STRUCTURAL CALCULATIONS

DATE: 13-08-2026

FOR

TIMBER FRAME DESIGN

PROJECT

THE BARN HOUSE RUSHTON ROAD NN14 6HH

PREPARED BY

MOHAMMED NAYEEM (MSc.)

Graduate Member of IStructE

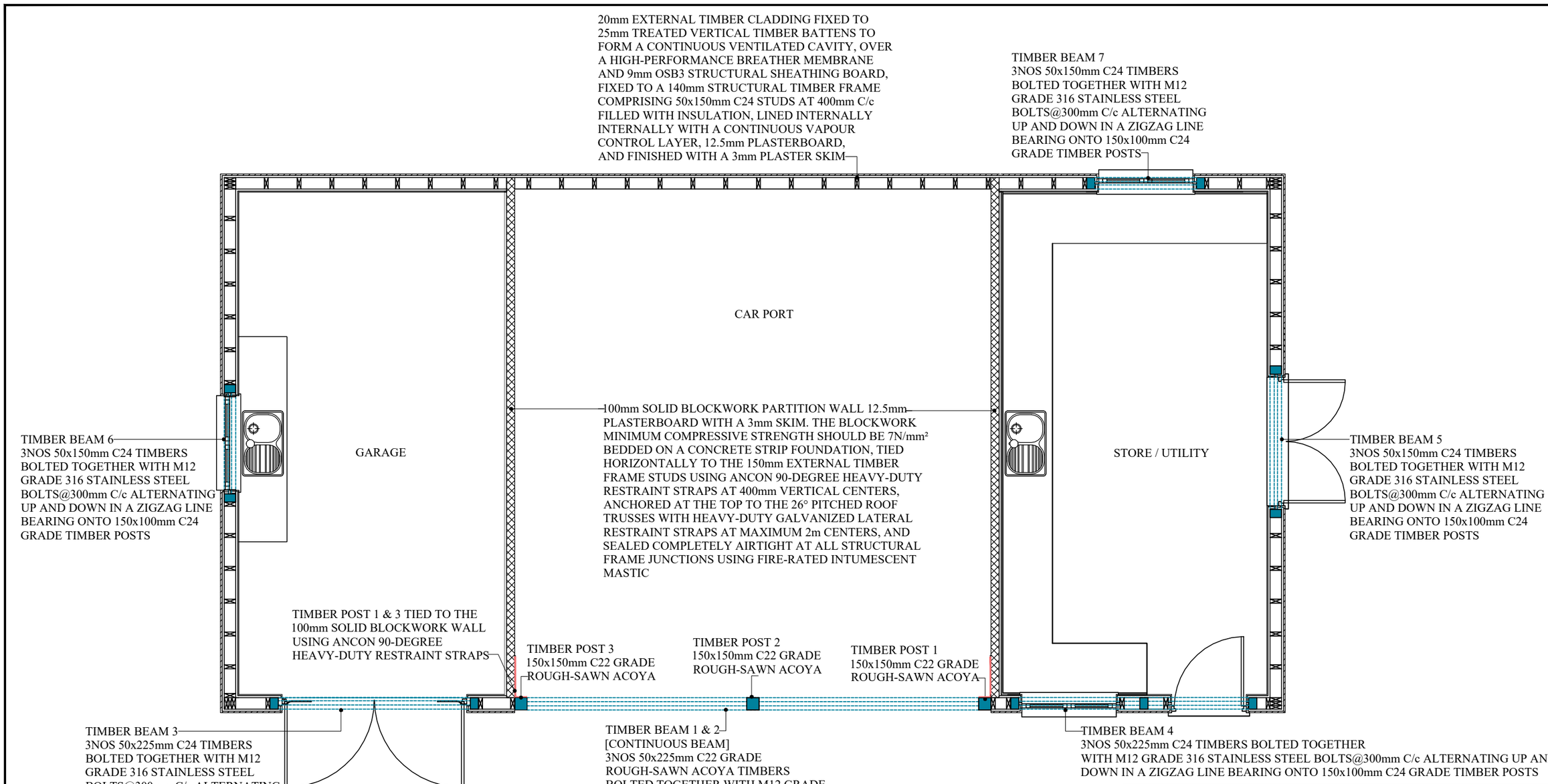
DADA STRUCTURAL CONSULTANTS

36 Merton Avenue, Uxbridge UB10 9BN

Tel: 07741025944

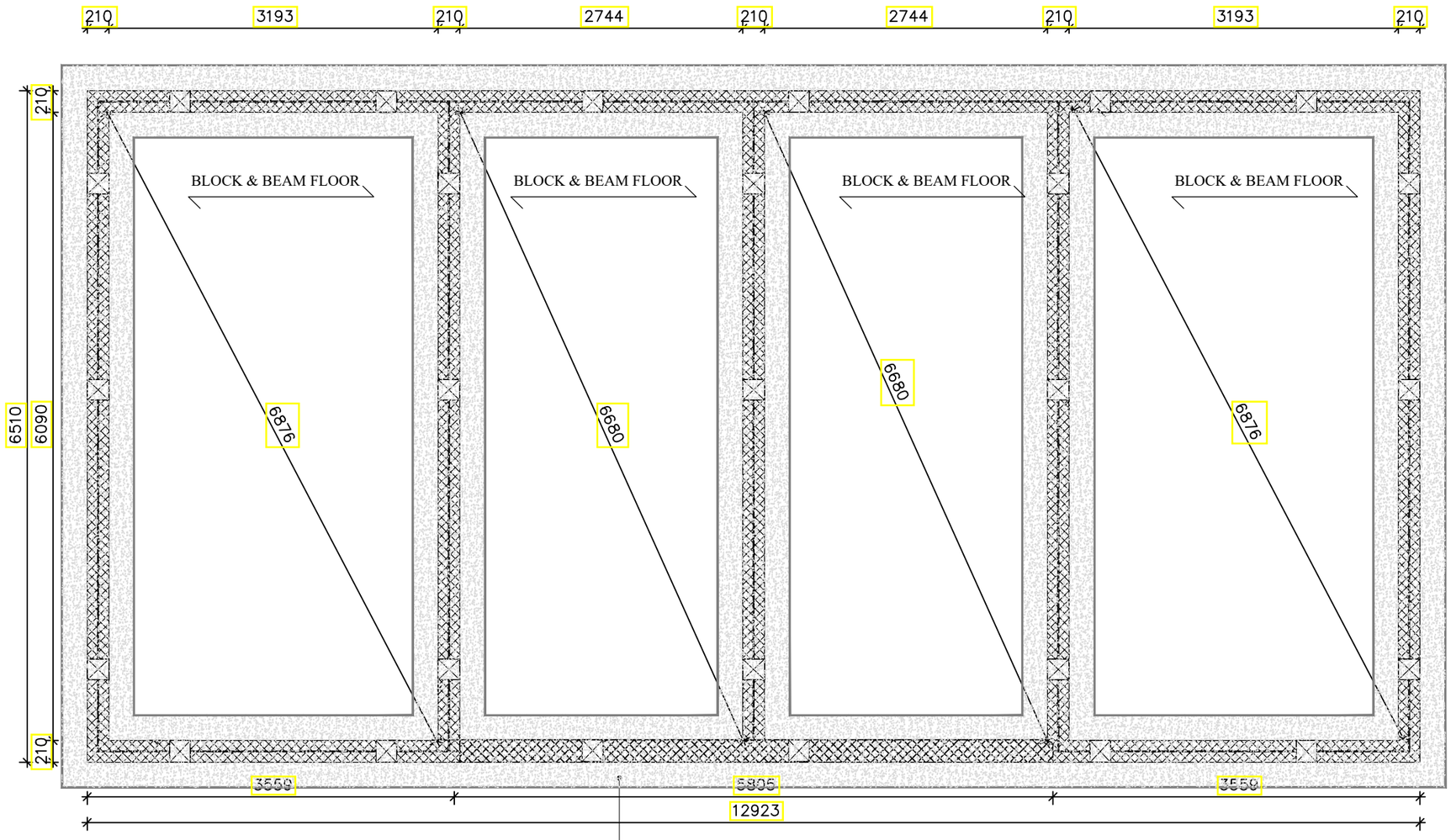
Website: www.dadastructuralconsultants.co.uk

E-mail: info@dadastructuralconsultants.co.uk



PROJECT	THE BARN HOUSE RUSHTON ROAD NN14 6HH
DESCRIPTION	PROPOSED GROUND FLOOR STRUCTURAL PLAN
SCALE	1:50@A3
DRAWINGS NO.	S01
DATE	13-08-2026
CLIENT	MR. ROSS HOWES
PREPARED BY	MOHAMMED NAYEEM MSc. (Hons.)

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700mm WIDE MASS CONCRETE TAKEN DOWN TO LOAD BEARING SUB-SOIL A WITH A MINIMUM 1200mm DEPTH (TO BE CONFIRMED BY BUILDING CONTROL OFFICER)

FOUNDATION IS TO BE 700mm WIDE TRENCH CONCRETE FOOTING WITH 1:2:4 CONCRETE MIX.FOUNDATION TO BE 1.2m DEEP (TO BE DETERMINED BY BUILDING CONTROL).ANY DRAINS PASSING THROUGH FOUNDATION ARE TO BE PROTECTED USING UPVC SLEEVING AND 150x100mm RC BLOCK LINTEL.

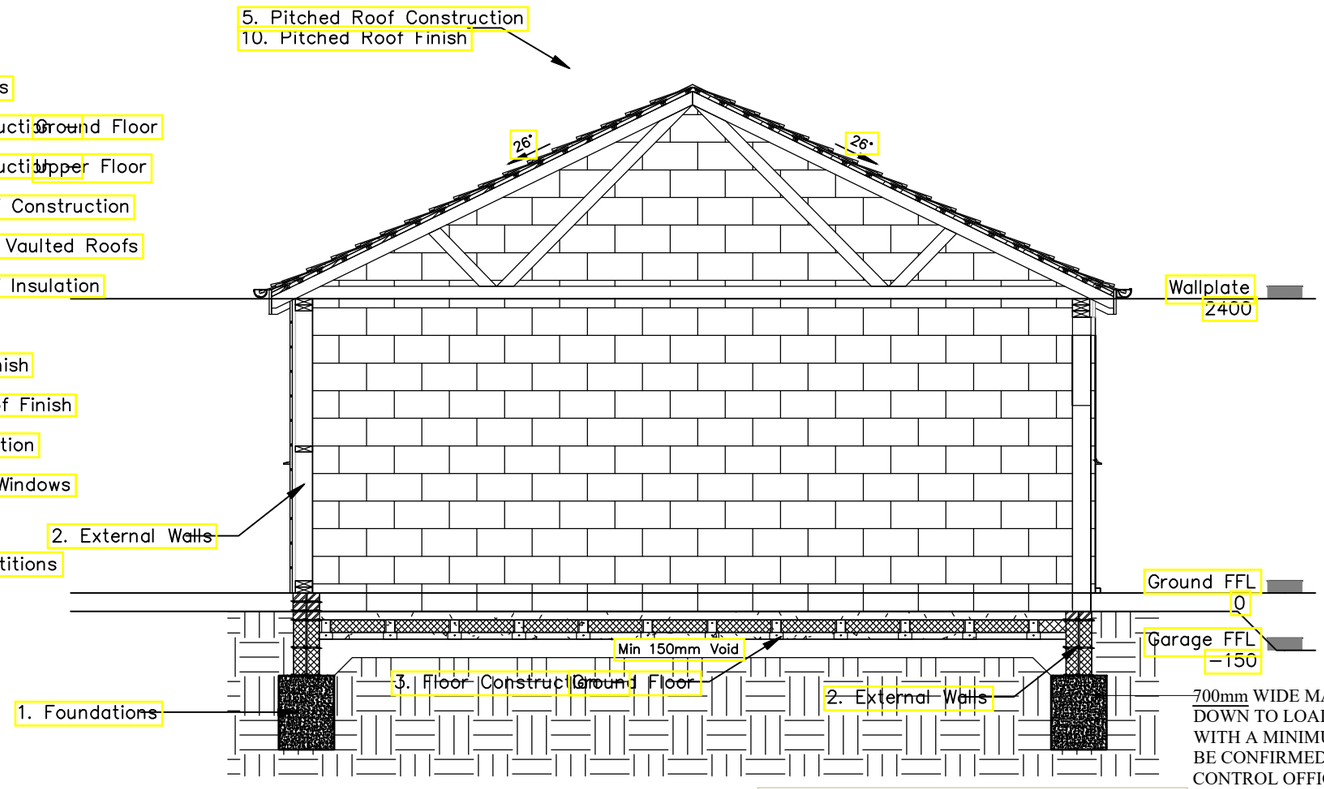
FOUNDATION DEPTH ACCORDING TO NHBC FOUNDATION CALCULATOR.FINAL DESIGN AND DEPTHS TO BE DETERMINED BY BUILDING CONTROL

PROJECT	THE BARN HOUSE RUSHTON ROAD NN14 6HH
DESCRIPTION	PROPOSED FOUNDATION STRUCTURAL PLAN
SCALE	1:50@A3
DRAWINGS NO.	S02
DATE	13-08-2026
CLIENT	MR. ROSS HOWES
PREPARED BY	MOHAMMED NAYEEM MSc. (Hons.)

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Construction Elements Refer to numbered note in construction specification

1. Foundations
2. External Walls
3. Floor Construction Ground Floor
4. Floor Construction Upper Floor
5. Pitched Roof Construction
6. Insulation in Vaulted Roofs
7. Pitched Roof Insulation
8. Flat Roof
9. Flat Roof Finish
10. Pitched Roof Finish
11. Roof Ventilation
12. Doors and Windows
13. Stairs
14. Internal Partitions



700mm WIDE MASS CONCRETE TAKEN DOWN TO LOAD BEARING SUB-SOIL A WITH A MINIMUM 1200mm DEPTH (TO BE CONFIRMED BY BUILDING CONTROL OFFICER)

FOUNDATION DEPTH ACCORDING TO NHBC FOUNDATION CALCULATOR.FINAL DESIGN AND DEPTHS TO BE DETERMINED BY BUILDING CONTROL

FOUNDATION IS TO BE 700mm WIDE TRENCH CONCRETE FOOTING WITH 1:2:4 CONCRETE MIX.FOUNDATION TO BE 1.2m DEEP(TO BE DETERMINED BY BUILDING CONTROL).ANY DRAINS PASSING THROUGH FOUNDATION ARE TO BE PROTECTED USING UPVC SLEEVING AND 150X100mm RC BLOCK LINTEL.

