# Timber Engineering Notebook series

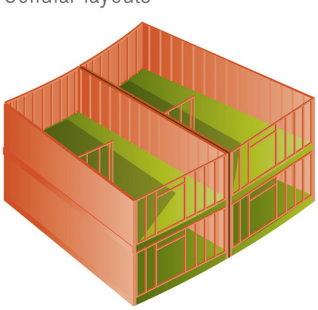
## No. 4: Timber frame structures – platform frame construction (part 2)

**This series is authored by the UKTFA**

The UK Timber Frame Association (UKTFA) represents over 85% of the UK's timber structural frame supply industry and is a trade organisation that provides business and technical support to the industry. The Association provides peer reviewed outputs on subjects related to the timber industry such as health and safety, fabric and technical performance, fire safety, promotion and training. These documents and other information are available at www.uktfa.com

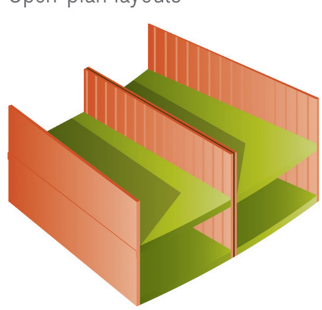
**Figure 1**
**Examples of building layouts and their influence on overall stability**

Cellular layouts



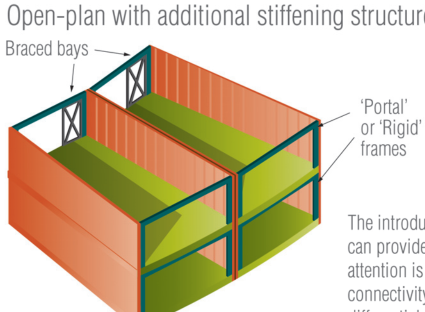
Cellular layouts are best suited to multi-storey platform timber frame. Internal load bearing walls may need to be strengthened to carry horizontal forces.

Open-plan layouts



Open-plan layouts with no transverse structure are not appropriate for platform timber frame and additional structure is required for stability design.

Open-plan with additional stiffening structure




The introduction of portal frames or braced bays, can provide solutions to open plan layouts but attention is needed to stiffness limits and connectivity of the framing types as well as differential movement of different materials.

### Introduction

In *Timber Engineering Notebook No. 3* (part 1 in this sub-series on platform frame construction), the composition and terminology used for platform timber frame building structures, and the structural engineering checks which are required to verify the adequacy of the vertical load paths and the strength and stiffness of the individual framing members, was introduced.

This Timber Engineering Notebook introduces the engineering checks for overall building stability and the stability checks required for the wall diaphragms which provide shear (or racking) resistance to a platform timber frame structure.

Robustness and disproportionate collapse design considerations for platform timber frame buildings will be discussed in part 3 of this sub-series.

### Overall stability

To achieve its stability, platform timber frame construction relies on the diaphragm action of floor structures to transfer horizontal forces to a distributed arrangement of loadbearing walls. The load bearing walls provide both vertical support and horizontal racking and shear resistance.

Due to the presence of open-plan or asymmetric layouts or the occurrence of large openings in loadbearing walls, it may be necessary to provide other means of providing stability to the building, for example by the use of 'portalised' or 'rigid' frames or discrete braced bays as indicated in Figure 1.

In these situations particular care needs to be taken by the engineer to ensure that the connections between elements are designed adequately, so that the loads can be distributed to the points of stiffness in the structure, e.g. floor diaphragm to braced bay connections, racking wall to rigid frame connections etc.

### Engineering principles

The horizontal load paths that require checking by the engineer are shown in Figure 2.

### Horizontal diaphragms and bracing

Horizontal diaphragms in platform frame buildings are provided by the intermediate floors (with a wood-based subdeck material fixed directly to the joists) and the roof structure (with either a wood-based 'sarking' board or discrete diagonal bracing members) (Figure 3). These horizontal structural diaphragms transfer

