Timber Engineering Notebook series

No. 5: Timber frame structures – platform frame construction (part 3)

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
Six storey timber frame test building 'TF2000', Cardington, Bedfordshire

MILNER ASSOCIATES

Introduction

This *Timber Engineering Notebook* provides guidance for engineers involved in the design of timber frame building structures on the structural stability issues commonly referred to as 'design for structural robustness' or 'design to avoid disproportionate collapse' in buildings.

Structural stability guidance is supported by tests which were conducted by BRE and TRADA on the TF2000 building (Figure 1).

Stability, robustness and disproportionate collapse

There are three different conditions that need to be satisfied by the structural engineer in the design of a multi-storey timber frame building:

- Strength and stiffness design (see *Timber Engineering Notebook No.3* for more information)
- Robustness design based on good practice but not case-specific calculations
- Disproportionate collapse design based on case-specific calculations for undefined accidental damage 'events'

Robustness of timber frame construction is generally achieved by the application of standard detailing that has become established over the last 30 years or more. UKTFA member companies and information on the UKTFA website (www.uktfa.com) provide typical detailing to achieve proven robust solutions.

Specific design checks for accidental damage should be carried out by the engineer to determine whether the building form is unduly sensitive to damage (caused accidentally or otherwise) such that collapse or partial collapse is not disproportionate to the original cause. The accidental design situation is an ultimate limit state and in general, serviceability criteria do not need to be considered.

In addition, as with all construction materials, there is no defined period for a building to survive when subject to accidental damage. The only requirement being that safe escape of occupants and safe access for emergency repair works should be possible.

Principal Code references

Actions and load combinations appropriate for the accidental load case are provided in BS EN 1991-1-7:2006 and its supporting documents.

Partial factors for actions are given in BS EN 1990 Eurocode 0 and its National Annexes. Partial factors for materials are given in BS EN 1995-1-1 and its National Annex and are reduced to 1.0 for the accidental load situation.

Building Regulations requirements for robustness

Regulations exist for accidental damage design limitation based on the principle that, in the event of damage occurring to a building, partial collapse is acceptable providing it is proportionate to the cause. In the Approved Document A to the Building Regulations

Table 1: Definition of Consequence Classes

Consequence Class	Description	Examples of buildings and civil engineering works
3	High consequence for loss of human life or economic, social or environmental consequences very great	Grandstands, public buildings where consequences of failure are high (e.g. a concert hall)
2	Medium consequence for loss of human life; economic, social or environmental consequences considerable	Residential and office buildings, public buildings where consequences of failure are medium (e.g. an office building)
1	Low consequence for loss of human life and economic, social or environmental consequences small or negligible	Agricultural buildings where people do not normally enter (e.g. storage buildings), greenhouses

Table 2: Building classes based on building type, number of storeys and occupancy

Building type	Consequence Class		
	1	2A	2B
Agricultural buildings	Typically single storey		
Single occupancy houses	1-4 storeys	5 storeys	6+ storeys
Hotels, flats and other residential buildings		1-4 storeys	5-15 storeys
Buildings to which the public are admitted		1-2 storeys and <2000m² floor area per storey	<5000m² floor area per storey
Educational		1 storey	2-15 storeys

2010, the area of floor collapse at any storey cannot exceed 15% of the floor area of that storey or 70m², whichever is smaller, and the collapse must not extend further than the immediate adjacent storey. It should be noted that the permitted collapse area differs from the 100m² given in BS EN 1991-1-7 clause 3.3(2) which is adopted in this Notebook.

In the Approved Document A, Section 5, buildings are classified into four building classes (called 'Risk Groups' in Scotland and 'Consequence Classes' in BS EN 1991-1-7) according to the perceived consequences of failure.

Figure 2
Consequence Classes applicable to single occupancy houses up to five storeys

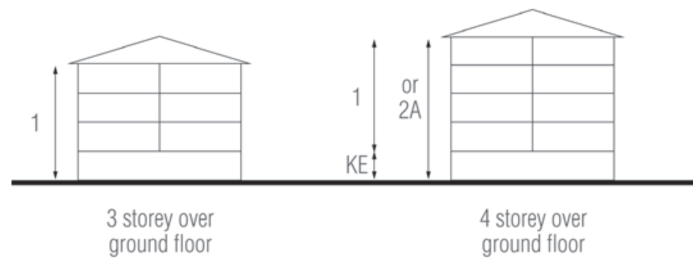


Figure 3
Consequence Classes applicable to single occupancy houses up to five storeys built over basements

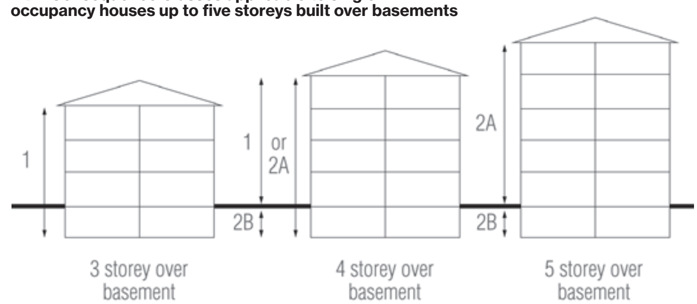
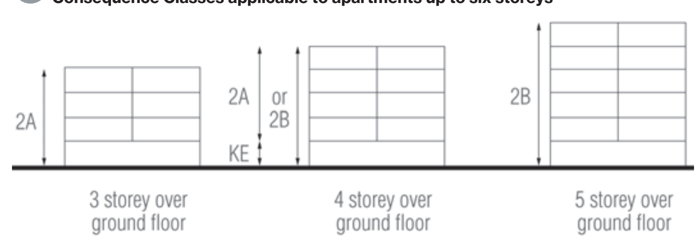


Figure 4
Consequence Classes applicable to apartments up to six storeys



Determination of the Consequence Class

The purpose of the Consequence Class is to ensure that buildings are designed and constructed to the appropriate requirements for robustness, as discussed in this Notebook.

