01527 579933
engineers@shire-uk.com | www.shire-uk.com
The Chapel | Barnsley Hall Road
Bromsgrove | Worcestershire | B61 0SZ

STRUCTURAL INSPECTION OF
TIMBER FRAMED BARN AT
CROWFIELDS FARM
FOCKBURY ROAD, DODFORD

ENGINEERING INNOVATION

INSPECT : INVESTIGATE : REPAIR

Consulting Civil & Structural Engineers | Geotechnical Investigations | Structural Inspections
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Project Summary

Visual Structural Inspection of timber framed barn for Class Q Planning application

Client Details

Date	**June 2023**
Client	**Mr D. Knowles**
Property Address	**Barn at Crowfields Farm, Fockbury Road, Dodford, Worcestershire B61 9AW**
Shire Ref number (our ref)	**S-22-186**
Report prepared by	**Richard Hartshorne – 07976 691472**

Contents

1. Introduction

1.1. Shire was appointed by Mr D. Knowles to carry out a visual structural inspection of a timber framed barn at Crowfields Farm, Fockbury Road, Dodford, Worcestershire for the purpose of submitting a Class Q Planning Application to convert the building to a dwellinghouse.

1.2. The barn is located on the Western edge of the site adjacent to open fields. To the East the barn faces towards the farmhouse.

1.3. The barn is a single storey timber framed building with a profiled metal single skin cladded roof and walls which are a combination of corrugated iron sheeting and timber boarding.

1.4. Any areas of the structure that were obscured or hidden from view at the time of the inspection have not been commented upon and we are therefore unable to confirm if these areas are free of defects or otherwise.

1.5. The visual inspection was carried out on Tuesday 6th June 2023 and the weather at the time of the inspection was dry and sunny.

Property Location

Location of Barn

2. External Inspection

2.1. The Northeast elevation of the building contains the primary access doors to the barn. There is a single wide opening into the first section of barn and a side entrance into the southern section of barn. This elevation is predominantly timber clad at the North end and corrugated iron clad at the South end. All elements are in reasonable working order.

2.2. The Southeast Gable elevation contains an opening to the adjacent field. The remainder of the gable is in corrugated iron sheeting with high level timber boarding. All structural elements are working satisfactorily.

2.3. The Southwest elevation of the barn comprises corrugated iron cladding with timber boarding over. All structural elements are working satisfactorily.

2.4. The Northwest gable elevation is not visible externally as it adjoins another building.

3. Internal Inspection

3.1. There is a floor slab throughout the full extent of the building. The floor slab is in good condition but does contain some minor shrinkage cracks. It is suitable to be overlain with insulation and screed. The slab has been subjected to significant storage loading and is capable of supporting domestic floor loading.

3.2. The timber sheeting rails and primary rafters are all in good condition and appear to be working satisfactorily.

3.3. The walls to the barn are lined with boarding and this is in good structural condition and provides stability to the barn.

4. Outline schedule of structural works to convert the building.

The following outline schedule of works has been compiled to give a general overview of the work and principles required to convert the existing steel framed barn into a dwelling house as part of a Class Q Planning Application conversion.

4.1 Building Works to convert the existing building into residential accommodation will include the following: The existing roof cladding will be retained and insulated as necessary, the purlins, primary timber frame, floor slabs and timber framed walling will all be retained and re-used and will not require modification. Within the existing building footprint, a new insulated timber studwork wall lining will be constructed, and the concrete floor will be overlain with a damp proof membrane, insulation, and screed.

4.2 The proposed works will require the following

4.2.1 The floor slab will need to be cleaned suitable to receive a damp proof membrane and insulation.

4.2.2 A new perimeter insulated inner wall will be constructed in timber studwork to comply with Building Regulations Parts A and L.

4.2.3 New windows and doors as required and permitted under Class Q, will require new timber lintols to be inserted into the external walls.

4.2.4 Install new wall and ceiling non-structural partitions to form internal rooms within the space.

5. Conclusions & Recommendations

5.1. The primary structure of the barn is suitable to be retained in the conversion to domestic living accommodation.

5.2. The building requires no structural changes and as noted above, in compliance with Q1(i)(i) and Q1(i)(ii), the extent of the works is only as reasonably necessary for the building to function as a dwellinghouse.

5.3. A design check was carried out to confirm that the existing structural elements are suitable for the conversion. A copy of the calculations is appended to this report.

Signed R. Y. Hartshorne

Richard Hartshorne BSc(Hons) CEng MIStructE MICE

APPENDIX A

STRUCTURAL CALCULATIONS

Shire

Job	S-22-186 / CA1				
Date	April '22	Page	01.	Rev	
Prepared by	R. Hoarse		Checked by		

CALCULATIONS FOR DESIGN CHECK

ON STRUCTURE AT BARN AT

CROWFIELDS FARM, DODFORD.

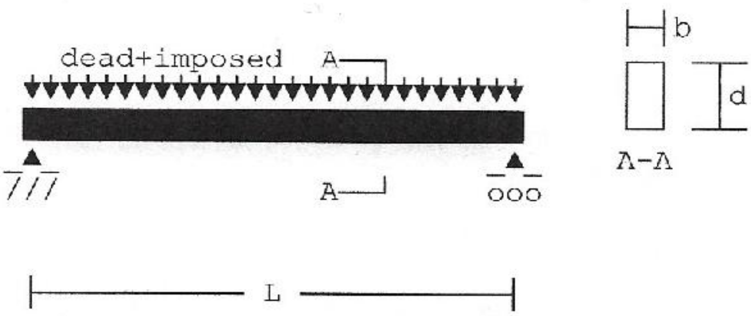
Job	S-22-186 / GA1				
Date	April '22	Page	02.	Rev	
Prepared by	R.Hoggs		Checked by		

LOADING SCHEDULE.

Proposed Roof Loads

Insulated sheeting	=	0.13
Rafters	=	0.05
		0.18 kN/m²
Services		0.02
Ceiling		0.13
		0.15 kN/m²
Imposed		0.75 kN/m².

Location: SHEETING SUPPORT TIMBERS



Domestic floor joist

These calculations follow the domestic floor joists example by V C Johnson in TRADA Design Aid DA1 and BS5268-2:2002.

The following assumptions are made in these calculations:

- that the timber has a moisture content of service class 1 or 2 (i.e. K2=1)
- the floor can adequately distribute any concentrated point load to at least two joists.
- the centres of joists do not exceed 610 mm
- that load sharing of the joists can occur & K8 = 1.1.

Effective span of joist	L=1.69 m
Centres of joists	crs=600 mm
Dead load including self weight	dead=0.33 kN/m²
Imposed udl load (on floor)	live=0.75 kN/m²
Imposed point load (on one joist)	PL=0 kN
Depth of section	d=75 mm
Width of section	b=50 mm

Joist laterally restrained with support restraint as Table 19.
Bearing length   lb=50 mm
Strength class C16 to Table 8.