**Figure 2**
**Horizontal structural load paths in platform timber frame**

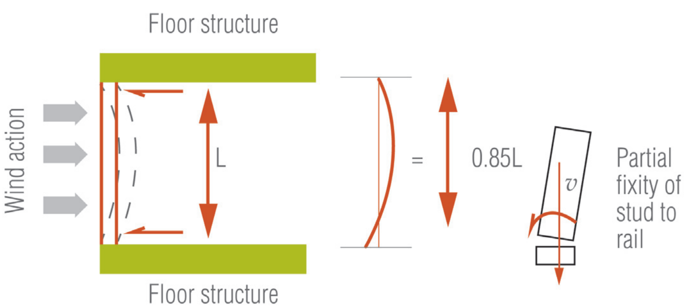



**Figure 2a**
**Wall studs subject to lateral actions – effective length of studs**

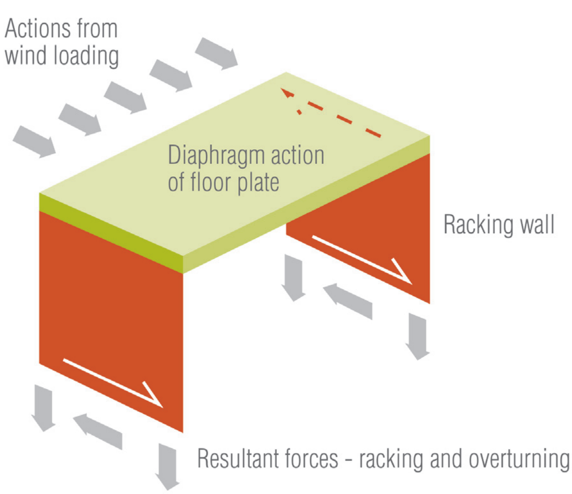



**Figure 2b**
**Transfer of lateral loads from a floor diaphragm into racking walls**

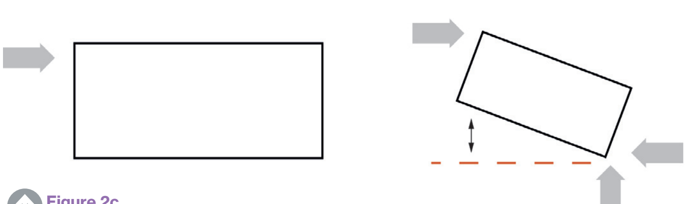


**Figure 2c**
**Racking wall – holding down fixings or connections to adjacent wall panels are required to provide overturning resistance. Fixings to underlying construction are necessary to prevent sliding**

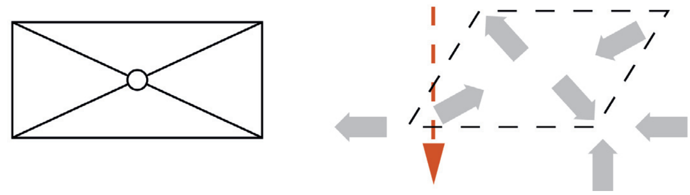


**Figure 2d**
**Braced frame – Mechanical fixings are required to provide resistance to uplift and sliding**

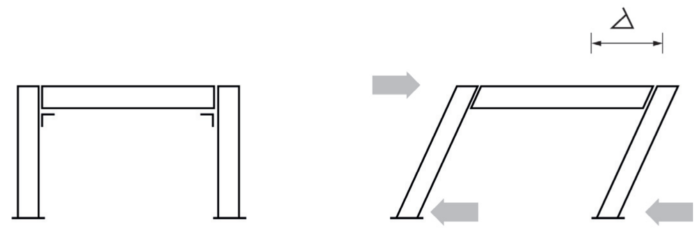



**Figure 2e**
**Rigid frame or 'sway' frame – sway must be limited to H/500 to be compatible with timber racking walls**

## Horizontal load transfer – structural notes

Horizontal loadpaths are to be checked for load transfer as follows:

Key items:

1. Wall stud bending and deflection. Partial stud fixity can be assumed leading to effective length of 0.85 L.
2. Check for stability of building against resistance to overturning.
3. Check of overall racking resistance of building and resolution of forces on the individual wall panels providing resistance against horizontal actions.
4. Ability of floor and roof diaphragms to transfer loads to racking walls.
5. Fixing of wall panels to floor structure and floor structure to lower wall panel or sole plate for sliding resistance.
6. Uplift resistance of racking panels and roof structures.
7. Deflection and sway of rigid frame components and fixity of bases.

**Figure 3**
**Platform timber frame under construction**

horizontal loads acting on the building to the foundations by means of their connections to the wall panels (or vertical diaphragms).

### Vertical diaphragms - racking walls

Vertical diaphragms or shear walls are commonly described as racking panels or walls in timber frame construction. They resist horizontal actions and are essential elements in the overall stability of the building.

The racking wall gains its strength from a wood-based board sheathing material or plasterboard lining material fixed to the wall studs which provides racking stability and sliding resistance by its connection to the horizontal diaphragms and foundations. Where different methods are used in a building to provide overall resistance to horizontal loads such as braced frames or

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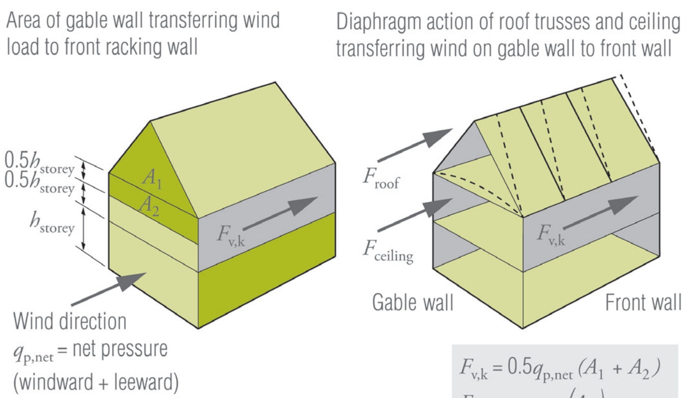



$$F_{v,k} = 0.5q_{p,net}(A_1 + A_2)$$
$$F_{roof} = q_{p,net}\left(\frac{A_1}{2}\right)$$
$$F_{ceiling} = q_{p,net}\left(\frac{A_1}{2} + A_2\right)$$

