290 Sidegate Lane
Ipswich
Suffolk
IP4 3DN

RKM
Consulting Building & Structural Engineers

[redacted]

Our Ref: RKM/02547
31 January 2023

Mrs Marie Mayhew
Primrose Hill Farm
Main Road
Hemingstone, Ipswich.
Suffolk
IP6 9RL

Dear Mrs Mayhew

PROPOSED CONVERSION OF AN EXISTING BARN TO DWELLING AT PRIMROSE HILL FARM, MAIN ROAD, HEMINGSTONE, IPSWICH, SUFFOLK. IP6 9RL.

Further to our site inspection of above, we write to confirm the following:-

1. The building is a detached single storey timber framed barn built circa 1960s. Previously used as a pig shed and currently used for storage of agricultural machinery. The building construction generally comprises of timber framed portals of 9.0m span x 4.0m high spaced at 4.0m centres with corrugated profile asbestos cement roof cladding. There are internal studwork partitions spaced at 6.0 maximum centres. The external walls are timber framed panels on 140mm thick x 1200mm high concrete blockwork dwarf walls. The internal floor is concrete ground bearing slab.
   (Three photographs of the building is appended with this brief report)

2. There are no large trees growing in the immediate vicinity of the building.

3. Investigations carried out by us on other sites in the vicinity of the above; indicate that the subsoils in this area generally comprise a layer of top soil on silty clay with chalk fragments.

4. The structural timber framed elements appear to be structurally adequate. We did not record any significant distortions other than the usual isolated areas of superficial inset infestation. There are no significant structural distortions to the existing concrete floor slab other than minor superficial cracks within isolated areas. During our inspection, we recorded minor diagonal and vertical cracks on isolated parts of the external blockwork wall.

5. We have structurally assessed the existing timber frames for long term use of the building for the proposed conversion and found to be structurally adequate.
   (As per the attached structural calculations appended with this report).

6. No Architectural drawings for the proposed conversion have been provided to date. However, if the intended proposal recommends formation of external window and door openings limited to 1800mm wide together with nominal removal of internal partitions including formation of internal openings, we would therefore confirm that the proposal will be carried out without altering or strengthening the existing structural elements. The existing asbestos roof and external wall covering should be carefully removed from the roof structure and timber framed external walls and replaced with propriety insulated roof and wall claddings. The new claddings are subject to the Local Authority Planning and building Control approval and agreement. Timber preservation treatment is required to be carried out to all retained structural members by an approved specialist. The recorded minor cracks along the external blockwork dwarf walls have no structural significant impact on the proposal as they are highly likely related to thermal movements than structural movements and this can be remedied during the conversion scheme works. The existing floor slab is adequate to support domestic floor loading but it is required to be insulated in order to meet the current Building Regulations.

We trust the foregoing brief report is adequate for your present purposes but if you have any queries or require further assistance please let us know. In the meantime we would reiterate that our comments are based on a single visual inspection and that we have not inspected the woodwork or other parts of the structure which are covered, unexposed or inaccessible and we are, therefore, unable to report that any such part of the property is free from defect.

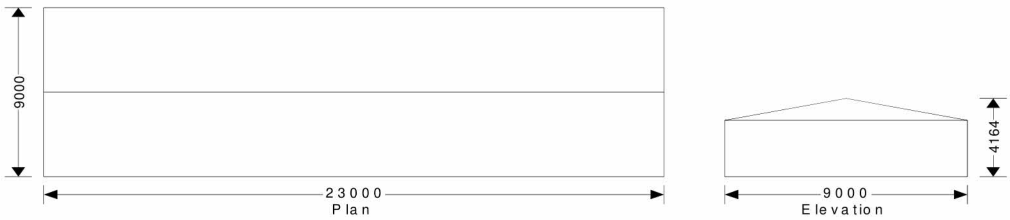
Yours sincerely

[redacted]

R K Mohsen B Eng (Hons.)
RKM

WIND LOADING (BS6399)

TEDDS calculation version 3.0.14



Building data

Type of roof	Duopitch
Length of building	$L = \mathbf{23000}$ mm
Width of building	$W = \mathbf{9000}$ mm
Height to eaves	$H = \mathbf{3000}$ mm
Pitch of roof	$\alpha_0 = \mathbf{14.5}$ deg
Reference height	$H_r = \mathbf{4164}$ mm

Dynamic classification

Building type factor (Table 1)	$K_b = \mathbf{1.0}$
Dynamic augmentation factor (1.6.1)	$C_r = [K_b \times (H_r / (0.1 \text{ m}))^{0.75}] / (800 \times \log(H_r / (0.1 \text{ m}))) = \mathbf{0.01}$