Classes are derived on the basis of building type, building height (number of storeys), floor plan area per storey (for retail) and occupancy. A structure could be comprised of different Consequence Classes.

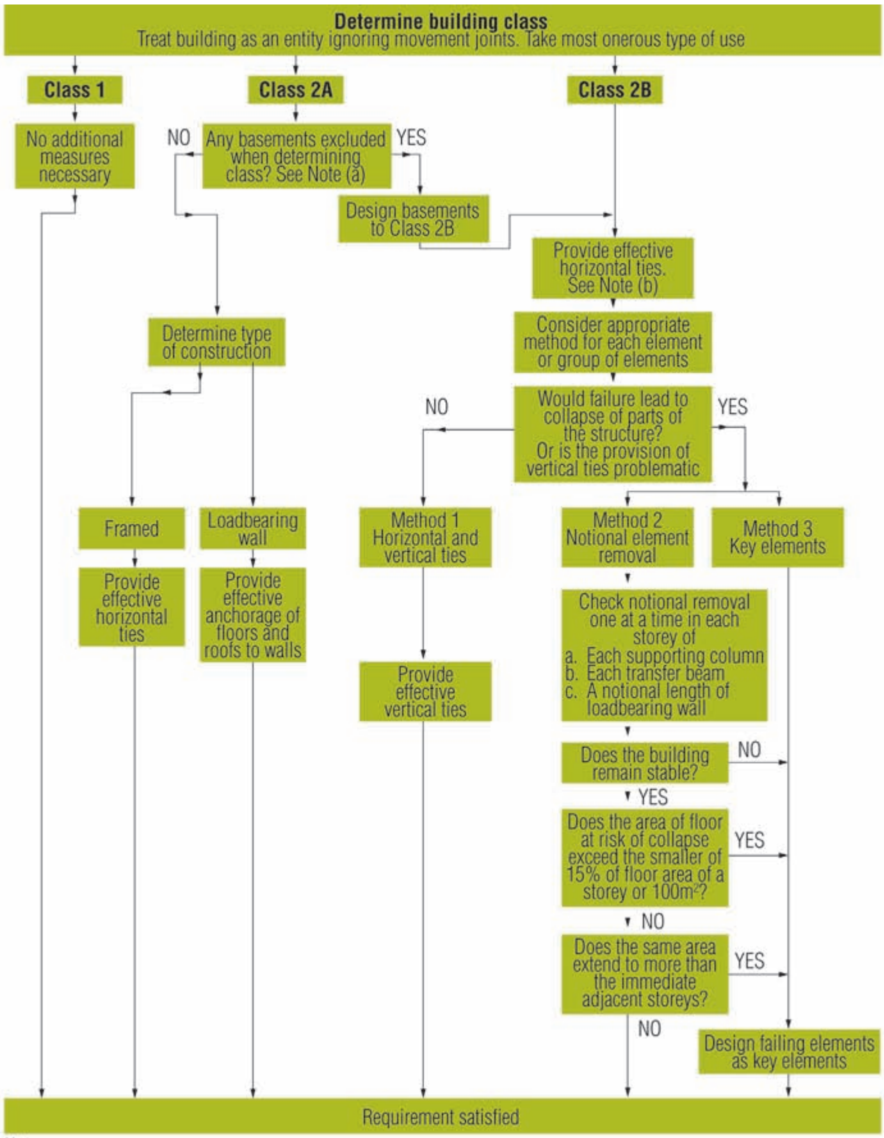
BS EN 1990 Table B1 gives guidelines for the choice of Consequence Class as indicated in Table 1.

Classification of structure

Table A.1 of BS EN 1991-1-7 categorises buildings into Consequence Classes based on their occupancy and number of storeys. Note that Consequence Class 2 in Table 1 covers a very broad spectrum of building types and so is divided into Classes 2A and 2B (Table 2). Most timber frame buildings will fall into Classes 1, 2A or 2B. Class 3 structures require a risk assessment-based approach which is not considered here.

Number of storeys

With the exception of buildings to which the public are admitted, the number of storeys is fundamental to a building's classification.



