## LOADINGS

<table>
<thead>
<tr>
<th><strong>Pitched Roof:</strong></th>
<th><strong>Flat Roof:</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td>Tiles = 0.70</td>
<td>Chippings = 0.20</td>
</tr>
<tr>
<td>Felt &amp; battens = 0.05</td>
<td>3 layer felt = 0.10</td>
</tr>
<tr>
<td>Roof timber = 0.15</td>
<td>Boards = 0.15</td>
</tr>
<tr>
<td>Insulation &amp; ceiling = 0.20</td>
<td>Joists = 0.15</td>
</tr>
<tr>
<td></td>
<td>Insulation + ceiling = 0.20</td>
</tr>
<tr>
<td></td>
<td>+ 1.10</td>
</tr>
<tr>
<td>L.L. Roof = 0.75</td>
<td>L.L. Roof = 0.75</td>
</tr>
<tr>
<td>L.L. Loft = 0.25</td>
<td>+ 0.80</td>
</tr>
<tr>
<td></td>
<td>+ 2.10 kN/m²</td>
</tr>
</tbody>
</table>

### Upper Floors – Timber:

| Boards = 0.15 | 65 screed = 1.50 |
| Joists = 0.15 | 150 Beam and Block = 1.75 |
| Ceiling = 0.20 | + 3.25 |
|                | 0.50 |
| L.L. Floor = 1.50 |

**L.L Floor (Domestic)**

| + 1.50 |
| + 2.00 kN/m² |

### External walls:

| Brickwork = 2.20 |
| Blockwork = 1.00 |
| Plaster = 0.20 |
|                | + 3.40 kN/m² |
|                | + 0.40 kN/m² |

### Block partitions:

| 100 blocks = 1.00 | 225 dense blocks = 4.60 |
| Plaster (both side) = 0.40 | Plaster (both sides) = 0.40 |
|                | + 1.40 kN/m² |
|                | + 5.00 kN/m² |

### Upper Floors – PC Conc:

| Screed = 1.80 | Tiles = 0.70 |
| PC Planks = 3.0 | Battens = 0.05 |
| Ceiling = 0.2 | 12mm ply = 0.10 |
|                | + 5.00 |
| 50 x 150 Studs = 0.10 |
| Plaster Bd + insul. = 0.20 |

**L.L Floor (Domestic)**

| 1.50 |
| + 1.30 |
| + 7.80 kN/m² |

**Partitions**

### Ground Floor:

| + 1.30 |
| + 6.05 kN/m² |

**Stud partitions**

### Tile Hung Stud:

| 0.70 |

### Steel design to:

| B.S. 449 Part 2 | 225 th brick walls = 5.0 kN/m² |
| Concrete design to:

| B.S. 8110 | 330 th brick walls = 7.5 kN/m² |
| Masonry design to:

| B.S. 5628 | 450 th brick walls = 10.0 kN/m² |
| Timber design to:

| B.S. 5268 |

### Steel design to B.S. 449 Part 2

- 225 th brick walls = 5.0 kN/m²
- 330 th brick walls = 7.5 kN/m²
- 450 th brick walls = 10.0 kN/m²

### Concrete design to B.S. 8110

- 330 th brick walls = 7.5 kN/m²
- 450 th brick walls = 10.0 kN/m²

### Masonry design to B.S. 5628

- 450 th brick walls = 10.0 kN/m²

### Timber design to B.S. 5268

- 450 th brick walls = 10.0 kN/m²

### Design spans stated are clear opening sizes between end supports rounded up to nearest 100mm taking end restraints in to account. Exact size of opening to be measured by building contractor on site and a minimum 150mm bearing length to be added to each end of beam/ lintel before ordering materials unless specified otherwise. Calculations are to be read in conjunction with all other pertinent works documentation including but not limited to all architectural drawings. When referring to R1 and R2, R1 is left or bottom end when looking at beam layout on plan, R2 is right or top end.

* Live Load Floor (Office Space/Commercial) = 2.5 KN/m

Unless otherwise stated foundations are designed for a maximum ground bearing capacity of 100 kN/m², which is to be verified on site.
CAVEAT

4 Point Plans cannot be held accountable for any mistakes, failures, or damages caused by erroneous data supplied by any third party where 4 Point Plans has not been appointed to conduct an initial survey or investigation.

If the architectural or structural design is altered and/or loads are changed and further geotechnical information brought to light 4 Point Plans is to be notified of these changes such that an assessment of the impact of these changes can be made.

It is held to be the duty of the project manager or whomsoever has been appointed as project overseer to check the data used in this assessment and design for correlation with current project information and practice proper record retention and dissemination to all relevant parties.

DESIGN VARIABLES AND ASSUMPTIONS

All assumptions are to be confirmed by approved building inspector and if not correct then report back to structural engineer for reassessment. The site contractor and associate basement sub-contractor are to remain vigilant during any excavations and to report back to engineer any evidence that suggests otherwise to the assumptions contained within this document.

Existing foundations are assumed sufficient to carry the existing building unless a report from an previously appointed engineer has identified weak foundations. All foundations intended to take additional loads, including notional horizontal loadings, are to be checked by an approved building inspector for adequacy.

All brickwork intended to take additional structural loads specially about the point of loading from steels and padstones is to be exposed and checked by approved inspector for adequacy to take additional loads. Any large cracks or wall discontinuities and/or evidence of any previous movement not apparent but exposed during construction is to be reported to engineer for further evaluation.

Unless otherwise stated foundations are designed for a maximum ground bearing capacity of 100 kN/m², which is to be verified on site. All mass concrete footings are to be taken to a minimum depth of 1.0m below external ground level unless otherwise stated or as required by the approved building inspector based on local site conditions including the presence of any medium or high water demand trees in the vicinity, whereby depths shall be in accordance with NHBC guidelines as to building close to trees (or as stated in this document).

DESIGN NOTES

Design spans stated are clear opening sizes between end supports rounded up to nearest 100mm taking end restraints in to account.

Exact size of opening to be measured by building contractor on site and a minimum 150mm bearing length to be added to each end of beam/lintel before ordering materials unless specified otherwise.

Calculations are to be read in conjunction with all other pertinent works documentation including but not limited to all architectural drawings.

When referring to R1 and R2, R1 is left or bottom end when looking at beam layout on plan, R2 is right or top end.

All steelwork, components, and details to be manufactured and/or fabricated to CE Execution class EXC2.1a except where explicitly stated in individual notes.
DESIGN OF RB1 – Ridge Beam - (Design Span 1.8m)

Load from New Dormer Roof Vaulted

\[
DL = \frac{1}{2} \times 1.1 \times 1.8 \times \frac{1}{\cos(42)} = 0.99 \text{ kN/m}
\]

\[
LL = \frac{1}{2} \times 0.75 \times 1.8 = 0.68 \text{ kN/m}
\]

Use min 47x147 C24 timber.

At each end sit on 2No 47x100 C24 studs well nailed together within new partition/dormer wall.

DESIGN OF RB2 – Roof Beam - (Design Span 1.8m)

Load from Existing Roof

\[
DL = \frac{1}{2} \times 1.1 \times 3.4 \times \frac{1}{\cos(42)} = 1.88 \text{ kN/m}
\]

\[
LL = \frac{1}{2} \times 1 \times 3.4 = 1.70 \text{ kN/m}
\]

Point Load from RB1

Refer to Calculation Sheet for Reaction Load

Use 2No 47x147 C24 timbers well nailed together. Encase to half hour fire resistance.

DESIGN OF 1L1 – Lintel - (Design Span 1.2m)

On inspection use 3No 47x147 C24 well nailed together.

DESIGN OF 1L2 – Lintel - (Design Span 1.2m)

On inspection use 3No 47x147 C24 well nailed together.

DESIGN OF 1L3 – Lintel - (Design Span 1.2m)

Load from Wall Over – Brick and brick

\[
DL-\text{Inner} = 2.4 \times 0.3 = 0.72 \text{ kN/m}
\]

\[
DL-\text{Outer} = 2.2 \times 0.3 = 0.66 \text{ kN/m}
\]

Load from New roof- Traditional Cut

\[
DL = \frac{1}{2} \times 1.1 \times 4.2 \times \frac{1}{\cos(42)} = 3.11 \text{ kN/m}
\]

\[
LL = \frac{1}{2} \times 1 \times 4.2 = 2.10 \text{ kN/m}
\]

On inspection use 2No 47x147 C24 well nailed together.

DESIGN OF 1L4– Lintel - (Design Span 1.2m)

On inspection use 2No 47x147 C24 well nailed together.
DESIGN OF GB1 – Roof Beam - (Design Span 1.6m)

Load from Porch roof- Traditional Cut

\[
\begin{align*}
DL &= \frac{1}{2} \times 1.1 \times 1.3 \div \cos(12) = 0.73 \text{ kN/m} \\
LL &= \frac{1}{2} \times 0.75 \times 1.3 = 0.49 \text{ kN/m}
\end{align*}
\]

Load from cat-slide roof- Traditional Cut

\[
\begin{align*}
DL &= \frac{1}{2} \times 1.1 \times 2.7 \div \cos(42) = 2.00 \text{ kN/m} \\
LL &= \frac{1}{2} \times 0.75 \times 2.7 = 1.01 \text{ kN/m}
\end{align*}
\]

Use 2No 47x147 C24 timbers well nailed together.

At end R2 (right on plan) hang from GB2 using suitable Joist Hanger. Encase to half hour fire resistance.