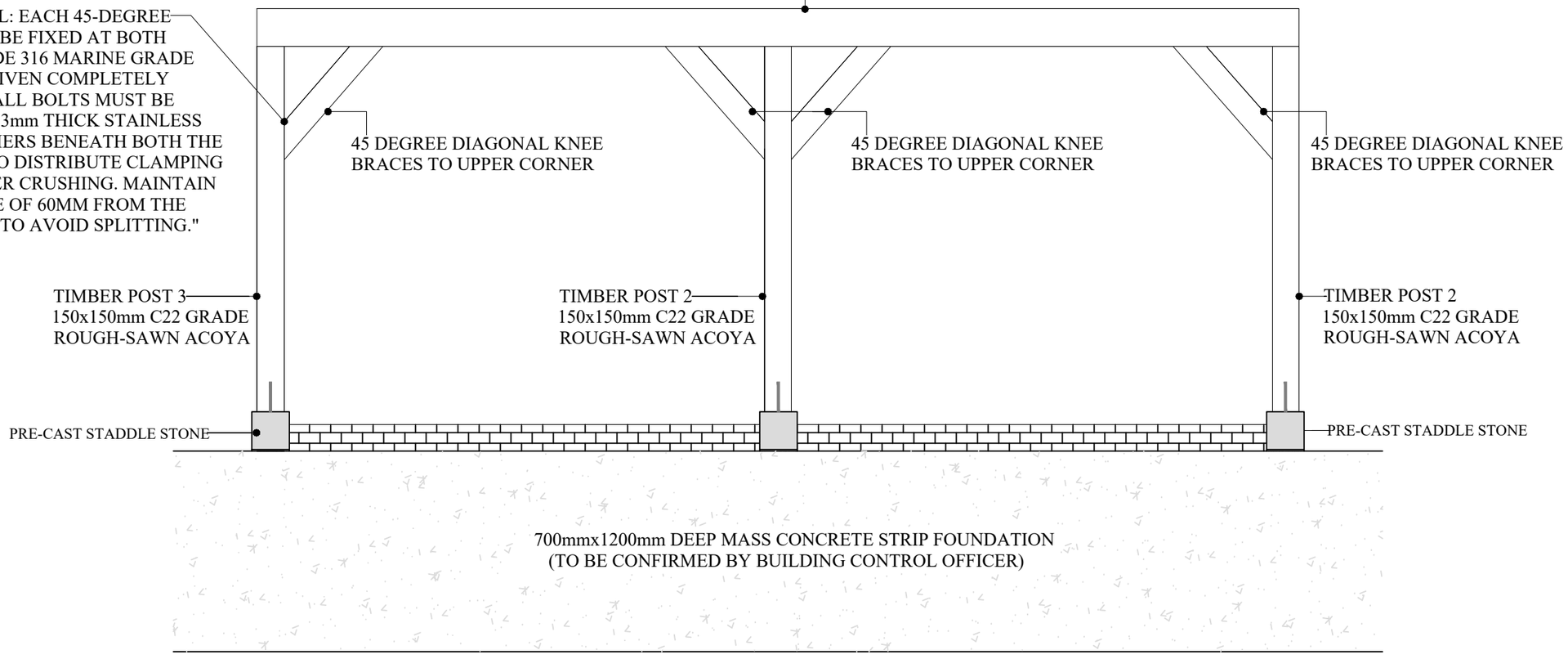
PROJECT	THE BARN HOUSE RUSHTON ROAD NN14 6HH
DESCRIPTION	PROPOSED SECTION
SCALE	1:50@A3
DRAWINGS NO.	S03
DATE	13-08-2026
CLIENT	MR. ROSS HOWES
PREPARED BY	MOHAMMED NAYEEM MSc. (Hons.)

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TIMBER BEAM:
3NOS 50x225mm C22 GRADE ROUGH-SAWN ACOYA TIMBERS BOLTED TOGETHER WITH M12 GRADE 316 STAINLESS STEEL BOLTS WITH 50x50x3mm SQAURE PLATE WASHERS SPACED AT 300mm CENTERS ALTERNATING UP AND DOWN IN A ZIGZAG LINE BEARING ONTO 150x150mm C22 GRADE ROUGH-SAWN ACOYA TIMBER POSTS

NOTE: FOR ALL THE FRAME CONNECTIONS REFER TO CONNECTION SPECIFICATIONS SEPARATELY PROVIDED WITHIN CALCULATIONS DOCUMENT ON PAGE NUMBER 22

"KNEE BRACE FIXING DETAIL: EACH 45-DEGREE DIAGONAL KNEE BRACE TO BE FIXED AT BOTH ENDS USING 2NOS M20 GRADE 316 MARINE GRADE STAINLESS STEEL BOLTS DRIVEN COMPLETELY THROUGH THE ASSEMBLY. ALL BOLTS MUST BE FITTED WITH 50mm x 50mm x 3mm THICK STAINLESS STEEL SQUARE PLATE WASHERS BENEATH BOTH THE BOLT HEAD AND THE NUT TO DISTRIBUTE CLAMPING FORCE AND PREVENT TIMBER CRUSHING. MAINTAIN A MINIMUM EDGE DISTANCE OF 60MM FROM THE END OF THE TIMBER BRACE TO AVOID SPLITTING."



PROJECT	THE BARN HOUSE RUSHTON ROAD NN14 6HH
DESCRIPTION	TIMBER FRAME CONNECTION DETAILS
SCALE	1:30@A3
DRAWINGS NO.	S03
DATE	13-08-2026
CLIENT	MR. ROSS HOWES
PREPARED BY	MOHAMMED NAYEEM MSc. (Hons.)

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REFERENCE:

CALCULATIONS ARE PREPARED IN ACCORDANCE WITH THE FOLLOWING CODES OF PRACTICE:

BS 5268 [TIMBER C22 / C24 GRADE]
CODE OF PRACTICE FOR USE OF TIMBER

BS 648 [WEIGHT OF BUILDING MATERIAL]
CODE OF PRACTICE FOR BUILDING MATERIAL

BS 5950 PART 1 – 2000
STRUCTURAL USE OF STEELWORK IN BUILDING

BS 5628 PART 1 – 1992
CODE OF PRACTICE FOR USE OF MASONRY

BS 6399 PART 1 – 1996
LOADING FOR BUILDINGS
[DEAD AND IMPOSED LOADS]

BS 6399 PART 3 - 1988 LOADING FOR BUILDINGS
[IMPOSED ROOF LOAD]

SPAN TABLES FOR SOLID TIMBER MEMBERS IN FLOORS, CEILINGS AND ROOFS FOR DWELLINGS --TRADA 2004

LOAD CALCULATIONS:

FLAT ROOF

	DEAD LOAD (kN/m^2)	**LIVE LOAD (kN/m^2)**
PLASTER BOARD	0.12	SNOW & REPAIR ACCESS = 0.75 kN/m^2
JOIST	0.18	-
FELT	0.15	-
CHIPPINGS	0.24	-
BOARDING	0.12	-
TOTAL	**0.81 kN/m^2**	**0.75 kN/m^2**

PITCHED ROOF

	DEAD LOAD (kN/m^2)	**LIVE LOAD (kN/m^2)**
RAFTERS /m2	0.12	SNOW & REPAIR ACCESS = 0.75 kN
BATTENS	0.04	-
FELT	0.05	-
TILES	0.68	-
	0.89 (On Slope)	-
	0.89 / Cos 26 Degrees	-
TOTAL	**= 1.00 (On Plan)**	**0.75 kN/m^2**

FLOOR

	DEAD LOAD (kN/m^2)	**LIVE LOAD (kN/m^2)**
CEILING	0.15	IMPOSED = 1.50 kN /m2
JOIST	0.15	-
BOARDING	0.20	-
TOTAL	**0.50 kN/m^2**	**1.50 kN/m^2**

CEILING

	DEAD LOAD (kN/m^2)	**LIVE LOAD (kN/m^2)**
CEILING	0.25	-
TOTAL	**0.25 kN/m^2**	-

BRICK WORK

	DEAD LOAD (kN/m^3)	**LIVE LOAD (kN/m^3)**
BRICKS	22.00	-
TOTAL	**22.00 kN/**m^3	-

BLOCK WORK

	DEAD LOAD (kN/m^3)	**LIVE LOAD (kN/m^3)**
BLOCKS	14.00	-
TOTAL	**14.00 kN/**m^3	-

INTERNAL STUDDING

	DEAD LOAD (kN/m^2)	**LIVE LOAD (kN/m^2)**
PLASTER BOARD	0.04	-
STUD	0.68	-
TOTAL	**0.72 kN/m^2**	-

EXTERNAL STUDDING

	DEAD LOAD (kN/m^2)	**LIVE LOAD (kN/m^2)**
PLASTER BOARD	0.12	-
CLADDING	0.68	-
FELT	0.05	-
BATTENS	0.04	-
STUD	0.12	-
TOTAL	**1.01 kN/m^2**	-

GENERAL AND SAFETY NOTES:

1. All dimensions, setting out and levels are to be verified on site with the architect prior to the commencement of any site work.

2. Building control approval must be obtained prior to the commencement of building works.

3.The contractor shall be responsible for and must take all necessary precautions to ensure the stability of the existing structure and earthworks on adjoining sites during the course of the contract.

4.Materials and constructions are to be in accordance with the relevant British Standards and Codes of Practice.

5.Any services or drainage which pass through the foundation are to be encased in a flexible sleeve.

6. It is the responsibility of the Client / Contractor to notify the designer of any discrepancies.

7. Structural Drawings provided here are based on architectural drawings and site survey therefore are indicative only and are NOT TO SCALE.

8. All existing foundations and lintels to be exposed and to be checked for adequacy by Builder / Building Control Officer and or replaced if necessary.

9. Depth of all footings is to be approved by the local building inspector. Minimum of 1 m deep, advise engineer if footings are located within 30m of large trees or hedges. Also notify if water is struck, tree roots are found or clay soil appears very dry and brittle.

LOAD CALCULATIONS FOR TIMBER BEAM 1 & 2:

CLEAR SPAN OF THE TIMBER BEAM 1 & 2 = 2678mm

LOADING:

DEAD LOAD ON TIMBER BEAM 1 & 2:

PITCHED ROOF = 1.00 kN/m^2 x SPAN OF ROOF IN MTS.

= 1.00 kN/m^2 x 3.00

= 3.00 kN/m

THEREFORE TOTAL DEAD LOAD ON TB12 = 3.00 kN/m.

LIVE LOAD ON TIMBER BEAM 1 & 2:

PITCHED ROOF = 0.75 kN/m^2 x SPAN OF ROOF IN MTS.

= 0.75 kN/m^2 x 3.00

= 2.25 kN/m

THEREFORE TOTAL LIVE LOAD ON TB12= 2.25 kN/m

STRUCTURAL REPORT FOR TIMBER BEAM 1 & 2:

Analysis for a simply-supported single-span timber beam to BS 5268

TEDDS calculation version 1.0.02

Span length & partial factors for loading

Span (mm)	Factors for moments & forces			Factors for deflection		
	γ_{fd}	γ_{fi}	γ_{fw}	γ_{dd}	γ_{di}	γ_{dw}
2678	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**

Loading data (unfactored)

Ref.	Category	Type	Load kN/m	Position mm	Load kN/m	Position mm
1	**"Dead"**	UDL	**3.0**	**0**	-	**2678**
2	**"Imposed"**	UDL	**2.3**	**0**	-	**2678**

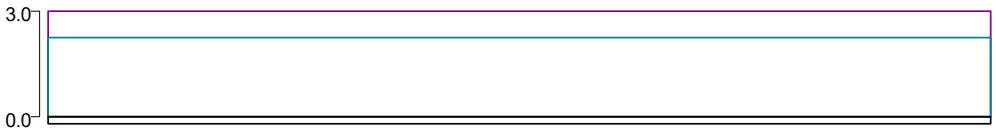
Analysis results - entire span

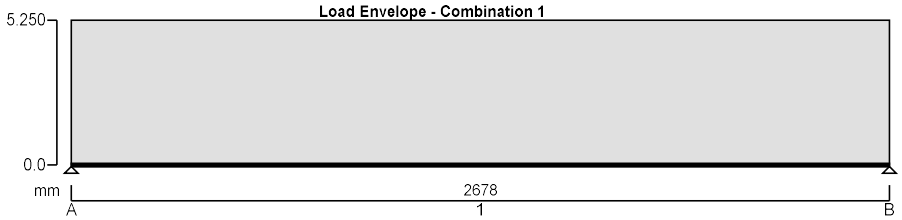
R_a	R_b	V	M		Deflection: δEI	
kN (fac)	kN (fac)	kN (fac)	kNm (fac)	Sense	kNm^3	Direction
7.0	**7.0**	**7.0**	**4.7**	**"Sagging"**	**3.52**	**"Down"**

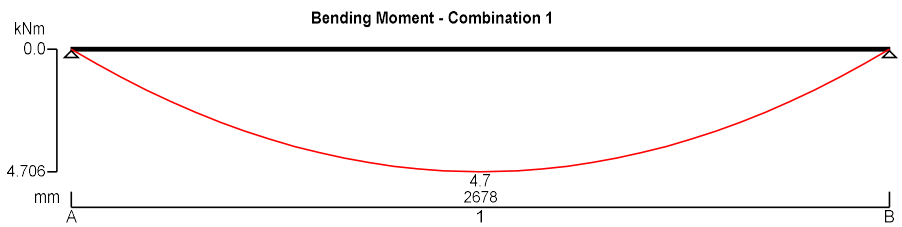
Unfactored support reactions

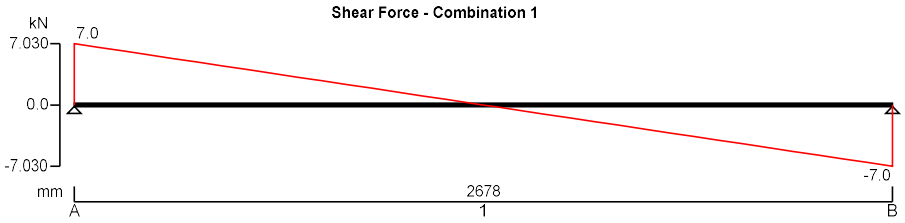
Support A	Dead load **-4.0** kN	Live load **-3.0** kN	Wind load **0.0** kN
Support B	Dead load **-4.0** kN	Live load **-3.0** kN	Wind load **0.0** kN

Beam Loads









Member design checks for a simply-supported single-span timber beam to BS 5268

Timber member design BS 5268-2:2002

Summary of results				
Section size	D = **225** mm	B = **150** mm	A= **33750** mm^2	
Section properties (x-x)	I_{xx} = **142382813** mm^4	Z_{xx} = **1265625** mm^3	r_{xx} = **65.0** mm	
(y-y)	I_{yy} = **63281250** mm^4	Z_{yy} = **843750** mm^3	r_{yy} = **43.3** mm	
Grade	**"C22"**	σ_c = **7.50** N/ mm^2		
Check	*Stress*	*Capacity*	*Notes*	*Result*
Bending stress	$\sigma_{m.a.para}$ = M / Z_{xx} = **3.72** N/mm^2	$\sigma_{m.adm.para}$ = **5.61** N/mm^2	Moment M = **4.7** kNm	**Pass**
Shear stress	τ_a = **0.31** N/mm^2	τ_{adm} = **0.64** N/mm^2	Shear V = **7.0** kN	**Pass**
Deflection	δ = **4.2** mm	14 mm		**Pass**
	δ / L_s = **0.00157**	0.003		**Pass**

LOAD CALCULATIONS FOR TIMBER BEAM 3:

CLEAR SPAN OF THE TIMBER BEAM 3 = 2250mm

LOADING:

DEAD LOAD ON TIMBER BEAM 3:

PITCHED ROOF = 1.00 kN/m^2 x SPAN OF ROOF IN MTS.