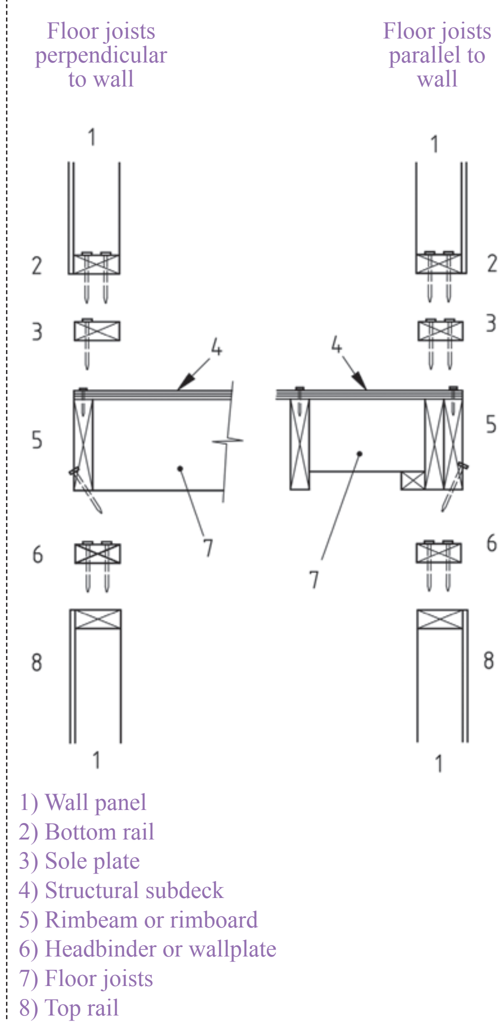
Notes

a Rules on the exclusion of basements vary in the building regulations of the various regions. When designing to EN1991-1-7[3.2] basements are included in the storey count. However in England and Wales and Scotland and Northern Ireland, if they meet Class 2B requirement, they may be excluded from the count.

b In Class 2B rather than using physical horizontal ties for platform timber frame buildings, effective anchorage of floors to walls as demonstrated by test has been proven to be acceptable

Figure 5 Disproportionate collapse – flow chart for design

Figure 6 Details of effective anchorage of floors to timber frame walls in buildings of Consequence Class 2a



Part storeys such as mezzanines and galleries or rooms-in-the-roof can generally be discounted if their floor area is less than 25% for a building with a large floor plan (≥800m²) or 50% of a building with a smaller floor plan (≤300m²).

Many four storey timber frame structures are constructed off a reinforced concrete or steel podium structure. If the podium structure is designed to meet the Class 2B requirements and to withstand the collapse of the timber frame structure above, by applying a static debris loading distributed over an area 25% larger than the original footprint of the timber frame building, then the podium structure may be deemed as a 'strong floor' and the four storey timber frame structure above can be designed to meet Class 2A requirements.

Examples of the required building classes to be applied to single occupancy dwellings and apartments are indicated in Figures 2-4 (where KE refers to a structure designed as a key element or 'strong floor' meeting the requirements of Consequence Class 2B).

Means of achieving compliance

A building's structural form will significantly affect its robustness. Cellular forms of construction such as those commonly seen in platform timber frame buildings *(Timber Engineering Notebook No. 3)* with many loadbearing walls, assure a sensible level of robustness and resistance to accidental damage, because loss of any one wall will generally not lead to the collapse of a large proportion of the structure.

In addition to the provision of horizontal ties to meet the criteria for Consequence Class 2A, there are three methods of achieving the requirements of Consequence Class 2B:

- Provision of both horizontal and vertical ties
- Notional removal of elements
- Provision of structurally adequate 'Key Elements'.

Figure 5 is a flow chart, guiding the engineer through the various robustness options to be applied to the different building classes.

Table 3: Required tying forces for internal ties for robust detailing of a timber frame building or a building with timber floors

Horizontal design tie force, $F_{t,hor,d}$ (BS EN 1991-1-7 clause A5)		Vertical design tie force, $F_{t,vert,d}$ (BS EN 1991-1-7 clause A6)	
Internal ties (kN)	**Perimeter ties (kN)**	**Distributed ties (N)**	**Concentrated ties (kN)**
$Max\ \{15\ or\ 0.8(g_k + \psi_1 q_k)\ s_t\ L$	$Max\ \{7.5\ or\ 0.4(g_k + \psi_1 q_k) s_t\ L$	$F_{t,vert,d} = (34A/8000).(h/t)^2\ or\ 100kN/m$ of wall whichever is the greater	A tensile force equal to the maximum vertical characteristic load from any one storey normally supported by the wall or column
g_k = characteristic dead weight of floor or roof per unit area (kN/m²) q_k = full characteristic imposed load on floor or roof per unit area (kN/m²) s_t = mean spacing of ties (m) L = full length of tied area of floor or roof (m) ψ_1 = frequent value of the variable action e.g. 0.5 for domestic floors		A = the cross sectional area in mm² of the loadbearing wall (excluding external skin of a cavity wall) measured on plan h = clear height of the wall in metres (Fig. 9) t = wall width in metres	

Building types of Consequence Class 2A

For Class 2A buildings, robustness will be achieved by providing effective horizontal ties, or 'effective anchorage' of suspended floors to walls. The structural layout of platform timber frame is cellular in nature and effective anchorage of each floor member to wall junction in accordance with Clause A5.2 of EN 1991-1-7 and the UK National Annex (which is satisfied by adopting the good building practice of providing lateral restraint to walls and designed mechanical anchorage details for floors to wall connections) is the preferred approach. For post and beam forms of construction, the provision of distinct horizontal ties is more likely to be needed.

For conventional timber frame buildings of cellular plan form, the effective anchorage of floors to walls will be achieved with a minimum density of nails in all horizontal interfaces equivalent of 3.1mm diameter at a maximum spacing of 300mm centres as shown in Figure 6 (taken from Annex B of PD 6693-1:2012).

The density of fixings has been derived from test evidence backed by calculations undertaken by the UKTFA and submitted to the UK code committee for approval and publication in PD 6693-1: 2012. The density of fixings is such that it gives an appropriate uniform distributed lateral tie force.