Site wind speed

Location	Ipswich
Basic wind speed (Figure 6 BS6399:Pt 2)	$V_b = \mathbf{23.2}$ m/s
Site altitude	$\Delta_S = \mathbf{26}$ m
Upwind distance from sea to site	$d_{sea} = \mathbf{17}$ km
Direction factor	$S_d = \mathbf{1.00}$
Seasonal factor	$S_s = \mathbf{1.00}$
Probability factor	$S_p = \mathbf{1.00}$
Critical gap between buildings	$g = \mathbf{10000000}$ mm
Topography not significant	
Altitude factor	$S_a = 1 + 0.001 \times \Delta_S / 1\text{m} = \mathbf{1.03}$
Site wind speed	$V_s = V_b \times S_a \times S_d \times S_s \times S_p = \mathbf{23.8}$ m/s
Terrain category	Country
Displacement height (sheltering effect excluded)	$H_d = 0$mm

The velocity pressure for the windward face of the building with a 0 degree wind is to be considered as 1 part as the height h is less than b (cl.2.2.3.2)

The velocity pressure for the windward face of the building with a 90 degree wind is to be considered as 1 part as the height h is less than b (cl.2.2.3.2)

Dynamic pressure - windward wall - Wind 0 deg and roof

Reference height (at which q is sought)	$H_{ref} = \mathbf{3000}$mm
Effective height	$H_e = \max(H_{ref} - H_d, 0.4 \times H_{ref}) = \mathbf{3000}$mm
Fetch factor (Table 22)	$S_c = \mathbf{0.826}$
Turbulence factor (Table 22)	$S_t = \mathbf{0.207}$
Gust peak factor	$g_t = \mathbf{3.44}$
Terrain and building factor	$S_b = S_c \times (1 + (g_t \times S_t) + S_h) = \mathbf{1.41}$
Effective wind speed	$V_e = V_s \times S_b = \mathbf{33.7}$ m/s

Dynamic pressure $q_s = 0.613\ kg/m^3 \times V_e^2 = \mathbf{0.695}\ kN/m^2$

Dynamic pressure - windward wall - Wind 90 deg and roof

Reference height (at which q is sought) $H_{ref} = \mathbf{4164}mm$

Effective height $H_e = max(H_{ref} - H_d, 0.4 \times H_{ref}) = \mathbf{4164}mm$

Fetch factor (Table 22) $S_c = \mathbf{0.891}$

Turbulence factor (Table 22) $S_t = \mathbf{0.198}$

Gust peak factor $g_t = \mathbf{3.44}$

Terrain and building factor $S_b = S_c \times (1 + (g_t \times S_t) + S_h) = \mathbf{1.50}$

Effective wind speed $V_e = V_s \times S_b = \mathbf{35.7}\ m/s$

Dynamic pressure $q_s = 0.613\ kg/m^3 \times V_e^2 = \mathbf{0.781}\ kN/m^2$

Size effect factors

Diagonal dimension for gablewall $a_{eg} = \mathbf{9.9}\ m$

External size effect factor gablewall $C_{aeg} = \mathbf{0.948}$

Diagonal dimension for side wall $a_{es} = \mathbf{23.2}\ m$

External size effect factor side wall $C_{aes} = \mathbf{0.884}$

Diagonal dimension for roof $a_{er} = \mathbf{23.5}\ m$

External size effect factor roof $C_{aer} = \mathbf{0.883}$

Room/storey volume for internal size effect factor $V_i = \mathbf{0.125}\ m^3$

Diagonal dimension for internal size effect factors $a_i = 10 \times (V_i)^{1/3} = \mathbf{5.000}\ m$

Internal size effect factor $C_{ai} = \mathbf{1.000}$

Pressures and forces

Net pressure $p = q_s \times c_{pe} \times C_{ae} - q_s \times c_{pi} \times C_{ai}$

Net force $F_w = p \times A_{ref}$

Roof load case 1 - Wind 0, c_{pi} 0.20, $-c_{pe}$

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m^2)	External size factor, C_{ae}	Net Pressure, p (kN/m^2)	Area, A_{ref} (m^2)	Net force, F_w (kN)
A (-ve)	-1.14	0.78	0.883	-0.94	7.16	-6.73
B (-ve)	-0.82	0.78	0.883	-0.72	12.62	-9.11
C (-ve)	-0.41	0.78	0.883	-0.44	87.12	-38.25
E (-ve)	-1.28	0.78	0.883	-1.04	7.16	-7.44
F (-ve)	-0.87	0.78	0.883	-0.76	12.62	-9.55
G (-ve)	-0.49	0.78	0.883	-0.50	87.12	-43.36

Total vertical net force $F_{w,v} = \mathbf{-110.81}\ kN$

Total horizontal net force $F_{w,h} = \mathbf{1.57}\ kN$

Walls load case 1 - Wind 0, c_{pi} 0.20, $-c_{pe}$

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m^2)	External size factor, C_{ae}	Net Pressure, p (kN/m^2)	Area, A_{ref} (m^2)	Net force, F_w (kN)
A	-1.30	0.78	0.948	-1.12	5.36	-5.99
B	-0.80	0.78	0.948	-0.75	24.81	-18.57
C	-0.50	0.78	0.948	-0.53	2.08	-1.09
w	0.68	0.70	0.884	0.28	69.00	19.39
l	-0.50	0.70	0.884	-0.45	69.00	-30.80