= 1.00 kN/m^2 x 3.00

= 3.00 kN/m

THEREFORE TOTAL DEAD LOAD ON TB3 = 3.00 kN/m.

LIVE LOAD ON TIMBER BEAM 3:

PITCHED ROOF = 0.75 kN/m^2 x SPAN OF ROOF IN MTS.

= 0.75 kN/m^2 x 3.00

= 2.25 kN/m

THEREFORE TOTAL LIVE LOAD ON TB3= 2.25 kN/m

STRUCTURAL REPORT FOR TIMBER BEAM 3:

Analysis for a simply-supported single-span timber beam to BS 5268

TEDDS calculation version 1.0.02

Span length & partial factors for loading

Span (mm)	Factors for moments & forces			Factors for deflection		
	γ_{fd}	γ_{fi}	γ_{fw}	γ_{dd}	γ_{di}	γ_{dw}
2250	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**

Loading data (unfactored)

Ref.	Category	Type	Load kN/m	Position mm	Load kN/m	Position mm
1	**"Dead"**	UDL	**3.0**	**0**	-	**2250**
2	**"Imposed"**	UDL	**2.3**	**0**	-	**2250**

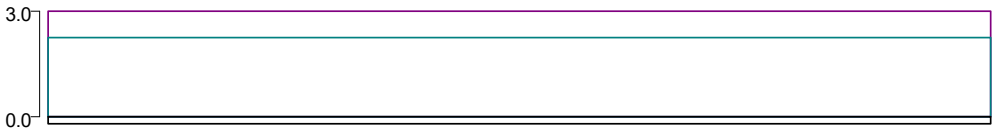
Analysis results - entire span

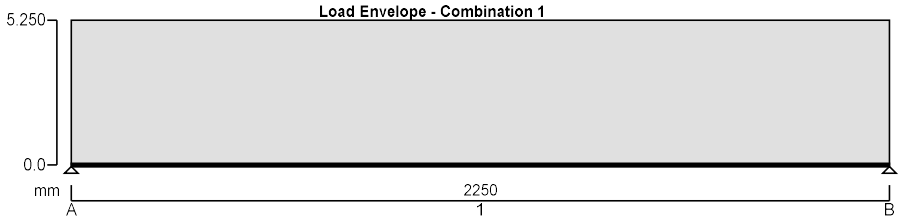
R_a	R_b	V	M		Deflection: δEI	
kN (fac)	kN (fac)	kN (fac)	kNm (fac)	Sense	kNm^3	Direction
5.9	**5.9**	**5.9**	**3.3**	**"Sagging"**	**1.75**	**"Down"**

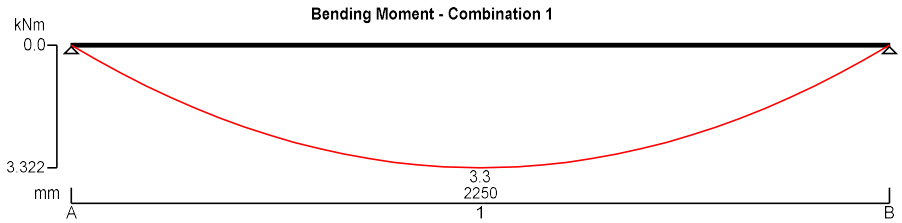
Unfactored support reactions

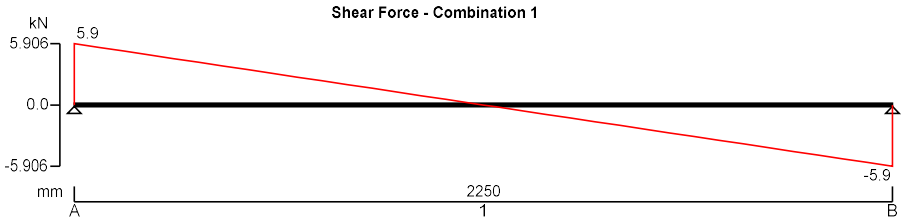
Support A	Dead load **-3.4** kN	Live load **-2.5** kN	Wind load **0.0** kN
Support B	Dead load **-3.4** kN	Live load **-2.5** kN	Wind load **0.0** kN

Beam Loads









Member design checks for a simply-supported single-span timber beam to BS 5268

Timber member design BS 5268-2:2002

Summary of results				
Section size	D = **225** mm	B = **150** mm	A= **33750** mm^2	
Section properties (x-x)	I_{xx} = **142382813** mm^4	Z_{xx} = **1265625** mm^3	r_{xx} = **65.0** mm	
(y-y)	I_{yy} = **63281250** mm^4	Z_{yy} = **843750** mm^3	r_{yy} = **43.3** mm	
Grade	**"C24"**	σ_c = **7.90** N/ mm^2		
Check	*Stress*	*Capacity*	*Notes*	*Result*
Bending stress	$\sigma_{m.a.para}$ = M / Z_{xx} = **2.63** N/mm^2	$\sigma_{m.adm.para}$ = **7.74** N/mm^2	Moment M = **3.3** kNm	**Pass**
Shear stress	τ_a = **0.26** N/mm^2	τ_{adm} = **0.71** N/mm^2	Shear V = **5.9** kN	**Pass**
Deflection	δ = **1.3** mm	14 mm		**Pass**
	δ / L_s = **0.00058**	0.003		**Pass**

LOAD CALCULATIONS FOR TIMBER BEAM 4:

CLEAR SPAN OF THE TIMBER BEAM 4 = 1400mm

.

SPAN 1 = 1400mm AND SPAN 2 = 1400mm

LOADING:

DEAD LOAD ON TIMBER BEAM 4:

PITCHED ROOF = 1.00 kN/m^2 x SPAN OF ROOF IN MTS.

= 1.00 kN/m^2 x 3.00

= 3.00 kN/m

THEREFORE TOTAL DEAD LOAD ON TB4 = 3.00 kN/m.

LIVE LOAD ON TIMBER BEAM 4:

PITCHED ROOF = 0.75 kN/m^2 x SPAN OF ROOF IN MTS.

= 0.75 kN/m^2 x 3.00

= 2.25 kN/m

THEREFORE TOTAL LIVE LOAD ON TB4 = 2.25 kN/m

STRUCTURAL REPORT FOR TIMBER BEAM 4:

Analysis for a simply-supported single-span timber beam to BS 5268

TEDDS calculation version 1.0.02

Span length & partial factors for loading

Span (mm)	Factors for moments & forces			Factors for deflection		
	γ_{fd}	γ_{fi}	γ_{fw}	γ_{dd}	γ_{di}	γ_{dw}
1400	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**

Loading data (unfactored)

Ref.	Category	Type	Load kN/m	Position mm	Load kN/m	Position mm
1	**"Dead"**	UDL	**3.0**	**0**	-	**1400**
2	**"Imposed"**	UDL	**2.3**	**0**	-	**1400**

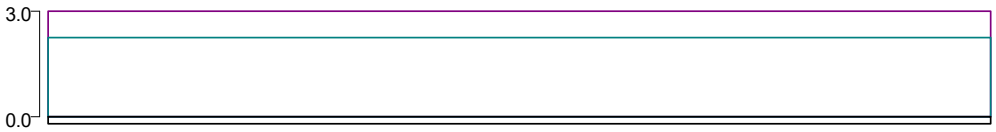
Analysis results - entire span

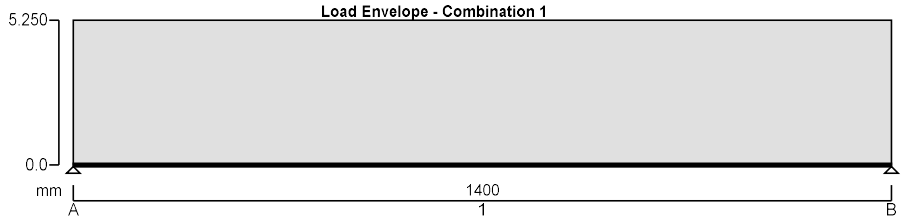
R_a	R_b	V	M		Deflection: δEI	
kN (fac)	kN (fac)	kN (fac)	kNm (fac)	Sense	kNm^3	Direction
3.7	**3.7**	**3.7**	**1.3**	**"Sagging"**	**0.26**	**"Down"**

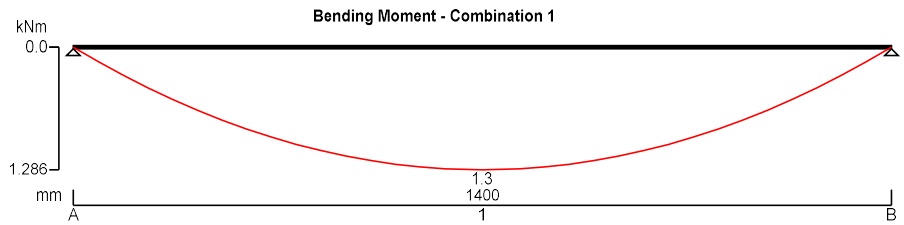
Unfactored support reactions

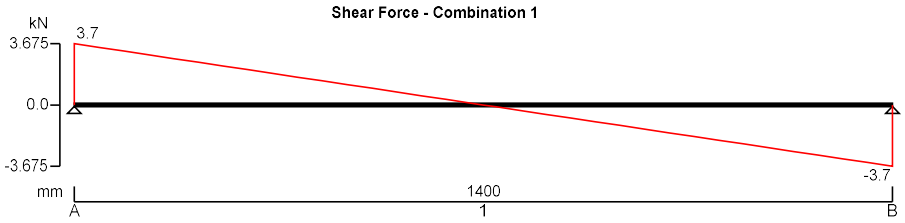
Support A	Dead load **-2.1** kN	Live load **-1.6** kN	Wind load **0.0** kN
Support B	Dead load **-2.1** kN	Live load **-1.6** kN	Wind load **0.0** kN

Beam Loads









Member design checks for a simply-supported single-span timber beam to BS 5268

Timber member design BS 5268-2:2002

Summary of results				
Section size	D = **225** mm	B = **150** mm	A= **33750** mm^2	
Section properties (x-x)	I_{xx} = **142382813** mm^4	Z_{xx} = **1265625** mm^3	r_{xx} = **65.0** mm	
(y-y)	I_{yy} = **63281250** mm^4	Z_{yy} = **843750** mm^3	r_{yy} = **43.3** mm	
Grade	**"C24"**	σ_c = **7.90** N/ mm^2		
Check	*Stress*	*Capacity*	*Notes*	*Result*
Bending stress	$\sigma_{m.a.para}$ = M / Z_{xx} = **1.02** N/mm^2	$\sigma_{m.adm.para}$ = **7.74** N/mm^2	Moment M = **1.3** kNm	**Pass**
Shear stress	τ_a = **0.16** N/mm^2	τ_{adm} = **0.71** N/mm^2	Shear V = **3.7** kN	**Pass**
Deflection	δ = **0.2** mm	14 mm		**Pass**
	δ / L_s = **0.00017**	0.003		**Pass**

LOAD CALCULATIONS FOR TIMBER BEAM 5:

CLEAR SPAN OF THE TIMBER BEAM 5 = 1565mm
.

LOADING:

DEAD LOAD ON TIMBER BEAM 5:

PITCHED ROOF = 1.00 kN/m^2 x SPAN OF ROOF IN MTS.

= 1.00 kN/m^2 x 3.00

= 3.00 kN/m

STUD WORK (100mm) = 1.01 kN/m^2 x HEIGHT OF STUD PARTITION IN MTS.

= 1.01 kN/m^2 x 2.00m

= 2.02 kN/m

THEREFORE TOTAL DEAD LOAD ON TB5 = 3.00 + 2.02 = 5.02kN/m.

LIVE LOAD ON TIMBER BEAM 5:

PITCHED ROOF = 0.75 kN/m^2 x SPAN OF ROOF IN MTS.