For timber structures the requirements for horizontal tie forces can also be achieved using Clause A5.1 of EN 1991-1-7 as amended by the UK National Annex to Eurocode 1-7 clauses NA 2.43 and NA.3. These design tie forces are given in Table 3.

Building types of Consequence Class 2B

For Class 2B structures, robustness can be achieved either by any of the following:

Method 1: By providing effective horizontal and vertical ties at all floor and roof levels (Table 3). For beam and post type structures or for loadbearing wall structures with an irregular or distributed arrangement of loadbearing walls, this can be achieved by designing connections between members for the required accidental tie forces. For platform timber frame structures with a regular arrangement of loadbearing walls, the horizontal tie requirement is satisfied by adopting the effective anchorage requirements for Consequence Class 2A together with one of the following methods:
Method 2: By checking for notional removal of a loadbearing wall, one at a time in each storey.
Method 3: By providing critical 'key' or 'protected' elements, which must be able to resist an accidental pressure of 34kN/m² in any direction (accidental loadcase).

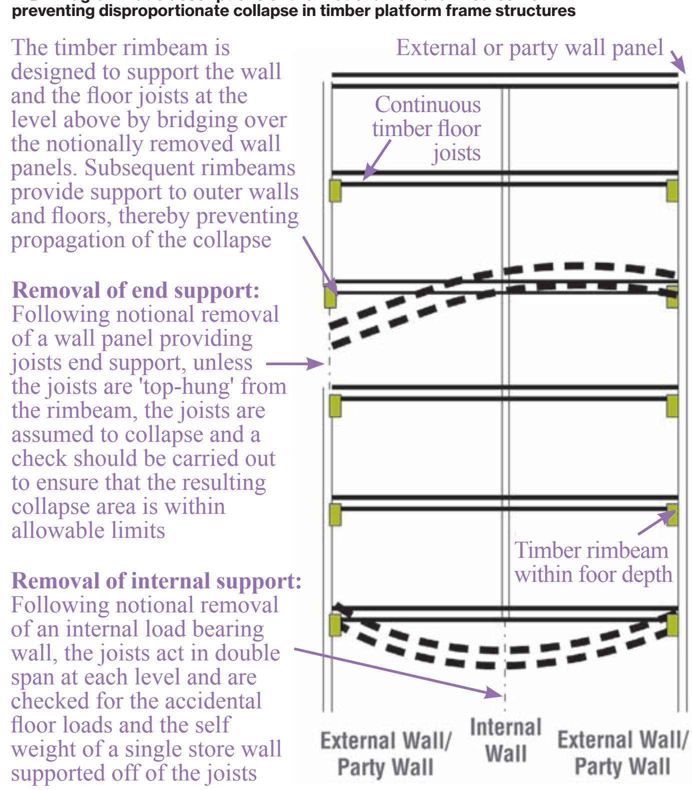
Where necessary, these methods may be used in combination.

Platform timber frame building structures which comprise loadbearing wall panels typically have a distributed arrangement of vertical walls. Notional removal of loadbearing elements (or notional panel removal), one at a time in each storey of the building in accordance with EN 1991-1-7 Clause A7, is therefore the preferred approach for platform frame construction.

In all cases the design process should involve checking the capacity of the component interfaces (e.g. panel rail to soleplate, soleplate to floor deck, floor joists to head binder and head binder to panel rail) against the variable horizontal wind forces. The timber frame designer should therefore be providing a robust connection at each and every junction as part of the normal design process.

In calculating the strength of connections to resist the tying forces, k_{mod} factors should be taken as the instantaneous values from Table 3.1 of EN 5 i.e. k_{mod} = 1.1 and γ_m = 1.0 (accidental condition).

Figure 7
Diagrammatic descriptions of the 'Notional removal method' for preventing disproportionate collapse in timber platform frame structures



Notional panel removal – structural notes

Removal of end support:

Following the notional removal of a wall panel providing joist end support, the following design checks should be carried out:

1. Check of collapse area: the joists immediately above the removed wall panel are assumed to collapse (unless the joists are 'top-hung' from the rim beam) and a check should be carried out to ensure that the resulting collapse area is within 15% of the total floor area or 100m² as defined by BS EN 1991-1-7 clause 3.3(2).

2. Rim beam design: The timber rim beam is designed to support a single storey of wall (plus any supported cladding) and the floor joists at the level above, by bridging over the notionally removed wall panels. Subsequent rim beams provide support to other walls and floors, thereby preventing propagation of the collapse.

Key item:
The nominal length of load bearing wall construction which needs to be considered for 'notional removal' (the notional panel) and therefore the rim beam design span is as follows:

- For an external wall, the clear length measured between lateral supports provided by intersecting return walls or members designed as key element posts.
- For an internal wall, a length not exceeding 2.25H
  Where: H is the clear height of the panel between lateral supports (the top of the structural deck level below to the underside of the structural joist level above).

Removal of internal support:

Following the notional removal of an intermediate load bearing wall, continuous floor joists (A-B) act as a double span at each level and the following design checks should be carried out:

Joist design:

Joists are checked for the design residual floor loads and the self weight of a single storey of wall (C-D) supported off of the joists.
When considering the design of the residual structure following notional panel removal (joists, rim beams and key element posts) the following design parameters should be considered:

BS EN 1990 design residual loadcases are:

For floors:

- 1.0 permanent + 0.5 imposed or
- 1.0 permanent + 0.2 wind + 0.3 imposed

For roofs:

- 1.0 permanent + 0.2 imposed or
- 1.0 permanent + 0.2 wind

Duration of load factors:

Kmod factors for the residual structure after the accidental event should be taken as the short term values from Table 3.1 of EN 5 (e.g 0.90 for LVL in Service Class 1) as described in Table NA.1 of the UK National Annex to BS 1995-1-1.

