 0.00 + 0.00

Design Moment =	0.59 kNm.
Design. Shear A =	1.02 kN
Design. Shear B =	1.02 kN

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TIMBER BEAM DESIGN to BS EN 1995-1-1

Beam Reference = Rafters

From previous calculation :-

Max. Shear = 1.02 kN   Maximum Moment = 0.59 kNm

Clear Span = 2200 mm.   Bearings = 100 mm minimum each side
Design span = 2.30 m = 2300 mm

Exposure condition = Service Class 1
(Intermediate Floors, Warm Roofs, internal and party timber frame walls)
Load Duration = Short Term   *(Snow, Maintenance on roofs.)*

Design Timber Grade = C24   Material = Solid Timber. Treated or untreated
Bending Strength, $f_{m,k}$ = 24.0 N/mm^2
Shear Strength, $f_{v,k}$ = 4.00 N/mm^2

Try 1 No. 200 deep x 50 wide member
Type of fire exposure = Not Exposed
Charring rate = General Rate = 0.60 mm/min
Required fire rating = 30 minutes
Effective size for design = 200 mm deep x 50 mm wide member
Elastic Modulus , W_{yy} = 3.3E+05 mm^3.   Partial Safety factor, γ_m = 1.3
K_h Factor = 1.00   Type of Beam= Load Sharing k_{sys} = 1.1
k_{mod} = 0.9 k_{crit} = 1   (No LTB due to boarding)

Bending

Design bending strength $f_{m,d}$ = $k_{mod} \cdot k_h \cdot k_{crit} \cdot k_{sys} \cdot f_{m,k} / \gamma_m$ = 18.28 N/mm^2

$M_{ult} = f_{m,d} \cdot W_{yy}$ = 6.1 kNm > 0.59 kNm **O.K.**

Shear k_{cr} = 0.67

Applied shear stress = $\dfrac{3V_d}{2bh\,k_{cr}}$ = 0.23 N/mm^2

Design shear strength $f_{v,d}$ = $k_{mod} \cdot k_{sys} \cdot f_{v,k} / \gamma_m$ = 3.05 N/mm^2

As 3.0 N/mm^2 > 0.23 N/mm^2 **O.K.**

TIMBER BEAM DESIGN to BS EN 1995-1-1

Beam Reference = Rafters

Deflection $A =$ 10000 mm^2 $E_{0,mean} =$ 11.00 kN/mm^2

$I_{yy} =$ 3.3E+07 mm^4 $k_{def} =$ 0.6 $\psi_2 =$ 0.0

Building use = Roofs Assume point loads approx 75 % imposed

Equivalent unfactored UDL Perm. = 0.43 kN/m M(Perm)= 0.28 kNm

Equivalent unfactored PL Perm. = 0.00 kN/m

$$w_{fin,G} = \frac{5wL^4}{384EI} + \frac{19.2\ M}{EA} \times (1 + k_{def})$$

$$w_{fin,G} = 0.43 + 0.05 \times 1.6 = 0.76\ mm$$

Equivalent unfactored UDL Variable = 0.21 kN/m M(Variable)= 0.14 kNm

Equivalent unfactored PL Variable = 0.00 kN/m

$$w_{fin,Q} = \frac{5wL^4}{384EI} + \frac{19.2\ M}{EA} \times (1 + \psi_2\ k_{def})$$

$$w_{fin,Q} = 0.21 + 0.02 \times 1 = 0.23\ mm$$

Final Deflection (including creep and shear) = 0.99 mm

Type of finish:- Brittle Finish Limiting value = L / 250

Allowable deflection = 8.8 mm

NB Deflection not checked in a fire situation as serviceability only.

As 8.8 mm > 0.99 mm **O.K.**

Therefore, use:-

1 No. 200 x 50 Joist/s, C24 Grade Timber

Centres = 400 mm

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TIMBER BEAM DESIGN to BS EN 1995-1-1

Beam Reference = Ceiling Joists

Actions

	Permanent Action, G_k (kN/m^2)	Leading Variable Action, $Q_{k\,1}$ (kN/m^2)	Variable Action, $Q_{k\,i}$ (kN/m^2)	Distance(m)		Permanent Action, G_k (kN/m)	Leading Variable Action, $Q_{k\,1}$ (kN/m)	Variable Action, $Q_{k\,i}$ (kN/m)
Loft Joists	0.35	0.35		c/c's	0.40	0.14	0.14	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
					TOTALS:	*0.14*	*0.14*	*0.00*

Clear Span = 4200 mm. Bearings = 100 mm minimum each side

Design span = **4300 mm** ***(NB - Design span only - not for setting out on site)***

Conservatively, use equation 6.10 = $1.35\ G_k + 1.5\ Q_{k\,1} + \Sigma\ \psi_{0\,i}\ 1.5\ Q_{k\,i}$

Conservatively, take $\psi_{0\,i}$ = 1.00

Design UDL= 0.19 + 0.21 + 0 = 0.40 kN/m

Unfactored UDL = 0.28 kN/m

Design Point Load 1= 0.00 kN

Distance from A to point load 1= 0 mm

Distance from B to point load 1= 4300 mm

Design Point Load 2= 0.00 kN

Distance from A to point load 2= 0 mm

Distance from B to point load 2= 4300 mm

Design Point Load 3= 0.00 kN

Distance from A to point load 3= 0 mm

Distance from B to point load 3= 4300 mm

Beam analysed with simple supports:-

Design Moment = 0.92 + 0.00 + 0.00 + 0.00

Design Shear Force At A = 0.86 + 0.00 + 0.00 + 0.00

Design Shear Force At B = 0.86 + 0.00 + 0.00 + 0.00

Design Moment =	0.92 kNm.
Design. Shear A =	0.86 kN
Design. Shear B =	0.86 kN

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TIMBER BEAM DESIGN to BS EN 1995-1-1

Beam Reference = Ceiling Joists

From previous calculation :-

Max. Shear = 0.86 kN Maximum Moment = 0.92 kNm

Clear Span = 4200 mm. Bearings = 100 mm minimum each side
Design span = 4.30 m = 4300 mm

Exposure condition = Service Class 1
(Intermediate Floors, Warm Roofs, internal and party timber frame walls)
Load Duration = Short Term *(Snow, Maintenance on roofs.)*

Design Timber Grade = C24 Material = Solid Timber. Treated or untreated

Bending Strength, $f_{m,k}$ = 24.0 N/mm^2

Shear Strength, $f_{v,k}$ = 4.00 N/mm^2

Try 1 No. 175 deep x 50 wide member
Type of fire exposure = Not Exposed
Charring rate = General Rate = 0.60 mm/min
Required fire rating = 30 minutes
Effective size for design = 175 mm deep x 50 mm wide member
Elastic Modulus , W_{yy} = 2.6E+05 mm^3. Partial Safety factor, γ_m = 1.3
K_h Factor = 1.00 Type of Beam= Load Sharing k_{sys} = 1.1
k_{mod} = 0.9 k_{crit} = 1 (No LTB due to boarding)

Bending

Design bending strength $f_{m,d}$ = $k_{mod} . k_h . k_{crit}. k_{sys} . f_{m,k} / \gamma_m$ = 18.28 N/mm^2

$M_{ult} = f_{m,d} . W_{yy}$ = 4.7 kNm > 0.92 kNm **O.K.**

Shear k_{cr} = 0.67

Applied shear stress = $\dfrac{3V_d}{2bh\,k_{cr}}$ = 0.22 N/mm^2

Design shear strength $f_{v,d}$ = $k_{mod} . k_{sys} . f_{v,k} / \gamma_m$ = 3.05 N/mm^2

As 3.0 N/mm^2 > 0.22 N/mm^2 **O.K.**

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TIMBER BEAM DESIGN to BS EN 1995-1-1

Beam Reference = Ceiling Joists

Deflection $A =$ 8750 mm^2 $E_{0,mean} =$ 11.00 kN/mm^2
$I_{yy} =$ 2.2E+07 mm^4 $k_{def} =$ 0.6 $\psi_2 =$ 0.0

Building use = Roofs Assume point loads approx 75 % imposed

Equivalent unfactored UDL Perm. = 0.14 kN/m M(Perm)= 0.32 kNm
Equivalent unfactored PL Perm. = 0.00 kN/m

$$w_{fin,G} = \frac{5wL^4}{384EI} + \frac{19.2\,M}{EA} \times (1 + k_{def})$$

$$w_{fin,G} = 2.53 + 0.06 \times 1.6 = 4.15 \text{ mm}$$

Equivalent unfactored UDL Variable = 0.14 kN/m M(Variable)= 0.32 kNm
Equivalent unfactored PL Variable = 0.00 kN/m

$$w_{fin,Q} = \frac{5wL^4}{384EI} + \frac{19.2\,M}{EA} \times (1 + \psi_2\, k_{def})$$

$$w_{fin,Q} = 2.54 + 0.06 \times 1 = 2.60 \text{ mm}$$

Final Deflection (including creep and shear) = 6.75 mm

Type of finish:- Brittle Finish Limiting value = L / 250
Allowable deflection = 16.8 mm
NB Deflection not checked in a fire situation as serviceability only.
As 16.8 mm > 6.75 mm O.K.

Therefore, use:-

1 No. 175 x 50 Joist/s, C24 Grade Timber

Centres = 400 mm

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SIMPLE BEAM ANALYSIS TO BS EN 1990

Ref = Steel Beam A

Actions

	Permanent Action, G_k (kN/m^2)	Leading Variable Action, $Q_{k\,1}$ (kN/m^2)	Variable Action, $Q_{k\,i}$ (kN/m^2)	Dist		Permanent Action, G_k (kN/m)	Variable Action, $Q_{k\,1}$ (kN/m)	Variable Action, $Q_{k\,i}$ (kN/m)
Cavity Wall	3.64			height	1.00	3.64	0.00	0.00
Bi Fold Doors	0.45			height	2.50	1.13	0.00	0.00
Pitched Roof	1.07		0.52	on slope	4.00	4.28	0.00	2.08
Flat Roof	0.65	0.60		5.5/2=	2.75	1.79	1.65	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
Self weight of beam	Using a	203x133x30				0.59		
					TOTALS:	*11.41*	*1.65*	*2.08*

Clear Span = 3570 mm. Bearings = 100 mm minimum each side

Design span = **3670 mm** ***(NB - Design span only - not for setting out on site)***

Span between restraints = 3670 mm

Conservatively, use equation 6.10 = $1.35\ G_k + 1.5\ Q_{k\,1} + \Sigma\ \psi_{0\,i}\ 1.5\ Q_{k\,i}$

Conservatively, take $\psi_{0\,i}$ = 1.00

Design UDL= 15.41 + 2.48 + 3.11769 = 21.00 kN/m

Unfactored UDL = 15.14 kN/m

Design Point Load 1= 0.00 kN

Distance from A to point load 1= 0 mm

Distance from B to point load 1= 3670 mm

Design Point Load 2= 0.00 kN

Distance from A to point load 2= 0 mm

Distance from B to point load 2= 3670 mm

Design Point Load 3= 0.00 kN

Distance from A to point load 3= 0 mm

Distance from B to point load 3= 3670 mm

Design Point Load 4= 0.00 kN

Distance from A to point load 4= 0 mm

Distance from B to point load 4= 3670 mm

Beam analysed with simple supports:-

Design Moment = 35.36 kNm ULS

Shear Force At A = 38.54 kN ULS Shear Force At B = 38.54 kN

Design Moment =	35.36	kNm
Design. Shear A =	38.54	kN
Design. Shear B =	38.54	kN

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DESIGN OF STEEL MEMBER TO BS EN 1993-1-1

Ref = Steel Beam A

Design Data

From previous calculation:- Moment, $M_{y,Ed}$ = 35.36 kNm

(Beam in pure bending only - no axial forces) Design Shear = 38.54 kN

Try a section Size =

2 No. **203x133x30** Design Span = 3670 mm

Between Restraints = 3670 mm

Grade of Steel = S355 Flange thickness, t_f= 9.60 mm

Web thickness, t_w= 6.40 mm Yield Strength, f_y = 355 N/mm^2 *(Table 3.1)*

Classification of Section

Web = $d/t = \dfrac{172.40}{6.40}$ = 26.94 mm ε = 0.81

Flange = $c = \dfrac{133.90 - 15.20 - 6.40}{2}$ = 56.15 mm

$c/t = \dfrac{56.15}{9.60}$ = 5.85 mm

(From table 5.2, for rolled Sections2)

web Class 1 limiting = 72 ε = 58.58 mm Therefore Class 1 **Suitable**

flange Class 1 limiting = 9 ε = 7.32 mm Therefore Class 1 **Suitable**

Moment Resistance

As member is Class 1 use Plastic modulus.