DESIGN OF GB2 – Wall Support Beam - (Design Span 4.2m)

Load from New roof- Traditional Cut

\[
\begin{align*}
DL &= \frac{1}{2} \times 1.1 \times 4.2 \div \cos(42) = 3.11 \text{ kN/m} \\
LL &= \frac{1}{2} \times 1 \times 4.2 = 2.10 \text{ kN/m}
\end{align*}
\]

Load from Wall Over – Tile Hung Stud, 1 to L

\[
\begin{align*}
DL &= 2.2 \times 1.15 = 2.53 \text{ kN/m}
\end{align*}
\]

Load from Wall Over – Brick and Block, 0 to 1

\[
\begin{align*}
DL-\text{Inner} &= 2.4 \times 2.2 = 5.28 \text{ kN/m} \\
DL-\text{Outer} &= 2.4 \times 2.2 = 5.28 \text{ kN/m}
\end{align*}
\]

Load from New Floor

\[
\begin{align*}
DL &= \frac{1}{2} \times 3.8 \times 0.5 = 0.95 \text{ kN/m} \\
LL &= \frac{1}{2} \times 3.8 \times 1.5 = 2.85 \text{ kN/m}
\end{align*}
\]

Load from Partitions Over

\[
\begin{align*}
DL &= 0.4 \times 2.4 = 0.96 \text{ kN/m}
\end{align*}
\]

Point Load from GB1 @ 2.7

Refer to Calculation Sheet for Reaction Load

Use 203x133x25 UB.

At end R1 (centre of house) rebuild existing wall end in engineering brick or infil cavity with engineering brick/concrete to create solid 275Wd x 300Lg pier, maintain existing DPMs and DPCs, and sit steel no 275Wd x 300Lg x 150 DP MC padstone. At end R2 sit on 300 Lg x 150 DP x 100Wd MC padstone within new 7N dense blockwork between new window and door (to foundation level). All brickwork and foundations below existing walls intended to take additional loads are to be exposed and checked for adequacy to carry additional loads.
Position steel under centre-line of existing cavity wall and under run of existing wall use 280Wd x 8mm MS top plate to support both leaves of masonry. Dry pack to underside of existing brickwork and Insert cavity tray above steels and any roof abutment flashing draining to weepholes at 450 crs horizontal.

Fix blocking tight to web and hold in place with M12 at 600 crs. Fix all new and existing timbers to blocking with joist hangers.

**DESIGN OF GB3 – Wall Support Beam - (Design Span 1.7m)**

**Load from Existing 1st Floor**

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</thead>
<tbody>
<tr>
<td>DL</td>
<td>1/2</td>
<td>x 3</td>
<td>x 0.5</td>
<td></td>
<td>0.75 kN/m</td>
</tr>
<tr>
<td>LL</td>
<td>1/2</td>
<td>x 3</td>
<td>x 1.5</td>
<td></td>
<td>2.25 kN/m</td>
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</tbody>
</table>

**Partition Walls Over**

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<tr>
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</thead>
<tbody>
<tr>
<td>DL</td>
<td>0.4</td>
<td>x 2.4</td>
<td></td>
<td>0.96 kN/m</td>
</tr>
</tbody>
</table>

Use min 2No 47x147 C24 timbers well nailed together. Encase to half hour fire resistance.

**DESIGN OF GB4 – Dormer Wall Support Beam - (Design Span 1.7m)**

**Load from Dormer Wall Over**

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<tr>
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</thead>
<tbody>
<tr>
<td>DL</td>
<td>1.15</td>
<td>x 2.7</td>
<td>3.11 kN/m</td>
</tr>
</tbody>
</table>

Distributed Load from RB1
Refer to Calculation Sheet for Reaction Load

Use min 2No 47x147 C24 timbers well nailed together. Encase to half hour fire resistance.

**DESIGN OF GL1 – Lintel - (Design Span 3.2m)**

**Load from Wall Over – Brick and Block**

<p>| | | | |</p>
<table>
<thead>
<tr>
<th></th>
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</thead>
<tbody>
<tr>
<td>DL-Inner</td>
<td>2.4</td>
<td>x 0.3</td>
<td>0.72 kN/m</td>
</tr>
<tr>
<td>DL-Outer</td>
<td>1.2</td>
<td>x 0.3</td>
<td>0.36 kN/m</td>
</tr>
</tbody>
</table>

**Load from Wall Over – Tile Hung Stud**

Average height of gable wall

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<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>DL</td>
<td>3.1</td>
<td>x 1.15</td>
<td>3.57 kN/m</td>
</tr>
</tbody>
</table>

**Load from Hanging Doors**

<p>| | | | |</p>
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<thead>
<tr>
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<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>DL</td>
<td>0.5</td>
<td>x 2.4</td>
<td>1.20 kN/m</td>
</tr>
</tbody>
</table>

Total working load across Lintel = (3.2+0.3 bearing) x (0.72+0.36+3.57+1.2) = 20.48 kN

For 100mm Cavity, use CATNIC CX90/100 or IG L5/100 Extra Heavy Duty insulated lintel or similar. Ensure wall over is partially supported on inner blockwork to spread load across lintel.

**DESIGN OF GL2 – Lintel - (Design Span 1.2m)**
Load from Wall Over – Brick and Block

- DL-Inner = 2.4 x 0.3 = 0.72 kN/m
- DL-Outer = 1.2 x 0.3 = 0.36 kN/m

Load from Wall Over – Tile Hung Stud – (in 45 degree load triangle)

- DL = 0.3 x 1.15 = 0.35 kN/m

For 100mm Cavity, use CATNIC CG90/100 or IG L1/S 100 Standard Duty insulated lintel or similar

DESIGN OF GL3 – Lintel - (Design Span 1.2m)

Load from New Floor

- DL = 1/2 x 3.8 x 0.5 = 0.95 kN/m
- LL = 1/2 x 3.8 x 1.5 = 2.85 kN/m

Load from Wall Over – Brick and Block

- DL-Inner = 2.4 x 0.3 = 0.72 kN/m
- DL-Outer = 1.2 x 0.3 = 0.36 kN/m

Load from Wall Over – Tile Hung Stud – (in 45 degree load triangle)

- DL = 0.3 x 1.15 = 0.35 kN/m

Total working load across Lintel = (1.2+0.3 bearing) x (0.95+ 2.85 + 0.72+0.36+0.35) = 7.85 kN

For 100mm Cavity, use CATNIC CG90/100 or IG L1/S 100 Standard Duty insulated lintel or similar

DESIGN OF GL4 – Lintel - (Design Span 1.2m)

Load from Wall Over – Brick and Block

- DL-Inner = 2.4 x 0.3 = 0.72 kN/m
- DL-Outer = 1.2 x 0.3 = 0.36 kN/m

Load from Wall Over – Tile Hung Stud – (in 45 degree load triangle)

- DL = 0.3 x 1.15 = 0.35 kN/m

Total working load across Lintel = (1.2+0.3 bearing) x (0.73+ 0.49 + 0.72+0.36) = 3.45 kN

For 100mm Cavity, use CATNIC CG90/100 or IG L1/S 100 Standard Duty insulated lintel or similar

DESIGN OF L5 – Lintel - (Design Span 1.2m)

Load from Wall Over – Brick and Brick Cavity (in 45 degree load triangle)

- DL-Inner = 2.4 x 0.6 = 1.44 kN/m
- DL-Outer = 2.4 x 0.6 = 1.44 kN/m

Load from New Floor
### DESIGN OF FB1 – Foundation Beam - (Design Span 3.0m)

**Load from 1st Floor**

\[
DL = 1/2 \times 3.8 \times 0.5 = 0.95 \text{ kN/m}
\]

\[
LL = 1/2 \times 3.8 \times 1.5 = 2.85 \text{ kN/m}
\]

**Load from Suspended Ground Floor**

\[
DL = 1/2 \times 3.8 \times 0.5 = 0.95 \text{ kN/m}
\]

\[
LL = 1/2 \times 3.8 \times 1.5 = 2.85 \text{ kN/m}
\]

**Load from Wall Over – Brick and Block**

\[
DL_{-inner} = 1.2 \times 2.7 = 3.24 \text{ kN/m}
\]

\[
DL_{-outer} = 2.4 \times 2.7 = 6.48 \text{ kN/m}
\]

**Load from Wall Over – Tile Hung Stud – (in 45 degree load triangle)**

\[
DL = 2.4 \times 1.15 = 2.76 \text{ kN/m}
\]

**Load from New roof- Traditional Cut**

\[
DL = 1/2 \times 1.1 \times 4.2 \times \cos(42) = 3.11 \text{ kN/m}
\]

\[
LL = 1/2 \times 1 \times 4.2 = 2.10 \text{ kN/m}
\]

Use 203x203 x46 UC wrapped in D98 mesh and encased in min 50mm concrete to create ground beam. Ate each end sit on foundation pad to a size and depth as shown on drawings.

### DESIGN OF FB2 – Foundation Beam - (Design Span 3.0m)

**Load from Wall Over – Brick and Block**

\[
DL_{-inner} = 1.2 \times 2.7 = 3.24 \text{ kN/m}
\]

\[
DL_{-outer} = 2.4 \times 2.7 = 6.48 \text{ kN/m}
\]

**Load from Gable Wall Over – Tile Hung Stud – (in 45 degree load triangle)**

\[
DL = 3.1 \times 1.15 = 3.57 \text{ kN/m}
\]

Use 203x203 x46 UC wrapped in D98 mesh and encased in min 50mm concrete to create ground beam. Ate each end sit on foundation pad to a size and depth as shown on drawings.