Figure 8
Notional panel removal – end span support wall

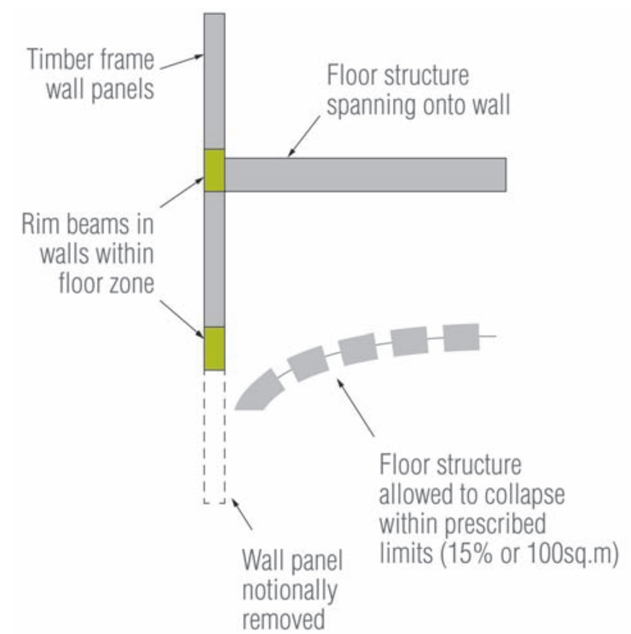


Figure 9
Height of wall or column between horizontal restraints (taken from Fig. A3 of EN 1991-1-7:2006)

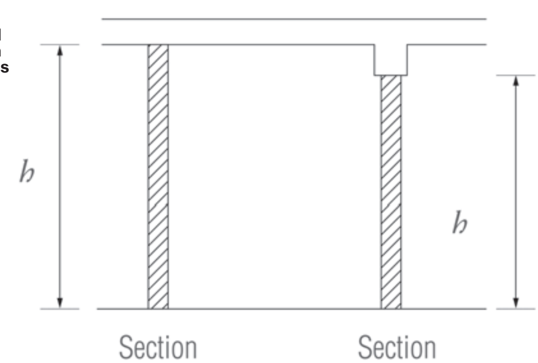
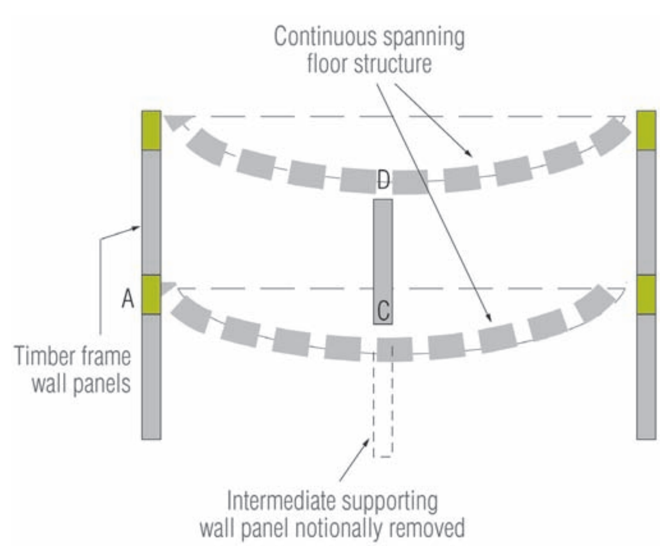


Figure 10
Notional panel removal – intermediate support wall

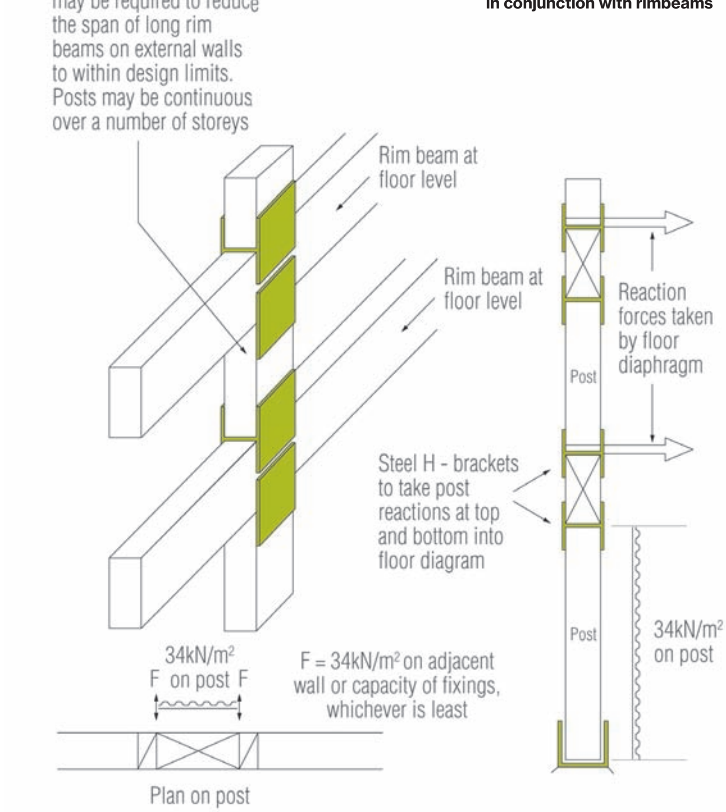


Notional removal of loadbearing element for Consequence Class 2B - The 'Rimbeam Method'
A separate engineered timber 'rim beam' is used to span between points of vertical lateral restraint (intersecting return walls or key element posts) and ensures that structural continuity is achieved in the event of notional removal of a loadbearing wall panel, by providing vertical load transfer as a bridging element over this panel (Figure 7).