Grade stresses

Bending parallel to grain	bparg=5.3 N/mm²
Shear parallel to grain	sparg=0.67 N/mm²
Compression perpend to grain	cperd=2.2 N/mm²
Mean modulus of elasticity	Emean=8800 N/mm²

Modification factors

Duration of loading	K3=1.25
Depth factor	K7=(300/d)^0.11=(300/75)^0.11 =1.1647
Load sharing (Clause 2.9)	K8=1.1

From BS5268-2 Table 18, bearing is < 75 mm from joist end.
Bearing Modification factor   K4=1.0

Permissible stresses

Bending parallel to grain sigmad=K3*K7*K8*bparg
=1.25*1.1647*1.1*5.3
=8.488 N/mm²

Compress perp to grain (no wane) sigcad=K3*K4*K8*cperd=1.25*1*1.1*2.2
=3.025 N/mm²

Shear parallel to grain torad=K3*K8*sparg=1.25*1.1*0.67
=0.92125 N/mm²

Strength

UDL case:
UDL per metre run on joist f1=(dead+live)*crs/1000
=(0.33+0.75)*600/1000
=0.648 kN/m
Bending moment M1=f1*L^2*10^6/8=0.648*1.69^2*10^6/8
=231344 Nmm
Point load case with udl (dead only):
UDL per metre run on joist (dead) f2=(dead)*crs/1000=(0.33)*600/1000
=0.198 kN/m
Bending moment (dead load only) M2=f2*L^2*10^6/8=0.198*1.69^2*10^6/8
=70688 Nmm
Bending moment point load M3=PL*L*10^6/4=0*1.69*10^6/4
=0 Nmm
Total bending moment M4=M2+M3=70688+0=70688 Nmm
Worst case moment (udl case governs) M=M1=231344 Nmm
Section modulus Z=b*d^2/6=50*75^2/6=46875 mm³
Bending stress sigma=M/Z=231344/46875=4.9353 N/mm²
Since sigma < sigmad (4.9353 N/mm² < 8.488 N/mm²) bending stress is less than permissible stress therefore OK.

Deflection

Second moment of area I=b*d^3/12=50*75^3/12=1.7578E6 mm⁴
UDL case:
Actual deflection including shear
del1=5*f1*(L*1000)^4/(384*Emean*I)+12*f1*(L*1000)^2/(5*Emean*b*d)
=5*0.648*(1.69*1000)^4/(384*8800*1.7578E6)+12*0.648*(1.69*1000)^2
/(5*8800*50*75)
=4.584 mm
Point load case:
Actual deflection including shear
del2=5*f2*(L*1000)^4/(384*Emean*I)+12*f2*(L*1000)^2/(5*Emean*b*d)
=5*0.198*(1.69*1000)^4/(384*8800*1.7578E6)+12*0.198*(1.69*1000)^2
/(5*8800*50*75)
=1.4007 mm
del3=PL*1000*(L*1000)^3/(48*Emean*I)
=0*1000*(1.69*1000)^3/(48*8800*1.7578E6)
=0 mm
del4=del2+del3=1.4007+0=1.4007 mm
UDL load case governs del=del1=4.584 mm
Limiting deflection Clause 2.10.7 dlim=0.003*L*1000=0.003*1.69*1000
=5.07 mm
Deflection does not exceed limit (4.584 mm ≤ 5.07 mm) therefore OK.

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Shear and bearing

UDL case:
Maximum applied shear force V1=f1*L/2=0.648*1.69/2=0.54756 kN
Point load case:
Maximum applied shear force V2=f2*L/2+PL=0.198*1.69/2+0
=0.16731 kN
Worst shear (udl case governs) V=V1=0.54756 kN
Shear stress tora=3*V*1000/(2*b*d)
=3*0.54756*1000/(2*50*75)
=0.21902 N/mm²
Since tora ≤ torad (0.21902 N/mm² ≤ 0.92125 N/mm²) shear stress does not exceed permissible therefore OK.
Bearing stress on support sigba=V*1000/(lb*b)
=0.54756*1000/(50*50)
=0.21902 N/mm²
Since sigba ≤ sigcad (0.21902 N/mm² ≤ 3.025 N/mm²) bearing stress does not exceed permissible therefore OK.

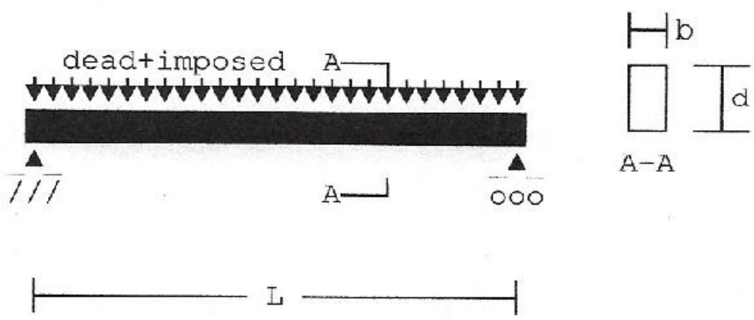
DESIGN SUMMARY

Joists: 75 mm x 50 mm @ 600 mm crs
Strength class C16 to Table 8.

Bending stress	4.9353 N/mm²
Permissible bending	8.488 N/mm²
Deflection	4.584 mm
Limiting deflection	5.07 mm
Shear stress	0.21902 N/mm²
Permissible shear	0.92125 N/mm²
Bearing stress	0.21902 N/mm²
Permissible bearing	3.025 N/mm²

No252

Location: RAFTERS



Domestic floor joist

These calculations follow the domestic floor joists example by V C Johnson in TRADA Design Aid DA1 and BS5268-2:2002.

The following assumptions are made in these calculations:

- that the timber has a moisture content of service class 1 or 2 (i.e. K2=1)
- the floor can adequately distribute any concentrated point load to at least two joists.
- the centres of joists do not exceed 610 mm
- that load sharing of the joists can occur & K8 = 1.1.

Effective span of joist	L=4.90 m
Centres of joists	crs=600 mm
Dead load including self weight	dead=0.33 kN/m²
Imposed udl load (on floor)	live=0.75 kN/m²
Imposed point load (on one joist)	PL=0 kN
Depth of section	d=200 mm
Width of section	b=50 mm

Joist laterally restrained with support restraint as Table 19.
Bearing length lb=50 mm
Strength class C24 to Table 8.

Grade stresses

Bending parallel to grain	bparg=7.5 N/mm²
Shear parallel to grain	sparg=0.71 N/mm²
Compression perpend to grain	cperd=2.4 N/mm²
Mean modulus of elasticity	Emean=10800 N/mm²

Modification factors

Duration of loading	K3=1.25
Depth factor	K7=(300/d)^0.11=(300/200)^0.11 =1.0456
Load sharing (Clause 2.9)	K8=1.1

From BS5268-2 Table 18, bearing is < 75 mm from joist end.
Bearing Modification factor K4=1.0

Permissible stresses

Bending parallel to grain — sigmad=K3*K7*K8*bparg
=1.25*1.0456*1.1*7.5
=10.783 N/mm²

Compress perp to grain (no wane) — sigcad=K3*K4*K8*cperd=1.25*1*1.1*2.4
=3.3 N/mm²

Shear parallel to grain — torad=K3*K8*sparg=1.25*1.1*0.71
=0.97625 N/mm²

Strength

UDL case:
UDL per metre run on joist — f1=(dead+live)*crs/1000
=(0.33+0.75)*600/1000
=0.648 kN/m

Bending moment — M1=f1*L^2*10^6/8=0.648*4.9^2*10^6/8
=1.9448E6 Nmm

Point load case with udl (dead only):
UDL per metre run on joist (dead) — f2=(dead)*crs/1000=(0.33)*600/1000
=0.198 kN/m

Bending moment (dead load only) — M2=f2*L^2*10^6/8=0.198*4.9^2*10^6/8
=594248 Nmm

Bending moment point load — M3=PL*L*10^6/4=0*4.9*10^6/4
=0 Nmm

Total bending moment — M4=M2+M3=594248+0=594248 Nmm
Worst case moment (udl case governs) — M=M1=1.9448E6 Nmm
Section modulus — Z=b*d^2/6=50*200^2/6=333333 mm³
Bending stress — sigma=M/Z=1.9448E6/333333
=5.8344 N/mm²

Since sigma < sigmad (5.8344 N/mm² < 10.783 N/mm²) bending stress is less than permissible stress therefore OK.