Overall loading

Equiv leeward net force for overall section $F_l = F_{w,wl} = \mathbf{-30.8}$ kN

Net windward force for overall section $F_w = F_{w,ww} = \mathbf{19.4}$ kN

Overall loading overall section $F_{w,w} = 0.85 \times (1 + C_r) \times (F_w - F_l + F_{w,h}) = \mathbf{44.5}$ kN

Roof load case 2 - Wind 0, c_{pi} -0.3, $+c_{pe}$

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m^2)	External size factor, C_{ae}	Net Pressure, p (kN/m^2)	Area, A_{ref} (m^2)	Net force, F_w (kN)
A (+ve)	0.19	0.78	0.883	0.37	7.16	2.62
B (+ve)	0.19	0.78	0.883	0.37	12.62	4.61
C (+ve)	0.19	0.78	0.883	0.37	87.12	31.84
E (+ve)	-1.28	0.78	0.883	-0.65	7.16	-4.65
F (+ve)	-0.87	0.78	0.883	-0.37	12.62	-4.62
G (+ve)	-0.49	0.78	0.883	-0.11	87.12	-9.34

Total vertical net force $F_{w,v} = \mathbf{19.81}$ kN

Total horizontal net force $F_{w,h} = \mathbf{14.44}$ kN

Walls load case 2 - Wind 0, c_{pi} -0.3, $+c_{pe}$

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m^2)	External size factor, C_{ae}	Net Pressure, p (kN/m^2)	Area, A_{ref} (m^2)	Net force, F_w (kN)
A	-1.30	0.78	0.948	-0.73	5.36	-3.90
B	-0.80	0.78	0.948	-0.36	24.81	-8.89
C	-0.50	0.78	0.948	-0.14	2.08	-0.28
w	0.68	0.70	0.884	0.63	69.00	43.37
l	-0.50	0.70	0.884	-0.10	69.00	-6.81

Overall loading

Equiv leeward net force for overall section $F_l = F_{w,wl} = \mathbf{-6.8}$ kN

Net windward force for overall section $F_w = F_{w,ww} = \mathbf{43.4}$ kN

Overall loading overall section $F_{w,w} = 0.85 \times (1 + C_r) \times (F_w - F_l + F_{w,h}) = \mathbf{55.6}$ kN

Roof load case 3 - Wind 90, c_{pi} 0.20, $-c_{pe}$

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m^2)	External size factor, C_{ae}	Net Pressure, p (kN/m^2)	Area, A_{ref} (m^2)	Net force, F_w (kN)
A (-ve)	-1.62	0.78	0.883	-1.27	3.87	-4.93

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m^2)	External size factor, C_{ae}	Net Pressure, p (kN/m^2)	Area, A_{ref} (m^2)	Net force, F_w (kN)
A (-ve)	-1.62	0.78	0.883	-1.27	3.87	-4.93
B (-ve)	-1.48	0.78	0.883	-1.18	3.87	-4.56
C (-ve)	-0.60	0.78	0.883	-0.57	30.97	-17.66
D (-ve)	-0.40	0.78	0.883	-0.44	175.10	-76.28

Total vertical net force $F_{w,v}$ = **-100.13** kN

Total horizontal net force $F_{w,h}$ = **0.00** kN

Walls load case 3 - Wind 90, c_{pi} 0.20, $-c_{pe}$

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m^2)	External size factor, C_{ae}	Net Pressure, p (kN/m^2)	Area, A_{ref} (m^2)	Net force, F_w (kN)
A	-1.30	0.70	0.884	-0.94	5.00	-4.69
B	-0.80	0.70	0.884	-0.63	19.99	-12.61
C	-0.50	0.70	0.884	-0.45	44.02	-19.65
w	0.60	0.78	0.948	0.29	32.24	9.29
l	-0.50	0.78	0.948	-0.53	32.24	-16.98

Overall loading

Equiv leeward net force for overall section $F_l = F_{w,wl}$ = **-17.0** kN

Net windward force for overall section $F_w = F_{w,ww}$ = **9.3** kN

Overall loading overall section $F_{w,w} = 0.85 \times (1 + C_r) \times (F_w - F_l + F_{w,h})$ = **22.6** kN

Roof load case 4 - Wind 90, c_{pi} -0.3, $-c_{pe}$

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m^2)	External size factor, C_{ae}	Net Pressure, p (kN/m^2)	Area, A_{ref} (m^2)	Net force, F_w (kN)
A (-ve)	-1.62	0.78	0.883	-0.88	3.87	-3.42
B (-ve)	-1.48	0.78	0.883	-0.79	3.87	-3.05
C (-ve)	-0.60	0.78	0.883	-0.18	30.97	-5.56
D (-ve)	-0.40	0.78	0.883	-0.05	175.10	-7.90