### DESIGN OF FB3 – Foundation Beam - (Design Span 2.0m)

As per FB1 and FB2
MASONRY WALL PANEL DESIGN (EN1996)

MASONRY WALL PANEL DESIGN (EN1996-1-1:2005)

In accordance with EN1996-1-1:2005 incorporating Corrigenda February 2006 and July 2009 and the UK national annex

TEDDS calculation version 1.2.03

Masonry panel details

Extension Front Wall - Unreinforced masonry wall with openings

Panel length  \( L = 5900 \text{ mm} \)

Panel height  \( h = 2800 \text{ mm} \)

Panel support conditions

Outer leaf  All edges supported, top, bottom and right continuous

Inner leaf  All edges supported continuously

Effective height of masonry walls - Section 5.5.1.2

Reduction factor

\[
\rho_2 = 1.000 \\
\rho_4 = \rho_2 / (1 + \left[ \rho_2 \times h / L_1^2 \right]) = 0.816
\]

Effective height of wall - eq 5.2

\[
h_{ef} = \rho_4 \times h = 2285 \text{ mm}
\]

Panel opening details

Spacing length  \( L_1 = 350 \text{ mm} \)

Opening width  \( w_1 = 1100 \text{ mm} \)

Height to underside of lintel  \( h_1 = 2400 \text{ mm} \)

Height of opening  \( o_1 = 1200 \text{ mm} \)

Spacing length  \( L_2 = 800 \text{ mm} \)

Opening width  \( w_2 = 3200 \text{ mm} \)

Height to underside of lintel  \( h_2 = 2400 \text{ mm} \)

Height of opening  \( o_2 = 2400 \text{ mm} \)
### Cavity wall construction details

- **Outer leaf thickness**: $t_1 = 100$ mm
- **Cavity thickness**: $t_c = 100$ mm
- **Inner leaf thickness**: $t_2 = 100$ mm

**Effective thickness of masonry walls - Section 5.5.1.3**

- Relative E factor: $k_{rel} = 1.000$
- Effective thickness - eq 5.11:
  
  $$t_{ef} = (k_{rel} \times t_1^3 + t_2^3)^{1/3} = 126 \text{ mm}$$

### Masonry outer leaf details

- **Masonry type**: Clay with water absorption less than 7% - Group 1
- **Mean compressive strength of masonry unit**: $f_{b1} = 5 \text{ N/mm}^2$
- **Density of masonry**: $\gamma_1 = 18 \text{ kN/m}^3$
- **Mortar type**: M6 - General purpose mortar
- **Compressive strength of masonry mortar**: $f_{m1} = 6 \text{ N/mm}^2$
- **Compressive strength factor**: $K = 0.50$
- **Characteristic compressive strength of masonry - eq 3.2**:
  
  $$f_{k1} = K \times f_{b1}^{0.7} \times f_{m1}^{0.3} = 2.641 \text{ N/mm}^2$$

- **Characteristic flexural strength of masonry having a plane of failure parallel to the bed joints - Table NA.6**
  
  $$f_{xk11} = 0.5 \text{ N/mm}^2$$

- **Characteristic flexural strength of masonry having a plane of failure perpendicular to the bed joints - Table NA.6**
  
  $$f_{xk21} = 1.5 \text{ N/mm}^2$$

### Masonry inner leaf details

- **Masonry type**: Autoclaved aerated concrete - Group 1
- **Mean compressive strength of masonry unit**: $f_{b2} = 3.6 \text{ N/mm}^2$
- **Density of masonry**: $\gamma_2 = 18 \text{ kN/m}^3$
- **Mortar type**: M6 - General purpose mortar
- **Compressive strength of masonry mortar**: $f_{m2} = 6 \text{ N/mm}^2$
- **Compressive strength factor**: $K = 0.55$
- **Characteristic compressive strength of masonry - eq 3.2**:
  
  $$f_{k2} = K \times f_{b2}^{0.7} \times f_{m2}^{0.3} = 2.308 \text{ N/mm}^2$$
Characteristic flexural strength of masonry having a plane of failure parallel to the bed joints - Table NA.6
\[ f_{xk12} = 0.25 \text{ N/mm}^2 \]

Characteristic flexural strength of masonry having a plane of failure perpendicular to the bed joints - Table NA.6
\[ f_{xk22} = 0.45 \text{ N/mm}^2 \]

Lateral loading details
Characteristic wind load on panel
\[ W_k = 0.500 \text{ kN/m}^2 \]

Vertical loading details
Permanent load on top of outer leaf
\[ G_{k1} = 3 \text{ kN/m} \]

Partial factors for material strength
Category of manufacturing control
Category II
Class of execution control
Class 2
Partial factor for masonry in compressive flexure
\[ \gamma_{Mc} = 3.00 \]
Partial factor for masonry in tensile flexure
\[ \gamma_{Mt} = 2.70 \]
Partial factor for masonry in shear
\[ \gamma_{Mv} = 2.50 \]

Slenderness ratio of masonry walls - Section 5.5.1.4
Allowable slenderness ratio
\[ SR_{all} = 27 \]
Slenderness ratio
\[ SR = \frac{h_{ef}}{r_{ef}} = 18.1 \quad \text{PASS - Slenderness ratio is less than maximum allowable} \]

Unreinforced masonry walls subjected to lateral loading - Section 6.3
Limiting height and length to thickness ratio for walls under serviceability limit state - Annex F
Length to thickness ratio
\[ L / t_{ef} = 46.828 \]
Limiting height to thickness ratio - Figure F.1
\[ 48.293 \]
Height to thickness ratio
\[ h / t_{ef} = 22.224 \quad \text{PASS - Limiting height to thickness ratio is not exceeded} \]

Partial safety factors for design loads
Partial safety factor for variable wind load
\[ \gamma_{WV} = 1.50 \]
Partial safety factor for permanent load
\[ \gamma_{G} = 1.00 \]

Design moments of resistance in panels
Considering outer leaf
Self weight at top of wall
\[ S_{wt1} = 0 \text{ kN/m} \]
Design compressive strength of masonry
\[ f_{d1} = \frac{f_{k1}}{\gamma_{Mc}} = 0.880 \text{ N/mm}^2 \]
Design vertical compressive stress
\[ \sigma_{d1} = \min\left(\gamma_{G} \times \left( G_{k1} + S_{wt1} \right) / t_1, 0.2 \times f_{d1} \right) = 0.03 \text{ N/mm}^2 \]
Design flexural strength of masonry parallel to bed joints
\[ f_{xd11} = \frac{f_{kx11}}{\gamma_{Mc}} = 0.167 \text{ N/mm}^2 \]
Apparent design flexural strength of masonry parallel to bed joints
\[ f_{xd11,app} = f_{xd11} + \sigma_{d1} = 0.197 \text{ N/mm}^2 \]
Design flexural strength of masonry perpendicular to bed joints
\[ f_{xd21} = \frac{f_{kx21}}{\gamma_{Mc}} = 0.5 \text{ N/mm}^2 \]
Elastic section modulus of wall
\[ Z_1 = t_1^2 / 6 = 1666667 \text{ mm}^3/m \]
Moment of resistance parallel to bed joints - eq.6.15
\[ M_{Rd11} = f_{xd11,app} \times Z_1 = 0.328 \text{ kNm/m} \]
Momemt of resistance perpendicular to bed joints - eq.6.15
\[ M_{Rd21} = f_{xd21} \times Z_1 = 0.833 \text{ kNm/m} \]
Considering inner leaf

Self weight at top of wall

\[ S_{\text{wt2}} = 0 \text{ kN/m} \]

Design compressive strength of masonry

\[ f_{\text{d2}} = \frac{f_{k2}}{\gamma_{Mc}} = 0.769 \text{ N/mm}^2 \]

Design vertical compressive stress

\[ \sigma_{d2} = \min\left(\frac{\gamma_{fG} \times (G_{k2} + S_{\text{wt2}})}{t_2}, 0.2 \times f_{d2}\right) = 0 \text{ N/mm}^2 \]

Design flexural strength of masonry parallel to bed joints

\[ f_{\text{xd12}} = f_{\text{sk12}} / \gamma_{Mc} = 0.083 \text{ N/mm}^2 \]

Apparent design flexural strength of masonry parallel to bed joints

\[ f_{\text{xd12,app}} = f_{\text{xd12}} + \sigma_{d2} = 0.083 \text{ N/mm}^2 \]

Design flexural strength of masonry perpendicular to bed joints

\[ f_{\text{xd22}} = f_{\text{sk22}} / \gamma_{Mc} = 0.15 \text{ N/mm}^2 \]

Elastic section modulus of wall

\[ Z_2 = t_2^2 / 6 = 1666667 \text{ mm}^3 / \text{m} \]

Moment of resistance parallel to bed joints - eq. 6.15

\[ M_{\text{Rd12}} = f_{\text{xd12,app}} \times Z_2 = 0.139 \text{ kNm/m} \]

Moment of resistance perpendicular to bed joints - eq. 6.15

\[ M_{\text{Rd22}} = f_{\text{xd22}} \times Z_2 = 0.25 \text{ kNm/m} \]

Design moment in panels

Calculate design wind load acting on each leaf

Outer leaf design wind load - parallel

\[ W_{k11} = M_{\text{Rd11}} \times W_k / (M_{\text{Rd11}} + M_{\text{Rd12}}) = 0.351 \text{ kN/m}^2 \]

Inner leaf design wind load - parallel

\[ W_{k12} = M_{\text{Rd12}} \times W_k / (M_{\text{Rd11}} + M_{\text{Rd12}}) = 0.149 \text{ kN/m}^2 \]

Outer leaf design wind load - perpendicular

\[ W_{k21} = M_{\text{Rd21}} \times W_k / (M_{\text{Rd21}} + M_{\text{Rd22}}) = 0.385 \text{ kN/m}^2 \]

Inner leaf design wind load - perpendicular

\[ W_{k22} = M_{\text{Rd22}} \times W_k / (M_{\text{Rd21}} + M_{\text{Rd22}}) = 0.115 \text{ kN/m}^2 \]

Orthogonal strength ratios

For outer leaf

\[ \mu_1 = \frac{f_{k11}}{f_{k21}} = 0.33 \]

For inner leaf

\[ \mu_2 = \frac{f_{k12}}{f_{k22}} = 0.56 \]

Sub panel no. 1 - Right, left and top supported

Ratio panel height to length

\[ h_{s1B} / L_{s1B} = 0.07 \]

Considering outer leaf

Parallel design moment of resistance

\[ M_{\text{Rd11}} = 0.328 \text{ kNm/m} \]
Using elastic analysis to determine bending moment coefficients for a vertically spanning sub panel
Bending moment coefficient \( \alpha_{s11B} = 0.5 \times (1 + 2 \times \beta_{s11B}) = 3.500 \)
Design moment in sub-panel \( M_{Ed11B} = \gamma_W \times \alpha_{s11B} \times W_{k11} \times h_{s1B}^2 = 0.295 \text{ kNm/m} \)
**PASS - Resistance moment exceeds design moment**