Deflection

Second moment of area — I=b*d^3/12=50*200^3/12=33.333E6 mm⁴
UDL case:
Actual deflection including shear
del1=5*f1*(L*1000)^4/(384*Emean*I)+12*f1*(L*1000)^2/(5*Emean*b*d)
=5*0.648*(4.9*1000)^4/(384*10800*33.333E6)+12*0.648*(4.9*1000)^2
/(5*10800*50*200)
=13.857 mm
Point load case:
Actual deflection including shear
del2=5*f2*(L*1000)^4/(384*Emean*I)+12*f2*(L*1000)^2/(5*Emean*b*d)
=5*0.198*(4.9*1000)^4/(384*10800*33.333E6)+12*0.198*(4.9*1000)^2
/(5*10800*50*200)
=4.2341 mm
del3=PL*1000*(L*1000)^3/(48*Emean*I)
=0*1000*(4.9*1000)^3/(48*10800*33.333E6)
=0 mm
del4=del2+del3=4.2341+0=4.2341 mm
UDL load case governs — del=del1=13.857 mm
Limiting deflection Clause 2.10.7 — dlim=14 mm
Deflection does not exceed limit (13.857 mm ≤ 14 mm) therefore OK.

Shear and bearing

UDL case:
Maximum applied shear force   V1=f1*L/2=0.648*4.9/2=1.5876 kN
Point load case:
Maximum applied shear force   V2=f2*L/2+PL=0.198*4.9/2+0
=0.4851 kN
Worst shear (udl case governs)   V=V1=1.5876 kN
Shear stress   tora=3*V*1000/(2*b*d)
=3*1.5876*1000/(2*50*200)
=0.23814 N/mm²
Since tora ≤ torad (0.23814 N/mm² ≤ 0.97625 N/mm²) shear stress does not exceed permissible therefore OK.
Bearing stress on support   sigba=V*1000/(lb*b)
=1.5876*1000/(50*50)
=0.63504 N/mm²
Since sigba ≤ sigcad (0.63504 N/mm² ≤ 3.3 N/mm²) bearing stress does not exceed permissible therefore OK.

DESIGN SUMMARY

Joists: 200 mm x 50 mm @ 600 mm crs
Strength class C24 to Table 8.

Bending stress	5.8344 N/mm²
Permissible bending	10.783 N/mm²
Deflection	13.857 mm
Limiting deflection	14 mm
Shear stress	0.23814 N/mm²
Permissible shear	0.97625 N/mm²
Bearing stress	0.63504 N/mm²
Permissible bearing	3.3 N/mm²

No252

Location: Wind loads

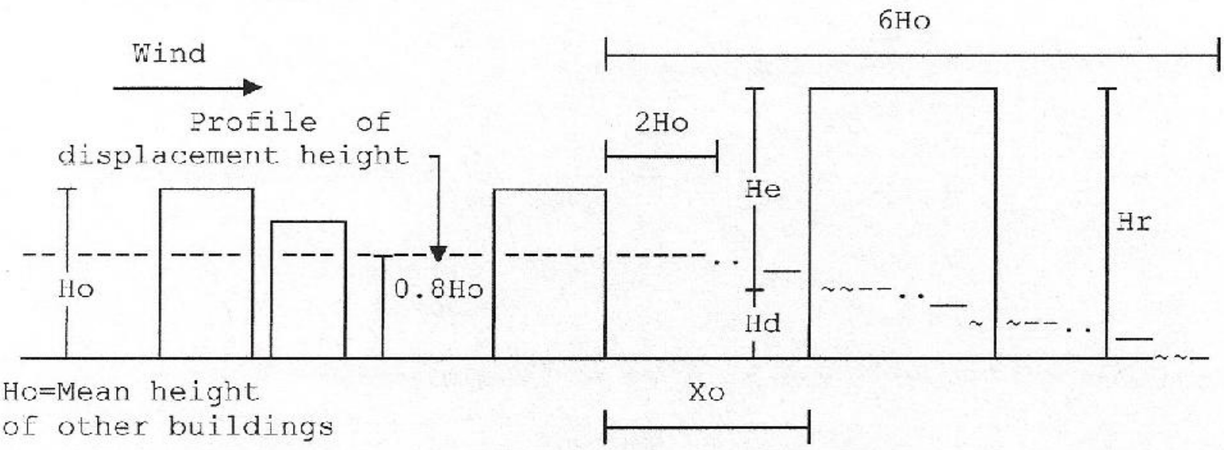
Wind load assessment based on BS6399-2:1997 (June 2002)

BS6399-2:1997 (June 02) offers two methods of assessing the wind load onto a structure. The method adopted in this program is the "standard method" to section 2.0 of this Code of Practice. To assess if BS6399-2 (Amendment June 02) is applicable, it is necessary to determine the "Dynamic augmentation factor" which determines if the chosen structure is "dynamic".

Dynamic augmentation factor

Chosen site is the country or terrain not defined as sea or town.

Displacement heights



Building reference height	Hr=3.1 m
Average height of buildings upwind of site	Ho=0 m
Upwind spacing of building	Xo=300 m
Displacement height	Hdw0=0 m
Displacement height	Hdw90=0 m
Wind angle Theta = 0°	
Effective height Case 1	He1w0=Hr-Hdw0=3.1-0=3.1 m
Effective height Case 2	He2=0.4*Hr=0.4*3.1=1.24 m
Maximum effective height	Hew0=3.1 m
Wind angle Theta = 90°	
Effective height Case 1	He1w90=Hr-Hdw90=3.1-0=3.1 m
Effective height Case 2	He2=0.4*Hr=0.4*3.1=1.24 m
Maximum effective height	Hew90=3.1 m
Building type factor	Kb=2

Chosen building type is a portal shed or similar light structure with few internal walls.

Building height	H=3.1 m
Dimensionless constant	Cho=0.1 m
Logarithm of height factor	lh=LOG(H/Cho)/LOG(10) =LOG(3.1/0.1)/LOG(10) =1.491
Dynamic Augmentation Factor	Cr=(Kb*(H/Cho)^0.75)/800/lh =(2*(3.1/0.1)^0.75)/800/1.491 =0.022

Since Dynamic Augmentation Factor Cr is less than or equal to 0.25 then this structure is not dynamic.