**Figure 4**
**Racking loads on timber frame roof, ceiling and wall diaphragms from wind loads on gable walls**

sway frames, care must be taken to ensure that the stiffness of individual vertical diaphragms is comparable. In these situations, the deflection limit of frames should be at least height/500 to ensure that they have a similar stiffness as the timber frame sheathed wall panels. Fig. 2 indicates the different stiffness characteristics of various vertical diaphragms.

Figure 4 shows the racking load on the front first floor wall of a house resulting from wind on the gable wall and the corresponding actions which must be resisted by the roof and ceiling diaphragms.

**Simplified analysis of wall diaphragms**
A method for the simplified analysis of wall diaphragms, consisting of timber framing connected on one or both faces to a wood-based sheathing material which, in turn, are connected to the underlying timber construction or foundations, is provided in PD 6693-1:2012 UK Non-Contradictory Complementary Information (NCCI) to Eurocode 5: Design of timber structures.

Plasterboard may also be considered to contribute to the racking resistance of a wall diaphragm within the limits set in PD 6693-1, but its contribution cannot be used in conjunction with wood-based sheathing boards on the same wall diaphragm.

**Requirements for timber framing to wall diaphragms**
The timber framing should consist of timber studs not exceeding 610mm centre to centre, between horizontal top and bottom rails. The timber framing members should be a minimum depth of 72mm and of minimum strength class C16.

The end nailing of the wall studs to the horizontal rails should comprise a minimum of two ringed-shank nails of diameter greater than or equal to 3.1mm and having a penetration into the studs greater than 45mm.

Shear buckling of the sheathing material can be disregarded provided that the following condition is met:

$$\frac{b_{net}}{t} \leq 100$$

Where:
$b_{net}$ is the clear distance between studs
$t$ is the thickness of the sheathing

The diameter of fasteners connecting the sheathing to the timber framing should be no greater than 0.09 x the stud thickness. Additionally where two sheathing sheets meet at a stud, the fastener edge distance for both the stud and the sheathing sheet should be a minimum of 3 x the fastener diameter. The fasteners fixing the sheathing to the framing should be equally spaced around the perimeter of each sheathing sheet at a maximum of 150mm. Fasteners fixing the sheathing to the framing on the internal studs should be equally spaced at not more than twice the perimeter fastener spacing.