$W_{pl,y}$ = 314000 mm^3 $W_{el,y}$ = 280000 mm^3 γ_{M0} = 1

$M_{c,Rd} = \dfrac{314000 \times 355}{1}$ = 111.47 kNm *(Cl 6.2.5)*

As 222.94 > 35.36 kNm Moment resistance of section **O.K.**

Shear Resistance

Root radius, r = 7.60 mm A = 3820 mm^2

$A_{v,z}$ = A - 2 b t_f + (t_w + 2r) t $A_{v,z}$ = 1456 mm^2 $\eta\, h_w\, t_w$ = 1103 mm^2

$V_{pl,z,Rd} = \dfrac{A_{v,z} (f_y / ((3)^{0.5}))}{\gamma_{M0}}$ = 298.52 kN

Combined bending/shear not considered as max. moment away from max. shear.

As 597.04 > 38.54 kN Shear resistance of section **O.K.**

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DESIGN OF STEEL MEMBER TO BS EN 1993-1-1 Ref = Steel Beam A

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Calculation of Mcrit

1. Destabilising Loads

$$M_{cr} = C_1 \frac{\pi^2 EI_z}{(kL)^2} \left\{ \sqrt{\left(\frac{k}{k_w}\right)^2 \frac{I_w}{I_z} + \frac{(kL)^2 GI_t}{\pi^2 EI_z} + (C_2 z_g)^2} - C_2 z_g \right\}$$

E= 210000 N/mm² G= 80770 N/mm²

Loading and support condition:- Simply supported, UDL over whole span

Factors based on bending moment:- C1= 1.127 C2= 0.454 C3= 1.0

$I_{w=}$	3.7E+10	mm^6	Warping constant from section property tables.
$I_{z=}$	3850000	mm^4	Minor second moment of area from section property tables.
$I_{t=}$	103000	mm^4	Torsion constant from section property tables
$k_{w=}$	1.0		Unless special provision for warping fixity is made, k_w should be taken as 1.0
$k_=$	1.2		Effective Length constant
$z_{g=}$	103.4	mm	Height of the destabilising load above the shear centre.
$z_{j=}$	0.0	mm	0 mm for bi-symmetric Sections2

$$M_{cr} = 463200 \times \left(\left(13839 + 20241 + 2204 \right)^{0.5} - 47 \right)$$

M_{cr}= **66.5 kNm** ***(Destabilising Load)***

2. Non-destabilising Loads

$$M_{cr} = C_1 \frac{\pi^2 EI_z}{L^2} \sqrt{\frac{I_w}{I_z} + \frac{L^2 GI_t}{\pi^2 EI_z}}$$

$$M_{cr} = 1 \times 591844 \times \left(23667 \right)^{0.5}$$

M_{cr}= **102.6 kNm** ***(Non-destabilising Load)***

For rolled profiles, λ_{LT0} = 0.4

Consider load to be Destabilising therefore use, M_{cr}= 66.49 kNm

W_y = 314000 mm^3

Non-dimensional slenderness $\lambda_{LT} = \left(\frac{W_y f_y}{M_{cr}}\right)^{0.5}$ = 1.295

As, 1.295 > 0.4 LTB verification is Required

DESIGN OF STEEL MEMBER TO BS EN 1993-1-1 Ref = Steel Beam A

Lateral Torsional Buckling (Cont)

$\frac{h}{b} = \frac{206.80}{133.90} = 1.54 < 2$ use buck. curve b *(Table 6.5)*

Imperfection factor, $\alpha_{LT} =$ 0.34 $\beta =$ 0.75

$\phi_{LT} = 0.5\,(1 + \alpha_{LT}\,(\lambda_{LT} - \lambda_{LT0}) + \beta\lambda_{LT}^2) = 1.28$

Reduction factor, $\chi_{LT} = \frac{1}{\phi_{LT} + (\phi_{LT}^2 - \beta\lambda_{LT}^2)^{0.5}}$ but $\chi_{LT} < 1.0$ and $\chi_{LT} < \frac{1}{\lambda_{LT}^2}$

$\chi_{LT} =$ 0.53 0.53 < 1 0.53 < 0.60

Conservatively, factor, f = 1 $\chi_{LT,mod} =$ 0.53 $\gamma_{M1} =$ 1.00

$M_{b,Rd} = \chi_{LT,mod} W_y f_y / \gamma_{M1} =$ 58.68 kNm

As 117.35 > 35.36 kNm Lateral Torsional Buckling resistance O.K.

Deflection Checks Take average partial safety factors as 1.425

Equivalent unfactored SLS udl to beam = $\frac{24.81 \times 8}{3.67 \times 3.67} =$ 14.74 kN/m

$I_{y=}$ 2.9E+07 mm^4 Major second moment of area from section property tables.

Deflection = 2.86 mm Deflection limits = $\frac{Span}{360} =$ 10.19 mm

Finishes = Brittle

As 2.86 < 10.19 mm Deflection due to SLS loading is O.K.

Frequency Checks

By inspection, frequency checks not required due to low deflections.

Therefore use	2 No.	203x133x30	S355	section

BEARING DESIGN TO BS EN 1996-1-1

Reference = Steel Beam A

Bearing at End "A" Factored Reaction = 38.54 kN ULS

$f_k = K f_b^{\alpha} f_m^{\beta}$ Masonry under padstone Aggregate Concrete blocks Group 1

Unit size below = 215 mm h x 100 mm width Shape Factor, δ = 1.38

Act. Comp. Str = 3.50 N/mm^2 Norm. Mean Comp. Strength, f_b = 4.83 N/mm^2

From National Annexe, Table NA4, K = 0.55 α = 0.7 β = 0.30

Mortar Designation = (iii) Mortar Type M 4 f_m = 4.00 N/mm^2

f_k = 2.51 N/mm^2 $f_d = f_k/\gamma_m$ γ_m (Table NA1) = 2.70

Bearing capacity, N_{Rdc} = $(1.2 + 0.4(a_1/h_c))f_dA_b$ but not more than $1.5f_dA_b$.

Edge distance, a_1 from edge of bearing = 600 mm

Height, h_c, of bearing base above floor = 2000 mm

Bearing capacity, N_{Rdc} = 1.32 f_dA_b < $1.5f_dA_b$. use 1.32 f_dA_b.

Bearing capacity, N_{Rdc} = 1.23 N/mm^2

Bearing size = 400 mm x 100 mm x 215 mm deep

$\dfrac{38.54 \times 10^3\ N}{40000\ mm^2}$ = 0.96 N/mm^2 < 1.23 N/mm^2 O.K.

Therefore, use a 400 mm x 100 mm x 215 mm dp padstone

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DESIGN OF STEEL MEMBER TO BS EN 1993-1-1 Ref = PFC Column A

Combined Axial Compression and Bending

Clear Height/length = 3000 mm.
Distance between restraints = 3000 mm.

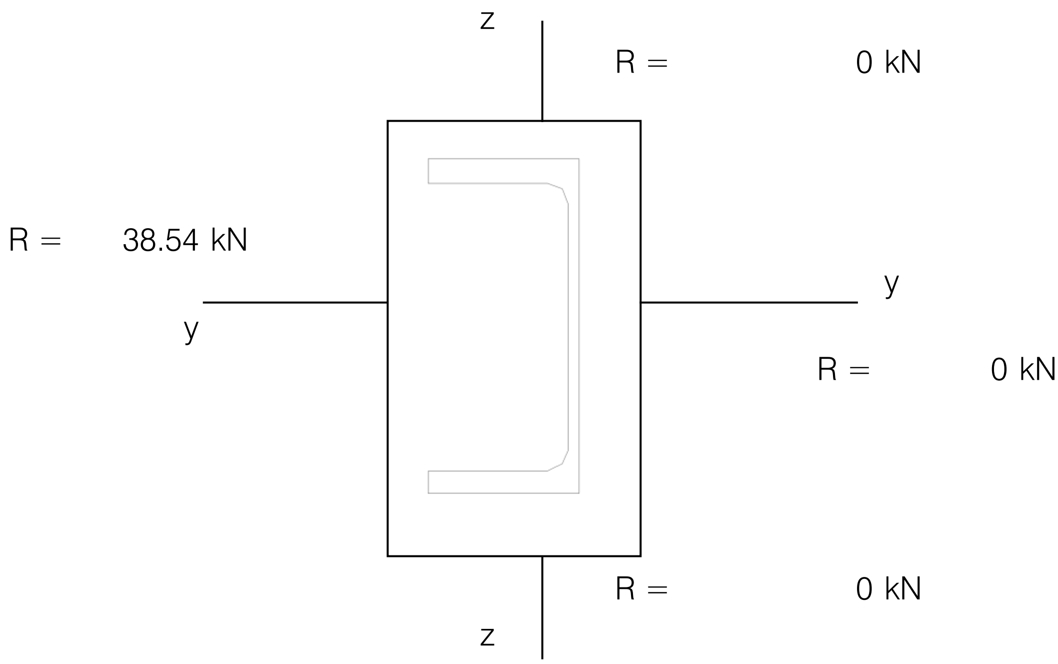
Consider column to be fully braced by surrounding structure.
Therefore consider nominal moment from eccentric connection only.

Try a section size **260x90x35 PFC** Grade of Steel = S355

Size of section =	depth, h =	260 mm	(about y-y direction(Major))
	width, b =	90 mm	(about z-z direction (Minor))

Ecc about y-y (Major) $= \frac{260}{2} + 100 =$ 230 mm = 0.23 m

Ecc about z-z (Minor) $= \frac{90}{2} + 100 =$ 145 mm = 0.15 m



Load Summary

$M_{y,Ed}$ (Major) = (	0.00	-	0.00	)	x	0.23 =	0.00 kNm
$M_{z,Ed}$ (Minor) = (	38.54	-	0.00	)	x	0.15 =	5.59 kNm
N_{Ed} Axial Load =	40 kN		Take minimum moment as				3 kNm

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DESIGN OF STEEL MEMBER TO BS EN 1993-1-1 Ref = PFC Column A

Combined Axial Compression and Bending

Design Data

Moment, $M_{y,Ed}$ = 3.00 kNm Moment, $M_{z,Ed}$ = 5.59 kNm
Axial Comp, $N_{b,Ed}$ = 39.92 kN
Length in y plane = 3000 mm Length in z plane = 3000 mm

Section size = 260x90x35 PFC Flange thickness, t_f= 14.00 mm
Web thickness, t_w= 8.00 mm Yield Strength, f_y = 355 N/mm^2 *(Table 3.1)*
Depth of Web, C = 208.00 mm A = 4440 mm^2 S/W = 0.34 kN/m
$W_{pl,y}$ = 425000 mm^3 $W_{el,y}$ = 364000 mm^3 γ_{M0} = 1
I_{y}= 4.7E+07 mm^4 I_{z}= 3.5E+06 mm^4 E = 210000 N/mm^2

Classification of Section for Bending & Compression ε = 0.81

$$\alpha = \frac{1}{2}\left(1 + \frac{F_c}{f_y\, t_w\, d}\right) = 0.53 > 0.50$$

$$\psi = \frac{2F_c}{A\, f_y} - 1 = -0.95 > -1$$

Web = $c/t = \frac{208.00}{8.00} = 26.00$ mm

Flange = $c = \frac{90.00 - 24.00 - 8.00}{2} = 29$ mm

$c/t = \frac{29.00}{14.00} = 2.07$ mm

(From table 5.2, for rolled sections)

Web	Class 1	a>0.5	limiting =	54.25 mm	Therefore	Class 1	a>0.5	Suitable
Flange	Class 1	a>0.5	limiting =	54.25 mm	Therefore	Class 1	a>0.5	Suitable

Therefore, section for design take as a Class 1 Section and use Plastic Design

Therefore use Wpl,y = 425000 mm^3

DESIGN OF STEEL MEMBER TO BS EN 1993-1-1 Ref = PFC Column A

Check Buckling of Member in Compression

Elastic critical Force, $N_{cr} = \dfrac{\pi^2 E I}{L_{cr}^{\ 2}}$ $N_{cr,y} =$ 10877.11 kN
$N_{cr,z} =$ 812.10 kN

Non-dimensional slenderness, $\left(\dfrac{A f_y}{N_{cr}}\right)^{0.5}$ $\lambda_y =$ 0.381
$\lambda_z =$ 1.393

Reduction factor, $\chi = \dfrac{1}{\phi + (\phi^2 - \lambda^2)^{0.5}}$ but $\chi_{LT} < 1.0$ *6.3.1.2*

$\dfrac{h}{b} = \dfrac{260.00}{90.00} =$ 2.89 > 1.2 $t_f =$ 14.00 mm

about y-y, use buck. curve a *(Table 6.2)* Imperfection factor, $\alpha_y =$ 0.21
about z-z, use buck. curve b *(Table 6.2)* Imperfection factor, $\alpha_z =$ 0.34

$\phi_y =$ 0.5 $(1 + \alpha(\lambda - 0.2) + \lambda^2) =$ 0.59 $\chi_y =$ 0.958
$\phi_z =$ 0.5 $(1 + \alpha(\lambda - 0.2) + \lambda^2) =$ 1.67 $\chi_z =$ 0.385

$\chi_{min} =$ 0.385 (Max 1.0)

Design buckling Resistance, $N_{b,Rd} = \chi A f_y / \gamma_{M1} =$ 606.21 kN *6.3.1.1*

Therefore 606.21 > 39.92 kN O.K.