Standard wind loads

```
Basic wind speed                    Vb=20.5 m/s
Structure is not located at the crest of a hill or escarpment and the
topography is not significant.
Site altitude above mean sea level deltaS=100 m
Altitude factor (long face)         Saw0=1+0.001*deltaS=1+0.001*100
                                        =1.1
Altitude factor (short face)        Saw90=1+0.001*deltaS=1+0.001*100
                                         =1.1
Wind acting on long face of building (i.e. angle Theta = 0°)
Direction factor                    Sdw0=1
Wind acting on short face of building (i.e. angle Theta = 90°)
Direction factor                    Sdw90=1
The structure is permanent or exposed to the wind for a continuous
period of more than 6 months.
Seasonal factor                     Ss=1.0
The basic wind speed has an annual risk of being exceeded of Q=0.02
Probability factor                  Sp=1
Country terrain with the closest distance to the sea being 100 km.
Wind acting on long face of building (i.e. angle Theta = 0°)
From Table 4 terrain & building factor Sbw0=1.33
Site wind speed @ height He         Vsw0=Vb*Saw0*Sdw0*Ss*Sp
                                        =20.5*1.1*1*1*1
                                        =22.55 m/s
Effective wind speed                Vew0=Vsw0*Sbw0=22.55*1.33
                                        =29.99 m/s
Dynamic pressure at height He       qsw0=0.613*Vew0^2/1000
                                        =0.613*29.99^2/1000
                                        =0.5514 kN/m²
Wind acting on short face of building (i.e. angle Theta =90°)
From Table 4 terrain & building factor Sbw90=1.33
Site wind speed @ height He         Vsw90=Vb*Saw90*Sdw90*Ss*Sp
                                         =20.5*1.1*1*1*1
                                         =22.55 m/s
Effective wind speed                Vew90=Vsw90*Sbw90=22.55*1.33
                                         =29.99 m/sc
Dynamic pressure at height He       qsw90=0.613*Vew90^2/1000
                                         =0.613*29.99^2/1000
                                         =0.5514 kN/m²
```

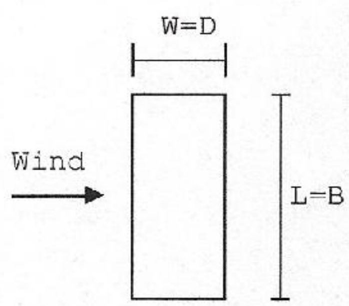
	Effective height (long face)	Hew0	3.1 m
	Effective height (short face)	Hew90	3.1 m
	Altitude factor (long face)	Saw0	1.1
DESIGN	Altitude factor (short face)	Saw90	1.1
SUMMARY	Direction factor (long face)	Sdw0	1
	Direction factor (short face)	Sdw90	1
	Seasonal factor	Ss	1
	Probability factor	Sp	1
	Dynamic wind pressure (long face)	qsw0	0.5514 kN/m²
	Dynamic wind pressure (short face)	qsw90	0.5514 kN/m²

Dynamic pressure & pressure coeffs on walls of rectangular buildings

Greater horiz.dimens of building L=15 m
Lesser horiz.dimens of building W=5 m
Height of wall including parapet Hr'=3.1 m
Wind acting on long face of building:
Dynamic pressure based on height above ground 3.1 m.
Design wind speed @ eaves Vsew0=22.55 m/s
Dynamic pressure @ eaves qew0=0.5514 kN/m²
Wind acting on short face of building:
Dynamic pressure based on height above ground 3.1 m.
Design wind speed @ eaves Vsew90=22.55 m/s
Dynamic pressure @ eaves qew90=0.5514 kN/m²

Wall size effect factor (θ=0°) CaLW1=1
Wall size effect factor (θ=90°) CaLW2=1

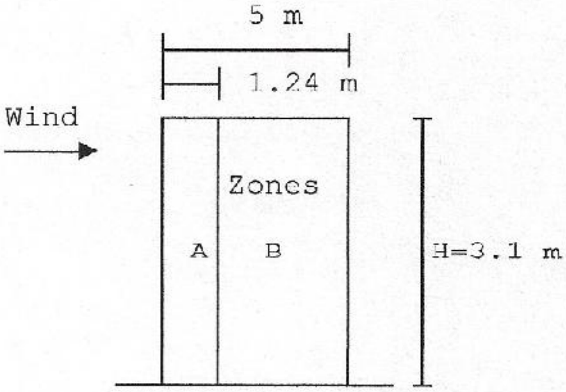
Case 1: Wind on long face of building (i.e. angle Theta = 0°)



PLAN VIEW

Inwind depth D=5 m
Cross wind width B=15 m

Span ratio Spratio=D/Hr'=5/3.1=1.613
Since B ≥ 2*Hr' (15 ≥ 6.2), then scaling length b=6.2 m.



ELEVATION ON SIDE FACE

Key zones for wall pressure data.

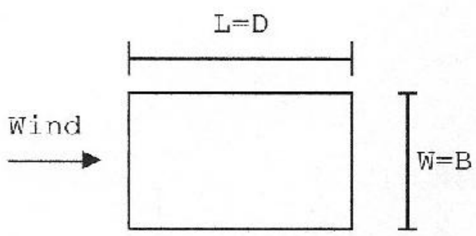
Scaling width > D
6.2 m > 5 m

Pressure coefficients on vertical walls are from Table 5.
Windward (front) face CpeLW1=0.7989
Leeward (rear) face CpeLW2=-0.5
Isolated Zone A side face CpeLW3=-1.3
Isolated Zone B side face CpeLW4=-0.8

External surface pressure on walls

```
Windward (front) face              PeLW1=qew0*CpeLW1*CaLW1
                                        =0.5514*0.7989*1
                                        =0.4405 kN/m²
Leeward (rear) face                PeLW2=qew0*CpeLW2*CaLW1=0.5514*-0.5*1
                                        =-0.2757 kN/m²
Isolated Zone A side face          PeLW3=qew0*CpeLW3*CaLW2=0.5514*-1.3*1
                                        =-0.7168 kN/m²
Isolated Zone B side face          PeLW4=qew0*CpeLW4*CaLW2=0.5514*-0.8*1
                                        =-0.4411 kN/m²
```

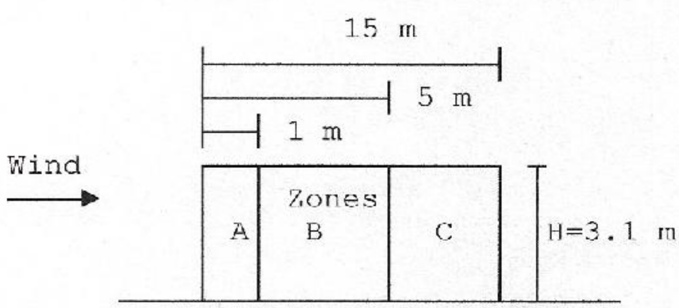
Case 2: Wind on short face of building (i.e. angle Theta = 90°)



PLAN VIEW

Inwind depth D=15 m
Cross wind width B=5 m

```
Span ratio                         Spratio=D/Hr'=15/3.1=4.839
Since span ratio is greater than or equal to 4     Spratio=4
Since B < 2*Hr' ( 5 < 6.2 ), then scaling length b=5 m.
```