**General arrangement of wall diaphragms**
A racking wall may comprise a single wall diaphragm or, if it

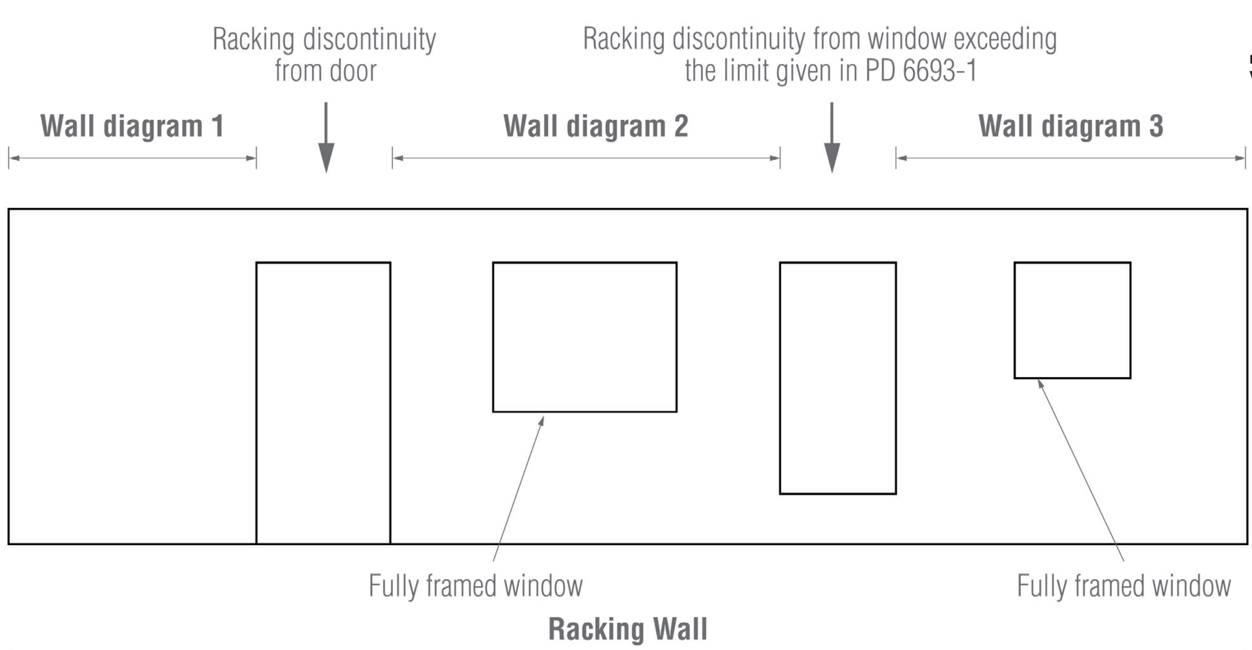



**Figure 5**
**Division of a racking wall into wall diaphragms**

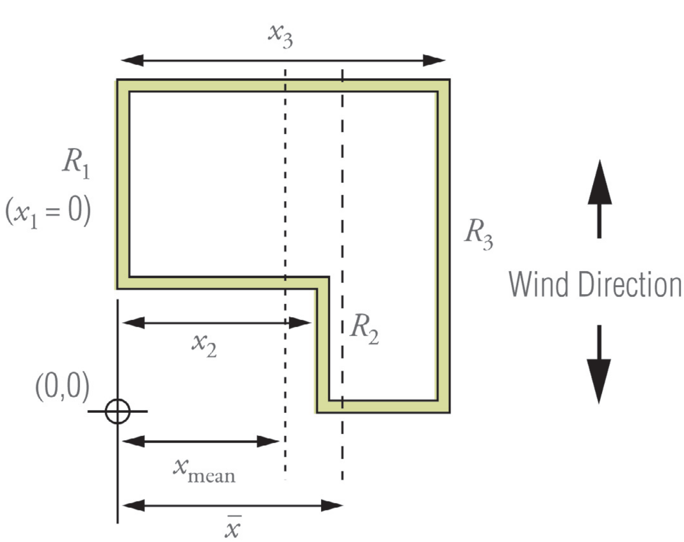
contains discontinuities such as door openings, may comprise more than one wall diaphragm (Figure 5). Limits are set out in PD 6693-1:2012.

**Distribution of horizontal wind loads between wall diaphragms**
Transfer of loads through ceiling and floors into wall diaphragms must be taken through the building storey levels so that there is a mechanical transfer of horizontal forces at the interface of floor/ceiling to wall diaphragm. Typically the wall diaphragms though the height of the building line up so that loads are accumulatively transferred down through the building.

The approach for platform frame engineering has been that a floor or ceiling diaphragm with standard construction detailing can be assumed to be acceptable to transfer loads without further design calculation, providing the aspect ratio of the diaphragm span being considered is no greater than 2 x the depth of the diaphragm. Eurocode 1995-1-1 takes this further in clause 9.2.3 where standard design approach is called simplified analysis for spans that lie between $2b$ and $6b$ where $b$ is the diaphragm depth. If the diaphragm dimensions fall outside of these general rules then specific engineering calculations to prove the diaphragm are needed.

A building should have a regularly distributed arrangement of racking walls in at least two orthogonal directions, such that there is not a significant eccentricity between the centroids of the wind action and the aggregated wall racking resistance.

If a significant eccentricity does exist (such as a three sided box with an open elevation e.g. a garage) then, where a sufficiently rigid horizontal diaphragm is provided, it is sufficient to assume that the structure acts as a rigid box, but the racking resistance of the orthogonal racking walls needs to be verified for the increased loads due to the sum of the direct and torsional forces acting on them.




$$R_1(\bar{x} - x_1) + R_2(\bar{x} - x_2) = R_3(x_3 - \bar{x})$$

$$\text{hence } \bar{x} = \frac{R_1 x_1 + R_2 x_2 + R_3 x_3}{R_1 + R_2 + R_3}$$

**Figure 6** Torsion-induced racking forces: Axis of rotation for wind perpendicular to the x-axis.

This torsional moment is equal to the wind load multiplied by the distance between the geometrical centre of the building and the building's centre of rotation (CR) measured perpendicular to the wind direction (Figure 6).

Torsional moment = $F_{v,d}(\bar{x} - x_{mean})$

The resulting torsional moment, is resisted by all the walls, with each wall contributing to the total moment in proportion to its (stiffness) x (lateral displacement) x (perpendicular distance to the centre of rotation), i.e.:

$$F_{v,d}(\bar{x} - x_{mean}) = k_x \sum R_{d,i} z_i^2$$

Where:
$F_{v,d}$ is the design racking load on building (sum of wind force on windward and leeward walls)
$\bar{x}$ is the distance of the building's centre of rotation from the origin, measured along the x-axis
$x_{mean}$ is the distance of geometrical centre of building from the origin, measured along x-axis
$k_x$ is a 'torsional' constant
$z_i$ is the perpendicular distance of any racking wall $i$ from CR, i.e.:

$(\bar{x} - x_i)$

or:

$(\bar{y} - y_i)$

as appropriate

The additional load which each wall perpendicular to the x-axis takes to resist the torsional moment is then:

$$F_{tor,d,I} = k_x R_{d,I} x_i$$

Hence the total load carried by each wall perpendicular to the x-axis is:

$$F_{d,I} = F_{v,d,i} + |F_{tor,d,i}|$$

The positive value of $F_{tor,d,i}$ should be used, for although $F_{tor,d,}$ will be positive for some walls, it will be negative when the wind is in the reverse direction.