Moment Resistance, $M_{z,Rd}$

As member is Class 1 use Plastic modulus.
$W_{pl,z} =$ 102000 mm^3 $W_{el,z} =$ 56300 mm^3 $\gamma_{M0} =$ 1

$M_{z,Rd} = \dfrac{102000 \times 355}{1} =$ 36.21 kNm *(Cl 6.2.5)*

As 36.21 > 5.59 kNm Moment resistance of section O.K.

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DESIGN OF STEEL MEMBER TO BS EN 1993-1-1 — Ref = PFC Column A

Lateral Torsional Buckling

$\lambda_{LT} =$ (Conservative for Column design) = $0.9\,\lambda_z$ — $i_z =$ 28.2 mm

$\lambda_z = \dfrac{L_{cr} / i_z}{\lambda_1}$ — $\lambda_1 = 76.37$ — $\lambda_{LT} = 1.25$ — For rolled profiles, $\lambda_{LT0} = 0.4$

$\dfrac{h}{b} = \dfrac{260.00}{90.00} = 2.89 > 2$ use buck. curve **d** *(Table 6.5)*

Imperfection factor, $\alpha_{LT} = 0.76$ — $\beta = 0.75$

$\phi_{LT} = 0.5\ (1 + \alpha_{LT}\,(\lambda_{LT} - \lambda_{LT0}) + \beta\lambda_{LT}^{2}) = 1.41$

Reduction factor, $\chi_{LT} = \dfrac{1}{\phi_{LT} + (\phi_{LT}^{2} - \beta\lambda_{LT}^{2})^{0.5}}$ but $\chi_{LT} < 1.0$ and $\chi_{LT} < \dfrac{1}{\lambda_{LT}^{2}}$

$\chi_{LT} = 0.43$

0.43 < 1 — 0.43 < 0.64 **O.K.**

Conservatively, take factor, f = 1 — $\chi_{LT,mod} = 0.43$ — $\gamma_{M1} = 1.00$

$M_{b,Rd} = \chi_{LT,mod}\, W_y\, f_y / \gamma_{M1} =$ **65.05 kNm**

As 65.05 > 3.00 kNm — Lateral Torsional Buckling resistance **O.K.**

Combined bending and compression buckling

Simplified version of equations 6.61 & 6.62:-

$$\frac{N_{ed}}{N_{b,z,Rd}} + \frac{M_{y,Ed}}{M_{b,Rd}} + 1.5\,\frac{M_{z,Ed}}{M_{z,Rd}} < 1$$

$$\frac{39.92}{606.21} + \frac{3.00}{65.05} + 1.5\,\frac{5.59}{36.21} = 0.34$$ **O.K.**

Therefore use a 260x90x35 PFC S355 section

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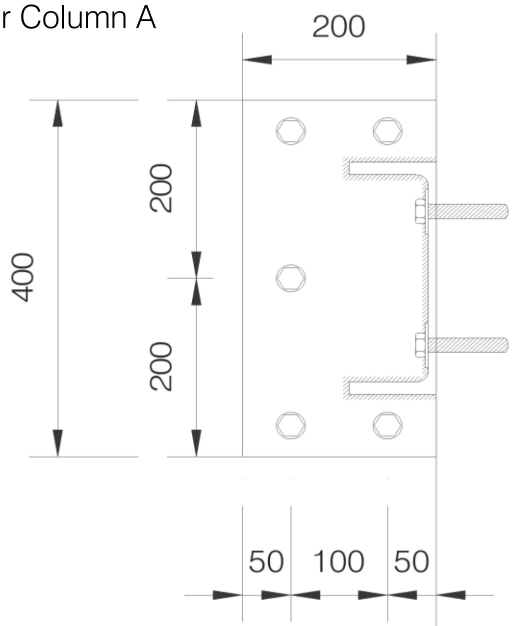
BASEPLATE DESIGN TO BS EN 1993

Reference:- Baseplate for Column A



Axial Load, N_{ed} = 39.92 kN (ULS) Horizontal Load = 10.00 kN

Try a 400 mm (D) x 200 mm(B) x 15 mm thick baseplate

Grade = S275 Type = Thickness up to 16mm f_y = 275 N/mm^2

Concrete Grade Below = 16 / 20

Section Size = 260x90x35 PFC

Area Column = 4440 mm^2 Perimeter column = 854 mm
Flange Thickness, t_f = 14.0 mm
Web thickness,t = 8.0 mm h = 260.0 mm b = 90.0 mm

Area of baseplate = 80000 mm^2

Bearing strength of concrete; f_{jd} = 0.67 x f_{ck} = 10.72 N/mm^2

Minimum Outstand, c = $0.50 \left(\frac{Ned}{2bf_{jd}} - t_f \right)$ = 3.35 mm

Actual outstand distance = $\frac{200 - 90.0}{2}$ = 55.00 mm O.K.

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BASEPLATE DESIGN TO BS EN 1993

Reference:- Baseplate for Column A

Minimum thickness, t_b = $c \left(\frac{3 f_{jd}}{f_y} \right)^{0.5}$ = 1.14 mm

but not less, than tf = 14.0 mm

Minimum thickness, t_b = 14.0 mm

Therefore 15 mm plate is O.K.

Check to Welds

Shear Load = 10.00 kN

Grade of Steel = S275 EN10025-2 Thickness of material = t<40mm

Simplified design from BS EN 1993:-

f_u = 430 N/mm^2 β = 0.85 γ_{M2} = 1.25

Design shear Strength, $f_{vw,d} = \frac{f_u / (3)^{0.5}}{\beta_w \gamma_{M2}}$ $f_{vw,d}$ = 233.66 N/mm^2

Try a fillet weld, leg size = 6 mm a = 4.2 mm

Design resistance of weld = 0.981 kN/mm

Effective length of welds = 520 mm Weld Resistance = 510.31 kN

As 510.31 > 10.00 kN Weld is O.K.

Summary:- Baseplate for Column A

Therefore use 400 mm x 200 mm x 15 mm thick baseplate

Grade = S275 Assume concrete below = C 16 / 20

Use a 6 mm thick fillet weld Minimum length = 520 mm

Holding down bolts , by inspection use 4 No. Hilti HY-200 HIT RODS M16

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PAD FOOTING DESIGN (CENTRALLY LOADED) Ref:- Pad under Column A

Assume 100 kN/m^2 allowable bearing pressure (to be checked on site)

Pressure = $\frac{P}{A}$

Worst Case Axial load =	39.92 kN
Worst Case Unfactored Axial Load =	26.62 kN
Additional Load =	0.00 kN

Pad Size = 1.00 m dp x 0.60 m wd x 0.60 m wide.

Take self weight relieved by overburden pressure = 6.00 kN/m^2

P = 2.16 + 26.62 + 0.00 = 28.78 kN

A = 0.36 m^2

Therefore:-

$$\frac{28.78}{0.36} = 79.93\ kN/m^2 < 100\ kN/m^2$$

Therefore Pad is O.K.

Use a 1.00 m dp x 0.60 m wd x 0.60 m wide pad footing

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SIMPLE BEAM ANALYSIS TO BS EN 1990

Ref = Steel Beam B

Actions

	Permanent Action, G_k (kN/m^2)	Leading Variable Action, $Q_{k\,1}$ (kN/m^2)	Variable Action, $Q_{k\,i}$ (kN/m^2)	Dist		Permanent Action, G_k (kN/m)	Variable Action, $Q_{k\,1}$ (kN/m)	Variable Action, $Q_{k\,i}$ (kN/m)
Cavity Wall	3.64			height	1.00	3.64	0.00	0.00
Bi Fold Doors	0.45			height	2.50	1.13	0.00	0.00
Pitched Roof	1.07		0.52	on slope	4.00	4.28	0.00	2.08
Flat Roof	0.65	0.60		3.6/2=	1.80	1.17	1.08	0.00
Loft	0.35	0.35		7/2=	3.50	1.22	1.23	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
Self weight of beam	Using a	203x133x30				0.29		
				TOTALS:		*11.73*	*2.31*	*2.08*

Clear Span = 4047 mm. Bearings = 100 mm minimum each side
Design span = **4147 mm** ***(NB - Design span only - not for setting out on site)***
Span between restraints = 4147 mm
Conservatively, use equation 6.10 = $1.35\ G_k + 1.5\ Q_{k\,1} + \Sigma\ \psi_{0\,i}\ 1.5\ Q_{k\,i}$
Conservatively, take $\psi_{0\,i}$ = 1.00
Design UDL= 15.83 + 3.46 + 3.11769 = 22.41 kN/m
Unfactored UDL = 16.11 kN/m
Design Point Load 1= 0.00 kN
Distance from A to point load 1= 0 mm
Distance from B to point load 1= 4147 mm
Design Point Load 2= 0.00 kN
Distance from A to point load 2= 0 mm
Distance from B to point load 2= 4147 mm
Design Point Load 3= 0.00 kN
Distance from A to point load 3= 0 mm
Distance from B to point load 3= 4147 mm
Design Point Load 4= 0.00 kN
Distance from A to point load 4= 0 mm
Distance from B to point load 4= 4147 mm

Beam analysed with simple supports:-

Design Moment = 48.16 kNm ULS
Shear Force At A = 46.46 kN ULS Shear Force At B = 46.46 kN

Design Moment =	**48.16**	kNm
Design. Shear A =	**46.46**	kN
Design. Shear B =	**46.46**	kN

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DESIGN OF STEEL MEMBER TO BS EN 1993-1-1 — Ref = Steel Beam B

Design Data

From previous calculation:- Moment, $M_{y,Ed}$ = 48.16 kNm

(Beam in pure bending only - no axial forces) Design Shear = 46.46 kN

Try a section Size =

1 No. 203x133x30 — Design Span = 4147 mm

Between Restraints = 4147 mm

Grade of Steel = S355 — Flange thickness, t_f = 9.60 mm

Web thickness, t_w = 6.40 mm — Yield Strength, f_y = 355 N/mm^2 *(Table 3.1)*

Classification of Section

Web = $d/t = \frac{172.40}{6.40}$ = 26.94 mm — ε = 0.81

Flange = $c = \frac{133.90 - 15.20 - 6.40}{2}$ = 56.15 mm

$c/t = \frac{56.15}{9.60}$ = 5.85 mm

(From table 5.2, for rolled Sections2)

web Class 1 limiting = 72 ε = 58.58 mm Therefore Class 1 Suitable

flange Class 1 limiting = 9 ε = 7.32 mm Therefore Class 1 Suitable

Moment Resistance As member is Class 1 use Plastic modulus.

$W_{pl,y}$ = 314000 mm^3 — $W_{el,y}$ = 280000 mm^3 — γ_{M0} = 1

$M_{c,Rd} = \frac{314000 \times 355}{1}$ = 111.47 kNm *(Cl 6.2.5)*

As 111.47 > 48.16 kNm — Moment resistance of section O.K.

Shear Resistance Root radius, r = 7.60 mm — A = 3820 mm^2

$A_{v,z} = A - 2 b t_f + (t_w + 2r) t$ — $A_{v,z}$ = 1456 mm^2 — $\eta h_w t_w$ = 1103 mm^2

$V_{pl,z,Rd} = \frac{A_{v,z} (f_y / ((3)^{0.5}))}{\gamma_{M0}}$ = 298.52 kN

Combined bending/shear not considered as max. moment away from max. shear.

As 298.52 > 46.46 kN — Shear resistance of section O.K.