Total vertical net force $F_{w,v}$ = **-19.29** kN

Total horizontal net force $F_{w,h}$ = **0.00** kN

Walls load case 4 - Wind 90, c_{pi} -0.3, $-c_{pe}$

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m^2)	External size factor, C_{ae}	Net Pressure, p (kN/m^2)	Area, A_{ref} (m^2)	Net force, F_w (kN)
A	-1.30	0.70	0.884	-0.59	5.00	-2.95
B	-0.80	0.70	0.884	-0.28	19.99	-5.66
C	-0.50	0.70	0.884	-0.10	44.02	-4.35
w	0.60	0.78	0.948	0.68	32.24	21.88
l	-0.50	0.78	0.948	-0.14	32.24	-4.39

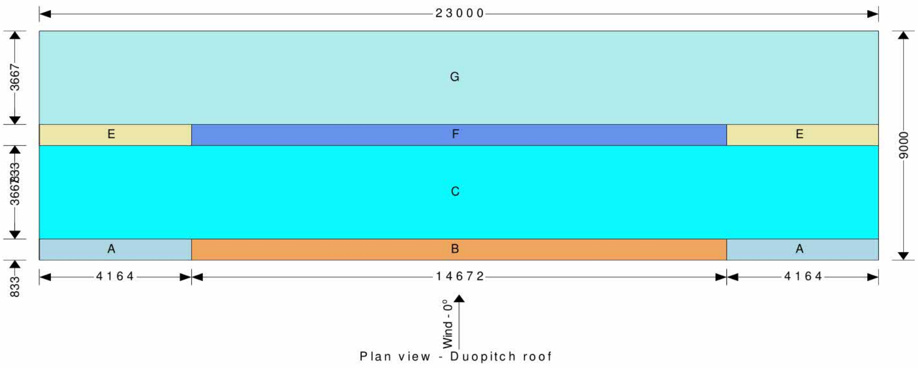
Overall loading

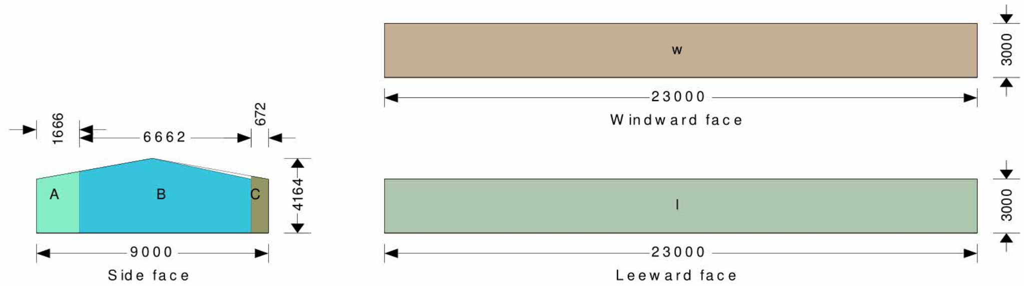
Equiv leeward net force for overall section $F_l = F_{w,wl}$ = **-4.4** kN

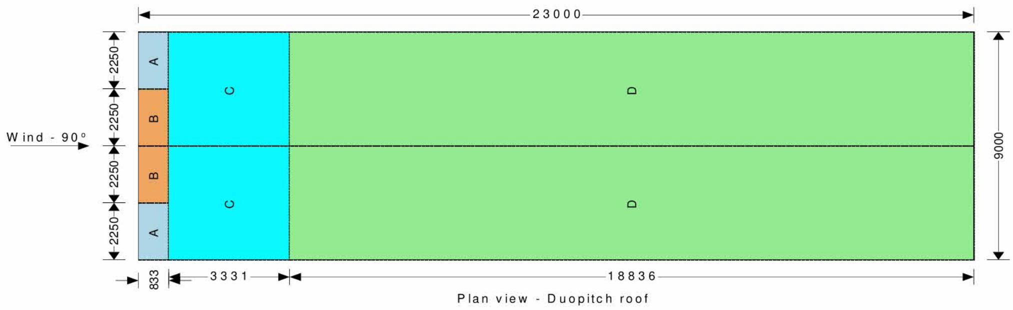
Net windward force for overall section $F_w = F_{w,ww}$ = **21.9** kN