= 0.75 kN/m^2 x 3.00

= 2.25 kN/m

THEREFORE TOTAL LIVE LOAD ON TB5 = 2.25 kN/m

STRUCTURAL REPORT FOR TIMBER BEAM 5:

Analysis for a simply-supported single-span timber beam to BS 5268

TEDDS calculation version 1.0.02

Span length & partial factors for loading

Span (mm)	Factors for moments & forces			Factors for deflection		
	γ_{fd}	γ_{fi}	γ_{fw}	γ_{dd}	γ_{di}	γ_{dw}
1565	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**

Loading data (unfactored)

Ref.	Category	Type	Load kN/m	Position mm	Load kN/m	Position mm
1	**"Dead"**	UDL	**5.0**	**0**	-	**1565**
2	**"Imposed"**	UDL	**2.3**	**0**	-	**1565**

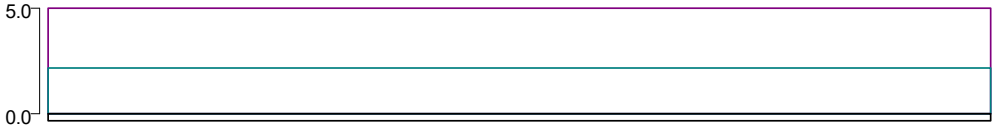
Analysis results - entire span

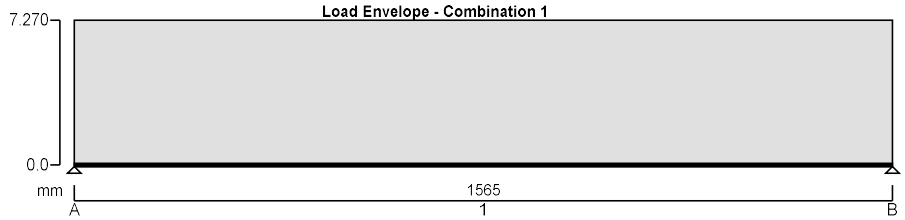
R_a kN (fac)	R_b kN (fac)	V kN (fac)	M kNm (fac)	M Sense	Deflection: δEI kNm^3	Deflection: δEI Direction
5.7	**5.7**	**5.7**	**2.2**	**"Sagging"**	**0.57**	**"Down"**

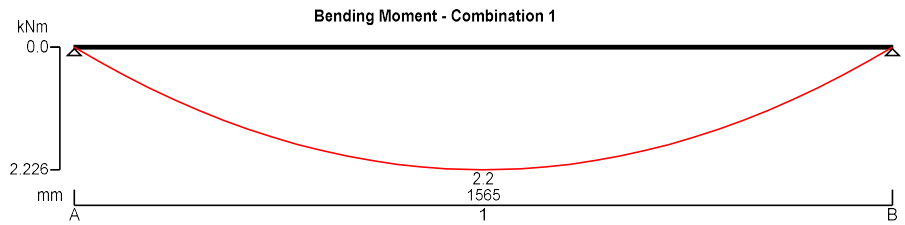
Unfactored support reactions

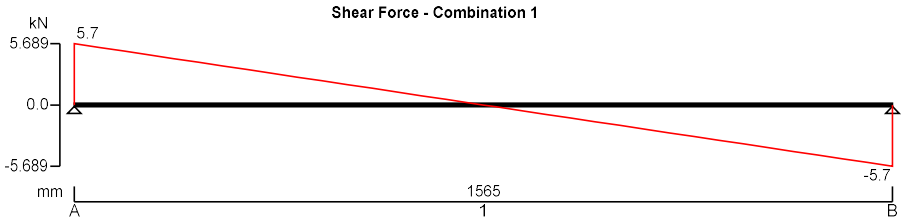
Support A	Dead load **-3.9** kN	Live load **-1.8** kN	Wind load **0.0** kN
Support B	Dead load **-3.9** kN	Live load **-1.8** kN	Wind load **0.0** kN

Beam Loads









Member design checks for a simply-supported single-span timber beam to BS 5268

Timber member design BS 5268-2:2002

Summary of results				
Section size	D = **150** mm	B = **150** mm	A= **22500** mm^2	
Section properties (x-x)	I_{xx} = **42187500** mm^4	Z_{xx} = **562500** mm^3	r_{xx} = **43.3** mm	
(y-y)	I_{yy} = **42187500** mm^4	Z_{yy} = **562500** mm^3	r_{yy} = **43.3** mm	
Grade	**"C24"**	σ_c = **7.90** N/ mm^2		
Check	*Stress*	*Capacity*	*Notes*	*Result*
Bending stress	$\sigma_{m.a.para}$ = M / Z_{xx} = **3.96** N/mm^2	$\sigma_{m.adm.para}$ = **8.09** N/mm^2	Moment M = **2.2** kNm	**Pass**
Shear stress	τ_a = **0.38** N/mm^2	τ_{adm} = **0.71** N/mm^2	Shear V = **5.7** kN	**Pass**
Deflection	δ = **1.4** mm	14 mm		**Pass**
	δ / L_s = **0.00091**	0.003		**Pass**

LOAD CALCULATIONS FOR TIMBER BEAM 6:

CLEAR SPAN OF THE TIMBER BEAM 6 = 1150mm
.

LOADING:

DEAD LOAD ON TIMBER BEAM 6:

PITCHED ROOF = 1.00 kN/m^2 x SPAN OF ROOF IN MTS.

= 1.00 kN/m^2 x 3.00

= 3.00 kN/m

STUD WORK (100mm) = 1.01 kN/m^2 x HEIGHT OF STUD PARTITION IN MTS.

= 1.01 kN/m^2 x 2.00m

= 2.02 kN/m

THEREFORE TOTAL DEAD LOAD ON TB6 = 3.00 + 2.02 = 5.02kN/m.

LIVE LOAD ON TIMBER BEAM 6:

PITCHED ROOF = 0.75 kN/m^2 x SPAN OF ROOF IN MTS.

= 0.75 kN/m^2 x 3.00

= 2.25 kN/m

THEREFORE TOTAL LIVE LOAD ON TB6 = 2.25 kN/m

STRUCTURAL REPORT FOR TIMBER BEAM 6:

Analysis for a simply-supported single-span timber beam to BS 5268

TEDDS calculation version 1.0.02

Span length & partial factors for loading

Span (mm)	Factors for moments & forces			Factors for deflection		
	γ_{fd}	γ_{fi}	γ_{fw}	γ_{dd}	γ_{di}	γ_{dw}
1150	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**

Loading data (unfactored)

Ref.	Category	Type	Load kN/m	Position mm	Load kN/m	Position mm
1	**"Dead"**	UDL	**5.0**	**0**	-	**1150**
2	**"Imposed"**	UDL	**2.3**	**0**	-	**1150**

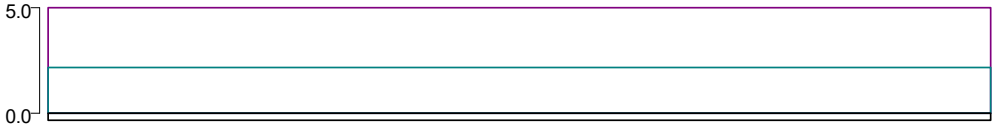
Analysis results - entire span

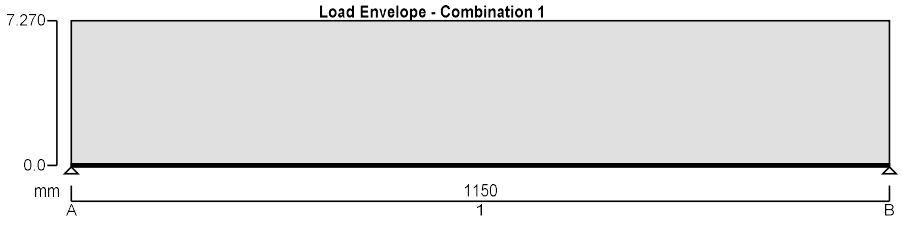
R_a kN (fac)	R_b kN (fac)	V kN (fac)	M kNm (fac)	M Sense	Deflection: δEI kNm^3	Deflection: δEI Direction
4.2	**4.2**	**4.2**	**1.2**	**"Sagging"**	**0.17**	**"Down"**

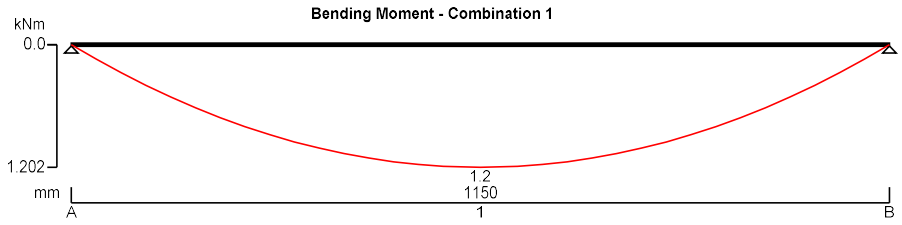
Unfactored support reactions

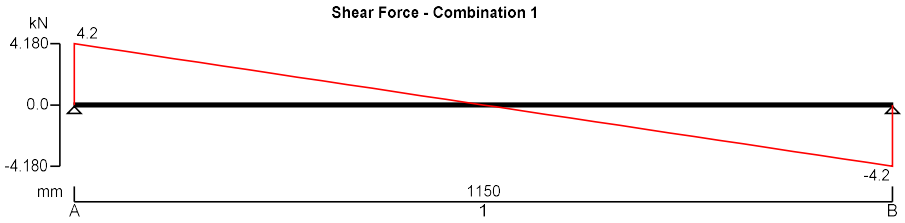
Support A	Dead load **-2.9** kN	Live load **-1.3** kN	Wind load **0.0** kN
Support B	Dead load **-2.9** kN	Live load **-1.3** kN	Wind load **0.0** kN

Beam Loads









Member design checks for a simply-supported single-span timber beam to BS 5268

Timber member design BS 5268-2:2002

Summary of results				
Section size	D = **150** mm	B = **150** mm	A= **22500** mm^2	
Section properties (x-x)	I_{xx} = **42187500** mm^4	Z_{xx} = **562500** mm^3	r_{xx} = **43.3** mm	
(y-y)	I_{yy} = **42187500** mm^4	Z_{yy} = **562500** mm^3	r_{yy} = **43.3** mm	
Grade	**"C24"**	σ_c = **7.90** N/ mm^2		
Check	*Stress*	*Capacity*	*Notes*	*Result*
Bending stress	$\sigma_{m.a.para}$ = M / Z_{xx} = **2.14** N/mm^2	$\sigma_{m.adm.para}$ = **8.09** N/mm^2	Moment M = **1.2** kNm	**Pass**
Shear stress	τ_a = **0.28** N/mm^2	τ_{adm} = **0.71** N/mm^2	Shear V = **4.2** kN	**Pass**
Deflection	δ = **0.5** mm	14 mm		**Pass**
	δ / L_s = **0.00040**	0.003		**Pass**

LOAD CALCULATIONS FOR TIMBER BEAM 7:

CLEAR SPAN OF THE TIMBER BEAM 7 = 1400mm

.

LOADING:

DEAD LOAD ON TIMBER BEAM 7:

PITCHED ROOF = 1.00 kN/m^2 x SPAN OF ROOF IN MTS.

= 1.00 kN/m^2 x 3.00

= 3.00 kN/m

THEREFORE TOTAL DEAD LOAD ON TB7 = 3.00 kN/m.

LIVE LOAD ON TIMBER BEAM 7:

PITCHED ROOF = 0.75 kN/m^2 x SPAN OF ROOF IN MTS.

= 0.75 kN/m^2 x 3.00

= 2.25 kN/m

THEREFORE TOTAL LIVE LOAD ON TB7 = 2.25 kN/m

STRUCTURAL REPORT FOR TIMBER BEAM 7:

Analysis for a simply-supported single-span timber beam to BS 5268

TEDDS calculation version 1.0.02

Span length & partial factors for loading

Span (mm)	Factors for moments & forces			Factors for deflection		
	γ_{fd}	γ_{fi}	γ_{fw}	γ_{dd}	γ_{di}	γ_{dw}
1400	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**

Loading data (unfactored)

Ref.	Category	Type	Load kN/m	Position mm	Load kN/m	Position mm
1	**"Dead"**	UDL	**3.0**	**0**	-	**1400**
2	**"Imposed"**	UDL	**2.3**	**0**	-	**1400**

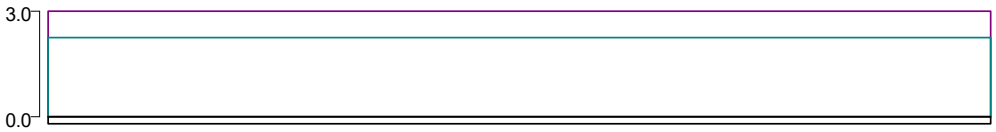
Analysis results - entire span

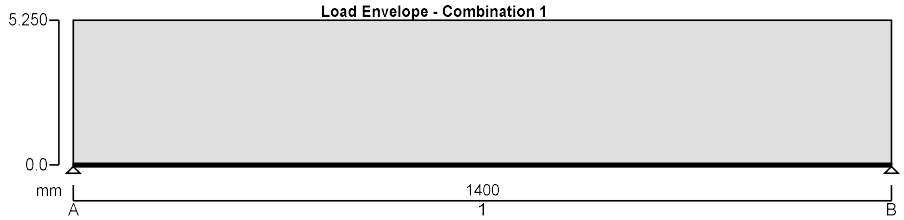
R_a	R_b	V	M		Deflection: δEI	
kN (fac)	kN (fac)	kN (fac)	kNm (fac)	Sense	kNm^3	Direction
3.7	**3.7**	**3.7**	**1.3**	**"Sagging"**	**0.26**	**"Down"**

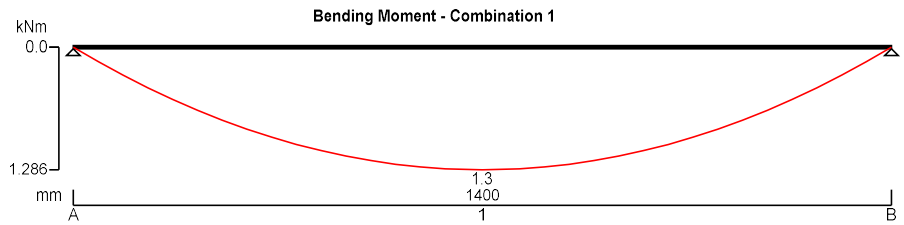
Unfactored support reactions

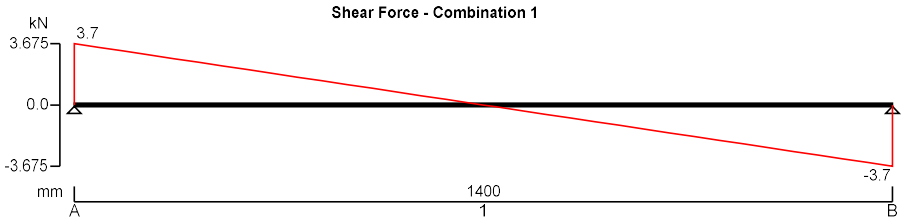
Support A	Dead load **-2.1** kN	Live load **-1.6** kN	Wind load **0.0** kN
Support B	Dead load **-2.1** kN	Live load **-1.6** kN	Wind load **0.0** kN