DESIGN OF STEEL MEMBER TO BS EN 1993-1-1   Ref = Steel Beam B

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Calculation of Mcrit

1. Destabilising Loads

$$M_{cr} = C_1 \frac{\pi^2 EI_z}{(kL)^2}\left\{\sqrt{\left(\frac{k}{k_w}\right)^2 \frac{I_w}{I_z} + \frac{(kL)^2 GI_t}{\pi^2 EI_z} + (C_2 z_g)^2} - C_2 z_g\right\}$$

E= 210000 N/mm^2 G= 80770 N/mm^2

Loading and support condition:- Simply supported, UDL over whole span

Factors based on bending moment:- C1= 1.127   C2= 0.454   C3= 1.0

$I_{w=}$	3.7E+10	mm^6	Warping constant from section property tables.
$I_{z=}$	3850000	mm^4	Minor second moment of area from section property tables.
$I_{t=}$	103000	mm^4	Torsion constant from section property tables
$k_{w=}$	1.0		Unless special provision for warping fixity is made, k_w should be taken as 1.0
$k_=$	1.2		Effective Length constant
$z_{g=}$	103.4	mm	Height of the destabilising load above the shear centre.
$z_{j=}$	0.0	mm	0 mm for bi-symmetric Sections2

$$M_{cr} = 362771 \times \left(\left(13839 + 25845 + 2204\right)^{0.5} - 47\right)$$

M_{cr}= 57.2 kNm   ***(Destabilising Load)***

2. Non-destabilising Loads

$$M_{cr} = C_1 \frac{\pi^2 EI_z}{L^2}\sqrt{\frac{I_w}{I_z} + \frac{L^2 GI_t}{\pi^2 EI_z}}$$

$$M_{cr} = 1 \times 463523 \times \left(27558\right)^{0.5}$$

M_{cr}= 86.7 kNm   ***(Non-destabilising Load)***

For rolled profiles, λ_{LT0} = 0.4

Consider load to be Destabilising   therefore use, M_{cr}= 57.22 kNm

W_y = 314000 mm^3

Non-dimensional slenderness $\lambda_{LT} = \left(\frac{W_y f_y}{M_{cr}}\right)^{0.5}$ = 1.396

As, 1.396 > 0.4   LTB verification is   Required

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DESIGN OF STEEL MEMBER TO BS EN 1993-1-1 Ref = Steel Beam B

Lateral Torsional Buckling (Cont)

$\frac{h}{b} = \frac{206.80}{133.90} = 1.54 < 2$ use buck. curve b *(Table 6.5)*

Imperfection factor, α_{LT} = 0.34 β = 0.75

$\phi_{LT} = 0.5\,(1 + \alpha_{LT}\,(\lambda_{LT} - \lambda_{LT0}) + \beta\lambda_{LT}^2) = 1.40$

Reduction factor, $\chi_{LT} = \frac{1}{\phi_{LT} + (\phi_{LT}^2 - \beta\lambda_{LT}^2)^{0.5}}$ but $\chi_{LT} < 1.0$ and $\chi_{LT} < \frac{1}{\lambda_{LT}^2}$

χ_{LT} = 0.47 0.47 < 1 0.47 < 0.51

Conservatively, factor, f = 1 $\chi_{LT,mod}$ = 0.47 γ_{M1} = 1.00

$M_{b,Rd} = \chi_{LT,mod} W_y f_y / \gamma_{M1}$ = **52.93 kNm**

As 52.93 > 48.16 kNm Lateral Torsional Buckling resistance O.K.

Deflection Checks Take average partial safety factors as 1.425

Equivalent unfactored SLS udl to beam = $\frac{33.80 \times 8}{4.15 \times 4.15}$ = 15.72 kN/m

I_y = 2.9E+07 mm^4 Major second moment of area from section property tables.

Deflection = 9.94 mm Deflection limits = $\frac{Span}{360}$ = 11.52 mm
Finishes = Brittle

As 9.94 < 11.52 mm Deflection due to SLS loading is O.K.

Frequency Checks
By inspection, frequency checks not required due to low deflections.

Therefore use	1 No.	203x133x30	S355	section

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BEARING DESIGN TO BS EN 1996-1-1

Reference = Steel Beam B

Bearing at End "A" Factored Reaction = 46.46 kN ULS
$f_k = K f_b^{\alpha} f_m^{\beta}$ Masonry under padstone Aggregate Concrete blocks Group 1
Unit size below = 215 mm h x 100 mm width Shape Factor, δ = 1.38
Act. Comp. Str = 3.50 N/mm^2 Norm. Mean Comp. Strength, f_b = 4.83 N/mm^2
From National Annexe, Table NA4, K = 0.55 α = 0.7 β = 0.30
Mortar Designation = (iii) Mortar Type M 4 f_m = 4.00 N/mm^2
f_k = 2.51 N/mm^2 $f_d = f_k/\gamma_m$ γ_m (Table NA1) = 2.70
Bearing capacity, N_{Rdc} = $(1.2 + 0.4(a_1/h_c))f_dA_b$ but not more than $1.5f_dA_b$.
Edge distance, a_1 from edge of bearing = 1300 mm
Height, h_c, of bearing base above floor = 2000 mm
Bearing capacity, N_{Rdc} = 1.46 f_dA_b < $1.5f_dA_b$. use 1.46 f_dA_b.
Bearing capacity, N_{Rdc} = 1.36 N/mm^2
Bearing size = 400 mm x 100 mm x 215 mm deep

$$\frac{46.46 \times 10^3\ N}{40000\ mm^2} = 1.16\ N/mm^2 < 1.36\ N/mm^2$$ **O.K.**

Therefore, use a 400 mm x 100 mm x 215 mm dp padstone

Bearing at End "B" Factored Reaction = 46.46 kN ULS
$f_k = K f_b^{\alpha} f_m^{\beta}$ Masonry under padstone Aggregate Concrete blocks Group 1
Unit size below = 215 mm h x 100 mm width Shape Factor, δ = 1.38
Act. Comp. Str = 3.50 N/mm^2 Norm. Mean Comp. Strength, f_b = 4.83 N/mm^2
From National Annexe, Table NA4, K = 0.55 α = 0.7 β = 0.30
Mortar Designation = (iii) Mortar Type M 4 f_m = 4.00 N/mm^2
f_k = 2.51 N/mm^2 $f_d = f_k/\gamma_m$ γ_m (Table NA1) = 2.70
Bearing capacity, N_{Rdc} = $(1.2 + 0.4(a_1/h_c))f_dA_b$ but not more than $1.5f_dA_b$.
Edge distance, a_1 from edge of bearing = 1300 mm
Height, h_c, of bearing base above floor = 2000 mm
Bearing capacity, N_{Rdc} = 1.46 f_dA_b < $1.5f_dA_b$. use 1.46 f_dA_b.
Bearing capacity, N_{Rdc} = 1.36 N/mm^2
Bearing size = 200 mm x 100 mm x 215 mm deep

$$\frac{46.46 \times 10^3\ N}{40000\ mm^2} = 1.16\ N/mm^2 < 1.36\ N/mm^2$$ **O.K.**

Therefore, use a 200 mm x 100 mm x 215 mm dp padstone

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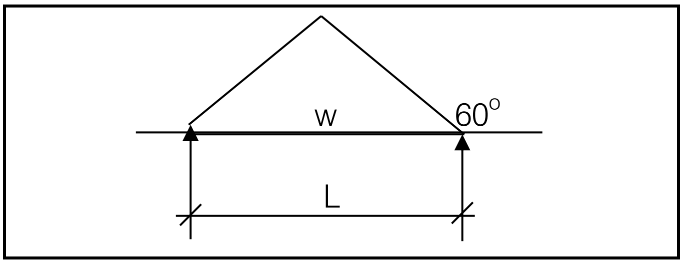
Title of Scheme *Lemsford Village Hall, Lemsford, Welwyn Garden City* Job No. *20655*

LINTEL DESIGN TO BS 5977 Location = Lintel 1

Length, L = 1585 mm = 1.585 m



External Wall Load =	3.64 kN/m²	Roof Dead Load =	0.65 kN/m²
Floor Dead Load =	0.00 kN/m²	Roof Live Load =	0.60 kN/m²
Floor Live Load =	0.00 kN/m²	Loft Dead Load =	0.00 kN/m²
Unfactored Point Load =	0.00 kN	Loft Dead Load =	0.00 kN/m²

Span of Floor Loading =		m	Floor Load =	0.00 kN/m
Span of Roof Loading =	5.384/2=	2.69 m	Roof Load =	3.36 kN/m
Span of Loft Loading =		m	Loft Load =	0.00 kN/m

Check Bearings - Span x 0.2 = 317 mm or 600mm <u>O.K.</u>

Actual Bearing at End 1 = 150 mm
Actual Bearing at End 2 = 150 mm
Length of Loading = 1.1 x 1.585 = 1.74 m

Wall Load = 3.64 x 1.51 x 1.74 = 9.58 kN

Floor Load =	0.00 kN	Loft Load =	0.00 kN
Roof Load =	5.86 kN	Point Load =	0.00 kN

Total Load on Lintel (Unfactored) = 15.44 kN

Try a Catnic Lintel, Reference:- CG90/100 Length = 1.885 m
Safe Working Load = 18.00 kN *(Taken from Manufacturers Literature)*

N.B. - Proposed Cavity Size to be confirmed before ordering of Lintel as Reference may change.

<u>As 18.00 > 15.44 kN, Lintel is O.K.</u>

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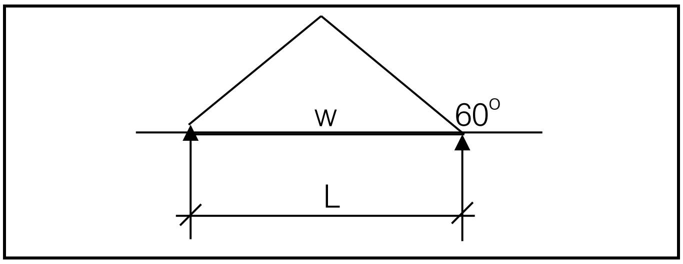
Title of Scheme	*Lemsford Village Hall, Lemsford, Welwyn Garden City*	Job No.	*20655*

<u>LINTEL DESIGN TO BS 5977</u> Location = Lintel 2

Length, L = 1510 mm = 1.51 m



External Wall Load =	3.64 kN/m^2	Roof Dead Load =	1.07 kN/m^2
Flat Roof Dead Load =	0.65 kN/m^2	Roof Live Load =	0.52 kN/m^2
Flat Roof Live Load =	0.60 kN/m^2	Loft Dead Load =	0.35 kN/m^2
Unfactored Point Load =	3.86 kN	Loft Dead Load =	0.35 kN/m^2

Span of Flat roof Loading =	5.4/2=	2.70 m	Floor Load =	3.37 kN/m
Span of Roof Loading =	4/2=	2.00 m	Roof Load =	3.18 kN/m
Span of Loft Loading =	6.7/2=	3.20 m	Loft Load =	2.24 kN/m

Check Bearings - Span x 0.2 = 302 mm or 600mm <u>O.K.</u>

Actual Bearing at End 1 = 150 mm
Actual Bearing at End 2 = 150 mm
Length of Loading = 1.1 x 1.51 = 1.66 m

Wall Load = 3.64 x 1.44 x 1.66 = 8.69 kN

Floor Load =	5.60 kN	Loft Load =	3.72 kN
Roof Load =	5.28 kN	Point Load =	3.86 kN

Total Load on Lintel (Unfactored) = 27.16 kN

Try a Catnic Lintel, Reference:- CH90/100 Length = 1.81 m
Safe Working Load = 32.00 kN *(Taken from Manufacturers Literature)*

N.B. - Proposed Cavity Size to be confirmed before ordering of Lintel as Reference may change.

<u>As 32.00 > 27.16 kN, Lintel is O.K.</u>

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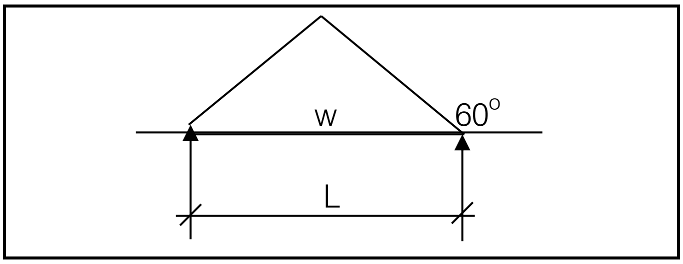
__LINTEL DESIGN TO BS 5977__ Location = Lintel 3

Length, L = 1980 mm = 1.98 m



External Wall Load =	3.64 kN/m^2	Roof Dead Load =	0.00 kN/m^2
Flat Roof Dead Load =	0.00 kN/m^2	Roof Live Load =	0.00 kN/m^2
Flat Roof Live Load =	0.00 kN/m^2	Loft Dead Load =	0.00 kN/m^2
Unfactored Point Load =	0.00 kN	Loft Dead Load =	0.00 kN/m^2

Span of Flat roof Loading =	m	Floor Load =	0.00 kN/m
Span of Roof Loading =	m	Roof Load =	0.00 kN/m
Span of Loft Loading =	m	Loft Load =	0.00 kN/m

Check Bearings - Span x 0.2 = 396 mm or 600mm __O.K.__

Actual Bearing at End 1 = 150 mm
Actual Bearing at End 2 = 150 mm
Length of Loading = 1.1 x 1.98 = 2.18 m

Wall Load = 3.64 x 1.89 x 2.18 = 14.95 kN

Floor Load =	0.00 kN	Loft Load =	0.00 kN
Roof Load =	0.00 kN	Point Load =	0.00 kN

Total Load on Lintel (Unfactored) = 14.95 kN

Try a Catnic Lintel, Reference:- CG90/100 Length = 2.28 m
Safe Working Load = 20.00 kN *(Taken from Manufacturers Literature)*

N.B. - Proposed Cavity Size to be confirmed before ordering of Lintel as Reference may change.