ELEVATION ON SIDE FACE

Key zones for wall pressure data.

```
Pressure coefficients on vertical walls are from Table 5.
Windward (front) face              CpeSW1=0.6
Leeward (rear) face                CpeSW2=-0.5
Isolated Zone A side face          CpeSW3=-1.3
Isolated Zone B side face          CpeSW4=-0.8
Isolated Zone C side face          CpeSW5=-0.5
```

External surface pressure on walls

Windward (front) face	PeSW1=qew90*CpeSW1*CaLW2=0.5514*0.6*1 =0.3308 kN/m²
Leeward (rear) face	PeSW2=qew90*CpeSW2*CaLW2 =0.5514*-0.5*1 =-0.2757 kN/m²
Isolated Zone A side face	PeSW3=qew90*CpeSW3*CaLW1 =0.5514*-1.3*1 =-0.7168 kN/m²
Isolated Zone B side face	PeSW4=qew90*CpeSW4*CaLW1 =0.5514*-0.8*1 =-0.4411 kN/m²
Isolated Zone C side face	PeSW5=qew90*CpeSW5*CaLW1 =0.5514*-0.5*1 =-0.2757 kN/m²

Internal pressure coefficients

Structure is an enclosed building having 2 opposite walls equally permeable and the other faces impermeable.

Internal pressure coefficient 1 Cpi(1)=0.2
Internal pressure coefficient 2 Cpi(2)=-0.3

Size effect factor for internal wall CaInt=1

Internal surface pressure

Walls:

Wind on long face of building, internal surface pressure 1	Pi1ww0=qew0*Cpi(1)*CaInt=0.5514*0.2*1 =0.1103 kN/m²
Wind on short face of building, internal surface pressure 1	Pi1ww90=qew90*Cpi(1)*CaInt =0.5514*0.2*1 =0.1103 kN/m²
Wind on long face of building, internal surface pressure 2	Pi2ww0=qew0*Cpi(2)*CaInt =0.5514*-0.3*1 =-0.1654 kN/m²
Wind on short face of building, internal surface pressure 2	Pi2ww90=qew90*Cpi(2)*CaInt =0.5514*-0.3*1 =-0.1654 kN/m²

Roof:

Wind on long face of building, internal surface pressure 1	PiW1w0=qsw0*Cpi(1)*CaInt=0.5514*0.2*1 =0.1103 kN/m²
Wind on short face of building, internal surface pressure 1	PiW1w90=qsw90*Cpi(1)*CaInt =0.5514*0.2*1 =0.1103 kN/m²

Wind on long face of building, internal surface pressure 2

PiW2w0=qsw0*Cpi(2)*CaInt
=0.5514*-0.3*1
=-0.1654 kN/m²

Wind on short face of building, internal surface pressure 2

PiW2w90=qsw90*Cpi(2)*CaInt
=0.5514*-0.3*1
=-0.1654 kN/m²

```
*******************************************************************
*                  NET PRESSURES SIGN CONVENTION                  *
* Wall/roof net pressures acting towards the surface are taken as *
* positive and wall/roof net pressures acting away from the surface *
* are taken as negative.                                          *
*******************************************************************
```

Wall net pressure Cpi=0.2	Wind on long face of building
Windward (front) face	N1LW1=PeLW1-Pi1ww0=0.4405-0.1103 =0.3302 kN/m²
Leeward (rear) face	N1LW2=PeLW2-Pi1ww0=-0.2757-0.1103 =-0.386 kN/m²
Isolated Zone A side face	N1LW3=PeLW3-Pi1ww0=-0.7168-0.1103 =-0.8271 kN/m²
Isolated Zone B side face	N1LW4=PeLW4-Pi1ww0=-0.4411-0.1103 =-0.5514 kN/m²

Wall net pressure Cpi=0.2	Wind on short face of building
Windward (front) face	N1SW1=PeSW1-Pi1ww90=0.3308-0.1103 =0.2206 kN/m²
Leeward (rear) face	N1SW2=PeSW2-Pi1ww90=-0.2757-0.1103 =-0.386 kN/m²
Isolated Zone A side face	N1SW3=PeSW3-Pi1ww90=-0.7168-0.1103 =-0.8271 kN/m²
Isolated Zone B side face	N1SW4=PeSW4-Pi1ww90=-0.4411-0.1103 =-0.5514 kN/m²
Isolated Zone C side face	N1SW5=PeSW5-Pi1ww90=-0.2757-0.1103 =-0.386 kN/m²

Wall net pressure Cpi=-0.3	Wind on long face of building
Windward (front) face	N2LW1=PeLW1-Pi2ww0=0.4405--0.1654 =0.6059 kN/m²
Leeward (rear) face	N2LW2=PeLW2-Pi2ww0=-0.2757--0.1654 =-0.1103 kN/m²
Isolated Zone A side face	N2LW3=PeLW3-Pi2ww0=-0.7168--0.1654 =-0.5514 kN/m²
Isolated Zone B side face	N2LW4=PeLW4-Pi2ww0=-0.4411--0.1654 =-0.2757 kN/m²

Wall net pressure Cpi=-0.3	Wind on short face of building
Windward (front) face	N2SW1=PeSW1-Pi2ww90=0.3308--0.1654 =0.4963 kN/m²

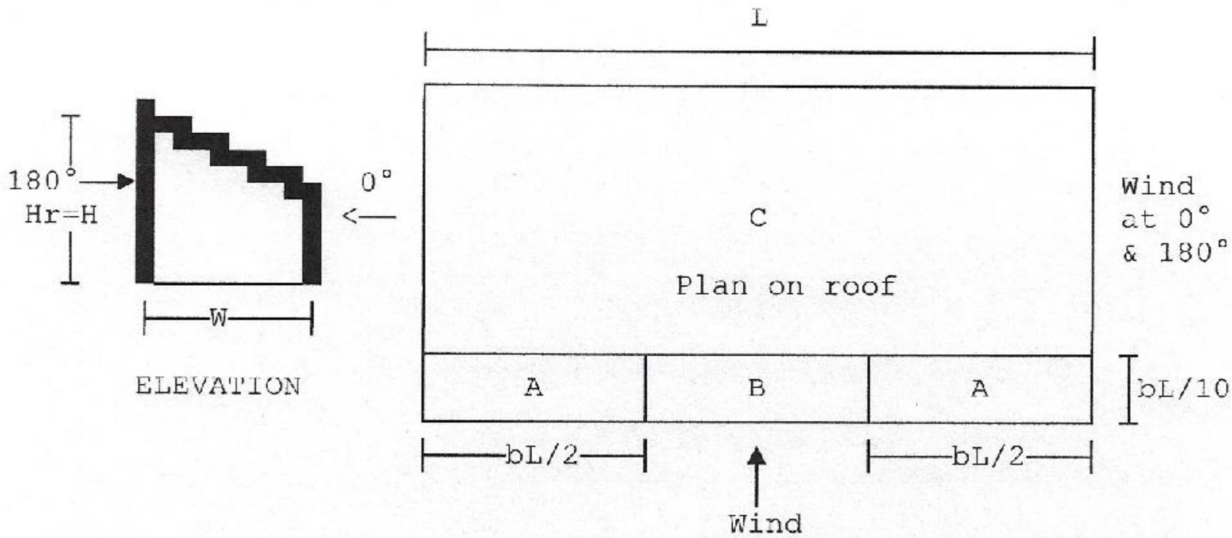
Office: 1154

Leeward (rear) face	N2SW2=PeSW2-Pi2ww90=-0.2757--0.1654 =-0.1103 kN/m²
Isolated Zone A side face	N2SW3=PeSW3-Pi2ww90=-0.7168--0.1654 =-0.5514 kN/m²
Isolated Zone B side face	N2SW4=PeSW4-Pi2ww90=-0.4411--0.1654 =-0.2757 kN/m²
Isolated Zone C side face	N2SW5=PeSW5-Pi2ww90=-0.2757--0.1654 =-0.1103 kN/m²

Coefficients Cpe for monopitch roofs of rectangular clad

read in conjunction with BS6399-2:1997 (June 2002)

Case 1: Wind angle Theta = 0°



Pitch of roof (5° to 75°) Malpha=6°
Since L ≥ 2Hr (15 ≥ 6.2), scaling length bL=6.2 m.