Check that $F_{d,i} < R_{d,i}$ for each wall parallel to the wind, then repeat the exercise for wind at 90° where:

$F_{v,d,I}$ is the design load on racking wall $i$
$R_{d,I}$ is the design racking resistance of wall $i$

**Design requirements for wall diaphragms under wind load**
For each wall diaphragm, it should be ensured that adequate racking, overturning and sliding resistance is provided to resist the wind actions applied.

This involves the following checks:

- That adequate racking resistance exists to resist the applied horizontal actions
- That compressive forces in the leeward end of wall diaphragms do not cause either buckling of the wall studs or excessive bearing stresses on the horizontal framing members
- That the panel holding down force at the windward end of the panel $f_{w,d}$ used to calculate $k_{i,w}$ does not exceed either the permanent load (uplift capacity) of either the underlying construction or the fixings used to transfer the required holding

down force between levels. The holding down force can be provided by the nailed wall panel to wall panel connections. For tall, narrow buildings or buildings with lightly-loaded racking walls, this may require that the free ends of racking walls are provided with some holding-down resistance - in the form of threaded rods or proprietary mechanical 'hold-downs' to enable loads to be transferred through floors or from racking walls to foundations. By spanning floor structures onto racking walls, the resistance to racking panel overturning is also increased
- That the sliding resistance of the wall panel to floor or foundation connection is adequate. Friction may be considered to contribute and the coefficient of friction may be taken as 0.4

These checks should be made at critical levels in the building e.g. at floor levels and foundations, taking into account the worst combination of design permanent and variable actions and both horizontal and vertical components of wind actions.

**Calculation of design racking strength of a wall diaphragm**
The design racking strength of a wall diaphragm (in kN) is calculated as:

$$F_{i,v,Rd} = K_{opening} K_{i,w} f_{p,d,t} L$$

Where:
$L$ is the length of the wall diaphragm (m)
$K_{i,w}$ is a modification factor taking into account wall length, vertical load and holding-down arrangements
$K_{opening}$ is a modification factor taking into account the effect of fully-framed window openings within prescribed limits given by PD 6693-1

The total design shear capacity per unit length of perimeter sheathing fasteners is calculated as follows:

$$f_{p,d,t} = f_{p,d,1} + K_{comb} f_{p,d,2}$$

Where:
$f_{p,d,1}$ is the design shear capacity per unit length of perimeter sheathing fasteners of the first or only sheathing layer (kN/m)
$f_{p,d,2}$ is the design shear capacity per unit length of perimeter sheathing fasteners of the second sheathing layer (kN/m)
$K_{comb}$ is the sheathing combination factor in the range 0–0.75 (PD 6693-1 Table 8)

In order to limit the racking deflection, the following condition should be applied:

$$K_{i,w} \cdot f_{p,d,t} \leq 8(1+K_{comb})(L/H)$$

Where:
$H$ is the height of the sheathed area of the wall diaphragm (m).

The deign shear capacity per unit length of the perimeter fasteners to a sheathing sheet $f_{p,d}$ should be calculated from:

$$f_{p,d} = \frac{F_{f,Rd}(1.15+s_n)}{s_n}$$

Where:
$F_{f,Rd}$ is the design lateral capacity of an individual fastener (kN)
$S_n$ is the sheathing perimeter fastener spacing (m).

The modification factor $K_{i,w}$ should be calculated from:

$$K_{i,w} = \min\left\{1 \left| \left[1+\left(\frac{H}{\mu L}\right)^2+\left(\frac{2M_{d,stb,n}}{\mu f_{p,d,t} L^2}\right)\right]^{0.5} - \frac{H}{\mu L}\right.\right\}$$

Where:
$M_{d,stb,n}$ is the net design stabilising moment acting on the wall diaphragm from design permanent actions, reduced by any vertical component of design wind actions due to roof uplift (a method for the calculation of design stabilising moment is provided in PD 6693-1 Figure 3)

And $\mu = \min[1, f_{wd}/f_{p,d,t}]$

Where:
$f_{wd}$ is the design withdrawal capacity of bottom rail to floor connections per unit length (KN/m)

Openings within wall diaphragms are accounted for by the factor $K_{opening}$. For a wall diaphragm with small or no openings $K_{opening}$ is taken as 1.0. For a wall diaphragm with fully framed openings $K_{opening}$ is taken as:

$$K_{opening} = 1 - 1.9p$$

Where:
$p$ = aggregate area of window openings in a wall diaphragm/HL

## Worked example

The design of a vertical wall diaphragm (racking panel) subjected to wind actions to PD6693-1:2012 covers the racking, sliding and overturning design of the first floor level rear wall of a two storey house under wind action.