**Considering inner leaf**
Parallel design moment of resistance \( M_{Rd11} = 0.139 \text{ kNm/m} \)

Using elastic analysis to determine bending moment coefficients for a vertically spanning sub panel
Bending moment coefficient \( \alpha_{s21B} = 0.5 \times (1 + 2 \times \beta_{s1B}) = 3.500 \)
Design moment in sub-panel \( M_{Ed21B} = \gamma_W \times \alpha_{s21B} \times W_{k12} \times h_{s1B}^2 = 0.125 \text{ kNm/m} \)
**PASS - Resistance moment exceeds design moment**

Sub panel no. 2 - Top, bottom and left supported

| Ratio panel height to length | \( h_{s2B} / L_{s2B} = 6.86 \) |

Considering outer leaf
Perpendicular design moment of resistance \( M_{Rd21} = 0.833 \text{ kNm/m} \)

Using yield line analysis to calculate bending moment coefficient
Bending moment coefficient \( \alpha_{s12B} = 7.735 \)
Design moment in sub-panel \( M_{Ed12B} = \gamma_W \times \alpha_{s12B} \times W_{k21} \times L_{s2B}^2 = 0.547 \text{ kNm/m} \)
**PASS - Resistance moment exceeds design moment**

**Considering inner leaf**
Perpendicular design moment of resistance \( M_{Rd22} = 0.250 \text{ kNm/m} \)

Using elastic analysis to determine bending moment coefficients for a horizontally spanning sub panel
Bending moment coefficient \( \alpha_{s22B} = 0.5 \times (1 + 2 \times \beta_{s2B}) = 2.071 \)
Design moment in sub-panel \( M_{Ed22B} = \gamma_W \times \alpha_{s22B} \times W_{k22} \times L_{s2B}^2 = 0.044 \text{ kNm/m} \)
**PASS - Resistance moment exceeds design moment**

Sub panel no. 3 - Right, left and bottom supported

| Ratio panel height to length | \( h_{s3B} / L_{s3B} = 1.09 \) |

Considering outer leaf
Perpendicular design moment of resistance \( M_{Rd21} = 0.833 \text{ kNm/m} \)

Using yield line analysis to calculate bending moment coefficient
Bending moment coefficient \( \alpha_{s13B} = 0.132 \)
Design moment in sub-panel \( M_{Ed13B} = \gamma_W \times \alpha_{s13B} \times W_{k21} \times L_{s3B}^2 = 0.092 \text{ kNm/m} \)
**PASS - Resistance moment exceeds design moment**

**Considering inner leaf**
Perpendicular design moment of resistance \( M_{Rd22} = 0.250 \text{ kNm/m} \)

Using yield line analysis to calculate bending moment coefficient
Bending moment coefficient \( \alpha_{s23B} = 0.119 \)
Design moment in sub-panel \( M_{Ed23B} = \gamma_W \times \alpha_{s23B} \times W_{k22} \times L_{s3B}^2 = 0.025 \text{ kNm/m} \)
**PASS - Resistance moment exceeds design moment**

Sub panel no. 4 - Top and bottom supported

| Ratio panel height to length | \( h_{s4B} / L_{s4B} = 3.00 \) |

Considering outer leaf
Parallel design moment of resistance \( M_{Rd11} = 0.328 \text{ kNm/m} \)
Using elastic analysis to determine bending moment coefficients for a vertically spanning sub panel

Bending moment coefficient
\[ \alpha_{s14B} = 0.125 \times (1 + 2 \times \beta_{s4B}) = 0.797 \]

Design moment in sub-panel
\[ M_{Ed14B} = \gamma_{W} \times \alpha_{s14B} \times W_{k11} \times h_{s4B}^2 = 2.418 \text{ kNm/m} \]

FAIL - Resistance moment is exceeded by design moment

Considering inner leaf
Parallel design moment of resistance
\[ M_{Rd12} = 0.139 \text{ kNm/m} \]

Using elastic analysis to determine bending moment coefficients for a vertically spanning sub panel

Bending moment coefficient
\[ \alpha_{s24B} = 0.125 \times (1 + 2 \times \beta_{s4B}) = 0.797 \]

Design moment in sub-panel
\[ M_{Ed24B} = \gamma_{W} \times \alpha_{s24B} \times W_{k12} \times h_{s4B}^2 = 1.025 \text{ kNm/m} \]

FAIL - Resistance moment is exceeded by design moment

Sub panel no. 5 - Top, bottom and right supported

Ratio panel height to length
\[ h_{s5B} / L_{s5B} = 5.33 \]

Considering outer leaf
Perpendicular design moment of resistance
\[ M_{Rd21} = 0.833 \text{ kNm/m} \]

Using elastic analysis to determine bending moment coefficients for a horizontally spanning sub panel

Bending moment coefficient
\[ \alpha_{s15B} = 0.5 \times (1 + 2 \times \beta_{s5B}) = 4.056 \]

Design moment in sub-panel
\[ M_{Ed15B} = \gamma_{W} \times \alpha_{s15B} \times W_{k21} \times L_{s5B}^2 = 0.474 \text{ kNm/m} \]

PASS - Resistance moment exceeds design moment

Considering inner leaf
Perpendicular design moment of resistance
\[ M_{Rd22} = 0.250 \text{ kNm/m} \]

Using elastic analysis to determine bending moment coefficients for a horizontally spanning sub panel

Bending moment coefficient
\[ \alpha_{s25B} = 0.5 \times (1 + 2 \times \beta_{s5B}) = 4.056 \]

Design moment in sub-panel
\[ M_{Ed25B} = \gamma_{W} \times \alpha_{s25B} \times W_{k22} \times L_{s5B}^2 = 0.142 \text{ kNm/m} \]

PASS - Resistance moment exceeds design moment

WINDPOST DESIGN (BS5950)

WINDPOST DESIGN (BS5950:PART 1:2000)

Section properties - 95 × 60 × 5 channel section windpost

Section depth
\[ d = 95 \text{ mm} \]

Section breadth
\[ b = 60 \text{ mm} \]

Section thickness
\[ t = 5 \text{ mm} \]

Radius of curves
\[ r = 5 \text{ mm} \]

Area of section
\[ \text{Area} = 9.771 \text{ cm}^2 \]

Section modulus
\[ Z = 28.929 \text{ cm}^3 \]

Moment of inertia
\[ I = 137.412 \text{ cm}^4 \]
Material properties
Design strength \( p_y = 180.00 \text{ N/mm}^2 \)
Youngs modulus \( E = 200000 \text{ N/mm}^2 \)

Section classification
Constant \( \varepsilon = [(275 / p_y) \times (E / 205000)]^{0.5} = 1.221 \)
Web with neutral axis at mid-depth \( d / t = 19.000 \leq 52 \times \varepsilon \quad (1) \text{ Plastic} \)
Outstand element of compression flange \( b / t = 12.000 \leq 11.0 \times \varepsilon \quad (3) \text{ Semi-compact} \)

Windpost design details
Height of windpost \( H = 2600 \text{ mm} \)
Length of loaded panel \( L = 5900 \text{ mm} \)
Windpost support type \( \text{Simply supported} \)

Loading details
Partial safety factor for wind loading \( \gamma_I = 1.40 \)
Unfactored wind loading \( w = 0.500 \text{ kN/m}^2 \)
Reaction at supports \( R = w \times \gamma_I \times H \times L / 2 = 5.369 \text{ kN} \)

Check bending stress
Bending moment \( M = w \times \gamma_I \times L \times H^2 / 8 = 3.490 \text{ kNm} \)
Applied bending stress \( p_{\text{app}} = M / Z = 120.635 \text{ N/mm}^2 \)
\( \text{PASS - Applied bending stress is less than maximum bending stress} \)

Check deflection
Maximum deflection \( \delta_{\max} = H / 360 = 7.222 \text{ mm} \)
Actual deflection \( \delta_{\text{act}} = 5 \times w \times L \times H^4 / (384 \times E \times I) = 6.387 \text{ mm} \)
\( \text{PASS - Actual deflection is less than maximum deflection} \)

Check wall ties
Brick tie capacity in tension and compression \( f_t = 4.500 \text{ kN} \)
Partial safety factor for wall ties \( \gamma_m = 3.00 \)
Total load on windpost \( F = w \times \gamma_I \times H \times L = 10.738 \text{ kN} \)
Maximum spacing between ties \( s = H \times f_t / (F \times \gamma_m) = 363 \text{ mm} \)
\( \text{Wall ties provided at a maximum of 225 mm centres} \)
Tie to ground plate/DPM-foundations and to wall plate as per manufacturer's instructions. Tie to wall top/steel GB2 with proprietary ties at 450 crs vertical avoiding cold-bridging issues.

**DESIGN OF FLOOR JOISTS:**

Largest single span between over living room = 3.8m.