Zone A depth	ZAdep=bL/10=6.2/10=0.62 m
Zone A length	ZAlen=bL/2=6.2/2=3.1 m
Zone B length	ZBlen=L-2*ZAlen=15-2*3.1 =8.8 m

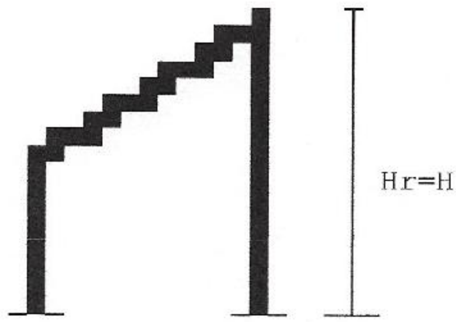
External pressure Coeff for zone A= Cpem01=-1.73
External pressure Coeff for zone B= Cpem02=-1.16
External pressure Coeff for zone C= Cpem03=-0.58

External + ve pressure Coeff.for zone A= Cpem04=0.02
External + ve pressure Coeff.for zone B= Cpem05=0.02
External + ve pressure Coeff.for zone C= Cpem06=0.02

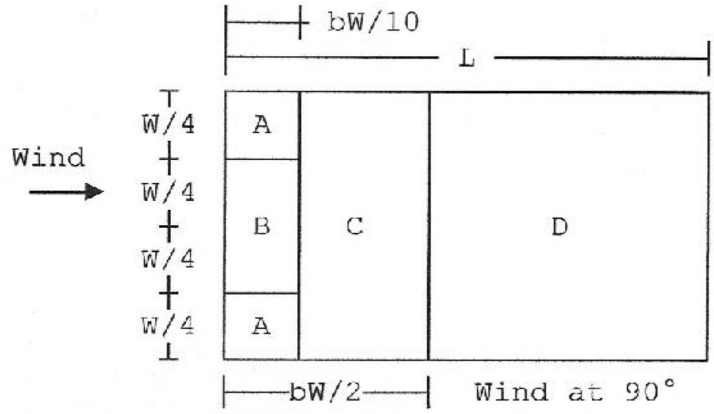
Case 2: Wind angle Theta = 180°

Zone A length ZAlen=bL/10=6.2/10=0.62 m
External pressure Coeff.for zone A= Cpem12=-2.42
External pressure Coeff.for zone B= Cpem13=-1.09
External pressure Coeff.for zone C= Cpem14=-0.81

Case 3: Wind angle Theta = 90°



SIDE ELEVATION



Since W < 2Hr (5 < 6.2), scaling length bW=5 m.

Zone A length ZAlen=+bW/10=+5/10=0.5 m
Zone B length ZBlen=+bW/10=+5/10=0.5 m
Zone C length ZClen=bW/2-bW/10=5/2-5/10
=2 m

External pressure Coeff for zone Au= Cpem07=-2.24
External pressure Coeff for zone Al= Cpem08=-2.05
External pressure Coeff for zone B= Cpem09=-1.1
External pressure Coeff for zone C= Cpem10=-0.71
External pressure Coeff for zone D= Cpem11=-0.71
External (+ve) pressure Coeff for zone Au= Cpem15=0.02
External (+ve) pressure Coeff for zone Al= Cpem16=0.02
External (+ve) pressure Coeff for zone B= Cpem17=0.02
External (+ve) pressure Coeff for zone C= Cpem18=0.02
External (+ve) pressure Coeff for zone D= Cpem19=0.02

Roof size effect factor (θ=0°) CaR0=1
Roof size effect factor (θ=90°) CaR90=1
Size effect factor for roof at 180 deg CaR180=1

External surface pressure on roof - Wind at 0° (long face)

Roof zone A	PeRA0=qsw0*Cpem01*CaR0=0.5514*-1.73*1 =-0.9539 kN/m²
Roof zone B	PeRB0=qsw0*Cpem02*CaR0=0.5514*-1.16*1 =-0.6396 kN/m²
Roof zone C	PeRC0=qsw0*Cpem03*CaR0=0.5514*-0.58*1 =-0.3198 kN/m²
Roof zone A (+Ve Value)	PRAA0=qsw0*Cpem04*CaR0=0.5514*0.02*1 =0.011 kN/m²
Roof zone B (+Ve Value)	PRAB0=qsw0*Cpem05*CaR0=0.5514*0.02*1 =0.011 kN/m²
Roof zone C (+Ve Value)	PRAC0=qsw0*Cpem06*CaR0=0.5514*0.02*1 =0.011 kN/m²

Roof net pressure Cpi=0.2	Wind at 0° (long face)
Roof zone A	N1RA0=PeRA0-PiW1w0=-0.9539-0.1103 =-1.064 kN/m²
Roof zone B	N1RB0=PeRB0-PiW1w0=-0.6396-0.1103 =-0.7499 kN/m²
Roof zone C	N1RC0=PeRC0-PiW1w0=-0.3198-0.1103 =-0.4301 kN/m²
Roof zone A (+Ve Value)	N1RAA0=PRAA0-PiW1w0=0.011-0.1103 =-0.0992 kN/m²
Roof zone B (+Ve Value)	N1RAB0=PRAB0-PiW1w0=0.011-0.1103 =-0.0992 kN/m²
Roof zone C (+Ve Value)	N1RAC0=PRAC0-PiW1w0=0.011-0.1103 =-0.0992 kN/m²

External surface pressure on roof - Wind at 90° (short face)	
Roof zone Au	PuRA9=qsw90*Cpem07*CaR90 =0.5514*-2.24*1 =-1.235 kN/m²
Roof zone Al	PlRA9=qsw90*Cpem08*CaR90 =0.5514*-2.05*1 =-1.13 kN/m²
Roof zone B	PeRB9=qsw90*Cpem09*CaR90 =0.5514*-1.1*1 =-0.6065 kN/m²
Roof zone C	PeRC9=qsw90*Cpem10*CaR90 =0.5514*-0.71*1 =-0.3915 kN/m²
Roof zone D	PeRD9=qsw90*Cpem11*CaR90 =0.5514*-0.71*1 =-0.3915 kN/m²
Roof zone Au (+ve value)	PpRu9=qsw90*Cpem15*CaR90 =0.5514*0.02*1 =0.011 kN/m²
Roof zone Al (+ve value)	PpRl9=qsw90*Cpem16*CaR90 =0.5514*0.02*1 =0.011 kN/m²
Roof zone B (+ve value)	PpRB9=qsw90*Cpem17*CaR90 =0.5514*0.02*1 =0.011 kN/m²
Roof zone C (+ve value)	PpRC9=qsw90*Cpem18*CaR90 =0.5514*0.02*1 =0.011 kN/m²
Roof zone D (+ve value)	PpRD9=qsw90*Cpem19*CaR90 =0.5514*0.02*1 =0.011 kN/m²