Using rim beams also allows joisted floor structures to be factory assembled as cassettes, with rimboards used to connect the joist ends together for transportation. The rim beam can later function as a vertical load transfer element in the completed structure.

The rim beams need to react onto wall intersections and the wall returns must be of 1200mm minimum length excluding any framed openings. Such walls can be non-loadbearing in the conventional sense, but must still be capable of transferring loads down through the structure (see structural notes on notional panel removal and Figures 8-10 which accompany them).

Figure 11 **Key element posts acting in conjunction with rimbeams**



Key element post method for Consequence Class 2B
'Key element posts' are often provided in combination with rim beams, to either reduce the length of wall panel that must be considered for notional panel removal, or to support transfer beams within the timber frame structure.

Key element posts can be an engineered timber (e.g. LVL) post within a timber frame wall panel with steel brackets connecting the key element post to the rimbeams at each floor level, to transfer the accidental loads back into the floor diaphragm.

Key element posts are designed for an accidental load of 34kN/m² (see BS EN 1991-1-7 clause 3.3) applied over the width of the post in one direction at a time. The additional reaction of adjacent wall panels, which are connected to the key element posts with nails, is likely to be small but should be considered (Figure 11).

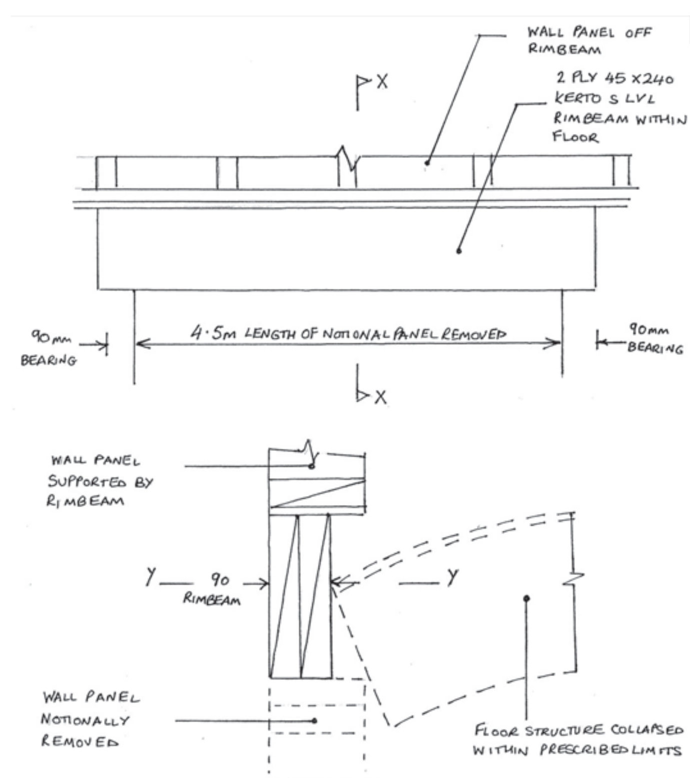
Principles of the rim beam design methodology are indicated in Figs 7-10 and the worked example.

Worked example

Engineered timber rim beam design example to BS EN 1995-1-1 Eurocode 5 and BS EN 1991-1-7 Eurocode 1:

The rim beam within the floor zone shown, is located at the end of a 5.0m span floor cassette. The floor structure has a self weight of 1.00kN/m² and a design imposed floor load of 1.50kN/m². The rim beam is located within an external wall panel which has a storey height of 2.75m and a self weight (including supported cladding) of 0.50 kN/m².

Following the notional removal of a 4.5m length of the external wall panel between intersecting return walls, check that the rim beam will meet the ULS requirements of EC0 and EC5 for the residual structure following an accidental event. The rim beam is assumed to be prevented from lateral buckling by the connections of the wall panels and soleplates above the beam.



1. BEAM GEOMETRIC PROPERTIES

BREADTH OF THE RIMBEAM $b = 2 \times 45$ — $b = 90\,mm$

DEPTH OF THE RIMBEAM — $h = 240\,mm$

CLEAR SPAN OF RIMBEAM — $L_c = 4500\,mm$

BEARING LENGTH OF THE RIMBEAM AT EACH END ONTO C16 TIMBER — $l_b = 90\,mm$

DESIGN SPAN OF THE BEAM $(=(l_c + l_b))$ — $l = 4590\,mm$

SECTION MODULUS OF THE BEAM ABOUT THE Y-Y AXIS $W_y = \frac{bh^2}{6}$ — $W_y = 8.64 \times 10^5\,mm^3$

2. TIMBER PROPERTIES

(FROM KERTO S LVL VTT CERTIFICATE N°184/03 MARCH 2009)