Beam Loads









Member design checks for a simply-supported single-span timber beam to BS 5268

Timber member design BS 5268-2:2002

Summary of results				
Section size	D = **150** mm	B = **150** mm	A= **22500** mm^2	
Section properties (x-x)	I_{xx} = **42187500** mm^4	Z_{xx} = **562500** mm^3	r_{xx} = **43.3** mm	
(y-y)	I_{yy} = **42187500** mm^4	Z_{yy} = **562500** mm^3	r_{yy} = **43.3** mm	
Grade	**"C24"**	σ_c = **7.90** N/ mm^2		
Check	*Stress*	*Capacity*	*Notes*	*Result*
Bending stress	$\sigma_{m.a.para}$ = M / Z_{xx} = **2.29** N/mm^2	$\sigma_{m.adm.para}$ = **8.09** N/mm^2	Moment M = **1.3** kNm	**Pass**
Shear stress	τ_a = **0.25** N/mm^2	τ_{adm} = **0.71** N/mm^2	Shear V = **3.7** kN	**Pass**
Deflection	δ = **0.7** mm	14 mm		**Pass**
	δ / L_s = **0.00048**	0.003		**Pass**

LOAD CALCULATIONS FOR 150 mm TIMBER POST 1, 2 & 3:

Try 150x150mm Square Timber Post 1, 2 & 3

Type of timber = 'Rough-Sawn Acoya'

Length of Timber Post = 2200mm [Builder to confirm on site prior to ordering]

Timber Grade = "C22"

Load from Timber Beam 1 & 2 = 14.00 kN

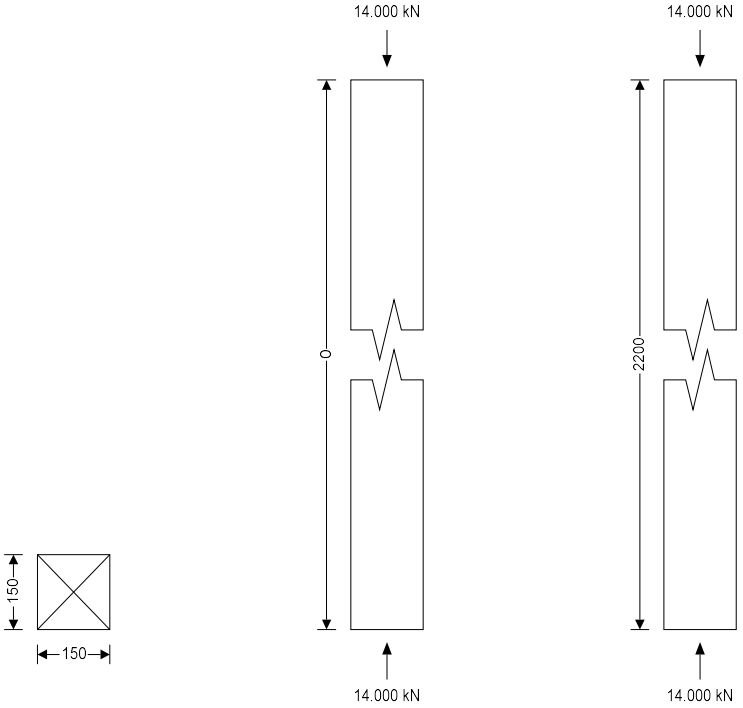
STRUCTURAL REPORT FOR 150mm TIMBER POST 1, 2 & 3:

TIMBER COLUMN DESIGN TO BS5268-2:2002

TEDDS calculation version 1.0.01

Column loading

Vertical applied load	F = **14.000** kN **(Compression)**
Duration of vertical applied load	**Long term**



Timber section details

Timber breadth	b = **150** mm
Timber depth	h = **150** mm
Number of timbers forming column	N = **1**
Timber strength class	**C22**

Section properties

Cross sectional area	$A = N \times b \times h$ = **22500** mm^2
Section modulus	$Z_x = N \times b \times h^2 / 6$ = **562500** mm^3
	$Z_y = N \times h \times b^2 / 6$ = **562500** mm^3
Moment of inertia	$I_x = N \times b \times h^3 / 12$ = **42187500** mm^4
	$I_y = N \times h \times b^3 / 12$ = **42187500** mm^4
Radius of gyration	$r_x = \sqrt{(I_x / A)}$ = **43.3** mm
	$r_y = \sqrt{(I_y / A)}$ = **43.3** mm

Column details

Service class of timber	**1**
Unbraced height of column in the y-axis	L_y = **2200** mm
Effective length of column in the y-axis	$L_{ey} = L_y \times 1.00$ = **2200** mm
Slenderness ratio	$\lambda = L_{ey} / r_y$ = **50.81**
Permissible column slenderness	λ_{max} = **180**

PASS - Slenderness ratio is less than permissible column slenderness

Modification factors

Service class for bending parallel to grain	K_{2m} = **1.00**

Service class for tension parallel to grain	K_{2t} = **1.00**
Service class for compression parallel to grain	K_{2c} = **1.00**
Duration of loading	K_3 = **1.00**
Depth of section	$K_7 = (300\ mm / h)^{0.11}$ = **1.08**
Load sharing	K_8 = **1.10**
Multiple member factor	K_9 = **1.00**
Compression member factor	K_{12} = **0.73** (From table 22 where $E / \sigma_{c\|\|} = 867$ and $\lambda = 51$)

Check compressive stress

Compression parallel to grain	σ_c = **7.500** N/mm^2
Permissible compressive stress	$\sigma_{c_adm} = \sigma_c \times K_{2c} \times K_3 \times K_8 \times K_{12}$ = **5.986** N/mm^2
Applied compressive stress	$\sigma_{c_max} = F / A$ = **0.622** N/mm^2

PASS - Applied compressive stress is within permissible limits

LOAD CALCULATIONS FOR 100 mm TIMBER POST FOR TIMBER BEAM 3,4,5,67:

Try 100x100mm Square Timber Post

Length of Timber Post = 2200mm

Timber Grade = “C24”

Load from Timber Beam 3 = 6.00 kN [Worst case loading]

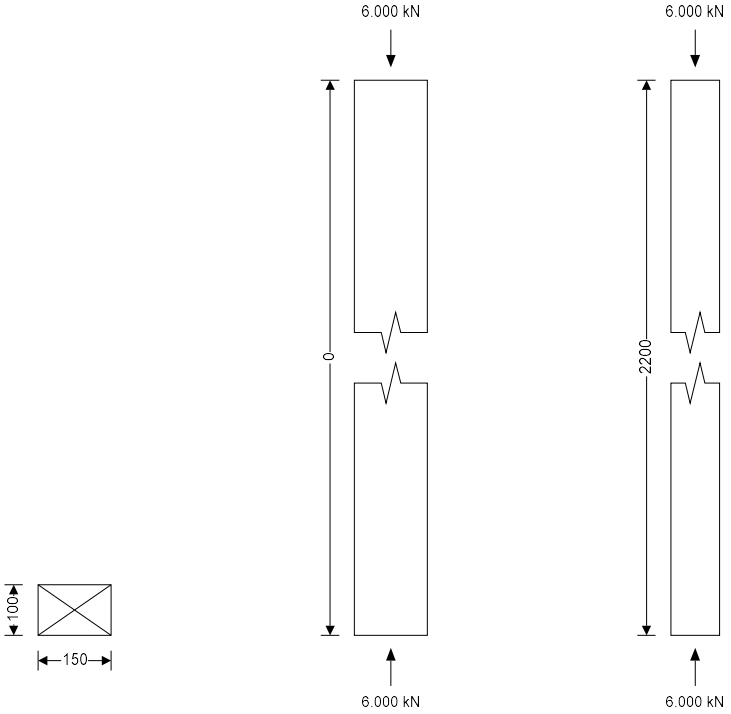
STRUCTURAL REPORT FOR 100mm TIMBER POST FOR TIMBER BEAM 3,4,5,6,7:

TIMBER COLUMN DESIGN TO BS5268-2:2002

TEDDS calculation version 1.0.01

Column loading

Vertical applied load	F = **6.000** kN **(Compression)**
Duration of vertical applied load	**Long term**



Timber section details

Timber breadth	b = **100** mm
Timber depth	h = **150** mm
Number of timbers forming column	N = **1**
Timber strength class	**C24**

Section properties

Cross sectional area	$A = N \times b \times h$ = **15000** mm^2
Section modulus	$Z_x = N \times b \times h^2 / 6$ = **375000** mm^3
	$Z_y = N \times h \times b^2 / 6$ = **250000** mm^3
Moment of inertia	$I_x = N \times b \times h^3 / 12$ = **28125000** mm^4
	$I_y = N \times h \times b^3 / 12$ = **12500000** mm^4
Radius of gyration	$r_x = \sqrt{(I_x / A)}$ = **43.3** mm
	$r_y = \sqrt{(I_y / A)}$ = **28.9** mm

Column details

Service class of timber	**1**
Unbraced height of column in the y-axis	L_y = **2200** mm
Effective length of column in the y-axis	$L_{ey} = L_y \times 1.00$ = **2200** mm
Slenderness ratio	$\lambda = L_{ey} / r_y$ = **76.21**
Permissible column slenderness	λ_{max} = **180**

PASS - Slenderness ratio is less than permissible column slenderness

Modification factors

Service class for bending parallel to grain	K_{2m} = **1.00**

Service class for tension parallel to grain	K_{2t} = **1.00**
Service class for compression parallel to grain	K_{2c} = **1.00**
Duration of loading	K_3 = **1.00**
Depth of section	$K_7 = (300\ mm / h)^{0.11}$ = **1.08**
Load sharing	K_8 = **1.10**
Multiple member factor	K_9 = **1.00**
Compression member factor	K_{12} = **0.55** (From table 22 where $E / \sigma_{c\|\|}$ = 911 and λ = 76)

Check compressive stress

Compression parallel to grain	σ_c = **7.900** N/mm^2
Permissible compressive stress	$\sigma_{c_adm} = \sigma_c \times K_{2c} \times K_3 \times K_8 \times K_{12}$ = **4.784** N/mm^2
Applied compressive stress	$\sigma_{c_max} = F / A$ = **0.400** N/mm^2

PASS - Applied compressive stress is within permissible limits

LOAD CALCULATIONS FOR STRIP FOUNDATION:

ASSUMIND 700mm WIDE MASS CONCRETE FOUNDATION

ASSUMIND GOOD STRATA AT 1200mm DEEP MASS CONCRETE FOUNDATION

ECCENTRICITY = 0mm [TO BE CHECKED BY BUILDER ON SITE]

LOADING:

DEAD LOAD ON FOUNDATION:

BRICK WORK = 22 kN/m^3 x THICKNES OF WALL x HEIGHT OF WALL IN MTS.
(100mm)

= 22 x 0.100 x 1.00m **(x2)**

= 4.40 kN/m

STUD WORK = 1.01 kN/m^2 x HEIGHT OF STUD PARTITION IN MTS.
(100mm)

= 1.01 kN/m^2 x 3.00m

= 3.03 kN/m

PITCHED ROOF = 1.00 kN/m^2 x SPAN OF ROOF IN MTS.

= 1.00 kN/m^2 x 3.00m

= 3.00 kN/m

THEREFORE TOTAL DEAD LOAD ON SF= 4.40 + 3.03 + 3.00 = 10.43 kN/m

LIVE LOAD ON STRIP FOUNDATION:

PITCHED ROOF = 0.75 kN/m^2 x SPAN OF ROOF IN MTS.

= 0.75 kN/m^2 x 3.00m

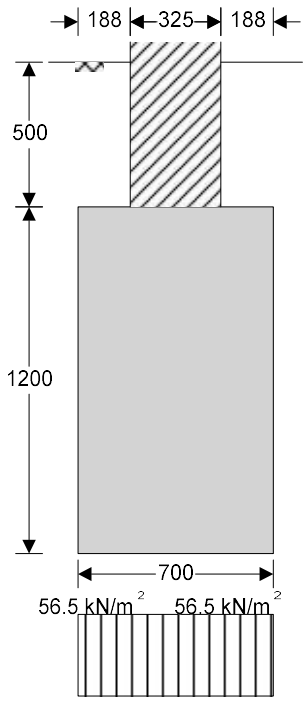
= 2.25 kN/m

THEREFORE TOTAL LIVE LOAD ON SF = 2.25 kN/m

STRUCTURAL REPORT FOR STRIP FOUNDATION:

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

TEDDS calculation version 2.0.02.00



Strip footing details

Width of strip footing	B = **700** mm
Depth of strip footing	h = **1200** mm
Depth of soil over strip footing	h_{soil} = **500** mm
Density of concrete	ρ_{conc} = **23.6** kN/m^3

Load details

Load width	b = **325** mm
Load eccentricity	e_P = **0** mm

Soil details

Dense, moderately graded, sub-angular, gravel

Mobilisation factor	m = **1.5**
Density of soil	ρ_{soil} = **20.0** kN/m^3
Design shear strength	ϕ' = **25.0** deg
Design base friction	δ = **19.3** deg
Allowable bearing pressure	$P_{bearing}$ = **100** kN/m^2

Axial loading on strip footing

Dead axial load	P_G = **10.5** kN/m
Imposed axial load	P_Q = **2.3** kN/m
Wind axial load	P_W = **0.0** kN/m
Total axial load	P = **12.8** kN/m

Foundation loads

Dead surcharge load	F_{Gsur} = **0.000** kN/m^2
Imposed surcharge load	F_{Qsur} = **0.000** kN/m^2
Strip footing self weight	$F_{swt} = h \times \rho_{conc}$ = **28.320** kN/m^2
Soil self weight	$F_{soil} = h_{soil} \times \rho_{soil}$ = **10.000** kN/m^2
Total foundation load	$F = B \times (F_{Gsur} + F_{Qsur} + F_{swt} + F_{soil})$ = **26.8** kN/m

Calculate base reaction

Total base reaction $T = F + P =$ **39.6** kN/m

Eccentricity of base reaction in x $e_T = (P \times e_P + M + H \times h) / T =$ **0** mm