__As 20.00 > 14.95 kN, Lintel is O.K.__

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__TIMBER POST TO BS EN 1995__

Post Reference = Ashlar Stud.

Consider studs held at top and bottom.
Take Studs as simply supported. Centres = 400 mm c/c

Wind Loading = 0.00 kN/m^2 (Estimated) γ_f = 1.5

Horizontal Load per post = 0.00 kN/m Vertical Span = 0.8 m

Moment = 0.00 kNm per post.

Axial Load =

	Perm Load (kN/m^2)	Var. Load (kN/m^2)	Distance(m)		Dead (kN/m)	Live (kN/m)
Pitched Roof	1.07	0.52	on slope	3.05	3.26	1.58
					0.00	0.00
					0.00	0.00
					0.00	0.00
				TOTALS:	*3.26*	*1.58*

γ_f = 1.35 Perm 1.5 Variable ULS load = 6.78 kN/m

Axial Load = 6.78 kN/m x 0.4 = 2.71 kN ULS

Therefore, design post with following loads:-

Major moment, My = 0.00 kNm ULS
Minor moment, Mz = 0 kNm ULS
Axial Compresion = 2.71 kNN ULS

See design over

__*TIMBER POST TO BS EN 1995*__

Post Reference = Ashlar Stud.

From previous calculation :-	Major moment, My =	0.00	kNm ULS
	Minor moment, Mz =	0	kNm ULS
	Axial Compresion =	2.71	kNN ULS

Design span = 0.80 m = 800 mm

Exposure condition = Service Class 2

(Ground Floors, Cold Roofs, external timber frame walls)

Load Duration = Medium Term *(Imposed Floor Loading)*

__*Design*__ Timber Grade = C24 Material = Solid Timber. Treated or untreated

Bending Strength, $f_{m,k}$ = 24.0 N/mm^2

Compressive Strength, $f_{c,0,k}$ = 21.0 N/mm^2

Try 1 No. 100 deep x 50 wide post

Type of fire exposure = Not Exposed

Charring rate = General Rate = 0.60 mm/min

Required fire rating = 30 minutes

Effective size for design = 100 mm deep x 50 mm wide post

Elastic Modulus , W_{yy} = 8.3E+04 mm^3. K_h Factor = 1.08

Elastic Modulus , W_{zz} = 4.2E+04 mm^3. K_h Factor = 1.25

Partial Safety factor, γ_m = 1.3 Type of Beam= Load Sharing k_{sys} = 1.1

k_{mod} = 0.8 k_{crit} = 1 (No LTB due to boarding)

__*Design Strengths*__

Design bending strength $f_{my,d}$ =	$k_{mod} \cdot k_h \cdot k_{crit} \cdot k_{sys} \cdot f_{m,k} / \gamma_m$ =	17.62	N/mm^2
Design bending strength $f_{mz,d}$ =	$k_{mod} \cdot k_h \cdot k_{crit} \cdot k_{sys} \cdot f_{m,k} / \gamma_m$ =	20.24	N/mm^2
Design compressive strength of timber, $f_{c,0,d}$=	$k_{mod} f_{c,0,}k / \gamma_m$ =	12.92	N/mm^2

__*Design Stresses*__

Design Compressive Stress, $\sigma_{c,0,d}$ =	F/A =	0.54	N/mm^2
Design Bending Stress in y-y, $\sigma_{m,y,d}$ =	M_y/W_{yy} =	0.00	N/mm^2
Design Bending Stress in z-z, $\sigma_{m,z,d}$ =	M_z/W_{zz} =	0.00	N/mm^2

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TIMBER POST TO BS EN 1995

Post Reference = Ashlar Stud.

Buckling about the z-z axis — Consider 0.5L as fully restrained by sheathing board

Radius of gyration, i_z = 14.43 mm

Eff. Length = 0.5 L = 400 mm λ_z = 27.71 Factor= 0.01696

$\lambda_{rel,z}$ = 0.47 > 0.30 *(Slender)*

Therefore, use

$$\frac{\sigma_{c,o,d}}{k_{c,z}\, f_{c,0,d}} + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} < 1$$

β_c = 0.20 Solid Timber. Treated or untreated

$k_z = 0.5\,(1 + \beta_c\,(\lambda_{rel,z} - 0.3) + \lambda_{rel,z}^{2})$ = 0.63 k_m = 0.7

$k_{c,z} = 1 / k_z + (k_z^{2} - \lambda_{rel,z}^{2})^{0.5}$ = 0.96

$$\frac{0.54}{12.39} + 0.70\,\frac{0.00}{17.62} + \frac{0.00}{20.24} = 0.04 < 1$$

O.K.

Buckling about the y-y axis

Radius of gyration, i_y = 28.87 mm

Eff. Length = 0.5 L = 400 mm λ_y = 13.86 Factor= 0.01696

$\lambda_{rel,y}$ = 0.23 < 0.30 *(Stocky - use different formula!)*

Therefore, use

$$\frac{\sigma_{c,o,d}}{k_{c,y}\, f_{c,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} < 1$$

β_c = 0.20 Solid Timber. Treated or untreated

$k_y = 0.5\,(1 + \beta_c\,(\lambda_{rel,y} - 0.3) + \lambda_{rel,y}^{2})$ = 0.52 k_m = 0.7

$k_{c,y} = 1 / k_y + (k_y^{2} - \lambda_{rel,y}^{2})^{0.5}$ = 1.01

$$\frac{0.54}{13.10} + \frac{0.00}{17.62} + 0.70\,\frac{0.00}{20.24} = 0.04 < 1$$

O.K.

The post is therefore adequate at ULS for direct compression and buckling about both axes.
Therefore, use

1 No. 100 x 50 Joist/s, C24 Grade Timber

Centres = 400 mm

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SIMPLE BEAM ANALYSIS TO BS EN 1990

Ref = Steel Beam C
Large Span

Actions

	Permanent Action, G_k (kN/m^2)	Leading Variable Action, $Q_{k\,1}$ (kN/m^2)	Variable Action, $Q_{k\,i}$ (kN/m^2)	Dist		Permanent Action, G_k (kN/m)	Variable Action, $Q_{k\,1}$ (kN/m)	Variable Action, $Q_{k\,i}$ (kN/m)
Pitched roof	1.07		0.52	on slope	3.80	4.06	0.00	1.97
Ashlar Wall	0.55			height	1.00	0.55	0.00	0.00
Ceiling Joists	0.35	0.35		6.8/2=	3.40	1.19	1.19	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
Self weight of beam	Using a	203x102x23				0.23		
					TOTALS:	*6.03*	*1.19*	*1.97*

Clear Span = 3930 mm. Bearings = 100 mm minimum each side
Design span = **4030 mm** *(NB - Design span only - not for setting out on site)*
Span between restraints = 4030 mm
Conservatively, use equation 6.10 = $1.35\ G_k + 1.5\ Q_{k\,1} + \Sigma\ \psi_{0\,i}\ 1.5\ Q_{k\,i}$
Conservatively, take $\psi_{0\,i}$ = 1.00
Design UDL= 8.13 + 1.79 + 2.96181 = 12.88 kN/m
Unfactored UDL = 9.19 kN/m
Design Point Load 1= 0.00 kN
Distance from A to point load 1= 0 mm
Distance from B to point load 1= 4030 mm
Design Point Load 2= 0.00 kN
Distance from A to point load 2= 0 mm
Distance from B to point load 2= 4030 mm
Design Point Load 3= 0.00 kN
Distance from A to point load 3= 0 mm
Distance from B to point load 3= 4030 mm
Design Point Load 4= 0.00 kN
Distance from A to point load 4= 0 mm
Distance from B to point load 4= 4030 mm

Beam analysed with simple supports:-

Design Moment = 26.15 kNm ULS
Shear Force At A = 25.96 kN ULS Shear Force At B = 25.96 kN

Design Moment =	**26.15**	kNm
Design. Shear A =	**25.96**	kN
Design. Shear B =	**25.96**	kN

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DESIGN OF STEEL MEMBER TO BS EN 1993-1-1 — Ref = Steel Beam C Large Span

Design Data

From previous calculation:- Moment, $M_{y,Ed}$ = 26.15 kNm

(Beam in pure bending only - no axial forces) Design Shear = 25.96 kN

Try a section Size =

1 No. 203x102x23 — Design Span = 4030 mm

Between Restraints = 4030 mm

Grade of Steel = S355 — Flange thickness, t_f= 9.30 mm

Web thickness, t_w= 5.40 mm — Yield Strength, f_y = 355 N/mm^2 *(Table 3.1)*

Classification of Section

Web = $d/t = \frac{169.40}{5.40} = 31.37$ mm — ε = 0.81

Flange = $c = \frac{101.80 - 15.20 - 5.40}{2} = 40.6$ mm

$c/t = \frac{40.60}{9.30} = 4.37$ mm

(From table 5.2, for rolled Sections2)

web	Class 1	limiting =	72 ε =	58.58 mm	Therefore	Class 1	Suitable
flange	Class 1	limiting =	9 ε =	7.32 mm	Therefore	Class 1	Suitable

Moment Resistance As member is Class 1 use Plastic modulus.

$W_{pl,y}$ = 234000 mm^3 — $W_{el,y}$ = 207000 mm^3 — γ_{M0} = 1

$M_{c,Rd} = \frac{234000 \times 355}{1} = 83.07$ kNm *(Cl 6.2.5)*

As 83.07 > 26.15 kNm — Moment resistance of section **O.K.**

Shear Resistance Root radius, r = 7.60 mm — A = 2940 mm^2

$A_{v,z}$ = A - 2 b t_f + (t_w + 2r) t — $A_{v,z}$ = 1238 mm^2 — $\eta\, h_w t_w$ = 915 mm^2

$V_{pl,z,Rd} = \frac{A_{v,z} (f_y / ((3)^{0.5}))}{\gamma_{M0}} = 253.76$ kN

Combined bending/shear not considered as max. moment away from max. shear.

As 253.76 > 25.96 kN — Shear resistance of section **O.K.**

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DESIGN OF STEEL MEMBER TO BS EN 1993-1-1 Ref = Steel Beam C Large Span

` `

Calculation of Mcrit

1. *Destabilising Loads*

$$M_{cr} = C_1 \frac{\pi^2 E I_z}{(kL)^2} \left\{ \sqrt{\left(\frac{k}{k_w}\right)^2 \frac{I_w}{I_z} + \frac{(kL)^2 G I_t}{\pi^2 E I_z} + (C_2 z_g)^2} - C_2 z_g \right\}$$

E= 210000 N/mm^2 G= 80770 N/mm^2

Loading and support condition:- Simply supported, UDL over whole span
Factors based on bending moment:- C1= 1.127 C2= 0.454 C3= 1.0

$I_{w=}$	1.5E+10	mm^6	Warping constant from section property tables.
$I_{z=}$	1640000	mm^4	Minor second moment of area from section property tables.
$I_{t=}$	70200	mm^4	Torsion constant from section property tables
$k_{w=}$	1.0		Unless special provision for warping fixity is made, k_w should be taken as 1.0
$k_=$	1.2		Effective Length constant
$z_{g=}$	101.6	mm	Height of the destabilising load above the shear centre.
$z_{j=}$	0.0	mm	0 mm for bi-symmetric Sections2

$$M_{cr} = 163634 \times \left(\left(13171 + 39051 + 2128 \right)^{0.5} - 46 \right)$$

M_{cr}= 30.6 kNm *(Destabilising Load)*

2. *Non-destabilising Loads*

$$M_{cr} = C_1 \frac{\pi^2 E I_z}{L^2} \sqrt{\frac{I_w}{I_z} + \frac{L^2 G I_t}{\pi^2 E I_z}}$$

$$M_{cr} = 1 \times 209080 \times \left(36265 \right)^{0.5}$$

M_{cr}= 44.9 kNm *(Non-destabilising Load)*

For rolled profiles, λ_{LT0} = 0.4

Consider load to be Destabilising therefore use, M_{cr}= 30.60 kNm
W_y = 234000 mm^3

Non-dimensional slenderness $\lambda_{LT} = \left(\frac{W_y f_y}{M_{cr}} \right)^{0.5}$ = 1.648

As, 1.648 > 0.4 LTB verification is Required

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DESIGN OF STEEL MEMBER TO BS EN 1993-1-1 Ref = Steel Beam C Large Span

Lateral Torsional Buckling (Cont)

$\frac{h}{b} = \frac{203.20}{101.80} = 2.00 < 2$ use buck. curve b *(Table 6.5)*

Imperfection factor, $\alpha_{LT} =$ 0.34 $\beta =$ 0.75

$\phi_{LT} = 0.5\ (1 + \alpha_{LT}\ (\lambda_{LT} - \lambda_{LT0}) + \beta\lambda_{LT}^2) = 1.73$

Reduction factor, $\chi_{LT} = \frac{1}{\phi_{LT} + (\phi_{LT}^2 - \beta\lambda_{LT}^2)^{0.5}}$ but $\chi_{LT} < 1.0$ and $\chi_{LT} < \frac{1}{\lambda_{LT}^2}$

$\chi_{LT} =$ 0.37 0.37 < 1 0.37 > 0.37

Conservatively, factor, f = 1 $\chi_{LT,mod} =$ 0.37 $\gamma_{M1} =$ 1.00

$M_{b,Rd} = \chi_{LT,mod} W_y f_y / \gamma_{M1} =$ 30.60 kNm

As 30.60 > 26.15 kNm Lateral Torsional Buckling resistance O.K.