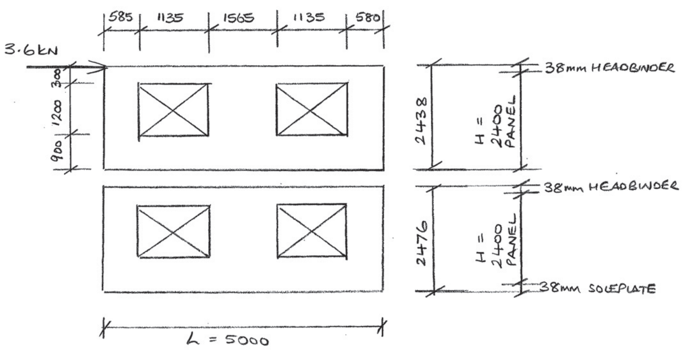
The overall height of the first floor wall is 2.438m between horizontal diaphragms. The wall panel itself is 2.4m in height and has a 38mm thick headbinder located on top. The wall functions in service class 2 conditions and supports a characteristic permanent action of 4.3 kN/m and a characteristic variable instantaneous (wind uplift) action of -1.1 kN/m. The wall panel self-weight is 0.6 kN/m. From an assessment of the overall wind actions acting on the structure, the characteristic wind action acting on the wall panel is 3.6kN. For simplicity, the wind action is assumed to be the same from both wind directions.

Fastener capacities for OSB sheathing to wall framing members, panel to panel connections and panel to soleplate connections can be derived from PD6693-1:2012 or can be taken from Tables 10.5, 6.5 and 6.8 of the Manual for the design of timber building structures to Eurocode 5. It should be noted when using these tables that for sheathing framing nails, the $k_{mod}$ value is taken as $\sqrt{(k_{mod,1}k_{mod,2})}$ in accordance with EC5 clause 2.3.2.1(2).

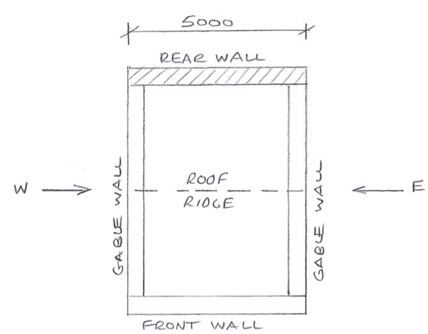
The following design fastener capacities are assumed in the calculations:

- Design lateral load capacity for 50mm long, 2.65mm diameter smooth nails at 150mm centres connecting sheathing to wall framing $F_{f,Rd,1}$ = 0.400 kN/nail
- Design lateral load capacity for 75mm long, 3.10mm diameter ring shanked nails at 150mm centres connecting wall panels to wall panels and at 600mm centres connecting wall panels to soleplates $F_{v,Rd}$ = 0.705 kN/nail
- Design withdrawal capacity for 90mm long, 3.10mm diameter ring shanked nails connecting wall panels to soleplates $F_{ax,Rd}$ = 0.393 kN/nail

Check that the wall will meet the ULS & SLS requirements of PD 6693-1 for resistance to horizontal (wind) actions:




ELEVATION ON REAR WALL




PLAN OF HOUSE

1. COMBINATION OF ACTIONS

(UK NATIONAL ANNEX TO BS EN 1990 : 2002 TABLE NA.A1.2(B))

ONLY ONE LOAD COMBINATION INVOLVING MINIMUM VERTICAL LOAD IS NECESSARY FOR THIS DESIGN EXAMPLE. FOR BUILDINGS OF MORE THAN 3 STOREYS THE LOAD COMBINATION FOR MAXIMUM VERTICAL LOAD MAY BE GOVERNING IN RESPECT OF THE COMPRESSION LOADS ON THE LEEWARD STUDS AND SHOULD BE CHECKED.

LOAD COMBINATION WITH MINIMUM VERTICAL LOAD

PERMANENT ACTIONS $\gamma_G = 1.0$ (FAVOURABLE)

DESIGN PERMANENT ACTION $= \gamma_G G_k = 1.0(4.3+0.6) = 4.9$ kN/m

LEADING VARIABLE ACTION $\gamma_Q = 1.5$

DESIGN VARIABLE ACTION $= \gamma_Q Q_{k(\text{UPLIFT})} = 1.5 \times {}^{-}1.1 = -1.65$ kN/m

2. RACKING DESIGN OF 1ST FLOOR STOREY REAR WALL

LOAD CAPACITIES OF NAILING SPECIFICATIONS IN RACKING WALL

SHEATHING NAILING $F_{f,Rd,1} = 0.400$ kN

PERIMETER SPACING $S_n = 0.15$ m

DESIGN SHEAR CAPACITY OF FASTENERS (PD 6693-1 : 2012 EQ(7))

$$f_{p,d,1} = \frac{F_{f,Rd}(1.15 + S_n)}{S_n} = \frac{0.400(1.15+0.15)}{0.15} = 3.47 \text{ kN/m}$$