**FLOOR JOIST DESIGN (BS5268)**

**TIMBER JOIST DESIGN (BS5268-2:2002)**

TEDDS calculation version 1.1.02

**Joist details**

- Joist breadth \( b = 47 \text{ mm} \)
- Joist depth \( h = 170 \text{ mm} \)
- Joist spacing \( s = 400 \text{ mm} \)
- Timber strength class \( C24 \)
- Service class of timber \( 1 \)

**Span details**

- Number of spans \( N_{span} = 1 \)
- Length of bearing \( L_b = 100 \text{ mm} \)
- Effective length of span \( L_s1 = 3800 \text{ mm} \)
Section properties
Second moment of area
\[ I = \frac{b \times h^3}{12} = 19242583 \text{ mm}^4 \]
Section modulus
\[ Z = \frac{b \times h^2}{6} = 226383 \text{ mm}^3 \]

Loading details
Joist self weight
\[ F_{\text{swt}} = b \times h \times \rho_{\text{char}} \times g_{\text{acc}} = 0.03 \text{ kN/m} \]
Dead load
\[ F_{d,\text{udl}} = 0.35 \text{ kN/m}^2 \]
Imposed UDL(Long term)
\[ F_{l,\text{udl}} = 1.50 \text{ kN/m}^2 \]
Imposed point load (Medium term)
\[ F_{l,\text{pt}} = 1.40 \text{ kN} \]

Modification factors
Service class for bending parallel to grain
\[ K_{2m} = 1.00 \]
Service class for compression
\[ K_{2c} = 1.00 \]
Service class for shear parallel to grain
\[ K_{2s} = 1.00 \]
Service class for modulus of elasticity
\[ K_{2e} = 1.00 \]
Section depth factor
\[ K_7 = 1.06 \]
Load sharing factor
\[ K_8 = 1.10 \]

Consider long term loads
Load duration factor
\[ K_3 = 1.00 \]
Maximum bending moment
\[ M = 1.385 \text{ kNm} \]
Maximum shear force
\[ V = 1.458 \text{ kN} \]
Maximum support reaction
\[ R = 1.458 \text{ kN} \]
Maximum deflection
\[ \delta = 10.334 \text{ mm} \]

Check bending stress
Bending stress
\[ \sigma_n = 7.500 \text{ N/mm}^2 \]
Permissible bending stress
\[ \sigma_{\text{adm}} = \sigma_n \times K_{2m} \times K_3 \times K_7 \times K_8 = 8.782 \text{ N/mm}^2 \]
Applied bending stress
\[ \sigma_{\text{max}} = \frac{M}{Z} = 6.119 \text{ N/mm}^2 \]
PASS - Applied bending stress within permissible limits

Check shear stress
Shear stress
\[ \tau = 0.710 \text{ N/mm}^2 \]
Permissible shear stress
\[ \tau_{\text{adm}} = \tau \times K_{2s} \times K_3 \times K_8 = 0.781 \text{ N/mm}^2 \]
Applied shear stress
\[ \tau_{\text{max}} = \frac{3}{2} \times \frac{V}{b \times h} = 0.274 \text{ N/mm}^2 \]
PASS - Applied shear stress within permissible limits

Check compression perpendicular to grain (no wane)
\[ \sigma_{\text{cp1}} = 2.400 \text{ N/mm}^2 \]
Permissible bearing stress
\[ \sigma_{\text{c,adm}} = \sigma_{\text{cp1}} \times K_{2c} \times K_3 \times K_8 = 2.640 \text{ N/mm}^2 \]
Applied bearing stress
\[ \sigma_{\text{c,max}} = \frac{R}{(b \times L_b)} = 0.310 \text{ N/mm}^2 \]
PASS - Applied bearing stress within permissible limits

Check deflection
Permissible deflection
\[ \delta_{\text{adm}} = \min(L_{s1} \times 0.003, 14 \text{ mm}) = 11.400 \text{ mm} \]
Bending deflection (based on \( E_{\text{mean}} \))
\[ \delta_{\text{bending}} = 10.026 \text{ mm} \]
Shear deflection
\[ \delta_{\text{shear}} = 0.308 \text{ mm} \]
Total deflection
\[ \delta = \delta_{\text{bending}} + \delta_{\text{shear}} = 10.334 \text{ mm} \]
PASS - Actual deflection within permissible limits

Consider medium term loads
Load duration factor
\[ K_3 = 1.25 \]
Maximum bending moment
\[ M = 1.632 \text{ kNm} \]
Maximum shear force \( V = 1.718 \text{kN} \)

Maximum support reaction \( R = 1.718 \text{kN} \)

Maximum deflection \( \delta = 10.252 \text{mm} \)

**Check bending stress**

Bending stress \( \sigma_m = 7.500 \text{ N/mm}^2 \)

Permissible bending stress \( \sigma_{m,adm} = \sigma_m \times K_2m \times K_3 \times K_7 \times K_8 = 10.977 \text{ N/mm}^2 \)

Applied bending stress \( \sigma_{m,max} = M / Z = 7.210 \text{ N/mm}^2 \)

**PASS - Applied bending stress within permissible limits**

**Check shear stress**

Shear stress \( \tau = 0.710 \text{ N/mm}^2 \)

Permissible shear stress \( \tau_{adm} = \tau \times K_{2s} \times K_3 \times K_8 = 0.976 \text{ N/mm}^2 \)

Applied shear stress \( \tau_{max} = 3 \times V / (2 \times b \times h) = 0.323 \text{ N/mm}^2 \)

**PASS - Applied shear stress within permissible limits**

**Check bearing stress**

Compression perpendicular to grain (no wane) \( \sigma_{cp1} = 2.400 \text{ N/mm}^2 \)

Permissible bearing stress \( \sigma_{c,adm} = \sigma_{cp1} \times K_{2c} \times K_3 \times K_8 = 3.300 \text{ N/mm}^2 \)

Applied bearing stress \( \sigma_{c,max} = R / (b \times L_b) = 0.366 \text{ N/mm}^2 \)

**PASS - Applied bearing stress within permissible limits**

**Check deflection**

Permissible deflection \( \delta_{adm} = \min(L_{s1} \times 0.003, 14 \text{ mm}) = 11.400 \text{ mm} \)

Bending deflection (based on \( E_{mean} \)) \( \delta_{bending} = 9.888 \text{ mm} \)

Shear deflection \( \delta_{shear} = 0.363 \text{ mm} \)

Total deflection \( \delta = \delta_{bending} + \delta_{shear} = 10.252 \text{ mm} \)

**PASS - Actual deflection within permissible limits**

Use 47x170 minimum C24 timbers at 400 crs. Triple up joists below all parallel stud partition walls and double up joists below bath and hot water boiler/tank. Provide solid noggins to new floor at max 2.5m centres. Strap to parallel walls with 30x5x1200 LG galvanised restraint straps at 2.0m crs across 3No joists. Use 22mm T&G chipboard flooring on joists extended to eaves and fix stud partition walls at first floor through T&G to joists below.

**DESIGN OF NEW ROOF MEMBERS**

**TIMBER RAFTER DESIGN (BS5268-2:2002)**

TEDDS calculation version 1.0.03
Rafter details
Breadth of timber sections \( b = 47 \text{ mm} \)
Depth of timber sections \( h = 125 \text{ mm} \)
Rafter spacing \( s = 400 \text{ mm} \)
Rafter slope \( \alpha = 41.0 \text{ deg} \)
Clear span of rafter on horizontal \( L_{ch} = 1900 \text{ mm} \)
Clear span of rafter on slope \( L_{cl} = L_{ch} / \cos(\alpha) = 2518 \text{ mm} \)
Rafter span Single span
Timber strength class C24

Section properties
Cross sectional area of rafter \( A = b \times h = 5875 \text{ mm}^2 \)
Section modulus \( Z = b \times h^2 / 6 = 122396 \text{ mm}^3 \)
Second moment of area \( I = b \times h^3 / 12 = 7649740 \text{ mm}^4 \)
Radius of gyration \( r = \sqrt{(I / A)} = 36.1 \text{ mm} \)

Loading details
Rafter self weight \( F_j = b \times h \times \rho_{char} \times g_{acc} = 0.02 \text{ kN/m} \)
Dead load on slope \( F_d = 0.90 \text{ kN/m}^2 \)
Imposed load on plan \( F_u = 0.75 \text{ kN/m}^2 \)
Imposed point load \( F_p = 0.90 \text{ kN} \)

Modification factors
Section depth factor \( K_7 = (300 \text{ mm} / h)^{0.11} = 1.10 \)
Load sharing factor \( K_8 = 1.10 \)

Consider long term load condition
Load duration factor \( K_3 = 1.00 \)
Total UDL perpendicular to rafter \( F = F_d \times \cos(\alpha) \times s + F_j \times \cos(\alpha) = 0.287 \text{ kN/m} \)
Notional bearing length \( L_b = F \times L_d / [2 \times (b \times \sigma_{cp1} \times K_8 - F)] = 3 \text{ mm} \)
Effective span \( L_{eff} = L_d + L_b = 2520 \text{ mm} \)
Check bending stress
Bending stress parallel to grain \( \sigma_m = 7.500 \text{ N/mm}^2 \)
Permissible bending stress \( \sigma_{m, \text{adm}} = \sigma_m \times K_3 \times K_7 \times K_e = 9.084 \text{ N/mm}^2 \)
Applied bending stress \( \sigma_{m, \text{max}} = F \times L_{\text{eff}}^2 / (8 \times Z) = 1.861 \text{ N/mm}^2 \)

**PASS - Applied bending stress within permissible limits**

Check compressive stress parallel to grain
Compression stress parallel to grain \( \sigma_c = 7.900 \text{ N/mm}^2 \)
Minimum modulus of elasticity \( E_{\min} = 7200 \text{ N/mm}^2 \)
Compression member factor \( K_{12} = 0.55 \)
Permissible compressive stress \( \sigma_{c, \text{adm}} = \sigma_c \times K_3 \times K_8 \times K_{12} = 6.024 \text{ N/mm}^2 \)
Applied compressive stress \( \sigma_{c, \text{max}} = F \times L_{\text{eff}} \times (\cot(\alpha) + 3 \times \tan(\alpha)) / (2 \times A) = 0.369 \text{ N/mm}^2 \)

**PASS - Applied compressive stress within permissible limits**

Check combined bending and compressive stress parallel to grain
Euler stress \( \sigma_e = \frac{\sigma_m^2 \times E_{\min}}{\lambda^2} = 14.565 \text{ N/mm}^2 \)
Euler coefficient \( K_{eu} = 1 - (1.5 \times \sigma_{c, \text{max}} \times K_{12} / \sigma_c) = 0.986 \)
Combined axial compression and bending check \( \sigma_{m, \text{max}} / (\sigma_{m, \text{adm}} \times K_{eu}) + \sigma_{c, \text{max}} / \sigma_{c, \text{adm}} = 0.253 < 1 \)