Deflection Checks Take average partial safety factors as 1.425

Equivalent unfactored SLS udl to beam = $\frac{18.35 \times 8}{4.03 \times 4.03} =$ 9.04 kN/m

$I_{y=}$ 2.1E+07 mm^4 Major second moment of area from section property tables.

Deflection = 7.01 mm Deflection limits = $\frac{Span}{360} =$ 11.19 mm

Finishes = Brittle

As 7.01 < 11.19 mm Deflection due to SLS loading is O.K.

Frequency Checks

By inspection, frequency checks not required due to low deflections.

Therefore use	1 No.	203x102x23	S355	section

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SIMPLE BEAM ANALYSIS TO BS EN 1990

Ref = Steel Beam C
Short Span

Actions

	Permanent Action, G_k (kN/m^2)	Leading Variable Action, $Q_{k\,1}$ (kN/m^2)	Variable Action, $Q_{k\,i}$ (kN/m^2)	Dist		Permanent Action, G_k (kN/m)	Variable Action, $Q_{k\,1}$ (kN/m)	Variable Action, $Q_{k\,i}$ (kN/m)
Pitched roof	1.07		0.52	on slope	3.80	4.06	0.00	1.97
Ashlar Wall	0.55			height	1.00	0.55	0.00	0.00
Ceiling Joists	0.35	0.35		6.8/2=	3.40	1.19	1.19	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
Self weight of beam	Using a	203x102x23				0.23		
					TOTALS:	*6.03*	*1.19*	*1.97*

Clear Span = 1670 mm. Bearings = 100 mm minimum each side
Design span = **1770 mm** ***(NB - Design span only - not for setting out on site)***
Span between restraints = 1770 mm
Conservatively, use equation 6.10 = $1.35\ G_k + 1.5\ Q_{k\,1} + \Sigma\ \psi_{0\,i}\ 1.5\ Q_{k\,i}$
Conservatively, take $\psi_{0\,i}$ = 1.00
Design UDL= 8.13 + 1.79 + 2.96181 = 12.88 kN/m
Unfactored UDL = 9.19 kN/m
Design Point Load 1= 0.00 kN
Distance from A to point load 1= 0 mm
Distance from B to point load 1= 1770 mm
Design Point Load 2= 0.00 kN
Distance from A to point load 2= 0 mm
Distance from B to point load 2= 1770 mm
Design Point Load 3= 0.00 kN
Distance from A to point load 3= 0 mm
Distance from B to point load 3= 1770 mm
Design Point Load 4= 0.00 kN
Distance from A to point load 4= 0 mm
Distance from B to point load 4= 1770 mm

Beam analysed with simple supports:-

Design Moment = 5.04 kNm ULS
Shear Force At A = 11.40 kN ULS Shear Force At B = 11.40 kN

Design Moment =	**5.04**	kNm
Design. Shear A =	**11.40**	kN
Design. Shear B =	**11.40**	kN

DESIGN OF STEEL MEMBER TO BS EN 1993-1-1 — Ref = Steel Beam C Short Span

Design Data

From previous calculation:- Moment, $M_{y,Ed}$ = 5.04 kNm

(Beam in pure bending only - no axial forces) Design Shear = 11.40 kN

Try a section Size =

1 No. 203x102x23 — Design Span = 1770 mm

Between Restraints = 1770 mm

Grade of Steel = S355 — Flange thickness, t_f = 9.30 mm

Web thickness, t_w = 5.40 mm — Yield Strength, f_y = 355 N/mm^2 *(Table 3.1)*

Classification of Section

Web = $d/t = \frac{169.40}{5.40}$ = 31.37 mm — ε = 0.81

Flange = $c = \frac{101.80 - 15.20 - 5.40}{2}$ = 40.6 mm

$c/t = \frac{40.60}{9.30}$ = 4.37 mm

(From table 5.2, for rolled Sections2)

web	Class 1	limiting =	72 ε =	58.58 mm	Therefore	Class 1	Suitable
flange	Class 1	limiting =	9 ε =	7.32 mm	Therefore	Class 1	Suitable

Moment Resistance — As member is Class 1 use Plastic modulus.

$W_{pl,y}$ = 234000 mm^3 — $W_{el,y}$ = 207000 mm^3 — γ_{M0} = 1

$M_{c,Rd} = \frac{234000 \times 355}{1}$ = 83.07 kNm *(Cl 6.2.5)*

As 83.07 > 5.04 kNm — Moment resistance of section O.K.

Shear Resistance — Root radius, r = 7.60 mm — A = 2940 mm^2

$A_{v,z}$ = A - 2 b t_f + (t_w + 2r) t — $A_{v,z}$ = 1238 mm^2 — $\eta\, h_w\, t_w$ = 915 mm^2

$V_{pl,z,Rd} = \frac{A_{v,z} (f_y / ((3)^{0.5}))}{\gamma_{M0}}$ = 253.76 kN

Combined bending/shear not considered as max. moment away from max. shear.

As 253.76 > 11.40 kN — Shear resistance of section O.K.

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DESIGN OF STEEL MEMBER TO BS EN 1993-1-1 Ref = Steel Beam C Short Span

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Calculation of Mcrit

1. Destabilising Loads

$$M_{cr} = C_1 \frac{\pi^2 EI_z}{(kL)^2}\left\{\sqrt{\left(\frac{k}{k_w}\right)^2 \frac{I_w}{I_z} + \frac{(kL)^2 GI_t}{\pi^2 EI_z} + (C_2 z_g)^2} - C_2 z_g\right\}$$

E= 210000 N/mm^2 G= 80770 N/mm^2

Loading and support condition:- Simply supported, UDL over whole span

Factors based on bending moment:- C1= 1.127 C2= 0.454 C3= 1.0

$I_{w=}$	1.5E+10	mm^6	Warping constant from section property tables.
$I_{z=}$	1640000	mm^4	Minor second moment of area from section property tables.
$I_{t=}$	70200	mm^4	Torsion constant from section property tables
$k_{w=}$	1.0		Unless special provision for warping fixity is made, k_w should be taken as 1.0
$k_=$	1.2		Effective Length constant
$z_{g=}$	101.6	mm	Height of the destabilising load above the shear centre.
$z_{j=}$	0.0	mm	0 mm for bi-symmetric Sections2

$$M_{cr} = 848276 \times \left(\left(13171 + 7533 + 2128\right)^{0.5} - 46\right)$$

M_{cr}= **89.0 kNm** ***(Destabilising Load)***

2. Non-destabilising Loads

$$M_{cr} = C_1 \frac{\pi^2 EI_z}{L^2}\sqrt{\frac{I_w}{I_z} + \frac{L^2 GI_t}{\pi^2 EI_z}}$$

$$M_{cr} = 1 \times 1083867 \times \left(14378\right)^{0.5}$$

M_{cr}= **146.5 kNm** ***(Non-destabilising Load)***

For rolled profiles, λ_{LT0} = 0.4

Consider load to be Destabilising therefore use, M_{cr}= 89.05 kNm

W_y = 234000 mm^3

Non-dimensional slenderness $\lambda_{LT} = \left(\frac{W_y f_y}{M_{cr}}\right)^{0.5}$ = 0.966

As, 0.966 > 0.4 LTB verification is Required

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DESIGN OF STEEL MEMBER TO BS EN 1993-1-1 — Ref = Steel Beam C Short Span

Lateral Torsional Buckling (Cont)

$\frac{h}{b} = \frac{203.20}{101.80} = 2.00 < 2$ use buck. curve b *(Table 6.5)*

Imperfection factor, α_{LT} = 0.34 β = 0.75

$\phi_{LT} = 0.5\,(1 + \alpha_{LT}(\lambda_{LT} - \lambda_{LT0}) + \beta\lambda_{LT}^2)$ = 0.95

Reduction factor, $\chi_{LT} = \frac{1}{\phi_{LT} + (\phi_{LT}^2 - \beta\lambda_{LT}^2)^{0.5}}$ but $\chi_{LT} < 1.0$ and $\chi_{LT} < \frac{1}{\lambda_{LT}^2}$

χ_{LT} = 0.72 0.72 < 1 0.72 < 1.07

Conservatively, factor, f = 1 $\chi_{LT,mod}$ = 0.72 γ_{M1} = 1.00

$M_{b,Rd} = \chi_{LT,mod} W_y f_y / \gamma_{M1}$ = **59.85 kNm**

As 59.85 > 5.04 kNm Lateral Torsional Buckling resistance O.K.

Deflection Checks Take average partial safety factors as 1.425

Equivalent unfactored SLS udl to beam = $\frac{3.54 \times 8}{1.77 \times 1.77}$ = 9.04 kN/m

I_y = 2.1E+07 mm^4 Major second moment of area from section property tables.

Deflection = 0.26 mm Deflection limits = $\frac{Span}{360}$ = 4.92 mm
Finishes = Brittle

As 0.26 < 4.92 mm Deflection due to SLS loading is O.K.

Frequency Checks
By inspection, frequency checks not required due to low deflections.

Therefore use	1 No.	203x102x23	S355	section

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BEARING DESIGN TO BS EN 1996-1-1

Reference = Steel Beam C Short Span

Bearing at End "A" Factored Reaction = 11.40 kN ULS

$f_k = K f_b^{\alpha} f_m^{\beta}$ Masonry under padstone Aggregate Concrete blocks Group 1

Unit size below = 215 mm h x 100 mm width Shape Factor, δ = 1.38

Act. Comp. Str = 3.50 N/mm^2 Norm. Mean Comp. Strength, f_b = 4.83 N/mm^2

From National Annexe, Table NA4, K = 0.55 α = 0.7 β = 0.30

Mortar Designation = (iii) Mortar Type M 4 f_m = 4.00 N/mm^2

f_k = 2.51 N/mm^2 $f_d = f_k/\gamma_m$ γ_m (Table NA1) = 2.70

Bearing capacity, N_{Rdc} = $(1.2 + 0.4(a_1/h_c))f_dA_b$ but not more than $1.5f_dA_b$.

Edge distance, a_1 from edge of bearing = 600 mm

Height, h_c, of bearing base above floor = 2000 mm

Bearing capacity, N_{Rdc} = 1.32 f_dA_b < $1.5f_dA_b$. use 1.32 f_dA_b.

Bearing capacity, N_{Rdc} = 1.23 N/mm^2

Bearing size = 300 mm x 100 mm x 150 mm deep

$$\frac{11.40 \times 10^3\ N}{30000\ mm^2} = 0.38\ N/mm^2 < 1.23\ N/mm^2$$ O.K.