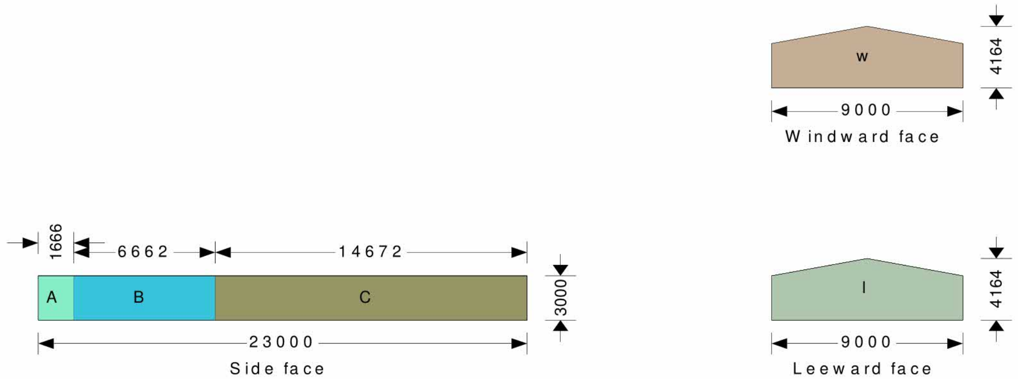
Tedds RKM 290 Sidegate Lane, Ipswich Suffolk IP4 3DN Mob: 07802 382636	Project Barn Conversion at Primrose Hill Farm, Main Road,				Job no. 02547	
	Calcs for Assessing the wind loading				Start page no./Revision 5	
	Calcs by RKM	Calcs date 01/02/2023	Checked by	Checked date	Approved by	Approved date

Overall loading overall section $F_{w,w} = 0.85 \times (1 + C_r) \times (F_w - F_l + F_{w,h}) = \mathbf{22.6}$ kN



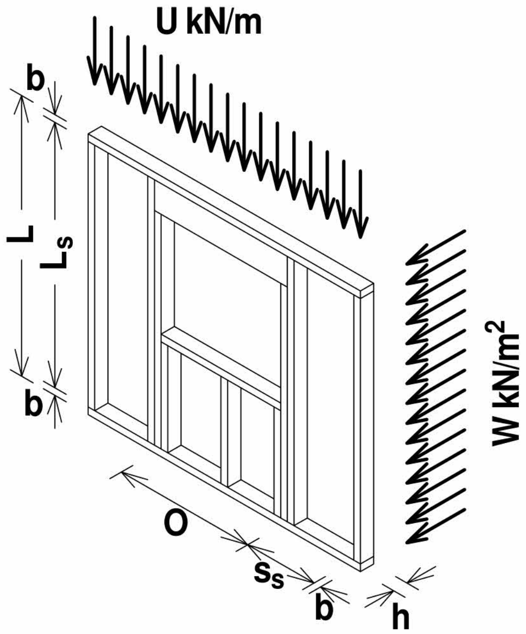






TIMBER STUD DESIGN (BS5268-2:2002)

TEDDS calculation version 1.0.05



Stud details

Stud breadth b = **50** mm

Stud depth h = **75** mm

Number of studs N_s = **15**

Strength class C16 timber (Table 8 BS5268:Pt 2:2002)

Section properties

Cross sectional area $A = N_s \times b \times h = \mathbf{56250}$ mm^2

Section modulus $Z = N_s \times b \times h^2 / 6 = \mathbf{703125}$ mm^3

Moment of inertia in the major axis $I_x = N_s \times b \times h^3 / 12 = \mathbf{26367187}$ mm^4

Moment of inertia in the minor axis $I_y = N_s \times h \times b^3 / 12 = \mathbf{11718750}$ mm^4

Radius of gyration in the major axis $r_x = \sqrt{(I_x / A)} = \mathbf{21.7}$ mm

Radius of gyration in the minor axis $r_y = \sqrt{(I_y / A)} = \mathbf{14.4}$ mm

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

Panel height L = **2400** mm

Stud length $L_s = L - (2 \times b) = \mathbf{2300}$ mm

Standard stud spacing s_s = **400** mm

Panel opening O = **1900** mm

Loaded panel length $s = \max(s_s, (O + s_s) / 2) = \mathbf{1150}$ mm

Effective length in the major axis $L_{ex} = 0.85 \times L_s = \mathbf{1955}$ mm

Slenderness ratio $\lambda = L_{ex} / r_x = \mathbf{90.30}$

Vertical loading details	**Dead loads**	**Imposed loads**
Wall UDL	U_{w_d} = **1.20** kN/m	

Roof UDL U_{r_d} = **5.00** kN/m U_{r_i} = **2.70** kN/m

Lateral loading details

Wind loading W = **0.78** kN/m^2

Wind load duration **Short term**

Modification factors

Section depth factor $K_7 = (300\text{ mm} / h)^{0.11}$ = **1.16**

Load sharing factor K_8 = **1.10**

Consider combined axial compression and bending under short term loads

Load duration factor K_3 = **1.50**

Vertical loading $F = (U_{w\_d} + U_{r\_d} + U_{r\_i}) \times s$ = **10.24** kN

Check bending stress

Bending parallel to grain σ_m = **5.300** N/mm^2

Permissible bending stress $\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8$ = **10.186** N/mm^2