**PASS - Combined compressive and bending stresses are within permissible limits**

Check shear stress
Shear stress parallel to grain \( \tau = 0.710 \text{ N/mm}^2 \)
Permissible shear stress \( \tau_{\text{adm}} = \tau \times K_3 \times K_8 = 0.781 \text{ N/mm}^2 \)
Applied shear stress \( \tau_{\text{max}} = 3 \times F \times L_{\text{eff}} / (4 \times A) = 0.092 \text{ N/mm}^2 \)

**PASS - Applied shear stress within permissible limits**

Check deflection
Permissible deflection \( \delta_{\text{adm}} = 0.003 \times L_{\text{eff}} = 7.561 \text{ mm} \)
Bending deflection \( \delta_b = 5 \times F \times L_{\text{eff}}^4 / (384 \times E_{\text{mean}} \times I) = 1.825 \text{ mm} \)
Shear deflection \( \delta_s = 12 \times F \times L_{\text{eff}}^2 / (5 \times E_{\text{mean}} \times A) = 0.069 \text{ mm} \)
Total deflection \( \delta_{\text{max}} = \delta_b + \delta_s = 1.894 \text{ mm} \)

**PASS - Total deflection within permissible limits**

Consider medium term load condition
Load duration factor \( K_3 = 1.25 \)
Total UDL perpendicular to rafter \( F = (F_u \times \cos(\alpha)^2 + F_d \times \cos(\alpha)) \times s + F_j \times \cos(\alpha) = 0.458 \text{ kN/m} \)
Notional bearing length \( L_{b} = F \times L_{cl} / [2 \times (b \times \sigma_{c, \text{adm}} \times K_8 - F)] = 5 \text{ mm} \)
Effective span \( L_{\text{eff}} = L_{cl} + L_{b} = 2522 \text{ mm} \)

Check bending stress
Bending stress parallel to grain \( \sigma_m = 7.500 \text{ N/mm}^2 \)
Permissible bending stress \( \sigma_{m, \text{adm}} = \sigma_m \times K_3 \times K_7 \times K_e = 11.355 \text{ N/mm}^2 \)
Applied bending stress \( \sigma_{m, \text{max}} = F \times L_{\text{eff}}^2 / (8 \times Z) = 2.974 \text{ N/mm}^2 \)

**PASS - Applied bending stress within permissible limits**

Check compressive stress parallel to grain
Compression stress parallel to grain \( \sigma_c = 7.900 \text{ N/mm}^2 \)
Minimum modulus of elasticity \( E_{\min} = 7200 \text{ N/mm}^2 \)
Compression member factor \( K_{12} = 0.55 \)
Permissible compressive stress \( \sigma_{c, \text{adm}} = \sigma_c \times K_3 \times K_8 \times K_{12} = 6.024 \text{ N/mm}^2 \)
Applied compressive stress \( \sigma_{c, \text{max}} = F \times L_{\text{eff}} \times (\cot(\alpha) + 3 \times \tan(\alpha)) / (2 \times A) = 0.369 \text{ N/mm}^2 \)
PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain

Euler stress
\[ \sigma_e = \pi^2 \times \frac{E_{\text{min}}}{l^2} = 14.545 \text{ N/mm}^2 \]

Euler coefficient
\[ K_{\text{eu}} = 1 - (1.5 \times \sigma_{c,\text{max}} \times K_{12} / \sigma_d) = 0.979 \]

Combined axial compression and bending check
\[ \sigma_{n,\text{max}} / (\sigma_{c,\text{adm}} \times K_{\text{eu}}) + \sigma_{c,\text{max}} / \sigma_{c,\text{adm}} = 0.329 < 1 \]

PASS - Combined compressive and bending stresses are within permissible limits

Check shear stress

Shear stress parallel to grain
\[ \tau = 0.710 \text{ N/mm}^2 \]

Permissible shear stress
\[ \tau_{\text{adm}} = \tau \times K_3 \times K_b = 0.976 \text{ N/mm}^2 \]

Applied shear stress
\[ \tau_{\text{max}} = 3 \times F \times L_{\text{eff}} / (4 \times A) = 0.147 \text{ N/mm}^2 \]

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection
\[ \delta_{\text{adm}} = 0.003 \times L_{\text{eff}} = 7.567 \text{ mm} \]

Bending deflection
\[ \delta_b = 5 \times F \times L_{\text{eff}} / (384 \times E_{\text{mean}} \times I) = 2.920 \text{ mm} \]

Shear deflection
\[ \delta_s = 12 \times F \times L_{\text{eff}} / (5 \times E_{\text{mean}} \times A) = 0.110 \text{ mm} \]

Total deflection
\[ \delta_{\text{max}} = \delta_b + \delta_s = 3.030 \text{ mm} \]

PASS - Total deflection within permissible limits

Consider short term load condition

Load duration factor
\[ K_3 = 1.50 \]

Total UDL perpendicular to rafter
\[ F = F_p \times \cos(\alpha) \times s + F_j \times \cos(\alpha) = 0.287 \text{ kN/m} \]

Notional bearing length
\[ L_b = [F \times L_\text{adj} + F_p \times \cos(\alpha)] / [2 \times (b \times \sigma_{c,\text{adm}} \times K_b - F)] = 6 \text{ mm} \]

Effective span
\[ L_{\text{eff}} = L_\text{adj} + L_b = 2523 \text{ mm} \]

Check bending stress

Bending stress parallel to grain
\[ \sigma_n = 7.500 \text{ N/mm}^2 \]

Permissible bending stress
\[ \sigma_{n,\text{adm}} = \sigma_n \times K_3 \times K_7 \times K_b = 13.626 \text{ N/mm}^2 \]

Applied bending stress
\[ \sigma_{n,\text{max}} = F \times L_{\text{eff}} \times (\delta \times Z) + F_p \times \cos(\alpha) \times L_{\text{eff}} / (4 \times Z) = 5.366 \text{ N/mm}^2 \]

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain

Compression stress parallel to grain
\[ \sigma_c = 7.900 \text{ N/mm}^2 \]

Minimum modulus of elasticity
\[ E_{\text{min}} = 7200 \text{ N/mm}^2 \]

Compression member factor
\[ K_{12} = 0.51 \]

Permissible compressive stress
\[ \sigma_{c,\text{adm}} = \sigma_c \times K_3 \times K_b \times K_{12} = 6.708 \text{ N/mm}^2 \]

Applied compressive stress
\[ \sigma_{c,\text{max}} = F \times L_{\text{eff}} \times (\delta \times Z) + F_p \times \cos(\alpha) \times L_{\text{eff}} / (4 \times Z) = 0.332 \text{ N/mm}^2 \]

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain

Euler stress
\[ \sigma_e = \pi^2 \times \frac{E_{\text{min}}}{l^2} = 14.534 \text{ N/mm}^2 \]

Euler coefficient
\[ K_{\text{eu}} = 1 - (1.5 \times \sigma_{c,\text{max}} \times K_{12} / \sigma_d) = 0.982 \]

Combined axial compression and bending check
\[ \sigma_{n,\text{max}} / (\sigma_{c,\text{adm}} \times K_{\text{eu}}) + \sigma_{c,\text{max}} / \sigma_{c,\text{adm}} = 0.450 < 1 \]

PASS - Combined compressive and bending stresses are within permissible limits

Check shear stress

Shear stress parallel to grain
\[ \tau = 0.710 \text{ N/mm}^2 \]

Permissible shear stress
\[ \tau_{\text{adm}} = \tau \times K_3 \times K_b = 1.172 \text{ N/mm}^2 \]

Applied shear stress
\[ \tau_{\text{max}} = 3 \times F \times L_{\text{eff}} / (4 \times A) + 3 \times F_p \times \cos(\alpha) / (2 \times A) = 0.266 \text{ N/mm}^2 \]

PASS - Applied shear stress within permissible limits
Check deflection

Permissible deflection
\[ \delta_{adm} = 0.003 \times L_{eff} = 7.570 \text{ mm} \]

Bending deflection
\[ \delta_b = L_{eff}^3 \times (5 \times F \times L_{eff} / 384 + F_p \times \cos(\alpha) / 48) / (E_{mean} \times I) = 4.584 \text{ mm} \]

Shear deflection
\[ \delta_s = 12 \times L_{eff} \times (F \times L_{eff}^{2} + 2 \times F_p \times \cos(\alpha)) / (5 \times E_{mean} \times A) = 0.199 \text{ mm} \]

Total deflection
\[ \delta_{max} = \delta_b + \delta_s = 4.783 \text{ mm} \]

**PASS - Total deflection within permissible limits**

CEILING JOIST DESIGN (BS5268)

TIMBER JOIST DESIGN (BS5268-2:2002)

TEDDS calculation version 1.1.02

Joist details
- Joist breadth \( b = 47 \text{ mm} \)
- Joist depth \( h = 125 \text{ mm} \)
- Joist spacing \( s = 400 \text{ mm} \)
- Timber strength class \( \text{C24} \)
- Service class of timber \( 1 \)

Span details
- Number of spans \( N_{\text{span}} = 1 \)
- Length of bearing \( L_b = 100 \text{ mm} \)
- Effective length of span \( L_{s1} = 2800 \text{ mm} \)

Section properties
- Second moment of area \( I = b \times h^3 / 12 = 7649740 \text{ mm}^4 \)
- Section modulus \( Z = b \times h^2 / 6 = 122396 \text{ mm}^3 \)
Loading details

Joist self weight \( F_{swt} = b \times h \times \rho_{char} \times g_{acc} = 0.02 \text{ kN/m} \)
Dead load \( F_{d.udl} = 0.25 \text{ kN/m}^2 \)
Imposed UDL(Long term) \( F_{i.udl} = 0.25 \text{ kN/m}^2 \)
Imposed point load (Medium term) \( F_{i.pt} = 0.90 \text{ kN} \)