Base reaction acts within middle third of base

Calculate base pressures

$q_1 = (T / B) \times (1 - 6 \times e_T / B) =$ **56.534** kN/m^2

$q_2 = (T / B) \times (1 + 6 \times e_T / B) =$ **56.534** kN/m^2

Minimum base pressure $q_{min} = \min(q_1, q_2) =$ **56.534** kN/m^2

Maximum base pressure $q_{max} = \max(q_1, q_2) =$ **56.534** kN/m^2

PASS - Maximum base pressure is less than allowable bearing pressure

Material details

Characteristic strength of concrete $f_{cu} =$ **30** N/mm^2

Calculate base lengths

Left hand length $B_L = B / 2 + e_P =$ **350** mm

Right hand length $B_R = B / 2 - e_P =$ **350** mm

Calculate rate of change of base pressure

Length of base reaction $B_x = B =$ **700** mm

Rate of change of base pressure $C_x = (q_1 - q_2) / B_x =$ **0.000** kN/m^2/m

Calculate minimum depth of unreinforced strip footing

Average pressure to left of strip footing $q_L = q_1 - C_x \times (B_L - b / 2) / 2 =$ **56.534** kN/m^2

Minimum depth to left of strip footing $h_{Lmin} = (B_L - b/2) \times \max(0.15 \times [(q_L / 1\ \text{kN/m}^2)^2 / (f_{cu} / 1\ \text{N/mm}^2)]^{1/4}, 1) =$ **188** mm

Average pressure to right of strip footing $q_R = q_2 + C_x \times (B_R - b / 2) / 2 =$ **56.534** kN/m^2

Minimum depth to right of strip footing $h_{Rmin} = (B_R - b/2) \times \max(0.15 \times [(q_R / 1\text{kN/m}^2)^2 / (f_{cu} / 1\text{N/mm}^2)]^{1/4}, 1) =$ **188** mm

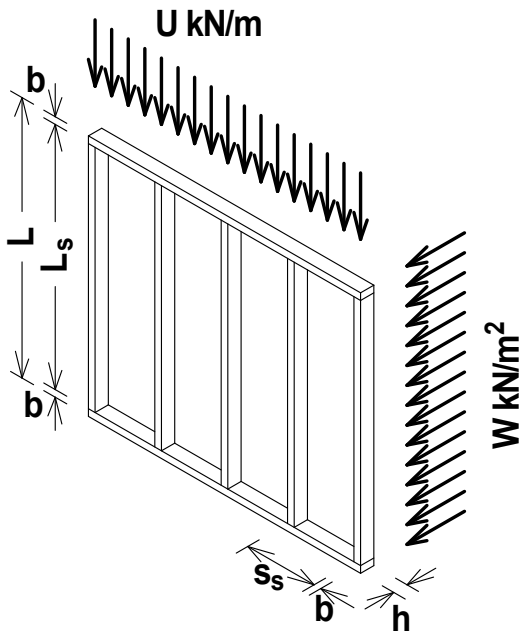
Minimum depth of unreinforced strip footing $h_{min} = \max(h_{Lmin}, h_{Rmin}, 300\ \text{mm}) =$ **300** mm

PASS - Unreinforced strip footing depth is greater than minimum

TIMBER STUD 50x150mm C24 DESIGN:

TIMBER STUD DESIGN (BS5268-2:2002)

TEDDS calculation version 1.0.03



Stud details

Stud breadth;	b = **50** mm
Stud depth;	h = **150** mm
Number of studs;	N_s = **1**

Strength class C24 timber (Table 8 BS5268:Pt 2:2002)

Section properties

Cross sectional area;	$A = N_s \times b \times h$ = **7500** mm^2
Section modulus;	$Z = N_s \times b \times h^2 / 6$ = **187500** mm^3
Moment of inertia in the major axis;	$I_x = N_s \times b \times h^3 / 12$ = **14062500** mm^4
Moment of inertia in the minor axis;	$I_y = N_s \times h \times b^3 / 12$ = **1562500** mm^4
Radius of gyration in the major axis;	$r_x = \sqrt{(I_x / A)}$ = **43.3** mm
Radius of gyration in the minor axis;	$r_y = \sqrt{(I_y / A)}$ = **14.4** mm

Panel details - Studs restrained by sheathing in the plane of the panel

Panel height;	L = **2400** mm
Stud length;	$L_s = L - (2 \times b)$ = **2300** mm
Standard stud spacing;	s_s = **400** mm
Panel opening;	O = **0** mm
Loaded panel length;	$s = \max(s_s, (O + s_s) / 2)$ = **400** mm
Effective length in the major axis;	$L_{ex} = 0.85 \times L_s$ = **1955** mm
Slenderness ratio;	$\lambda = L_{ex} / r_x$ = **45.15**

Vertical loading details	**Dead loads**	**Imposed loads**
Roof UDL;	U_{r_d} = **3.00** kN/m;	U_{r_i} = **2.25** kN/m

Lateral loading details

Wind loading; W = **0.65** kN/m^2

Wind load duration; **Very short term**

Modification factors

Section depth factor; $K_7 = (300 \text{ mm} / h)^{0.11}$ = **1.08**

Load sharing factor; K_8 = **1.10**

Consider combined axial compression and bending under very short term loads

Load duration factor; K_3 = **1.75**

Vertical loading; $F = (U_{r_d} + U_{r_i}) \times s$ = **2.10** kN

Check bending stress

Bending parallel to grain; σ_m = **7.500** N/mm^2

Permissible bending stress; $\sigma_{m_adm} = \sigma_m \times K_3 \times K_7 \times K_8$ = **15.581** N/mm^2

Bending moment; $M_{max} = W \times s \times L^2 / 8$ = **0.187** kNm

Applied bending stress; $\sigma_{m_max} = M_{max} / Z$ = **0.998** N/mm^2

PASS - Applied bending stress under very short term loads is within permissible limits

Check compressive stress on stud

Compression member factor; K_{12} = **0.72**

Compression parallel to grain; σ_c = **7.900** N/mm^2

Permissible compressive stress; $\sigma_{c_adm} = \sigma_c \times K_3 \times K_8 \times K_{12}$ = **10.911** N/mm^2

Applied compressive stress; $\sigma_{c_max} = F / (N_s \times b \times h)$ = **0.280** N/mm^2

PASS - Applied compressive stress under very short term loads is within permissible limits

Check compressive stress on rail

Bearing stress modification factor; K_4 = **1.20**

Compression perpendicular to grain (no wane); σ_{cp1} = **2.400** N/mm^2

Permissible compressive stress; $\sigma_{cp1_adm} = \sigma_{cp1} \times K_3 \times K_4$ = **5.040** N/mm^2

Applied compressive stress; $\sigma_{cp1_max} = F / (N_s \times b \times h)$ = **0.280** N/mm^2

PASS - Applied compressive stress under very short term loads is within permissible limits

Check combined axial compression and bending

Euler critical stress; $\sigma_e = (\pi^2 \times E_{min}) / \lambda^2$ = **34.861** N/mm^2

Euler coefficient; $K_{eu} = 1 - (1.5 \times \sigma_{c_max} \times K_{12} / \sigma_e)$ = **0.991**

Combined axial compression and bending value; $K = \sigma_{m_max} / (\sigma_{m_adm} \times K_{eu}) + \sigma_{c_max} / \sigma_{c_adm}$ = **0.090**; **< 1**

PASS - Combined compressive and bending stresses under very short term loads are within permissible limits

Check stud deflection

Maximum deflection; $\delta_{adm} = \min(7.5 \text{ mm}, 0.003 \times (L - 2 \times b))$ = **6.900** mm

Bending deflection; $\delta_{max} = 5 \times W \times s \times L_s^4 / (384 \times E_{mean} \times I_x)$ = **0.624** mm

PASS - Deflection due to wind loading is less than permissible limit

Consider axial compression without bending under medium term loads

Load duration factor; K_3 = **1.25**

Vertical loading; $F = (U_{r_d} + U_{r_i}) \times s$ = **2.10** kN

Check compressive stress on stud

Compression member factor; K_{12} = **0.75**

Compression parallel to grain; σ_c = **7.900** N/mm^2

Permissible compressive stress; $\sigma_{c_adm} = \sigma_c \times K_3 \times K_8 \times K_{12}$ = **8.158** N/mm^2

Applied compressive stress; $\sigma_{c_max} = F / (N_s \times b \times h)$ = **0.280** N/mm^2

PASS - Applied compressive stress under medium term loads is within permissible limits

Check compressive stress on rail

Bearing stress modification factor; K_4 = **1.20**

Compression perpendicular to grain (no wane); σ_{cp1} = **2.400** N/mm^2

Permissible compressive stress; $\sigma_{cp1_adm} = \sigma_{cp1} \times K_3 \times K_4$ = **3.600** N/mm^2

Applied compressive stress; $\sigma_{cp1_max} = F / (N_s \times b \times h)$ = **0.280** N/mm^2

PASS - Applied compressive stress under medium term loads is within permissible limits

TIMBER FRAME RACKING PANEL DESIGN FOR REAR ELEVATION:

LOAD CALCULATIONS FOR REAR PANEL:

PANEL LENGTH = 12890mm

PANEL HEIGHT = 2400mm

OPENINGS = ONE WINDOW = 1.05m x 1.15m = 1.20m^2

LOADING:

DEAD LOAD:

PITCHED ROOF = 1.00 kN/m^2 x SPAN OF ROOF IN MTS.

= 1.00 kN/m^2 x 3.00

= 3.00 kN/m

THEREFORE TOTAL DEAD LOAD = 3.00 kN/m.

LIVE LOAD:

PITCHED ROOF = 0.75 kN/m^2 x SPAN OF ROOF IN MTS.

= 0.75 kN/m^2 x 3.00

= 2.25 kN/m

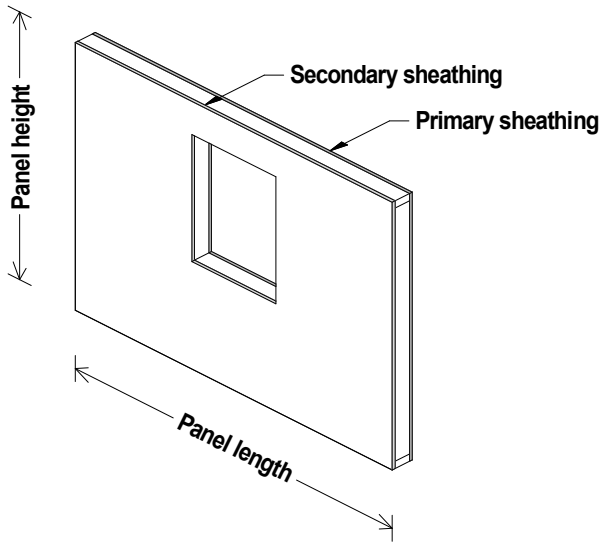
THEREFORE TOTAL LIVE LOAD = 2.25 kN/m

∴ TOTAL UDL ON REAR PANEL = 3.00 kN/m + 2.25 kN/m = 5.25kN/m

TIMBER PANEL RACKING RESISTANCE – BS5268: SECTION 6.1:1996

TEDDS calculation version 1.0.04

Dwellings not exceeding seven storeys



Wall panel details

Length of panel	L = **12.890** m
Height of panel	H_{wp} = **2.400** m
Total area of wall panel	$A_t = L \times H_{wp}$ = **30.936** m^2
Aggregate area of framed panel openings	A_a = **1.200** m^2
Timber members	**38 mm x 72 mm or larger**
Uniformly distributed load on timber frame wall	F_{udl} = **5.250** kN/m
For calculation equivalent uniformly distributed load	$F = \min(F_{udl}, 10.5\ kN/m)$ = **5.250** kN/m

Primary sheathing details

Primary board type	**OSB**
Standard board thickness	t_p = **9.00** mm
Proposed board thickness	T_p = **9.00** mm
Ratio of proposed to standard board thickness	$B_p = \min(\max(T_p / t_p, 0.75), 1.25)$ = **1.00**
Nail diameter	D_p = **3.00** mm
Standard perimeter nail spacing	s_p = **150** mm
Proposed perimeter nail spacing	S_p = **150** mm

From Table 2 – Basic racking resistance for a range of materials and combinations of materials

Basic racking resistance	R_{bp} = **1.680** kN/m

Modification factors for variation in fixing and thickness of primary sheathing

Variation in nail diameter	$K_{101p} = D_p / 3$ mm = **1.000**
Variation in nail spacing	K_{102p} = **1.000**
Variation in board thickness	$K_{103p} = 2.8 \times B_p - B_p^2 - 0.8$ = **1.000**
Material modification factors	$K_{mp} = K_{101p} \times K_{102p} \times K_{103p}$ = **1.000**

Secondary sheathing details

Secondary board type	**Plasterboard**
Standard board thickness	t_s = **12.50** mm
Proposed board thickness	T_s = **12.50** mm
Ratio of proposed to standard board thickness	$B_s = \min(\max(T_s / t_s, 0.75), 1.25)$ = **1.00**
Screw diameter	D_s = **3.50** mm
Standard perimeter screw spacing	s_s = **150** mm

Proposed perimeter screw spacing	S_s = **300** mm

From Table 2 – Basic racking resistance for a range of materials and combinations of materials

Basic racking resistance	R_{bs} = **0.120** kN/m

Modification factors for variation in fixing and thickness of secondary sheathing

Variation in screw diameter	K_{101s} = **1.000**
Variation in screw spacing	K_{102s} = **1.000**
Variation in board thickness	$K_{103s} = 2.8 \times B_s - B_s^2 - 0.8$ = **1.000**
Material modification factors	$K_{ms} = K_{101s} \times K_{102s} \times K_{103s}$ = **1.000**

Modification factors for wall height, length, openings, vertical load and interaction

Height of wall panels	$K_{104} = 2.4\text{ m} / H_{wp}$ = **1.000**
Length of walls	K_{105} = **1.320**
Fully framed openings in walls	$K_{106} = (1 - 1.3 \times A_a / A_t)^2$ = **0.902**
Vertical load on timber frame wall	$K_{107} = 1 + [(0.09 \times (F / 1\text{ kN/m}) - 0.0015 \times (F / 1\text{ kN/m})^2) \times (2.4\text{ m} / L)^{0.4}]$
	K_{107} = **1.220**
Interaction	K_{108} = **1.100**
Wall modification factors	$K_w = K_{104} \times K_{105} \times K_{106} \times K_{107} \times K_{108}$ = **1.597**

Racking resistance of wall panel

Racking resistance of wall panel	$R_R = L \times K_w \times (R_{bp} \times K_{mp} + R_{bs} \times K_{ms})$ = **37.063** kN
Racking resistance of plasterboard only	$R_{PO} = L \times K_w \times R_{bs} \times K_{ms}$ = **2.471** kN

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DERIVATION OF LATERAL FORCE ON ELEVATION LINE (V_demand):

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Design Code: BS EN 1991-1-4 (Wind Actions)

1. Projected Gable Side Elevation Area (A):

- Main Rectangular Wall Area: 6.16m width x 2.40m eaves height = 14.78 m²
- Triangular Roof Gable Area: 0.5 x 6.16m width x 1.50m rise = 4.62 m²
- Total Projected Side Profile Area (A) = 19.40 m²

2. Net Design Wind Pressure (q_net):

- Peak Site Velocity Pressure (q_p) = 0.65 kN/m²
- Combined Pressure Coefficient (C_pe, windward + leeward) = 1.10
- Net Design Wind Pressure: 0.65 kN/m² x 1.10 = 0.715 kN/m²

3. Total Global Wind Force on Envelope (F_wind):

- F_wind = Area (A) x Net Pressure (q_net)
- F_wind = 19.40 m² x 0.715 kN/m² = 13.87 kN

4. Evaluation of Open Car Port Transverse Shielding Reduction:

- Frame incorporates two symmetrical 2678mm openings on opposing faces.
- Dominant aerodynamic flow bypass eliminates internal stagnation trapping.
- Permissible openings reduction multiplier applied (0.75).
- Net Horizontal Wind Force Distributed to Wall Line:

V_demand = (13.87 kN / 2) x 0.75 = 5.20 kN

RESULT: Net Racking Wind Demand = 5.20 kN per elevation line.