Bending moment $M_{max} = W \times s \times L^2 / 8$ = **0.646** kNm

Applied bending stress $\sigma_{m\_max} = M_{max} / Z$ = **0.919** N/mm^2

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

Check compressive stress on stud

Compression member factor K_{12} = **0.35**

Compression parallel to grain σ_c = **6.800** N/mm^2

Permissible compressive stress $\sigma_{c\_adm} = \sigma_c \times K_3 \times K_8 \times K_{12}$ = **3.907** N/mm^2

Applied compressive stress $\sigma_{c\_max} = F / (N_s \times b \times h)$ = **0.182** N/mm^2

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

Check compressive stress on rail

Bearing stress modification factor K_4 = **1.00**

Compression perpendicular to grain (no wane) σ_{cp1} = **2.200** N/mm^2

Permissible compressive stress $\sigma_{cp1\_adm} = \sigma_{cp1} \times K_3 \times K_4$ = **3.300** N/mm^2

Applied compressive stress $\sigma_{cp1\_max} = F / (N_s \times b \times h)$ = **0.182** N/mm^2

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

Check combined axial compression and bending

Euler critical stress $\sigma_e = (\pi^2 \times E_{min}) / \lambda^2$ = **7.021** N/mm^2

Euler coefficient $K_{eu} = 1 - (1.5 \times \sigma_{c\_max} \times K_{12} / \sigma_e)$ = **0.986**

Combined axial compression and bending value $K = \sigma_{m\_max} / (\sigma_{m\_adm} \times K_{eu}) + \sigma_{c\_max} / \sigma_{c\_adm}$ = **0.138** **< 1**

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

Check stud deflection

Euler critical stress $\sigma_e = (\pi^2 \times E_{min}) / \lambda^2$ = **7.021** N/mm^2

Maximum deflection $\delta_{adm} = \min(6.9\text{ mm}, 0.003 \times (L - 2 \times b))$ = **6.900** mm

Bending deflection $\delta_{max} = 0.005 \times \lambda \times (\sigma_{c\_max} + \sigma_{m\_max}) / (\sigma_e - \sigma_{c\_max}) \times (Z / A)$ = **0.908** 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 $F = (U_{w\_d} + U_{r\_d} + U_{r\_i}) \times s$ = **10.24** kN

Check compressive stress on stud

Compression member factor K_{12} = **0.39**

Compression parallel to grain σ_c = **6.800** N/mm^2

Permissible compressive stress $\sigma_{c\_adm} = \sigma_c \times K_3 \times K_8 \times K_{12} = \mathbf{3.656}$ N/mm^2

Applied compressive stress $\sigma_{c\_max} = F / (N_s \times b \times h) = \mathbf{0.182}$ N/mm^2

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

Check compressive stress on rail

Bearing stress modification factor $K_4 = \mathbf{1.00}$

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

Permissible compressive stress $\sigma_{cp1\_adm} = \sigma_{cp1} \times K_3 \times K_4 = \mathbf{2.750}$ N/mm^2

Applied compressive stress $\sigma_{cp1\_max} = F / (N_s \times b \times h) = \mathbf{0.182}$ N/mm^2

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

Consider axial compression without bending under long term loads

Load duration factor $K_3 = \mathbf{1.00}$

Vertical loading $F = (U_{w\_d} + U_{r\_d}) \times s = \mathbf{7.13}$ kN

Check compressive stress on stud

Compression member factor $K_{12} = \mathbf{0.44}$

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

Permissible compressive stress $\sigma_{c\_adm} = \sigma_c \times K_3 \times K_8 \times K_{12} = \mathbf{3.307}$ N/mm^2

Applied compressive stress $\sigma_{c\_max} = F / (N_s \times b \times h) = \mathbf{0.127}$ N/mm^2

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

Check compressive stress on rail

Bearing stress modification factor $K_4 = \mathbf{1.00}$

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

Permissible compressive stress $\sigma_{cp1\_adm} = \sigma_{cp1} \times K_3 \times K_4 = \mathbf{2.200}$ N/mm^2

Applied compressive stress $\sigma_{cp1\_max} = F / (N_s \times b \times h) = \mathbf{0.127}$ N/mm^2

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

RACKING LOADS DESIGN – BS6399-2:1997

TEDDS calculation version 1.0.06

Considering wind loads to the front elevation

General details

Building type	Dwelling
Overall height of building	$H = \mathbf{4.000}$ m
Number of storeys	1
Depth of building	$D = \mathbf{23.000}$ m
Breadth of building	$B = \mathbf{9.000}$ m
Roof type	Duopitch
Roof pitch	$\alpha = \mathbf{14.5}$ deg

Windloading details

Dynamic augmentation factor	$C_r = \mathbf{0.010}$
Dynamic pressure eaves level	$q_{se} = \mathbf{0.780}$ kN/m^2
Dynamic pressure roof level	$q_{sr} = \mathbf{0.780}$ kN/m^2