CHARACTERISTIC BENDING STRENGTH — $f_{m,k} = 44\,N/mm^2$

CHARACTERISTIC SHEAR STRENGTH — $f_{v,k} = 4.1\,N/mm^2$

CHARACTERISTIC BEARING STRENGTH — $f_{c,90,k} = 6.0\,N/mm^2$

5th PERCENTILE MOE PARALLEL TO GRAIN — $E_{0.05} = 13800\,N/mm^2$

MEAN SHEAR MODULUS — $G_{0,mean} = 400\,N/mm^2$

MEAN DENSITY — $\rho_m = 510\,kg/m^3$

(BS EN 338 : 2009 TABLE 1 FOR C16 STRENGTH CLASS)

CHARACTERISTIC BEARING STRENGTH — $f_{c,90,k,C16} = 2.2\,N/mm^2$

3. PARTIAL SAFETY FACTORS

(UK NATIONAL ANNEX TO BS EN 1990:2002 TABLE NA.A1.3 FOR ACCIDENTAL ACTIONS)

PERMANENT ACTIONS — $\gamma_{G,ULS} = 1.0$

(UK NATIONAL ANNEX TO BS EN 1990:2002 TABLE NA.A1.1 FOR CATEGORY A VARIABLE ACTIONS)

FACTOR FOR THE FREQUENT VALUE OF THE LEAD VARIABLE ACTION ψ_1 — $\psi_1 = 0.5$

(UK NATIONAL ANNEX TO BS EN 1995-1-1 TABLE NA.3)

MATERIAL PARTIAL SAFETY FACTOR FOR LVL AND C16 — $\gamma_M = 1.0$

4. ACTIONS

CHARACTERISTIC SELF WEIGHT OF THE BEAM

$$G_{k,swt} = b \cdot h \cdot g \cdot \rho_m$$

$$= 90 \times 240 \times 510 \times 9.81 \times 10^{-9} \qquad G_{k,swt} = 0.11\,kN/m$$

CHARACTERISTIC PERMANENT ACTION DUE TO SUPPORTED WALL SELF WEIGHT

$$G_{k,wall} = 2.75 \times 0.50\,kN/m^2 \qquad G_{k,wall} = 1.38\,kN/m$$

CHARACTERISTIC PERMANENT ACTION DUE TO SUPPORTED FLOOR SELF WEIGHT

$$G_{k,floor} = 1/2 \times 5.00 \times 1.00\,kN/m^2 \qquad G_{k,floor} = 2.50\,kN/m$$

TOTAL CHARACTERISTIC PERMANENT ACTION — $G_k = 4.00\,kN/m$

DESIGN ACTION DUE TO PERMANENT ACTION

$$F_{d,p} = 1.0 \times 4.00 \qquad F_{d,p} = 4.00\,kN/m$$

CHARACTERISTIC VARIABLE ACTION DUE TO SUPPORTED FLOOR

$$Q_{k,floor} = 1/2 \times 5.00 \times 1.50\,kN/m^2 \qquad Q_{k,floor} = 3.75\,kN/m$$

DESIGN ACTION DUE TO VARIABLE ACTION

$$F_{d,q} = \psi_1 \times Q_{k,floor} = 0.5 \times 3.75 \qquad F_{d,q} = 1.88\,kN/m$$

5. MODIFICATION FACTORS

(THE DESIGNS CONSIDERED ARE POST-ACCIDENTAL EVENT (i.e. FOR THE RESIDUAL STRUCTURE AND THE APPROPRIATE k_{mod} VALUES ARE THEREFORE THOSE APPLICABLE TO SHORT TERM DURATION LOADS AS PROVIDED BY TABLE NA.1 OF UK NA TO BS EN 1995-1-1)

FACTOR FOR SHORT TERM DURATION LOADING AND SC1 (BS EN 1995-1-1 TABLE 3.1) — $k_{mod,sht} = 0.90$

SIZE FACTOR FOR DEPTH LESS THAN 300mm (BS EN 1995-1-1 EX 3.3 AND KERTO VTT) — $k_h = \min\begin{cases}\left(\frac{300}{240}\right)^{0.12} = 1.03 \\ 1.2\end{cases}$

LATERAL STABILITY OF THE BEAM (BS EN 1995-1-1 EX 6.3.3) — $k_{crit} = 1.0$

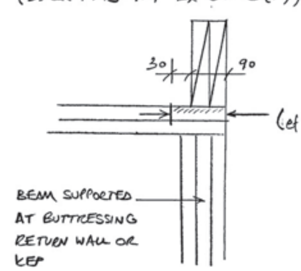
BEARING FACTOR $k_{c,90}$ FOR LVL (KERTO VTT CERTIFICATE) — $k_{c,90,LVL} = 1.0$

BEARING FACTOR $k_{c,90}$ FOR C16 (BS EN 1995-1-1 EX 6.1.5(3)) — $k_{c,90,C16} = 1.25$

LOAD SHARING FACTOR CONSIDERING A 2 PLY LVL RIMBEAM (BS EN 1995-1-1 EX 6.6(2)) — $k_{sys} = 1.1$

6. BEAM BEARING DETAIL AT SUPPORT

(BS EN 1995-1-1 EX 6.1.5(1))



EFFECTIVE CONTACT AREA IN COMPRESSION PERPENDICULAR TO GRAIN

$$A_{ef} = l_b \times l_{ef} = 90 \times (90 + 30) \qquad A_{ef} = 10800\,mm^2$$

7. BENDING STRENGTH

DESIGN BENDING MOMENT FOR THE RESIDUAL LOADCASE

$$M_d = \frac{(F_{d,p} + F_{d,q}) \cdot l^2}{8} = (4.00 + 1.88) \cdot \frac{4.590^2}{8} = 15.5\,kNm$$