Therefore, use a 300 mm x 100 mm x 150 mm dp padstone

Reference = Steel Beams C

Bearing at End "B" Factored Reaction = 37.35 kN ULS

$f_k = K f_b^{\alpha} f_m^{\beta}$ Masonry under padstone Aggregate Concrete blocks Group 1

Unit size below = 215 mm h x 100 mm width Shape Factor, δ = 1.38

Act. Comp. Str = 3.50 N/mm^2 Norm. Mean Comp. Strength, f_b = 4.83 N/mm^2

From National Annexe, Table NA4, K = 0.55 α = 0.7 β = 0.30

Mortar Designation = (iii) Mortar Type M 4 f_m = 4.00 N/mm^2

f_k = 2.51 N/mm^2 $f_d = f_k/\gamma_m$ γ_m (Table NA1) = 2.70

Bearing capacity, N_{Rdc} = $(1.2 + 0.4(a_1/h_c))f_dA_b$ but not more than $1.5f_dA_b$.

Edge distance, a_1 from edge of bearing = 600 mm

Height, h_c, of bearing base above floor = 2000 mm

Bearing capacity, N_{Rdc} = 1.32 f_dA_b < $1.5f_dA_b$. use 1.32 f_dA_b.

Bearing capacity, N_{Rdc} = 1.23 N/mm^2

Bearing size = 400 mm x 100 mm x 215 mm deep

$$\frac{37.35 \times 10^3\ N}{40000\ mm^2} = 0.93\ N/mm^2 < 1.23\ N/mm^2$$ O.K.

Therefore, use a 400 mm x 100 mm x 215 mm dp padstone

SIMPLE BEAM ANALYSIS TO BS EN 1990   Ref = Steel Beam D

Actions

	Permanent Action, G_k (kN/m^2)	Leading Variable Action, $Q_{k\,1}$ (kN/m^2)	Variable Action, $Q_{k\,i}$ (kN/m^2)	Dist		Permanent Action, G_k (kN/m)	Variable Action, $Q_{k\,1}$ (kN/m)	Variable Action, $Q_{k\,i}$ (kN/m)
Pitched roof	1.07		0.52	on slope	3.80	4.06	0.00	1.97
Ashlar Wall	0.55			height	1.00	0.55	0.00	0.00
Ceiling Joists	0.35	0.35		6.8/2=	3.40	1.19	1.19	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
Self weight of beam	Using a	178x102x19				0.19		
					TOTALS:	*5.99*	*1.19*	*1.97*

Clear Span = 2900 mm.   Bearings = 100 mm minimum each side

Design span = **3000 mm**   ***(NB - Design span only - not for setting out on site)***

Span between restraints = 3000 mm

Conservatively, use equation 6.10 = $1.35\ G_k + 1.5\ Q_{k\,1} + \Sigma\ \psi_{0\,i}\ 1.5\ Q_{k\,i}$

Conservatively, take $\psi_{0\,i}$ = 1.00

Design UDL= 8.08 + 1.79 + 2.96181 = 12.83 kN/m

Unfactored UDL = 9.15 kN/m

Design Point Load 1= 0.00 kN

Distance from A to point load 1= 0 mm

Distance from B to point load 1= 3000 mm

Design Point Load 2= 0.00 kN

Distance from A to point load 2= 0 mm

Distance from B to point load 2= 3000 mm

Design Point Load 3= 0.00 kN

Distance from A to point load 3= 0 mm

Distance from B to point load 3= 3000 mm

Design Point Load 4= 0.00 kN

Distance from A to point load 4= 0 mm

Distance from B to point load 4= 3000 mm

Beam analysed with simple supports:-

Design Moment = 14.43 kNm ULS

Shear Force At A = 19.24 kN ULS   Shear Force At B = 19.24 kN

Design Moment =	**14.43**	kNm
Design. Shear A =	**19.24**	kN
Design. Shear B =	**19.24**	kN

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DESIGN OF STEEL MEMBER TO BS EN 1993-1-1 Ref = Steel Beam D

Design Data

From previous calculation:- Moment, $M_{y,Ed}$ = 14.43 kNm

(Beam in pure bending only - no axial forces) Design Shear = 19.24 kN

Try a section Size =

1 No. 178x102x19 Design Span = 3000 mm

Between Restraints = 3000 mm

Grade of Steel = S355 Flange thickness, t_f= 7.90 mm

Web thickness, t_w= 4.80 mm Yield Strength, f_y = 355 N/mm^2 *(Table 3.1)*

Classification of Section

Web = $d/t = \frac{146.80}{4.80}$ = 30.58 mm ε = 0.81

Flange = $c = \frac{101.20 - 15.20 - 4.80}{2}$ = 40.6 mm

$c/t = \frac{40.60}{7.90}$ = 5.14 mm

(From table 5.2, for rolled Sections2)

web Class 1 limiting = 72 ε = 58.58 mm Therefore Class 1 Suitable

flange Class 1 limiting = 9 ε = 7.32 mm Therefore Class 1 Suitable

Moment Resistance As member is Class 1 use Plastic modulus.

$W_{pl,y}$ = 171000 mm^3 $W_{el,y}$ = 153000 mm^3 γ_{M0} = 1

$M_{c,Rd} = \frac{171000 \times 355}{1}$ = 60.71 kNm *(Cl 6.2.5)*

As 60.71 > 14.43 kNm Moment resistance of section O.K.

Shear Resistance Root radius, r = 7.60 mm A = 2430 mm^2

$A_{v,z}$ = A - 2 b t_f + (t_w + 2r) t $A_{v,z}$ = 989 mm^2 $\eta\, h_w\, t_w$ = 705 mm^2

$V_{pl,z,Rd} = \frac{A_{v,z} (f_y / ((3)^{0.5}))}{\gamma_{M0}}$ = 202.71 kN

Combined bending/shear not considered as max. moment away from max. shear.

As 202.71 > 19.24 kN Shear resistance of section O.K.

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DESIGN OF STEEL MEMBER TO BS EN 1993-1-1 Ref = Steel Beam D

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Calculation of Mcrit

$$M_{cr} = C_1 \frac{\pi^2 EI_z}{(kL)^2}\left\{\sqrt{\left(\frac{k}{k_w}\right)^2 \frac{I_w}{I_z} + \frac{(kL)^2 GI_t}{\pi^2 EI_z} + (C_2 z_g)^2} - C_2 z_g\right\}$$

1. Destabilising Loads

E= 210000 N/mm^2 G= 80770 N/mm^2

Loading and support condition:- Simply supported, UDL over whole span
Factors based on bending moment:- C1= 1.127 C2= 0.454 C3= 1.0

$I_{w=}$	1E+10 mm^6	Warping constant from section property tables.
$I_{z=}$	1370000 mm^4	Minor second moment of area from section property tables.
$I_{t=}$	44100 mm^4	Torsion constant from section property tables
$k_{w=}$	1.0	Unless special provision for warping fixity is made, k_w should be taken as 1.0
$k_=$	1.2	Effective Length constant
$z_{g=}$	88.9 mm	Height of the destabilising load above the shear centre.
$z_{j=}$	0.0 mm	0 mm for bi-symmetric Sections2

$$M_{cr} = 246671 \times \left(\left(10511 + 16274 + 1629\right)^{0.5} - 40\right)$$

M_{cr}= **31.6 kNm** *(Destabilising Load)*

2. Non-destabilising Loads

$$M_{cr} = C_1 \frac{\pi^2 EI_z}{L^2}\sqrt{\frac{I_w}{I_z} + \frac{L^2 GI_t}{\pi^2 EI_z}}$$

$$M_{cr} = 1 \times 315179 \times \left(18601\right)^{0.5}$$

M_{cr}= **48.4 kNm** *(Non-destabilising Load)*

For rolled profiles, λ_{LT0} = 0.4

Consider load to be Destabilising therefore use, M_{cr}= 31.62 kNm
W_y = 171000 mm^3

Non-dimensional slenderness $\lambda_{LT} = \left(\frac{W_y f_y}{M_{cr}}\right)^{0.5}$ = 1.385

As, 1.385 > 0.4 LTB verification is Required

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DESIGN OF STEEL MEMBER TO BS EN 1993-1-1 Ref = Steel Beam D

Lateral Torsional Buckling (Cont)

$\frac{h}{b} = \frac{177.80}{101.20} = 1.76 < 2$ use buck. curve b *(Table 6.5)*

Imperfection factor, $\alpha_{LT} =$ 0.34 $\beta =$ 0.75

$\phi_{LT} = 0.5\,(1 + \alpha_{LT}(\lambda_{LT} - \lambda_{LT0}) + \beta\lambda_{LT}^2) = 1.39$

Reduction factor, $\chi_{LT} = \frac{1}{\phi_{LT} + (\phi_{LT}^2 - \beta\lambda_{LT}^2)^{0.5}}$ but $\chi_{LT} < 1.0$ and $\chi_{LT} < \frac{1}{\lambda_{LT}^2}$

$\chi_{LT} = 0.48$ 0.48 < 1 0.48 < 0.52

Conservatively, factor, f = 1 $\chi_{LT,mod} =$ 0.48 $\gamma_{M1} =$ 1.00

$M_{b,Rd} = \chi_{LT,mod} W_y f_y / \gamma_{M1} =$ **29.13 kNm**

As 29.13 > 14.43 kNm Lateral Torsional Buckling resistance O.K.

Deflection Checks Take average partial safety factors as 1.425

Equivalent unfactored SLS udl to beam = $\frac{10.13 \times 8}{3.00 \times 3.00} =$ 9.00 kN/m

$I_y =$ 1.4E+07 mm^4 Major second moment of area from section property tables.

Deflection = 3.33 mm Deflection limits = $\frac{Span}{360} =$ 8.33 mm
Finishes = Brittle

As 3.33 < 8.33 mm Deflection due to SLS loading is O.K.

Frequency Checks
By inspection, frequency checks not required due to low deflections.

Therefore use	1 No.	178x102x19	S355	section

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BEARING DESIGN TO BS EN 1996-1-1

Reference = Steel Beam D

Bearing at End "A" Factored Reaction = 19.24 kN ULS

$f_k =$ $K f_b^{\alpha} f_m^{\beta}$ Masonry under padstone Aggregate Concrete blocks Group 1

Unit size below = 215 mm h x 100 mm width Shape Factor, $\delta =$ 1.38

Act. Comp. Str = 3.50 N/mm² Norm. Mean Comp. Strength, $f_b =$ 4.83 N/mm²

From National Annexe, Table NA4, K = 0.55 $\alpha =$ 0.7 $\beta =$ 0.30

Mortar Designation = (iii) Mortar Type M 4 $f_m =$ 4.00 N/mm²

$f_k =$ 2.51 N/mm² $f_d =$ f_k/γ_m γ_m (Table NA1) = 2.70

Bearing capacity, $N_{Rdc} =$ $(1.2 + 0.4(a_1/h_c))f_dA_b$ but not more than $1.5f_dA_b$.

Edge distance, a_1 from edge of bearing = 600 mm

Height, h_c, of bearing base above floor = 2000 mm

Bearing capacity, $N_{Rdc} =$ 1.32 f_dA_b < $1.5f_dA_b$. use 1.32 f_dA_b.

Bearing capacity, $N_{Rdc} =$ 1.23 N/mm²

Bearing size = 300 mm x 100 mm x 150 mm deep

$$\frac{19.24 \times 10^3\ N}{30000\ mm^2} = 0.64\ N/mm^2 < 1.23\ N/mm^2$$ **O.K.**

Therefore, use a 300 mm x 100 mm x 150 mm dp padstone

Reference = Steel Beam C (large Span) & D

Bearing at End "B" Factored Reaction = 45.19 kN ULS

$f_k =$ $K f_b^{\alpha} f_m^{\beta}$ Masonry under padstone Aggregate Concrete blocks Group 1

Unit size below = 215 mm h x 100 mm width Shape Factor, $\delta =$ 1.38

Act. Comp. Str = 3.50 N/mm² Norm. Mean Comp. Strength, $f_b =$ 4.83 N/mm²

From National Annexe, Table NA4, K = 0.55 $\alpha =$ 0.7 $\beta =$ 0.30

Mortar Designation = (iii) Mortar Type M 4 $f_m =$ 4.00 N/mm²

$f_k =$ 2.51 N/mm² $f_d =$ f_k/γ_m γ_m (Table NA1) = 2.70

Bearing capacity, $N_{Rdc} =$ $(1.2 + 0.4(a_1/h_c))f_dA_b$ but not more than $1.5f_dA_b$.

Edge distance, a_1 from edge of bearing = 2000 mm

Height, h_c, of bearing base above floor = 2000 mm

Bearing capacity, $N_{Rdc} =$ 1.60 f_dA_b > $1.5f_dA_b$. use 1.50 f_dA_b.