Wall details	**Area on elevation**	**Pressure coefficient**	**Size effect factor**
Ground floor w/w elevation	$Area_{w0} = \mathbf{30.000}$ m^2	$C_{pew} = \mathbf{0.600}$	$C_{aw0} = \mathbf{1.000}$
Ground floor l/w elevation	$Area_{l0} = \mathbf{55.000}$ m^2	$C_{pel} = \mathbf{-0.500}$	$C_{al0} = \mathbf{1.000}$

Roof details	**Area on plan**	**Pressure coefficient**
Roof zone A	$Plan_A = \mathbf{6.400}$ m^2	$C_{peA} = \mathbf{0.190}$
Roof zone B	$Plan_B = \mathbf{0.800}$ m^2	$C_{peB} = \mathbf{0.190}$
Roof zone C	$Plan_C = \mathbf{96.300}$ m^2	$C_{peC} = \mathbf{0.190}$
Roof zone E	$Plan_E = \mathbf{6.400}$ m^2	$C_{peE} = \mathbf{-1.280}$
Roof zone F	$Plan_F = \mathbf{0.800}$ m^2	$C_{peF} = \mathbf{-0.870}$
Roof zone G	$Plan_G = \mathbf{96.300}$ m^2	$C_{peG} = \mathbf{-0.495}$

Shielding effect of masonry cladding on windward elevation

Masonry wall with buttresses or returns at one end only

Total area of elevation	$Area_w = \mathbf{30.000}$ m^2
Total area of openings	$Open_w = \mathbf{10.800}$m^2
Percentage of openings	$p_w = Open_w / Area_w = \mathbf{36.0}$ %

From BS 5268:Section 6.1:1996 - Table 1

Windward modification factor $K_{100w} = \mathbf{0.744}$

Shielding effect of masonry cladding on leeward elevation

Masonry wall with buttresses or returns at one end only

Total area of elevation	$Area_l = \mathbf{55.000}$ m^2
Total area of openings	$Open_l = \mathbf{4.500}$m^2
Percentage of openings	$p_l = Open_l / Area_l = \mathbf{8.2}$ %

From BS 5268:Section 6.1:1996 - Table 1

Leeward modification factor $K_{100l} = \mathbf{0.633}$

Calculate racking load at ground floor level

Comb w/w loading coefficient	$C_w = C_{aw0} \times (1 + C_r) = \mathbf{1.010}$
Comb l/w loading coefficient	$C_l = C_{aw0} \times (1 + C_r) = \mathbf{1.010}$
Area of w/w elevation	$TotalArea_{w0} = Area_{w0} / 2 = \mathbf{15.000}$ m^2
Loads applied to w/w elevation	$P_{w0} = 0.85 \times K_{100w} \times TotalArea_{w0} \times q_{se} \times C_{pew} \times C_w = \mathbf{4.484}$ kN
Area of l/w elevation	$TotalArea_{l0} = Area_{l0} / 2 = \mathbf{27.500}$ m^2

Loads applied to l/w elevation $P_{l0} = 0.85 \times K_{100l} \times TotalArea_{l0} \times q_{se} \times C_{pel} \times C_{l}$ = **-5.826** kN

Loads applied to roof Zone A $P_{rA} = 0.85 \times Plan_{A} \times \tan(\alpha) \times q_{sr} \times C_{peA} \times C_{w}$ = **0.211** kN

Loads applied to roof Zone B $P_{rB} = 0.85 \times Plan_{B} \times \tan(\alpha) \times q_{sr} \times C_{peB} \times C_{w}$ = **0.026** kN

Loads applied to roof Zone C $P_{rC} = 0.85 \times Plan_{C} \times \tan(\alpha) \times q_{sr} \times C_{peC} \times C_{w}$ = **3.169** kN

Loads applied to roof Zone E $P_{rE} = 0.85 \times Plan_{E} \times \tan(\alpha) \times q_{sr} \times C_{peE} \times C_{l}$ = **-1.419** kN

Loads applied to roof Zone F $P_{rF} = 0.85 \times Plan_{F} \times \tan(\alpha) \times q_{sr} \times C_{peF} \times C_{l}$ = **-0.121** kN

Loads applied to roof Zone G $P_{rG} = 0.85 \times Plan_{G} \times \tan(\alpha) \times q_{sr} \times C_{peG} \times C_{l}$ = **-8.255** kN

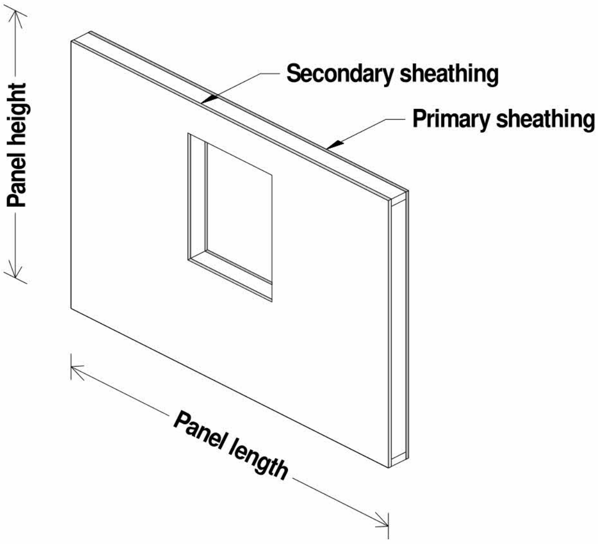
Racking load at ground floor $P_{0} = P_{w0} - P_{l0} + P_{rA} + P_{rB} + P_{rC} - P_{rE} - P_{rF} - P_{rG}$ = **23.510** kN