Modification factors

Service class for bending parallel to grain \( K_{2m} = 1.00 \)
Service class for compression \( K_{2c} = 1.00 \)
Service class for shear parallel to grain \( K_{2s} = 1.00 \)
Service class for modulus of elasticity \( K_{2e} = 1.00 \)
Section depth factor \( K_{7} = 1.10 \)
Load sharing factor \( K_{8} = 1.10 \)

Consider long term loads
Load duration factor \( K_{3} = 1.00 \)
Maximum bending moment \( M = 0.216 \text{ kNm} \)
Maximum shear force \( V = 0.308 \text{ kN} \)
Maximum support reaction \( R = 0.308 \text{ kN} \)
Maximum deflection \( \delta = 2.198 \text{ mm} \)

Check bending stress

Bending stress \( \sigma_{m} = 7.500 \text{ N/mm}^2 \)
Permissible bending stress \( \sigma_{m, adm} = \sigma_{m} \times K_{2m} \times K_{3} \times K_{7} \times K_{8} = 9.084 \text{ N/mm}^2 \)
Applied bending stress \( \sigma_{m, max} = M / Z = 1.763 \text{ N/mm}^2 \)

PASS - Applied bending stress within permissible limits

Check shear stress

Shear stress \( \tau = 0.710 \text{ N/mm}^2 \)
Permissible shear stress \( \tau_{adm} = \tau \times K_{2s} \times K_{3} \times K_{8} = 0.781 \text{ N/mm}^2 \)
Applied shear stress \( \tau_{max} = 3 \times V / (2 \times b \times h) = 0.079 \text{ N/mm}^2 \)

PASS - Applied shear stress within permissible limits

Check bearing stress

Compression perpendicular to grain (no wane) \( \sigma_{cp1} = 2.400 \text{ N/mm}^2 \)
Permissible bearing stress \( \sigma_{c, adm} = \sigma_{cp1} \times K_{2c} \times K_{3} \times K_{8} = 2.640 \text{ N/mm}^2 \)
Applied bearing stress \( \sigma_{c, max} = R / (b \times L_{b}) = 0.066 \text{ N/mm}^2 \)

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection \( \delta_{adm} = \min(L_{s1} \times 0.003, 14 \text{ mm}) = 8.400 \text{ mm} \)
Bending deflection (based on \( E_{\text{mean}} \)) \( \delta_{bending} = 2.133 \text{ mm} \)
Shear deflection \( \delta_{shear} = 0.065 \text{ mm} \)
Total deflection \( \delta = \delta_{bending} + \delta_{shear} = 2.198 \text{ mm} \)

PASS - Actual deflection within permissible limits

Consider medium term loads
Load duration factor \( K_{3} = 1.25 \)
Maximum bending moment \( M = 0.748 \text{ kNm} \)
Maximum shear force \( V = 1.068 \text{ kN} \)
Maximum support reaction \( R = 1.068 \text{ kN} \)
Maximum deflection \( \delta = 6.372 \text{ mm} \)
Check bending stress

Bending stress \( \sigma_m = 7.500 \text{ N/mm}^2 \)

Permissible bending stress
\( \sigma_{m,\text{adm}} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8 = 11.355 \text{ N/mm}^2 \)

Applied bending stress
\( \sigma_{m,\text{max}} = \frac{M}{Z} = 6.109 \text{ N/mm}^2 \)

**PASS - Applied bending stress within permissible limits**

Check shear stress

Shear stress \( \tau = 0.710 \text{ N/mm}^2 \)

Permissible shear stress
\( \tau_{\text{adm}} = \tau \times K_{2s} \times K_3 \times K_8 = 0.976 \text{ N/mm}^2 \)

Applied shear stress
\( \tau_{\text{max}} = \frac{3 \times V}{(2 \times b \times h)} = 0.273 \text{ N/mm}^2 \)

**PASS - Applied shear stress within permissible limits**

Check bearing stress

Compression perpendicular to grain (no wane) \( \sigma_{cp1} = 2.400 \text{ N/mm}^2 \)

Permissible bearing stress
\( \sigma_{\text{c,adm}} = \sigma_{cp1} \times K_{2c} \times K_3 \times K_8 = 3.300 \text{ N/mm}^2 \)

Applied bearing stress
\( \sigma_{c,\text{max}} = \frac{R}{b \times L_b} = 0.227 \text{ N/mm}^2 \)

**PASS - Applied bearing stress within permissible limits**

Check deflection

Permissible deflection
\( \delta_{\text{adm}} = \text{min}(L_{s1} \times 0.003, 14 \text{ mm}) = 8.400 \text{ mm} \)

Bending deflection (based on \( E_{\text{mean}} \))
\( \delta_{\text{bending}} = 6.146 \text{ mm} \)

Shear deflection
\( \delta_{\text{shear}} = 0.226 \text{ mm} \)

Total deflection
\( \delta = \delta_{\text{bending}} + \delta_{\text{shear}} = 6.372 \text{ mm} \)

**PASS - Actual deflection within permissible limits**

Use min 47x125 C24 rafters and ceiling joists bolted together with M12 and timber connectors. Fasten to 100x47 treated wall plate and to ridge board with BAT universal framing anchors or similar and strap wall plate to wall below with 30x5x1200 LG galvanised restraint straps at 1.2m crs. Provide solid noggins at 2.5m crs and all usual diagonal bracing and strap rafters to gable end wall with restraint straps at 1.2m crs along rake of gable.
**Beam: RB1**

<table>
<thead>
<tr>
<th>Load name</th>
<th>Loading w1</th>
<th>Start x1</th>
<th>Loading w2</th>
<th>End x2</th>
<th>R1comp</th>
<th>R2comp</th>
</tr>
</thead>
<tbody>
<tr>
<td>U G o.w.</td>
<td>0.15</td>
<td>0</td>
<td>L</td>
<td>0.14</td>
<td>0.14</td>
<td></td>
</tr>
<tr>
<td>U G Dormer Roof</td>
<td>0.99</td>
<td>0</td>
<td>L</td>
<td>0.89</td>
<td>0.89</td>
<td></td>
</tr>
<tr>
<td>U QH Dormer Roof</td>
<td>0.68</td>
<td>0</td>
<td>L</td>
<td>0.61</td>
<td>0.61</td>
<td></td>
</tr>
</tbody>
</table>

Total load (unfactored): 3.28 kN

<table>
<thead>
<tr>
<th>Load types: U:UDL; Load positions are measured in m. from R1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load durations: G:Dead; Qx: Imposed; QH: Roof</td>
</tr>
</tbody>
</table>

Maximum B.M. = 1.04 kNm (6.10) at 0.90 m. from R1

Maximum S.F. = 2.30 kN (6.10) at R1

Mid-span deflections: Dead: 0.16 x 10^8/EI (E in N/mm^2, I in cm^4)

Live: 0.09 x 10^8/EI

Total: 0.25 x 10^8/EI

---

**Timber beam calculation to BS EN1995-1-1 using C24 timber**

**Use 47 x 147 C24 2.9 kg/m approx**

- \( W_{el,y} = 169.3 \text{ cm}^3 \)
- \( I_y = 1,244 \text{ cm}^4 \)
- Depth factor, \( k_{h,y} = (150/147)^{0.2} = 1.00 \) [EC5 eq.(3.1)]

**Timber grade: C24**

- Grade bending stress, \( f_{m,k} = 24.0 \text{ N/mm}^2 \)
- Grade shear stress, \( f_{v,k} = 2.5 \text{ N/mm}^2 \)

**Material partial factor, \( \gamma_M = 1.3 \) [EC5 UK Table NA.3]**

- \( E_{0.05} = 7,400 \text{ N/mm}^2 \)
- \( E_{0,mean} = 11,000 \text{ N/mm}^2 \)

**Loading modification factor, \( k_{mod} = 0.6 \) (Service class 1; Live load duration: Permanent) [EC5 Tables 2.2/3.1]**

**Load sharing factor, \( k_{sys} = 1.0 \)**

**Deflection modification factor, \( k_{def} = 0.60 \) [EC5 Table 3.2]**

**Bending**

- Design bending strength, \( f_{m,y,d} = f_{m,k}k_{mod}k_{h,y}k_{sys}/\gamma_M = 24.0 \times 0.60 \times 1.00 \times 1.0/1.30 = 11.12 \text{ N/mm}^2 \)
- Design bending stress, \( \sigma_{m,y,d} = 1.04 \times 1000/169.3 = 6.12 \text{ N/mm}^2 \) OK
- Bending resistance = 11.12 x 169.3/1000 = 1.88 kNm

**Shear**

- Design shear strength, \( f_{v,d} = f_{v,k}k_{mod}k_{sys}/\gamma_M = 2.50 \times 0.60 \times 1.0/1.30 = 1.15 \text{ N/mm}^2 \)
- Design shear stress, \( \sigma_{v,y,d} = 2.30 \times 1000 \times 3/(2 \times 47 \times 147) = 0.50 \text{ N/mm}^2 \) OK
- Shear resistance = 47 x 147 x 1.15 x 2/(3 x 1000) = 5.31 kN

**Deflection**

- Final deflection limit = 0.003L = 5.40 mm
Instantaneous mid-span shear deflection = 1.2 x 0.737 x 10^6/((E/16) x 47 x 147) = 0.19 mm

Final shear deflection is assumed to increase in proportion to total bending deflection

<table>
<thead>
<tr>
<th>Mid-span deflections:</th>
<th>x 1e8/EI</th>
<th>Inst. mm</th>
<th>(\psi_2)</th>
<th>k_{inf}</th>
<th>Fin. mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dead:</td>
<td>0.16</td>
<td>1.14</td>
<td>1.00</td>
<td>0.60</td>
<td>1.82</td>
</tr>
<tr>
<td>Live QH:</td>
<td>0.09</td>
<td>0.68</td>
<td>0.00</td>
<td>0.60</td>
<td>0.68</td>
</tr>
<tr>
<td>Shear deflection:</td>
<td>0.19</td>
<td></td>
<td></td>
<td></td>
<td>0.26</td>
</tr>
<tr>
<td>Total mm:</td>
<td>2.00</td>
<td></td>
<td></td>
<td></td>
<td>2.76</td>
</tr>
</tbody>
</table>
**4 Point Plans Ltd**  
Parville, Church Steet, Barton St David, Somerset TA11 6BU