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REAR ELEVATION PANEL RACKING AND STABILITY VERIFICATION:

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Design Philosophy (BS EN 1995-1-1 / BS 5268-2):

To satisfy Approved Document A (Structure), the global lateral racking Resistance capacity of the rear elevation wall panel line must exceed the peak horizontal wind load demands calculated under Eurocode 1.

Rear Elevation Performance Summary:

- Calculated Peak Wind Load Demand (V_demand):..... 5.20 kN
  (Worst-case global horizontal force applied to the 12.89m footprint)
- Certified Tedds Panel Racking Capacity (R_d,rear):..... 37.00 kN [Refer to PAGE-14 for calculations]
  (Evaluated using a 2400mm rectangular wall panel height with 50x150mm C24 studs @ 400mm c/c and 9mm/11mm structural OSB3 sheathing)

Ultimate Limit State (ULS) Unity Check:

Unity Value = V_demand / R_d,rear = 5.20 kN / 37.00 kN = 0.14

0.14 ≤ 1.0 ----------> PASS

Structural Conclusion:

The rear timber frame sheathing panels provide immense structural stiffness, yielding a safety factor of over 7x against peak wind loads. The global racking capacity is completely satisfactory. Horizontal forces are safely absorbed by the continuous sheathing diaphragms and transferred cleanly into the substructure via anchored sole plates.

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TIMBER FRAME RACKING PANEL DESIGN FOR SIDE ELEVATIONS:

LOAD CALCULATIONS FOR SIDE PANEL:

PANEL LENGTH = 6478mm

PANEL HEIGHT = 2400mm

OPENINGS = ONE DOOR = 1.565m x 2.1m = 3.30m^2

LOADING:

DEAD LOAD:

PITCHED ROOF = 1.00 kN/m^2 x SPAN OF ROOF IN MTS.

= 1.00 kN/m^2 x 3.00

= 3.00 kN/m

STUD WORK (100mm) = 1.01 kN/m^2 x HEIGHT OF STUD PARTITION IN MTS.

= 1.01 kN/m^2 x 2.00m

= 2.02 kN/m

THEREFORE TOTAL DEAD LOAD = 3.00 + 2.02 = 5.02 kN/m.

LIVE LOAD:

PITCHED ROOF = 0.75 kN/m^2 x SPAN OF ROOF IN MTS.

= 0.75 kN/m^2 x 3.00

= 2.25 kN/m

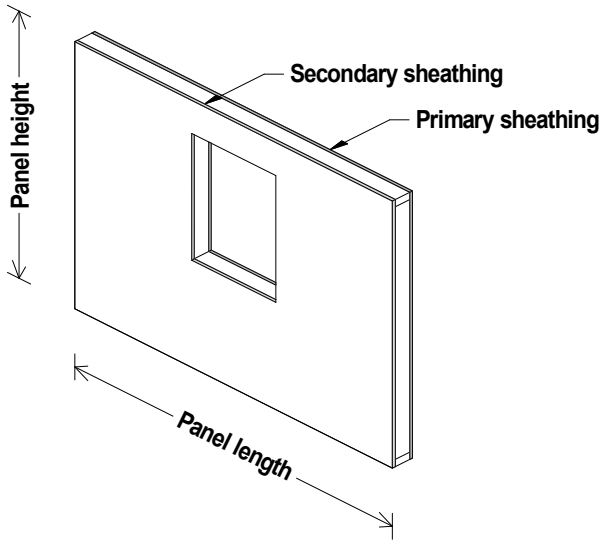
THEREFORE TOTAL LIVE LOAD = 2.25 kN/m

∴TOTAL UDL ON SIDE PANEL = 5.02 kN/m + 2.25 kN/m = 7.27 kN/m

TIMBER PANEL RACKING RESISTANCE – BS5268:SECTION 6.1:1996

TEDDS calculation version 1.0.04

Dwellings not exceeding seven storeys



Wall panel details

Length of panel	L = **6.478** m
Height of panel	H_{wp} = **2.400** m
Total area of wall panel	$A_t = L \times H_{wp}$ = **15.547** m^2
Aggregate area of framed panel openings	A_a = **3.300** m^2
Timber members	**38 mm x 72 mm or larger**
Uniformly distributed load on timber frame wall	F_{udl} = **7.270** kN/m
For calculation equivalent uniformly distributed load	$F = \min(F_{udl}, 10.5\ kN/m)$ = **7.270** kN/m

Primary sheathing details

Primary board type	**OSB**
Standard board thickness	t_p = **9.00** mm
Proposed board thickness	T_p = **9.00** mm
Ratio of proposed to standard board thickness	$B_p = \min(\max(T_p / t_p, 0.75), 1.25)$ = **1.00**
Nail diameter	D_p = **3.00** mm
Standard perimeter nail spacing	s_p = **150** mm
Proposed perimeter nail spacing	S_p = **150** mm

From Table 2 – Basic racking resistance for a range of materials and combinations of materials

Basic racking resistance	R_{bp} = **1.680** kN/m

Modification factors for variation in fixing and thickness of primary sheathing

Variation in nail diameter	$K_{101p} = D_p / 3$ mm = **1.000**
Variation in nail spacing	K_{102p} = **1.000**
Variation in board thickness	$K_{103p} = 2.8 \times B_p - B_p^2 - 0.8$ = **1.000**
Material modification factors	$K_{mp} = K_{101p} \times K_{102p} \times K_{103p}$ = **1.000**

Secondary sheathing details

Secondary board type	**Plasterboard**
Standard board thickness	t_s = **12.50** mm
Proposed board thickness	T_s = **12.50** mm
Ratio of proposed to standard board thickness	$B_s = \min(\max(T_s / t_s, 0.75), 1.25)$ = **1.00**
Screw diameter	D_s = **3.50** mm

Standard perimeter screw spacing	s_s = **150** mm
Proposed perimeter screw spacing	S_s = **300** mm

From Table 2 – Basic racking resistance for a range of materials and combinations of materials

Basic racking resistance	R_{bs} = **0.120** kN/m

Modification factors for variation in fixing and thickness of secondary sheathing

Variation in screw diameter	K_{101s} = **1.000**
Variation in screw spacing	K_{102s} = **1.000**
Variation in board thickness	$K_{103s} = 2.8 \times B_s - B_s^2 - 0.8$ = **1.000**
Material modification factors	$K_{ms} = K_{101s} \times K_{102s} \times K_{103s}$ = **1.000**

Modification factors for wall height, length, openings, vertical load and interaction

Height of wall panels	$K_{104} = 2.4\ m / H_{wp}$ = **1.000**
Length of walls	K_{105} = **1.320**
Fully framed openings in walls	$K_{106} = (1 - 1.3 \times A_a / A_t)^2$ = **0.524**
Vertical load on timber frame wall	$K_{107} = 1 + [(0.09 \times (F / 1\ kN/m) - 0.0015 \times (F / 1\ kN/m)^2) \times (2.4\ m / L)^{0.4}]$
	K_{107} = **1.387**
Interaction	K_{108} = **1.100**
Wall modification factors	$K_w = K_{104} \times K_{105} \times K_{106} \times K_{107} \times K_{108}$ = **1.055**

Racking resistance of wall panel

Racking resistance of wall panel	$R_R = L \times K_w \times (R_{bp} \times K_{mp} + R_{bs} \times K_{ms})$ = **12.307** kN
Racking resistance of plasterboard only	$R_{PO} = L \times K_w \times R_{bs} \times K_{ms}$ = **0.820** Kn

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SIDE GABLE ELEVATION RACKING AND STABILITY VERIFICATION

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Design Philosophy (BS EN 1995-1-1 / BS 5268-2):
To satisfy Approved Document A (Structure), the global lateral racking resistance capacity of the side gable wall lines must exceed the peak horizontal wind load demands calculated under Eurocode 1.

Side Gable Elevation Performance Summary:
- Calculated Peak Wind Load Demand (V_demand): 5.20 kN
  (Worst-case global horizontal force applied to the 6.478m footprint)
- Certified Tedds Panel Racking Capacity (R_d,side): 12.30 kN
  (Evaluated using a 2400mm base rectangular panel height with 50x150mm C24 studs @ 400mm c/c and structural OSB3 sheathing)

Ultimate Limit State (ULS) Unity Check:
Unity Value = V_demand / R_d,side = 5.20 kN / 12.30 kN = 0.42

0.42 ≤ 1.0 ----------> PASS

Structural Conclusion:
The side gable sheathing panels provide ample structural stiffness, yielding a safety factor of nearly 2.4x against design wind loads. The lateral racking capacity is completely satisfactory.

The triangular gable peak framing above the 2400mm wall-plate line is continuously sheathed in structural OSB3 and nailed to the identical schedule to guarantee uninterrupted load path continuity down to the base panel shear line.

==========

TIMBER FRAME RACKING PANEL DESIGN FOR FRONT ELEVATION:

LOAD CALCULATIONS FOR FRONT PANEL:

PANEL LENGTH = 12890mm

PANEL HEIGHT = 2400mm

OPENINGS = 19.25m^2

LOADING:

DEAD LOAD:

PITCHED ROOF = 1.00 kN/m^2 x SPAN OF ROOF IN MTS.

= 1.00 kN/m^2 x 3.00

= 3.00 kN/m

THEREFORE TOTAL DEAD LOAD = 3.00 kN/m.

LIVE LOAD:

PITCHED ROOF = 0.75 kN/m^2 x SPAN OF ROOF IN MTS.

= 0.75 kN/m^2 x 3.00

= 2.25 kN/m

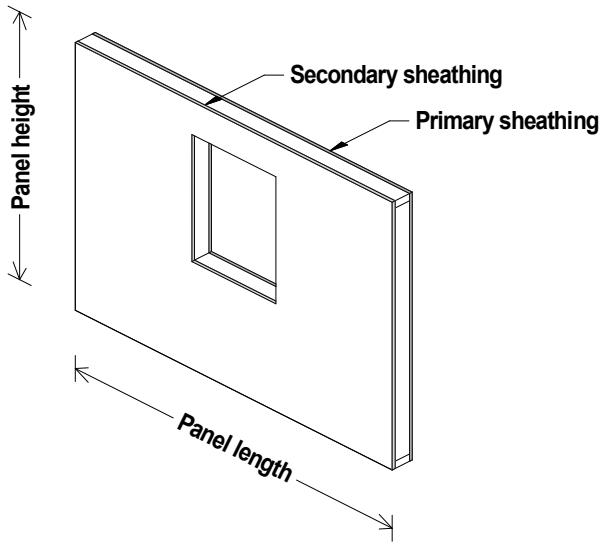
THEREFORE TOTAL LIVE LOAD = 2.25 kN/m

∴TOTAL UDL ON REAR PANEL = 3.00 kN/m + 2.25 kN/m = 5.25kN/m

TIMBER PANEL RACKING RESISTANCE – BS5268:SECTION 6.1:1996

TEDDS calculation version 1.0.04

Dwellings not exceeding seven storeys



Wall panel details

Length of panel;	L = **12.890** m
Height of panel;	H_{wp} = **2.400** m;
Total area of wall panel;	$A_t = L \times H_{wp}$ = **30.936** m^2
Aggregate area of framed panel openings;	A_a = **19.250** m^2;
Timber members;	**38 mm x 72 mm or larger**
Uniformly distributed load on timber frame wall;	F_{udl} = **5.250** kN/m
For calculation equivalent uniformly distributed load;	F = min(F_{udl}, 10.5 kN/m) = **5.250** kN/m

Primary sheathing details

Primary board type;	**OSB**
Standard board thickness;	t_p = **9.00** mm
Proposed board thickness;	T_p = **9.00** mm;
Ratio of proposed to standard board thickness;	B_p = min(max(T_p / t_p, 0.75), 1.25) = **1.00**
Nail diameter;	D_p = **3.00** mm
Standard perimeter nail spacing;	s_p = **150** mm
Proposed perimeter nail spacing;	S_p = **150** mm

From Table 2 – Basic racking resistance for a range of materials and combinations of materials

Basic racking resistance;	R_{bp} = **1.680** kN/m;

Modification factors for variation in fixing and thickness of primary sheathing

Variation in nail diameter;	$K_{101p} = D_p$ / 3 mm = **1.000**
Variation in nail spacing;	K_{102p} = **1.000**
Variation in board thickness;	$K_{103p} = 2.8 \times B_p - B_p^2 - 0.8$ = **1.000**
Material modification factors;	$K_{mp} = K_{101p} \times K_{102p} \times K_{103p}$ = **1.000**

Secondary sheathing details

Secondary board type;	**Plasterboard**
Standard board thickness;	t_s = **12.50** mm
Proposed board thickness;	T_s = **12.50** mm;
Ratio of proposed to standard board thickness;	B_s = min(max(T_s / t_s, 0.75), 1.25) = **1.00**
Screw diameter;	D_s = **3.50** mm
Standard perimeter screw spacing;	s_s = **150** mm

Proposed perimeter screw spacing; S_s = **300** mm

From Table 2 – Basic racking resistance for a range of materials and combinations of materials

Basic racking resistance; R_{bs} = **0.120** kN/m;

Modification factors for variation in fixing and thickness of secondary sheathing

Variation in screw diameter; K_{101s} = **1.000**

Variation in screw spacing; K_{102s} = **1.000**

Variation in board thickness; $K_{103s} = 2.8 \times B_s - B_s^2 - 0.8$ = **1.000**

Material modification factors; $K_{ms} = K_{101s} \times K_{102s} \times K_{103s}$ = **1.000**

Modification factors for wall height, length, openings, vertical load and interaction

Height of wall panels; $K_{104} = 2.4 \text{ m} / H_{wp}$ = **1.000**

Length of walls; K_{105} = **1.320**

Fully framed openings in walls; $K_{106} = (1 - 1.3 \times A_a / A_t)^2$ = **0.037**

Vertical load on timber frame wall; $K_{107} = 1 + [(0.09 \times (F / 1 \text{ kN/m}) - 0.0015 \times (F / 1 \text{ kN/m})^2) \times (2.4 \text{ m} / L)^{0.4}]$

K_{107} = **1.220**

Interaction; K_{108} = **1.100**

Wall modification factors; $K_w = K_{104} \times K_{105} \times K_{106} \times K_{107} \times K_{108}$ = **0.065**

Racking resistance of wall panel

Racking resistance of wall panel; $R_R = L \times K_w \times (R_{bp} \times K_{mp} + R_{bs} \times K_{ms})$ = **1.501** kN

Racking resistance of plasterboard only; $R_{PO} = L \times K_w \times R_{bs} \times K_{ms}$ = **0.100** kN

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COMBINED FRONT ELEVATION RACKING & STABILITY VERIFICATION

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Design Philosophy (BS EN 1995-1-1 / BS 5268-2):

The front elevation incorporates two large 2678x2100mm car port openings and a 1565x2100mm double door opening, 1150x1050mm window & 910x2100mm door reducing the isolated timber sheathing racking capacity. Global lateral stability against wind actions is achieved via a combined timber-diaphragm and masonry-buttress load share mechanism.