Bearing capacity, $N_{Rdc} =$ 1.39 N/mm²

Bearing size = 450 mm x 100 mm x 215 mm deep

$$\frac{45.19 \times 10^3\ N}{45000\ mm^2} = 1.00\ N/mm^2 < 1.39\ N/mm^2$$ **O.K.**

Therefore, use a 450 mm x 100 mm x 215 mm dp padstone

SIMPLE BEAM ANALYSIS TO BS EN 1990 Ref = Steel Beam E

Actions

	Permanent Action, G_k (kN/m^2)	Leading Variable Action, $Q_{k\,1}$ (kN/m^2)	Variable Action, $Q_{k\,i}$ (kN/m^2)	Dist		Permanent Action, G_k (kN/m)	Variable Action, $Q_{k\,1}$ (kN/m)	Variable Action, $Q_{k\,i}$ (kN/m)
Pitched roof	1.07		0.52	nominal	0.40	0.43	0.00	0.21
Cavity Wall	3.64			height	3.00	10.92	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
Self weight of beam	Using a	152x89x16				0.16		
					TOTALS:	*11.50*	*0.00*	*0.21*

Clear Span = 1000 mm. Bearings = 100 mm minimum each side

Design span = **1100 mm** ***(NB - Design span only - not for setting out on site)***

Span between restraints = 1100 mm

Conservatively, use equation 6.10 = $1.35\ G_k + 1.5\ Q_{k\,1} + \Sigma\ \psi_{0\,i}\ 1.5\ Q_{k\,i}$

Conservatively, take $\psi_{0\,i}$ = 1.00

Design UDL= 15.53 + 0.00 + 0.31177 = 15.84 kN/m

Unfactored UDL = 11.71 kN/m

Design Point Load 1= 11.40 kN Steel Beam C (short span)

Distance from A to point load 1= 90 mm

Distance from B to point load 1= 1010 mm

Design Point Load 2= 0.00 kN

Distance from A to point load 2= 0 mm

Distance from B to point load 2= 1100 mm

Design Point Load 3= 0.00 kN

Distance from A to point load 3= 0 mm

Distance from B to point load 3= 1100 mm

Design Point Load 4= 0.00 kN

Distance from A to point load 4= 0 mm

Distance from B to point load 4= 1100 mm

Beam analysed with simple supports:-

Design Moment = 3.34 kNm ULS

Shear Force At A = 19.18 kN ULS Shear Force At B = 9.64 kN

Design Moment =	**3.34**	kNm
Design. Shear A =	**19.18**	kN
Design. Shear B =	**9.64**	kN

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<u>DESIGN OF STEEL MEMBER TO BS EN 1993-1-1</u> Ref = Steel Beam E

Design Data

From previous calculation:- Moment, $M_{y,Ed}$ = 3.34 kNm
(Beam in pure bending only - no axial forces) Design Shear = 19.18 kN
Try a section Size =
1 No. 152x89x16 Design Span = 1100 mm
Between Restraints = 1100 mm
Grade of Steel = S355 Flange thickness, t_f= 7.70 mm
Web thickness, t_w= 4.50 mm Yield Strength, f_y = 355 N/mm^2 *(Table 3.1)*

Classification of Section

Web = $d/t = \frac{121.80}{4.50}$ = 27.07 mm ε = 0.81

Flange = $c = \frac{88.70 - 15.20 - 4.50}{2}$ = 34.5 mm

$c/t = \frac{34.50}{7.70}$ = 4.48 mm

(From table 5.2, for rolled Sections2)

web	Class 1	limiting =	72 ε =	58.58 mm	Therefore	Class 1	Suitable
flange	Class 1	limiting =	9 ε =	7.32 mm	Therefore	Class 1	Suitable

Moment Resistance As member is Class 1 use Plastic modulus.
$W_{pl,y}$ = 123000 mm^3 $W_{el,y}$ = 109000 mm^3 γ_{M0} = 1

$M_{c,Rd} = \frac{123000 \times 355}{1}$ = 43.67 kNm *(Cl 6.2.5)*

As 43.67 > 3.34 kNm Moment resistance of section O.K.

Shear Resistance Root radius, r = 7.60 mm A = 2030 mm^2

$A_{v,z}$ = A - 2 b t_f + (t_w + 2r) t $A_{v,z}$ = 816 mm^2 $\eta\, h_w t_w$ = 548 mm^2

$V_{pl,z,Rd} = \frac{A_{v,z}\,(f_y / ((3)^{0.5}))}{\gamma_{M0}}$ = 167.19 kN

Combined bending/shear not considered as max. moment away from max. shear.

As 167.19 > 19.18 kN Shear resistance of section O.K.

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DESIGN OF STEEL MEMBER TO BS EN 1993-1-1 Ref = Steel Beam E

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Calculation of Mcrit

1. *Destabilising Loads*

$$M_{cr} = C_1 \frac{\pi^2 E I_z}{(kL)^2} \left\{ \sqrt{\left(\frac{k}{k_w}\right)^2 \frac{I_w}{I_z} + \frac{(kL)^2 G I_t}{\pi^2 E I_z} + (C_2 z_g)^2} - C_2 z_g \right\}$$

E= 210000 N/mm^2 G= 80770 N/mm^2

Loading and support condition:- Simply supported, UDL over whole span
Factors based on bending moment:- C1= 1.127 C2= 0.454 C3= 1.0

$I_{w=}$	5E+09	mm^6	Warping constant from section property tables.
$I_{z=}$	898000	mm^4	Minor second moment of area from section property tables.
$I_{t=}$	35600	mm^4	Torsion constant from section property tables
$k_{w=}$	1.0		Unless special provision for warping fixity is made, k_w should be taken as 1.0
$k_=$	1.2		Effective Length constant
$z_{g=}$	76.2	mm	Height of the destabilising load above the shear centre.
$z_{j=}$	0.0	mm	0 mm for bi-symmetric Sections2

$$M_{cr} = 1202627 \times \left(\left(8018 + 2695 + 1197 \right)^{0.5} - 35 \right)$$

M_{cr}= 89.6 kNm ***(Destabilising Load)***

2. *Non-destabilising Loads*

$$M_{cr} = C_1 \frac{\pi^2 E I_z}{L^2} \sqrt{\frac{I_w}{I_z} + \frac{L^2 G I_t}{\pi^2 E I_z}}$$

$$M_{cr} = 1 \times 1536631 \times \left(7439 \right)^{0.5}$$

M_{cr}= 149.4 kNm ***(Non-destabilising Load)***

For rolled profiles, λ_{LT0} = 0.4

Consider load to be Destabilising therefore use, M_{cr}= 89.64 kNm
W_y = 123000 mm^3

Non-dimensional slenderness $\lambda_{LT} = \left(\frac{W_y f_y}{M_{cr}} \right)^{0.5}$ = 0.698

As, 0.698 > 0.4 LTB verification is Required

<u>DESIGN OF STEEL MEMBER TO BS EN 1993-1-1</u> Ref = Steel Beam E

Lateral Torsional Buckling (Cont)

$\frac{h}{b} = \frac{152.40}{88.70} = 1.72 < 2$ use buck. curve b *(Table 6.5)*

Imperfection factor, $\alpha_{LT} =$ 0.34 $\beta =$ 0.75

$\phi_{LT} = 0.5\ (1 + \alpha_{LT}\ (\lambda_{LT} - \lambda_{LT0}) + \beta\lambda_{LT}^2) = 0.73$

Reduction factor, $\chi_{LT} = \frac{1}{\phi_{LT} + (\phi_{LT}^2 - \beta\lambda_{LT}^2)^{0.5}}$ but $\chi_{LT} < 1.0$ and $\chi_{LT} < \frac{1}{\lambda_{LT}^2}$

$\chi_{LT} = 0.87$ 0.87 < 1 0.87 < 2.05

Conservatively, factor, f = 1 $\chi_{LT,mod} =$ 0.87 $\gamma_{M1} =$ 1.00

$M_{b,Rd} = \chi_{LT,mod}\ W_y\ f_y / \gamma_{M1} =$ **38.02 kNm**

As 38.02 > 3.34 kNm <u>Lateral Torsional Buckling resistance O.K.</u>

Deflection Checks Take average partial safety factors as 1.425

Equivalent unfactored SLS udl to beam = $\frac{2.34 \times 8}{1.10 \times 1.10}$ = 15.49 kN/m

$I_{y=}$ 8.3E+06 mm^4 Major second moment of area from section property tables.

Deflection = 0.17 mm Deflection limits = $\frac{Span}{360}$ = 3.06 mm

Finishes = Brittle

As 0.17 < 3.06 mm <u>Deflection due to SLS loading is O.K.</u>

Frequency Checks

By inspection, frequency checks not required due to low deflections.

Therefore use	1 No.	152x89x16	S355	section

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BEARING DESIGN TO BS EN 1996-1-1

Reference = Steel Beam E

Bearing at End "A" Factored Reaction = 19.18 kN ULS

$f_k = K f_b^{\alpha} f_m^{\beta}$ Masonry under padstone Aggregate Concrete blocks Group 1

Unit size below = 215 mm h x 100 mm width Shape Factor, δ = 1.38

Act. Comp. Str = 3.50 N/mm^2 Norm. Mean Comp. Strength, f_b = 4.83 N/mm^2

From National Annexe, Table NA4, K = 0.55 α = 0.7 β = 0.30

Mortar Designation = (iii) Mortar Type M 4 f_m = 4.00 N/mm^2

f_k = 2.51 N/mm^2 $f_d = f_k/\gamma_m$ γ_m (Table NA1) = 2.70

Bearing capacity, N_{Rdc} = $(1.2 + 0.4(a_1/h_c))f_dA_b$ but not more than $1.5f_dA_b$.

Edge distance, a_1 from edge of bearing = 0 mm

Height, h_c, of bearing base above floor = 2000 mm

Bearing capacity, N_{Rdc} = 1.20 f_dA_b < $1.5f_dA_b$. use 1.20 f_dA_b.

Bearing capacity, N_{Rdc} = 1.12 N/mm^2

Bearing size = 400 mm x 100 mm x 215 mm deep

$$\frac{19.18 \times 10^3\ N}{40000\ mm^2} = 0.48\ N/mm^2 < 1.12\ N/mm^2$$ O.K.

Therefore, use a 400 mm x 100 mm x 215 mm dp padstone

Bearing at End "B" Factored Reaction = 9.64 kN ULS

$f_k = K f_b^{\alpha} f_m^{\beta}$ Masonry under padstone Aggregate Concrete blocks Group 1

Unit size below = 215 mm h x 100 mm width Shape Factor, δ = 1.38

Act. Comp. Str = 3.50 N/mm^2 Norm. Mean Comp. Strength, f_b = 4.83 N/mm^2

From National Annexe, Table NA4, K = 0.55 α = 0.7 β = 0.30

Mortar Designation = (iii) Mortar Type M 4 f_m = 4.00 N/mm^2

f_k = 2.51 N/mm^2 $f_d = f_k/\gamma_m$ γ_m (Table NA1) = 2.70

Bearing capacity, N_{Rdc} = $(1.2 + 0.4(a_1/h_c))f_dA_b$ but not more than $1.5f_dA_b$.

Edge distance, a_1 from edge of bearing = 0 mm

Height, h_c, of bearing base above floor = 2000 mm

Bearing capacity, N_{Rdc} = 1.20 f_dA_b < $1.5f_dA_b$. use 1.20 f_dA_b.

Bearing capacity, N_{Rdc} = 1.12 N/mm^2

Bearing size = 300 mm x 100 mm x 150 mm deep

$$\frac{9.64 \times 10^3\ N}{30000\ mm^2} = 0.32\ N/mm^2 < 1.12\ N/mm^2$$ O.K.

Therefore, use a 300 mm x 100 mm x 150 mm dp padstone

RCA STRUCTURES

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WALL LOADINGS FOR STRIP FOUNDATION CHECK

	Perm.G_k (kN/m^2)	Var. Q_k (kN/m^2)	Length/Height		Perm. Gk (kN/m)	Var. Q_k (kN/m)
Pitched Roof	1.07	0.52	on slope	4.00	4.28	2.08
Loft	0.35	0.35	6.7/2=	3.35	1.17	1.17
Flat Roof	0.65	0.60	5.4/2=	2.70	1.75	1.62
Cavity Wall	3.64		height	2.50	9.10	0.00
Wall below DPC	6.58		height	0.50	3.29	0.00
					0.00	0.00
					0.00	0.00
					0.00	0.00
Total				=	19.59	4.87

MINIMUM FOUNDATION WIDTH

Take Ground bearing Pressure as:- 100 kN/m^2

TBC on site.

Wall Ref.	Perm. Load G_k (kN/m)	Variable Load Q_k (kN/m)	Total Load (kN/m)	Width Required (m)	Width Used (m)	Check
Rear Wall	19.59	4.87	24.46	0.24	0.45	O.K.