DESIGN BENDING STRESS

$$\sigma_{m,y,d} = \frac{M_d}{W_y} = \frac{15.5 \times 10^6}{8.64 \times 10^5} = 17.9\,N/mm^2$$

DESIGN BENDING STRENGTH

$$f_{m,y,d} = \frac{k_{mod,sht} \cdot k_{sys} \cdot k_h \cdot f_{m,k}}{\gamma_M} = \frac{0.90 \times 1.1 \times 1.03 \times 44}{1.0} = 44.9\,N/mm^2$$

$f_{m,y,d} > \sigma_{m,y,d}$ BENDING STRENGTH IS SATISFACTORY

8. SHEAR STRENGTH

DESIGN SHEAR FORCE FOR THE RESIDUAL LOADCASE

$$V_d = (F_{d,p} + F_{d,q}) \cdot \frac{l}{2} = (4.00 + 1.88) \cdot \frac{4.590}{2} = 13.5\,kN$$

DESIGN SHEAR STRESS (BS EN 1995-1-1 EX 6.60)

$$\tau_{v,d} = \frac{3}{2} \cdot \frac{V_d}{b \cdot h} = \frac{3}{2} \cdot \frac{13.5 \times 10^3}{90 \times 240} = 0.94\,N/mm^2$$

DESIGN SHEAR STRENGTH

$$f_{v,d} = \frac{k_{mod,sht} \cdot k_{sys} \cdot f_{v,k}}{\gamma_M} = \frac{0.90 \times 1.1 \times 4.1}{1.0} = 4.1\,N/mm^2$$

$f_{v,d} > \tau_{v,d}$ SHEAR STRENGTH IS SATISFACTORY

9. BEARING STRENGTH

DESIGN BEARING FORCE AT SUPPORTS = DESIGN SHEAR FORCE V_d

FOR THE LVL RIMBEAM :

DESIGN BEARING STRESS $\sigma_{c,90,d} = \frac{V_d}{b \cdot L_b} = \frac{13.5 \times 10^3}{90 \times 90} = 1.67\ N/mm^2$

DESIGN BEARING STRENGTH $f_{c,0,d(LVL)} = \frac{k_{mod,sht} \cdot k_{sys} \cdot k_{c,90} \cdot f_{c,90,k,LVL}}{\gamma_M}$

$= \frac{0.90 \times 1.1 \times 1.0 \times 6.0}{1.0}$

$= 5.94\ N/mm^2$

$f_{c,0,d(LVL)} > \sigma_{c,90,d}$ BEARING OF LVL IS SATISFACTORY

FOR THE C16 PLATES :

DESIGN BEARING STRESS $\sigma_{c,90,d} = \frac{V_d}{A_{ef}} = \frac{13.5 \times 10^3}{10800} = 1.25\ N/mm^2$

DESIGN BEARING STRENGTH $f_{c,0,d(C16)} = \frac{k_{mod,sht} \cdot k_{c,90} \cdot f_{c,90,k,C16}}{\gamma_m}$

$= \frac{0.90 \times 1.25 \times 2.2}{1.0}$

$= 2.48\ N/mm^2$

$f_{c,0,d(C16)} > \sigma_{c,90,d}$ BEARING ON PLATES IS SATISFACTORY

THEREFORE THE 2PLY 45 x 240 LVL RIM BEAM SATISFIES THE ULS REQUIREMENTS OF EC0 AND EC5 FOR THE RESIDUAL STRUCTURE FOLLOWING AN ACCIDENTAL EVENT.

Other methods of designing against disproportionate collapse

There are other methods of achieving support for walls and floors following the notional removal of wall panels. These include:

- The use of room-size floor cassettes with cassette edge boards acting as rim beams, bridging over removed wall panels. This is possible where small repeatable room sizes are present (e.g. hotel type accommodation)
- Loose floor construction with top-hung joists or joists supported in joist hangers from loose rim beams which bridge over removed wall panels
- The use of wall panels designed as deep beams in lieu of rim beams - applicable where there are no openings in wall panels such as hotel bedroom dividing walls
- Where joist span lengths are repetitive, using alternate double-spanning joists with selected walls designed as deep beams (a check should be made that overall building instability does not occur due to a lack of restraint at the back span of any cantilevered joists)
- Cantilevered joists supporting walls where the floor joists are designed to support the point load reaction from a single storey of wall panel plus any supported claddings following notional panel removal

Relevant codes of practice

The Building Regulations 2010, Approved Document A - Structure (2004 ed. incorporating 2010 amendments)

BS EN 1990:2002 Eurocode 0: Basis of structural design

BS EN 1991-1-7:2006 Eurocode 1: Actions on structures: General actions – Accidental actions

NA to BS EN 1991-1-7:2006 Eurocode 1: Actions on structures: General actions – Accidental actions

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

BS EN 1995-1-1 UK National Annex to Eurocode 5: 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

NHBC Technical Guidance Note 2004 – The Building Regulations (2004 ed. Requirement A3 – Disproportionate collapse)

Definitions

Accidental event – an event due to a specified or un-specified cause which results in accidental damage being suffered to the structure of a building.

Key element posts – reinforced load bearing elements of structure which are designed for combinations of permanent and variable actions and for accidental actions applied in any direction, to satisfy robustness checks during disproportionate collapse design.

References and further reading

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