Roof net pressure Cpi=0.2	Wind at 90° (short face)
Roof zone Au	N1Au9=PuRA9-PiW1w90=-1.235-0.1103 =-1.345 kN/m²
Roof zone Al	N1Al9=PlRA9-PiW1w90=-1.13-0.1103 =-1.241 kN/m²
Roof zone B	N1RB9=PeRB9-PiW1w90=-0.6065-0.1103 =-0.7168 kN/m²

Roof zone C	N1RC9=PeRC9-PiW1w90=-0.3915-0.1103 =-0.5018 kN/m²
Roof zone D	N1RD9=PeRD9-PiW1w90=-0.3915-0.1103 =-0.5018 kN/m²
Roof zone Au (+ve value)	N1Rup9=PpRu9-PiW1w90=0.011-0.1103 =-0.0992 kN/m²
Roof zone Al (+ve value)	N1Rlp9=PpRl9-PiW1w90=0.011-0.1103 =-0.0992 kN/m²
Roof zone B (+ve value)	N1RBp9=PpRB9-PiW1w90=0.011-0.1103 =-0.0992 kN/m²
Roof zone C (+ve value)	N1RCp9=PpRC9-PiW1w90=0.011-0.1103 =-0.0992 kN/m²
Roof zone D (+ve value)	N1RDp9=PpRD9-PiW1w90=0.011-0.1103 =-0.0992 kN/m²

External surface pressure on roof - Wind at 180°

Roof zone A	PeRA8=qsw0*Cpem12*CaR180 =0.5514*-2.42*1 =-1.334 kN/m²
Roof zone B	PeRB8=qsw0*Cpem13*CaR180 =0.5514*-1.09*1 =-0.601 kN/m²
Roof zone C	PeRC8=qsw0*Cpem14*CaR180 =0.5514*-0.81*1 =-0.4466 kN/m²

Roof net pressure Cpi=0.2 Wind at 180°

Roof zone A	N1RA8=PeRA8-PiW1w0=-1.334-0.1103 =-1.445 kN/m²
Roof zone B	N1RB8=PeRB8-PiW1w0=-0.601-0.1103 =-0.7113 kN/m²
Roof zone C	N1RC8=PeRC8-PiW1w0=-0.4466-0.1103 =-0.5569 kN/m²

Roof net pressure Cpi=-0.3 Wind at 0° (long face)

Roof zone A	N2RA0=PeRA0-PiW2w0=-0.9539--0.1654 =-0.7885 kN/m²
Roof zone B	N2RB0=PeRB0-PiW2w0=-0.6396--0.1654 =-0.4742 kN/m²
Roof zone C	N2RC0=PeRC0-PiW2w0=-0.3198--0.1654 =-0.1544 kN/m²
Roof zone A (+Ve Value)	N2RAA0=PRAA0-PiW2w0=0.011--0.1654 =0.1764 kN/m²
Roof zone B (+Ve Value)	N2RAB0=PRAB0-PiW2w0=0.011--0.1654 =0.1764 kN/m²
Roof zone C (+Ve Value)	N2RAC0=PRAC0-PiW2w0=0.011--0.1654 =0.1764 kN/m²

Roof net pressure Cpi=-0.3	Wind at 90° (short face)
Roof zone A upper	N2Au9=PuRA9-PiW2w90=-1.235--0.1654 =-1.07 kN/m²
Roof zone A lower	N2Al9=PlRA9-PiW2w90=-1.13--0.1654 =-0.9649 kN/m²
Roof zone B	N2RB9=PeRB9-PiW2w90=-0.6065--0.1654 =-0.4411 kN/m²
Roof zone C	N2RC9=PeRC9-PiW2w90=-0.3915--0.1654 =-0.2261 kN/m²
Roof zone D	N2RD9=PeRD9-PiW2w90=-0.3915--0.1654 =-0.2261 kN/m²
Roof zone Au (+ve value)	N2Rup9=PpRu9-PiW2w90=0.011--0.1654 =0.1764 kN/m²
Roof zone Al (+ve value)	N2Rlp9=PpRl9-PiW2w90=0.011--0.1654 =0.1764 kN/m²
Roof zone B (+ve value)	N2RBp9=PpRB9-PiW2w90=0.011--0.1654 =0.1764 kN/m²
Roof zone C (+ve value)	N2RCp9=PpRC9-PiW2w90=0.011--0.1654 =0.1764 kN/m²
Roof zone D (+ve value)	N2RDp9=PpRD9-PiW2w90=0.011--0.1654 =0.1764 kN/m²

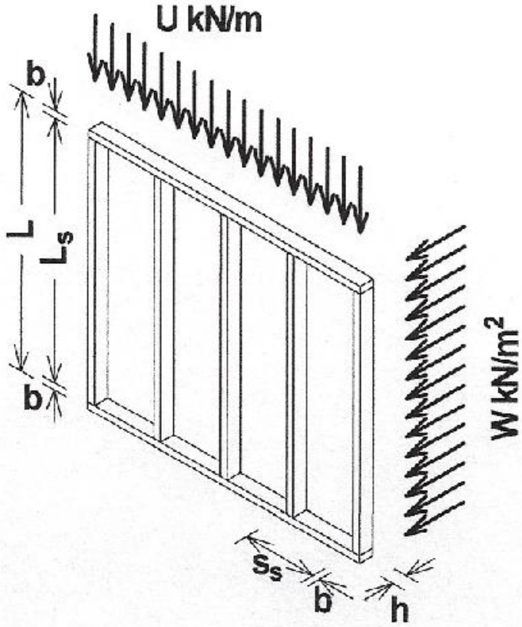
Roof net pressure Cpi=-0.3	Wind at 180°
Roof zone A	N2RA8=PeRA8-PiW2w0=-1.334--0.1654 =-1.169 kN/m²
Roof zone B	N2RB8=PeRB8-PiW2w0=-0.601--0.1654 =-0.4356 kN/m²
Roof zone C	N2RC8=PeRC8-PiW2w0=-0.4466--0.1654 =-0.2812 kN/m²

Shireconsulting Shire Consulting The Chapel Barnsley Hall Road Bromsgrove	Project Crowfields Farm Barn				Job Ref. S-22-186/CA1	
	Section Wall Studs				Sheet no./rev. 20	
	Calc. by RYH	Date 12/04/22	Chk'd by	Date	App'd by	Date

TIMBER STUD DESIGN (BS5268)

TIMBER STUD DESIGN (BS5268-2:2002)