TIMBER PANEL RACKING RESISTANCE – BS5268:SECTION 6.1:1996

TEDDS calculation version 1.0.05

Dwellings not exceeding seven storeys

Gable walls



Wall panel details

Length of panel	L = **5.000** m
Height of panel	H_{wp} = **2.500** m
Total area of wall panel	$A_t = L \times H_{wp}$ = **12.500** m^2
Aggregate area of framed panel openings	A_a = **2.000** m^2
Timber members	**38 mm x 72 mm or larger**
Uniformly distributed load on timber frame wall	F_{udl} = **0.000** kN/m
For calculation equivalent uniformly distributed load	F = min(F_{udl}, 10.5 kN/m) = **0.000** 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 $K_{104} = 2.4\ \text{m} / H_{wp}$ = **0.960**

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

Fully framed openings in walls $K_{106} = (1 - 1.3 \times A_a / A_t)^2$ = **0.627**

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} = **1.000**

Interaction K_{108} = **1.100**

Wall modification factors $K_w = K_{104} \times K_{105} \times K_{106} \times K_{107} \times K_{108}$ = **0.874**

Racking resistance of wall panel

Racking resistance of wall panel $R_R = L \times K_w \times (R_{bp} \times K_{mp} + R_{bs} \times K_{ms})$ = **7.869** kN

Racking resistance of plasterboard only $R_{PO} = L \times K_w \times R_{bs} \times K_{ms}$ = **0.525** kN

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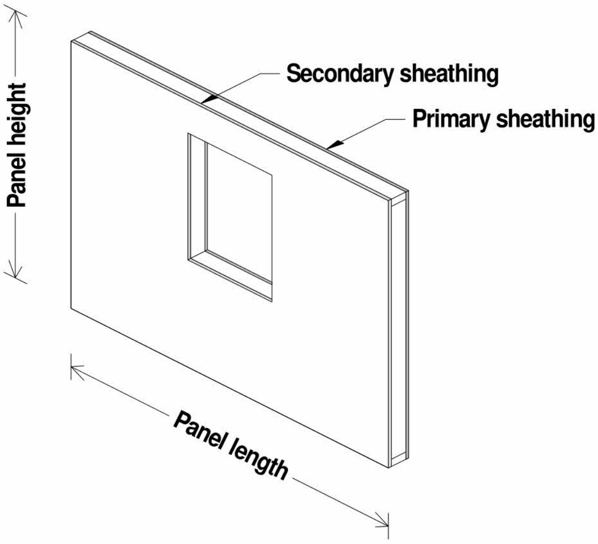
TIMBER PANEL RACKING RESISTANCE – BS5268:SECTION 6.1:1996

TEDDS calculation version 1.0.05

Dwellings not exceeding seven storeys

Gable walls



Wall panel details

Length of panel	L = **6.000** m
Height of panel	H_{wp} = **2.500** m
Total area of wall panel	$A_t = L \times H_{wp}$ = **15.000** m^2
Aggregate area of framed panel openings	A_a = **3.600** m^2
Timber members	**38 mm x 72 mm or larger**
Uniformly distributed load on timber frame wall	F_{udl} = **4.500** kN/m
For calculation equivalent uniformly distributed load	$F = \min(F_{udl}, 10.5\ kN/m)$ = **4.500** 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 $B_s = \min(\max(T_s / t_s, 0.75), 1.25) = \mathbf{1.00}$

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 $K_{103s} = 2.8 \times B_s - B_s^2 - 0.8 = \mathbf{1.000}$

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

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

Height of wall panels $K_{104} = 2.4\text{ m} / H_{wp} = \mathbf{0.960}$

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

Fully framed openings in walls $K_{106} = (1 - 1.3 \times A_a / A_t)^2 = \mathbf{0.473}$

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} = **1.260**

Interaction K_{108} = **1.100**

Wall modification factors $K_w = K_{104} \times K_{105} \times K_{106} \times K_{107} \times K_{108} = \mathbf{0.831}$

Racking resistance of wall panel

Racking resistance of wall panel $R_R = L \times K_w \times (R_{bp} \times K_{mp} + R_{bs} \times K_{ms}) = \mathbf{8.976}$ kN

Racking resistance of plasterboard only $R_{PO} = L \times K_w \times R_{bs} \times K_{ms} = \mathbf{0.598}$ kN