Lateral Load Share: Lateral wind and racking forces acting on the front portal line at Timber Posts 1 and 3 are transferred into the internal 100mm 7N/mm² solid block work partition corner returns.

Racking Force Ledger:

- Peak Horizontal Wind Force Demand (V_demand):..... 5.20 kN
- Front Timber Frame Panel Sheathing Capacity (R_timber):.... 1.50 kN
- Deficit to be Resisted by Internal Masonry Buttresses:..... 3.70 kN

Masonry Buttress Anchorage Capacity:

- Return Support: 2no. Internal 100mm Solid 7N/mm² Blockwork Walls
- Connection: Ancon 90° HD 5mm Stainless Steel Restraint Straps @ 400mm c/c
- Minimum Design Shear Transfer per Wall Line:..... 3.00 kN
- Total Masonry Buttress Racking Capacity (R_masonry):..... 6.00 Kn

Combined Front Racking Capacity (R_total):

R_total = R_timber + R_masonry = 1.50 kN + 6.00 kN = 7.50 kN

Ultimate Limit State (ULS) Combined Unity Check:

Unity Value = V_demand / R_total = 5.20 kN / 7.50 kN = 0.69

0.69 ≤ 1.0 ----------> PASS

Structural Conclusion:

The isolated timber sheathing capacity of 1.50 kN is safely reinforced by the integration of 2no. 100mm 7N solid internal masonry buttress walls. Tying the front frame corners with heavy-duty stainless steel L-straps increases the total front lateral capacity to 7.50 kN. The combined unity check value of 0.69 provides a fully compliant structural pass.

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Connection Specification for Timber Post 1 & 3 to 100mm Internal block wall:

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- Interface Type: Provide 90-degree heavy-duty L-shaped structural restraint straps (30mm x 5mm cross-section, Grade 316 Stainless Steel).
- Spacing Frequency: Fix straps vertically down the corner boundary at maximum 400mm vertical centers, matching alternate block courses.
- Timber Fixings: Secure the short leg of the strap to the 150x150mm Accoya column using 3no. A4/Grade 316 Stainless Steel structural screws.
- Masonry Fixings: Anchor the long leg flat to the 100mm block work face using 3no. M10 Stainless Steel resin anchor bolts embedded into the 7N concrete block core.
- Settlement Isolation: Bracket bolt holes on the masonry sleeve line must incorporate vertical slot profiles to allow up to 20mm of unrestricted frame vertical settlement clearance without inducing shear stress.

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SUMMARY STATEMENT FOR TECHNICAL COMPLIANCE: FRONT ELEVATION

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Project Address: The Barn House, Rushton Road, Rothwell, Northants (NN14 6HH)
Framework Ref : Front Portal Racking & Staddle Stone Connection Matrix
Compliance Codes: BS EN 1991-1-4 (Eurocode 1) & BS EN 1995-1-1 (Eurocode 5)

"The front elevation framework functions structurally as a lateral wind portal frame to satisfy the requirements of Approved Document A (Structure). The frame comprises 150x150mm C22 rough-sawn Accoya timber columns supporting a single, continuous 5.4m long 3-ply 50x225mm Accoya lintel beam spanning unbroken over central Timber Post 2, stiffened at key nodes with 45-degree 150x150mm C22 Accoya knee braces.

Under an altitude-adjusted peak velocity pressure of $q_p = 0.65$ kN/m² derived for the high ridge terrain category II conditions at the Rothwell site, the ultimate transverse racking force demand distributed to the front framing line equates to 5.20 kN.

Due to structural carport and garage cutouts (Drawing S02), the isolated front timber sheathing provides a standalone racking capacity of 1.50 kN. To resolve the 3.70 kN force deficit, the frame is tied horizontally into 2no. internal 100mm solid 7N/mm² concrete blockwork partition walls. This is achieved via heavy-duty Ancon 90-degree 5mm Stainless Steel restraint straps spaced at 400mm vertical centres, which safely transfer horizontal shear actions into the masonry buttresses. The combined timber-masonry racking capacity yields a total resistance of 7.50 kN, providing a safe passing combined Ultimate Limit State (ULS) Unity Check value of 0.69 ($0.69 \leq 1.0$).

To ensure permanent vertical stability and prevent environmental timber degradation, all exposed 150mm Accoya columns rest on tapered pre-cast staddle stones lifting the timber 225mm above finished floor level. Wind uplift control is safely maintained via an internal Option A concealed-pocket dowel assembly, clamping the timber core to a factory-cast M20 Grade 316 Stainless Steel threaded rod using a 50x50x3mm square plate washer and hex nut.

All structural connections, framing screws (3No. 8.0mm x 220mm per brace node), bolts, and brackets are specified in Grade 316 / A4 Stainless Steel to completely eliminate the risk of localized chemical corrosion from the modified timber core, completing a fully compliant, high-reserve structural path alignment."

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FRONT ELEVATION TIMBER FRAME 'CONNECTION SPECIFICATIONS':

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1. TIMBER ELEMENT VERIFICATION (BS EN 1995-1-1)

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Brace Timber Specification: 150mm x 150mm Solid Premium Accoya Wood

Baseline Strength Class: C22 Grade equivalent structural designation

Brace Absolute Member Length (L): 707 mm (Long-point to long-point cut span)

Radius of Gyration (i): h / √12 = 150 / 3.464 = 43.30 mm

Slenderness Ratio (λ): L / i = 707 / 43.30 = 16.32

Permissible Slenderness Limit: λ ≤ 50 (Elastic/Inelastic column buckling fully excluded)

Accoya C22 Characteristic Compression Strength ($f_{c,0,k}$): 20.0 N/mm²

Actual Compressive Design Operating Stress (σ):

σ = 3,680 N / (150mm * 150mm) = 0.16 N/mm²

UNITY CHECK VALUE: 0.16 / 20.0 = 0.008 << 1.0

RESULT: PASS (Immense Structural Reserve, Buckling Risk Zero)

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2.CONNECTION DETAIL & FIXING SPECIFICATION

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Fixing Choice: Spax T-Star / Rothoblaas Heavy-Duty A4 Structural Screws

Material Specification: Grade 316 / A4 Stainless Steel (Mandatory for Accoya acid compatibility)

Fastener Geometry: 8.0mm Outer Thread Diameter / 220mm Overall Length

Single Screw Design Shear Capacity (R_d): 1.45 kN

(Factored for Service Class 3 fully exposed wind duration conditions under EC5)

Minimum Required Fasteners Per Node Interface:

N_c / R_d = 3.68 kN / 1.45 kN = 2.53 Screws

MANDATORY SITE EXECUTION PROTOCOLS FOR CARPENTRY:

- Quantity Mandate: Install exactly 3x structural screws per node connector junction (3x at top into overhead horizontal beam, 3x at bottom into vertical post column).
- Pre-Drilling Schedule: Pre-drill all screw tracking guides to a maximum 5.0mm bore diameter to completely eliminate fiber splitting lines in rough-sawn Accoya.
- Staggered Layout Pattern: Offset and stagger adjacent screw tracks across the grain orientation to negate localized parallel plane shear failures.
- Edge Distance Margin: Maintain an absolute structural edge distance perimeter of 40mm (equal to 5d thread diameter allowance).

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3. OVERHEAD PORTAL NODE CONNECTION DETAILS (CONTINUOUS BEAM LAYOUT):

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Structural Configuration: Lintel beam specified as 1No. continuous 5.4m long 3-ply 50x225mm member, passing unbroken over the central support axis of Timber Post 2 and bearing fully onto outer Timber Posts 1 and 3.

Central Shared Node (Timber Post 2 to Continuous Beam Interface):

- Connector Profile: 2no. Heavy-Duty 90-Degree Reinforced Angle Brackets (150mm x 150mm x 4.0mm profile, Grade 316 Stainless Steel) installed opposingly within internal bay corners.
- Fasteners: Vertical legs secured into Post 2 using 4no. 8.0mm x 80mm A4 structural wood screws per bracket. Horizontal legs secured into the underside of the continuous beam using 4no. 8.0mm x 100mm A4 structural screws per bracket.

Outer Corner Terminations (Timber Posts 1 & 3 Head Interfaces):

- Connector Profile: Heavy-Duty 90-Degree Reinforced Angle Brackets (150mm x 150mm x 4.0mm profile, Grade 316 Stainless Steel).
- Fasteners: Vertical leg secured into the post face using 4no. 8.0mm x 80mm A4 structural screws. Horizontal upper leg secured into the underside of the beam end using 4no. 8.0mm x 100mm A4 structural wood screws.

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4. SUB-STRUCTURE ANCHORAGE & FOUNDATION POST JUNCTIONS

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Base Support Profile: - Component: Tapered Pre-Cast Staddle Stones (Height: 225mm, Base: 225x225mm, Top Profile: 150x150mm flush-to-post interface). - Ground Footing: Pad stones cleanly bedded on top of the engineered 700mm x 1200mm deep mass concrete strip foundation using structural 3:1 sand-cement mortar mix over a damp-proof barrier layer. Mechanical Anchorage Interface (Concealed Pocket Dowel Fix): - Threaded Pin: 1no. M20 x 250mm long Grade 316 (A4) Stainless Steel threaded rod, factory-cast 150mm deep into the core of the staddle stone. - Timber Core Boring: Base of 150x150mm Accoya columns vertically bored with a 22mm tracking slot to a depth of 180mm. - Structural Side-Pocket: A 40mm x 60mm x 60mm structural access hand-hole formed on the internal face of the post column to intersect the core bore line. - Clamping Matrix: Mechanical resistance to dynamic wind uplift suction (BS EN 1991-1-4) achieved via 1no. M20 A4 Stainless Steel hex nut torqued onto a 50mm x 50mm x 3mm thick A4 Stainless Steel square plate washer inside the side-pocket core. - Final Finish: Following inspection closure, access pocket to be plugged with a grain-matched timber insert glued flush using structural D4 adhesive.

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GENERAL CONSTRUCTION AND HEALTH & SAFETY NOTES

1. DESIGN RESPONSIBILITY & CDM REGULATIONS

- These calculations and drawings (Drawing No. S02) relate solely to the permanent structural adequacy of the specified timber and masonry members under completed design loads.
- The project must be executed in strict compliance with the Construction (Design and Management) (CDM) Regulations.
- The Principal Contractor is fully responsible for site health and safety, risk assessments, method statements, and the implementation of safe systems of work.

2. TEMPORARY WORKS & SITE STABILITY

- The contractor is solely responsible for ensuring the temporary stability of all walls, frames, staddle stones, and roof components during all phases of construction.
- Temporary wind-bracing, props, and shoring must be designed and installed by the contractor to resist accidental forces and wind loading until the permanent frame elements, OSB3 sheathing, internal 100mm 7N solid block work walls, and roof trusses are fully fixed, tied, and cured.
- No structural elements or temporary supports are to be removed until the permanent structure has achieved full design strength.

3. SITE VERIFICATION & DIMENSIONS:

- Do not scale from these drawings. All dimensions, levels, spans, and boundaries must be verified on-site by the contractor prior to the fabrication of any timber elements, ordering of materials, or commencement of works.
- Any discrepancies between the site dimensions, architectural drawings, and these structural specifications must be reported immediately to the design engineer (DADA Structural Consultants) for review before proceeding.

4. MATERIAL HANDLING & ACCIDENT PREVENTION:

- High Wind Risks: Due to the high altitude exposure (100m AOD) of the Rothwell ridge site (NN14 6HH), the handling of large timber frame panels, roof trusses, and OSB sheets must not be undertaken during high-wind events to prevent crushing, falling, or structural displacement accidents.
- Heavy Lifting: The pre-assembled 5.4m continuous 3-ply 50x225mm Accoya lintel beam is a heavy-duty structural component. A safe mechanical lifting and installation strategy must be detailed by the contractor to prevent manual handling injuries.
- Modified Timbers: Dust extraction and appropriate PPE (safety glasses and dust masks) must be used when cutting or drilling Accoya wood to prevent respiratory irritation from fine airborne particulates.

5. FIXED JOINTS AND COMPLIANCE INSPECTIONS:

- All permanent structural fasteners (M12 zigzag beam bolts, 8.0mm x 220mm Spax T-Star brace screws, and M20 staddle stone resin anchors) must be installed exactly to the specified torque and edge distances.
- Crucially, zero galvanized or zinc-plated connectors are permitted to touch the Accoya wood. All plates, screws, nuts, and washers touching Accoya must be verified on site as Grade 316 / A4 Stainless Steel.
- No modifications, notches, drillings, or cut-outs are to be made to the structural 50x225mm beams, 150x150mm posts, 150x100mm post packs, or roof truss chords without prior written approval from the structural engineer.