TEDDS calculation version 1.0.05



Stud details

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

Strength class C16 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 = **3100** mm
Stud length	$L_s = L - (2 \times b)$ = **3000** mm
Standard stud spacing	s_s = **600** mm
Panel opening	O = **0** mm
Loaded panel length	$s = \max(s_s, (O + s_s) / 2)$ = **600** mm
Effective length in the major axis	$L_{ex} = 0.85 \times L_s$ = **2550** mm
Slenderness ratio	$\lambda = L_{ex} / r_x$ = **58.89**

Vertical loading details	**Dead loads**	**Imposed loads**
Roof UDL	U_{r_d} = **0.81** kN/m	U_{r_i} = **1.84** kN/m

Lateral loading details

Wind loading — $W = \mathbf{0.61}$ kN/m^2

Wind load duration — **Very short term**

Modification factors

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

Load sharing factor — $K_8 = \mathbf{1.10}$

Consider combined axial compression and bending under very short term loads

Load duration factor — $K_3 = \mathbf{1.75}$

Vertical loading — $F = (U_{r\_d} + U_{r\_i}) \times s = \mathbf{1.59}$ kN

Check bending stress

Bending parallel to grain — $\sigma_m = \mathbf{5.300}$ N/mm^2

Permissible bending stress — $\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8 = \mathbf{11.011}$ N/mm^2

Bending moment — $M_{max} = W \times s \times L^2 / 8 = \mathbf{0.436}$ kNm

Applied bending stress — $\sigma_{m\_max} = M_{max} / Z = \mathbf{2.326}$ N/mm^2

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

Check compressive stress on stud

Compression member factor — $K_{12} = \mathbf{0.57}$

Compression parallel to grain — $\sigma_c = \mathbf{6.800}$ N/mm^2

Permissible compressive stress — $\sigma_{c\_adm} = \sigma_c \times K_3 \times K_8 \times K_{12} = \mathbf{7.428}$ N/mm^2

Applied compressive stress — $\sigma_{c\_max} = F / (N_s \times b \times h) = \mathbf{0.212}$ 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 — $K_4 = \mathbf{1.20}$

Compression perpendicular to grain (no wane) — $\sigma_{cp1} = \mathbf{2.200}$ N/mm^2

Permissible compressive stress — $\sigma_{cp1\_adm} = \sigma_{cp1} \times K_3 \times K_4 = \mathbf{4.620}$ N/mm^2

Applied compressive stress — $\sigma_{cp1\_max} = F / (N_s \times b \times h) = \mathbf{0.212}$ 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 — $\sigma_e = (\pi^2 \times E_{min}) / \lambda^2 = \mathbf{16.506}$ N/mm^2

Euler coefficient — $K_{eu} = 1 - (1.5 \times \sigma_{c\_max} \times K_{12} / \sigma_e) = \mathbf{0.989}$

Combined axial compression and bending value — $K = \sigma_{m\_max} / (\sigma_{m\_adm} \times K_{eu}) + \sigma_{c\_max} / \sigma_{c\_adm} = \mathbf{0.242} < 1$

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

Check stud deflection

Euler critical stress — $\sigma_e = (\pi^2 \times E_{min}) / \lambda^2 = \mathbf{16.506}$ N/mm^2

Maximum deflection — $\delta_{adm} = \min(9.0 \text{ mm}, 0.003 \times (L - 2 \times b)) = \mathbf{9.000}$ mm

Bending deflection — $\delta_{max} = 5 \times W \times s \times L_s^4 / (384 \times E_{mean} \times I_x) = \mathbf{3.094}$ mm

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

Consider axial compression without bending under medium term loads

Load duration factor — $K_3 = \mathbf{1.25}$

Vertical loading — $F = (U_{r\_d} + U_{r\_i}) \times s = \mathbf{1.59}$ kN

Check compressive stress on stud

Compression member factor — $K_{12} = \mathbf{0.63}$

Compression parallel to grain — $\sigma_c = \mathbf{6.800}$ N/mm^2

Permissible compressive stress — $\sigma_{c\_adm} = \sigma_c \times K_3 \times K_8 \times K_{12} = \mathbf{5.929}$ N/mm^2

Applied compressive stress $\sigma_{c\_max} = F / (N_s \times b \times h) = \mathbf{0.212}$ N/mm²

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

Check compressive stress on rail

Bearing stress modification factor $K_4 = \mathbf{1.20}$

Compression perpendicular to grain (no wane) $\sigma_{cp1} = \mathbf{2.200}$ N/mm²

Permissible compressive stress $\sigma_{cp1\_adm} = \sigma_{cp1} \times K_3 \times K_4 = \mathbf{3.300}$ N/mm²

Applied compressive stress $\sigma_{cp1\_max} = F / (N_s \times b \times h) = \mathbf{0.212}$ N/mm²

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

TIMBER FRAME RACKING PANEL DESIGN (BS5268)

TIMBER PANEL RACKING RESISTANCE – BS5268:SECTION 6.1:1996

TEDDS calculation version 1.0.05

Dwellings not exceeding seven storeys

Perimeter wall

Wall panel details

Length of panel	$L = \mathbf{3.650}$ m
Height of panel	$H_{wp} = \mathbf{2.700\ m}$
Total area of wall panel	$A_t = L \times H_{wp} = \mathbf{9.855\ m^2}$
Aggregate area of framed panel openings	$A_a = \mathbf{0.000}$ m^2
Timber members	**38 mm x 72 mm or larger**
Uniformly distributed load on timber frame wall	$F_{udl} = \mathbf{0.810}$ kN/m
For calculation equivalent uniformly distributed load	$F = \min(F_{udl}, 10.5\ \text{kN/m}) = \mathbf{0.810\ kN/m}$

Primary sheathing details

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

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

Basic racking resistance	$R_{bp} = \mathbf{1.680\ kN/m}$

Modification factors for variation in fixing and thickness of primary sheathing

Variation in nail diameter	$K_{101p} = D_p / 3\ \text{mm} = \mathbf{1.000}$
Variation in nail spacing	$K_{102p} = 1 / (0.6 \times (S_p / s_p) + 0.4) = \mathbf{1.000}$
Variation in board thickness	$K_{103p} = 2.8 \times B_p - B_p^2 - 0.8 = \mathbf{1.000}$
Material modification factors	$K_{mp} = K_{101p} \times K_{102p} \times K_{103p} = \mathbf{1.000}$

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

Height of wall panels	$K_{104} = 2.4\ \text{m} / H_{wp} = \mathbf{0.889}$
Length of walls	$K_{105} = (L / 2.4\ \text{m})^{0.4} = \mathbf{1.183}$
Fully framed openings in walls	$K_{106} = (1 - 1.3 \times A_a / A_t)^2 = \mathbf{1.000}$
Vertical load on timber frame wall	$K_{107} = 1 + [(0.09\times (F / 1\ \text{kN/m}) - 0.0015\times (F / 1\ \text{kN/m})^2)\times (2.4\ \text{m} / L)^{0.4}]$
	$K_{107} = \mathbf{1.061}$
Interaction	$K_{108} = \mathbf{1.100}$
Wall modification factors	$K_w = K_{104} \times K_{105} \times K_{106} \times K_{107} \times K_{108} = \mathbf{1.227}$

Racking resistance of wall panel

Racking resistance of wall panel	$R_R = L \times K_w \times R_{bp} \times K_{mp} = \mathbf{7.522}$ kN