AS THERE IS NO SECONDARY SHEATHING (PD 6693-1 : 2012 TABLE 8) $f_{p,d,2} = 0$, $k_{comb} = 0$

TOTAL DESIGN SHEAR CAPACITY PER UNIT LENGTH OF PERIMETER SHEATHING FASTENERS (PD 6693-1 : 2012 EQ(6))

$$f_{p,d,t} = f_{p,d,1} + k_{comb} f_{p,d,2} = 3.47 \text{ kN/m}$$

DESIGN SHEAR CAPACITY OF WALL PANEL – WALL PANEL CONNECTIONS @ 150cc

$$= \frac{F_{v,Rd}}{S} = \frac{0.705}{0.15} = 4.7 \text{ kN/m}$$

AS THIS CAPACITY EXCEEDS THE CAPACITY OF THE PERIMETER SHEATHING FASTENERS IT WILL NOT BE THE CRITICAL CONNECTION INTERFACE FOR THE ASSESSMENT OF THE PANEL RACKING RESISTANCE AND NO FURTHER CALCULATION CHECKS ARE NECESSARY FOR THIS NAILED CONNECTION.

DESIGN SHEAR RESISTANCE OF WALL PANEL – SOLEPLATE CONNECTIONS @ 600cc

$$= \frac{F_{v,Rd}}{S} = \frac{0.705}{0.6} = 1.18 \text{ kN/m}$$

DESIGN WITHDRAWAL CAPACITY OF WALL PANEL – SOLEPLATE CONNECTIONS @ 600cc

$$f_{w,d} = \frac{F_{ax,Rd}}{S} = \frac{0.393}{0.6} = 0.66 \text{ kN/m}$$

MODIFICATION FACTOR $\mu$ (PD 6693-1 : 2012 EQ(10))

$$\mu = \min\begin{cases}1.0\\ \dfrac{f_{w,d}}{f_{p,d,t}}\end{cases} = \min\begin{cases}1.0\\ \dfrac{0.66}{3.47}\end{cases} = 0.19$$

GENERAL ARRANGEMENT AND NUMBER OF WALL DIAPHRAGMS IN RACKING WALL (PD 6693-1 : 2012 CL 20.2.2)

MAXIMUM WINDOW HEIGHT $= 1.2\text{m} < 0.65H$

MINIMUM HEIGHT UNDER WINDOW $= 0.9\text{m} \geq 0.25H$

THEREFORE THERE ARE NO RACKING DISCONTINUITIES AND THE RACKING WALL CAN BE CONSIDERED AS A SINGLE WALL DIAPHRAGM OF 5.0m LENGTH

PERCENTAGE OPENINGS IN DIAPHRAGM (PD 6693-1 : 2012 EQ(18))

$$p = \frac{A}{HL} = \frac{2(1.2)(1.135)}{2.4 \times 5.0} = 0.23$$

OPENING MODIFICATION FACTOR (PD 6693-1 : 2012 EQ(17))

$$k_{opening} = 1 - 1.9p = 0.57$$

DESIGN LOADING ACTING ON WALL DIAPHRAGM

DESIGN HORIZONTAL WIND ACTION $= \gamma_Q Q_{k,\text{WIND}} = 1.5 \times 3.6 = 5.4$ kN

DESIGN DESTABILISING MOMENTS DUE TO WIND ACTION: (PD 6693-1 : 2012 FIG 3)

LEVER ARM FOR WIND ACTION TO BASE OF SHEATHING $= 2.438$ m

DESIGN DESTABILISING MOMENT $M_{d,dst,base} = 2.438 \times 5.4 = 13.17$ kNm

LEVER ARM FOR WIND ACTION TO TOP OF SHEATHING $= 0.038$m

DESIGN DESTABILISING MOMENT $M_{d,dst,top} = 0.038 \times 5.4 = 0.21$ kNm

DESIGN STABILISING MOMENTS FROM VERTICAL ACTIONS ON WALL DIAPHRAGM:

DESIGN UDL $w_{t,d} = \gamma_G G_k + \gamma_Q Q_k = 4.9 + {}^{-}1.65 = 3.25$ kN/m

TOTAL DESIGN VERTICAL LOAD ON WALL DIAPHRAGM $= w_{t,d} \times L = 3.25 \times 5.0 = 16.3$ kN

DESIGN STABILISING MOMENT (PD6693-1 : 2012 FIG 3)

$$M_{d,stb} = 0.5\, w_{t,d} L^2 = 0.5(3.25)(5.0)^2 = 40.6 \text{ kNm}$$

IN ACCORDANCE WITH PD6693-1 : 2012 CL 20.4.4, ADDITIONAL PASSIVE LOAD CAN BE MOBILISED FROM WINDWARD RETURN WALLS UP TO AN OUTSTANDING DISTANCE OF H. THIS HAS THE EFFECT OF INCREASING THE DESIGN STABILISING MOMENT ON THE WALL DIAPHRAGM. HOWEVER, FOR SIMPLICITY IN THIS WORKED EXAMPLE, THIS ADDITIONAL CONTRIBUTION IS IGNORED.

3. DESIGN OF WALL DIAPHRAGM

DESIGN AGAINST DIAPHRAGM OVERTURNING

DESIGN DESTABILISING MOMENT AT BOTTOM LEEWARD CORNER $M_{d,dst,base} = 13.17$ kNm

DESIGN STABILISING MOMENT AT BOTTOM LEEWARD CORNER $M_{d,stb} = 40.6$ kNm

$M_{d,dst,base} < M_{d,stb}$ THEREFORE THE OVERTURNING RESISTANCE REQUIREMENT IS SATISFIED

DESIGN AGAINST DIAPHRAGM SLIDING

COEFFICIENT OF FRICTION (PD 6693-1:2012 CL 20.4.2) $= 0.4$

SLIDING RESISTANCE = FRICTIONAL RESISTANCE + SHEAR RESISTANCE FROM FLOOR FIXINGS

$= (0.4 \times 16.3) + (5.0 \times 1.18) = 12.4$ kN

SLIDING RESISTANCE > DESIGN HORIZONTAL WIND ACTION OF 5.4 kN. THEREFORE THE SLIDING RESISTANCE REQUIREMENT IS SATISFIED

RACKING RESISTANCE CHECK

DESIGN RACKING STRENGTH (PD 6693-1:2012 EQ (5))

$$F_{i,v,Rd} = K_{opening}\, k_{i,w}\, f_{p,d,t}\, L$$

MODIFICATION FACTOR (PD 6693-1:2012 EQ(8))

$$k_{i,w} = \min\left\{1, \left[1+\left(\frac{H}{\mu L}\right)^2 + \left(2M_{d,stb,n}/\mu f_{p,d,t} L^2\right)\right]^{0.5} - \left(\frac{H}{\mu L}\right)\right\}$$

(PD 6693-1:2012 EQ(9)) and $M_{d,stb,n} = M_{d,stb} - M_{d,dst,top}$

$= 40.6 - 0.21 = 40.4$ kNm

$$k_{i,w} = \min\left\{\begin{array}{l}1.0\\ \left[1+\left(\frac{2.4}{0.19\times5.0}\right)^2 + \left(\frac{2\times40.4}{0.19\times3.47\times5.0^2}\right)\right]^{0.5} - \left(\frac{2.4}{0.19\times5.0}\right)\end{array}\right. = 0.97$$

DESIGN RACKING STRENGTH (PD 6693-1:2012 EQ (5))

$F_{i,v,Rd} = 0.57 \times 0.97 \times 3.47 \times 5.0$

$= 9.6$ kN

DESIGN RACKING STRENGTH > DESIGN HORIZONTAL WIND ACTION OF 5.4 kN, THEREFORE THE RACKING RESISTANCE REQUIREMENT IS SATISFIED

SERVICEABILITY CHECK (PD 6693-1:2012 CL 20.5.2.3)

$k_{i,w} f_{p,d,t} = 0.97 \times 3.47 = 3.37$

$8(1+k_{comb})(L/H) = 8(1+0)(5.0/2.4) = 16.67 > 3.37$

THEREFORE THE SERVICEABILITY REQUIREMENT FOR RACKING PANEL DEFLECTION IS SATISFIED

The design of the ground floor rear wall panel and the gable wall panels would follow similar principles. In the case of the gable walls, these would likely be subjected to variable floor actions rather than variable roof actions. The ground floor wall diaphragms would be checked for the sum of the total wind actions acting at first and ground floor levels and the total design destabilising moments as described in PD 6693-1:2012 Figure 3, together with any revised opening modification factors which may be applicable.

## Relevant codes of practice

**BS EN 1995-1-1 Eurocode 5:** Design of Timber Structures – Part 1-1: General – Common rules and rules for buildings

**UK National Annex to Eurocode 5: BS EN 1995-1-1:** Design of Timber Structures – Part 1-1: General – Common rules and rules for buildings

**PD 6693-1:2012:** UK Non-Contradictory Complementary Information (NCCI) to Eurocode 5: Design of timber structures

## Definitions

**Racking wall** – a timber frame wall panel sheathed with a wood-based board material or plasterboard located generally in a direction parallel to the wind load.

**Soleplate** – a timber section which is fixed to the foundation or structural subdeck to provide a locating position for the wall panel.

**Headbinder** – a timber section which connects together adjacent wall panels to enable them to function as a continuous wall diaphragm and, in combination with the top wall panel rails, act as 'spreader' beams to distribute floor joist loads to the wall studs.

## References and further reading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