Site: 1 Green Bushes, Turners Hill, Crawley, RH10 7SL  
Job: Two Storey Side Extension  
Job number: 16048  
Client copy  

**EuroBeam 1.20a 100086**

<table>
<thead>
<tr>
<th>Beam: RB2</th>
<th>Span: 1.8 m.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load name</td>
<td>Loading w1</td>
</tr>
<tr>
<td>U G o.w.</td>
<td>0.15</td>
</tr>
<tr>
<td>U G Existing Roof</td>
<td>1.88</td>
</tr>
<tr>
<td>U QH Existing Roof</td>
<td>1.70</td>
</tr>
<tr>
<td>P G Beam: RB1 : R2</td>
<td>1.03</td>
</tr>
<tr>
<td>P QH Beam: RB1 : R2</td>
<td>0.61</td>
</tr>
</tbody>
</table>

Total load (unfactored): 8.35 kN  
(6.10): 11.83 kN

| Load types: U:UDL; P:Point load;  Load positions are measured in m. from R1  |
| Load durations: G:Dead; Qx: Imposed; QA: Residential; QH: Roof |

Maximum B.M. = 3.18 kNm (6.10) at 0.90 m. from R1  
Maximum S.F. = -5.91 kN (6.10) at R2  
Mid-span deflections:  
Dead: 0.40 x 10^\text{8}/EI (E in N/mm^2, I in cm^4)  
Live: 0.31 x 10^\text{8}/EI  
Total: 0.71 x 10^\text{8}/EI

**Timber beam calculation to BS EN1995-1-1 using C24 timber**

**Use 2no 47 x 147 C24 5.8 kg/m approx**

- $W_{el,y} = 338.5 \text{ cm}^3$, $l_y = 2,488 \text{ cm}^4$  
- Depth factor, $k_{h,y} = (150/147)^{0.2} = 1.00$ [EC5 eq.(3.1)]

**Timber grade: C24**

- Grade bending stress, $f_{m,k} = 24.0 \text{ N/mm}^2$  
- Grade shear stress, $f_{v,k} = 2.5 \text{ N/mm}^2$  
- Material partial factor, $\gamma_m = 1.3$ [EC5 UK Table NA.3]  
- $E_0.05 = 7,400 \text{ N/mm}^2$; $E_0,\text{mean} = 11,000 \text{ N/mm}^2$  
- Loading modification factor, $k_{mod} = 0.6$ (Service class 1; Live load duration: Permanent) [EC5 Tables 2.2/3.1]  
- Load sharing factor, $k_{sys} = 1.0$  
- Deflection modification factor, $k_{def} = 0.60$ [EC5 Table 3.2]

**Bending**

- Design bending strength, $f_{m,y,d} = f_{m,k}k_{mod}k_k\gamma_m = 24.0 \times 0.60 \times 1.00 \times 1.0/1.30 = 11.12 \text{ N/mm}^2$  
- Design bending stress, $\sigma_{m,y,d} = 3.18 \times 1000/338.5 = 9.39 \text{ N/mm}^2$ OK  
- Bending resistance = 11.12 x 338.5/1000 = 3.77 kNm  

**Shear**

- Design shear strength, $f_{v,d} = f_{v,k}k_{sys}/\gamma_m = 2.50 \times 0.60 \times 1.0/1.30 = 1.15 \text{ N/mm}^2$  
- Design shear stress, $\sigma_{v,y,d} = 5.91 \times 1000 / (2 \times 94 \times 147) = 0.64 \text{ N/mm}^2$ OK  
- Shear resistance = 94 x 147 x 1.15 x 2/(3 x 1000) = 10.6 kN

**Shear force and bending moment**

**Timber beam loading**

<table>
<thead>
<tr>
<th>Beam loading</th>
<th>2.34 (D)</th>
<th>1.84 (L)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.03 (D)</td>
<td>0.61 (L)</td>
<td>1.64</td>
</tr>
</tbody>
</table>

**Total kN**

- Dead
- Live

### Shear force and bending moment graph

- S.F. kN
- B.M. kNm

---

N.B. Thousands separator: ','  Decimal separator: '.'
Deflection

Final deflection limit = 0.003L = 5.40 mm

Instantaneous mid-span shear deflection = \(1.2 \times 2.25 \times 10^6/(E/16) \times 94 \times 147\) = 0.28 mm

Final shear deflection is assumed to increase in proportion to total bending deflection

<table>
<thead>
<tr>
<th>Mid-span deflections: (x 1e8/EI)</th>
<th>Inst. mm</th>
<th>(\psi_2)</th>
<th>(k_{def})</th>
<th>Fin. mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dead:</td>
<td>0.40</td>
<td>1.47</td>
<td>1.00</td>
<td>0.60</td>
</tr>
<tr>
<td>Live QH:</td>
<td>0.31</td>
<td>1.12</td>
<td>0.00</td>
<td>0.60</td>
</tr>
<tr>
<td>Shear deflection:</td>
<td>0.28</td>
<td></td>
<td>0.38</td>
<td></td>
</tr>
<tr>
<td>Total mm.</td>
<td>2.87</td>
<td></td>
<td>3.85</td>
<td>OK</td>
</tr>
</tbody>
</table>
Beam: GB1  
\[ \text{Span: 1.6 m.} \]

<table>
<thead>
<tr>
<th>Load name</th>
<th>Loading w1</th>
<th>Start x1</th>
<th>Loading w2</th>
<th>End x2</th>
<th>R1comp</th>
<th>R2comp</th>
</tr>
</thead>
<tbody>
<tr>
<td>U G o.w.</td>
<td>0.15</td>
<td>0</td>
<td></td>
<td></td>
<td>0.12</td>
<td>0.12</td>
</tr>
<tr>
<td>U G Cat-Slide Roof</td>
<td>2</td>
<td>0</td>
<td></td>
<td></td>
<td>1.60</td>
<td>1.60</td>
</tr>
<tr>
<td>U QH Cat-Slide Roof</td>
<td>1.01</td>
<td>0</td>
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<td></td>
<td>0.81</td>
<td>0.81</td>
</tr>
<tr>
<td>U G Porch Roof</td>
<td>0.73</td>
<td>0</td>
<td></td>
<td></td>
<td>0.58</td>
<td>0.58</td>
</tr>
<tr>
<td>U QH Porch Roof</td>
<td>0.49</td>
<td>0</td>
<td></td>
<td></td>
<td>0.39</td>
<td>0.39</td>
</tr>
</tbody>
</table>

Total load (unfactored): 7.01 kN  
(6.10): 9.82 kN 

Load types: U:UDL; Load positions are measured in m. from R1  
Load durations: G:Dead; Qx: Imposed; QH: Roof  

Maximum B.M. = 1.96 kNm (6.10) at 0.80 m. from R1  
Maximum S.F. = 4.91 kN (6.10) at R1  
Mid-span deflections: Dead: \(0.25 \times 10^{-8}/EI\) (\(E\) in N/mm\(^2\), \(I\) in cm\(^4\))  
Live: \(0.13 \times 10^{-8}/EI\)  
Total: \(0.37 \times 10^{-8}/EI\)

---

**Timber beam calculation to BS EN1995-1-1 using C24 timber**  
**Use 2no 47 x 147 C24**  
5.8 kg/m approx  
\(W_{e,y} = 338.5\, \text{cm}^3\)  
\(I_y = 2,488\, \text{cm}^4\)  
Depth factor, \(k_{h,y} = (150/147)^{0.2} = 1.00\) [EC5 eq.(3.1)]  

**Timber grade: C24**  
Grade bending stress, \(f_{m,k} = 24.0\, \text{N/mm}^2\)  
Grade shear stress, \(f_{v,k} = 2.5\, \text{N/mm}^2\)  
Material partial factor, \(\gamma_M = 1.3\) [EC5 UK Table NA.3]  
\(E_{0.05} = 7,400\, \text{N/mm}^2\);  
\(E_{0,\text{mean}} = 11,000\, \text{N/mm}^2\)  
Loading modification factor, \(k_{\text{mod}} = 0.6\) (Service class 1; Live load duration: Permanent) [EC5 Tables 2.2/3.1]  
Load sharing factor, \(k_{\text{sys}} = 1.0\)  
Deflection modification factor, \(k_{\text{def}} = 0.60\) [EC5 Table 3.2]  

**Bending**  
Design bending strength, \(f_{m,y,d} = f_{m,k}k_{\text{mod}}k_{\text{sys}}\gamma_M = 24.0 \times 0.60 \times 1.0 \times 1.30 = 11.12\, \text{N/mm}^2\)  
Design bending stress, \(\sigma_{m,y,d} = 1.96 \times 1000/338.5 = 5.80\, \text{N/mm}^2\) OK  
Bending resistance = \(11.12 \times 338.5/1000 = 3.77\, \text{kNm}\)  

**Shear**  
Design shear stress, \(f_{v,y,d} = f_{v,k}k_{\text{sys}}\gamma_M = 2.50 \times 0.60 \times 1.0/1.30 = 1.15\, \text{N/mm}^2\)  
Design shear stress, \(\sigma_{v,y,d} = 4.91 \times 1000 \times 3/(2 \times 94 \times 147) = 0.53\, \text{N/mm}^2\) OK  
Shear resistance = 94 x 147 x 1.15 x 2/(3 x 1000) = 10.6 kN
Deflection

- Final deflection limit = 0.003L = 4.80 mm
- Instantaneous mid-span shear deflection = \(1.2 \times 1.40 \times 10^6 / ((E/16) \times 94 \times 147)\) = 0.18 mm
- Final shear deflection is assumed to increase in proportion to total bending deflection

<table>
<thead>
<tr>
<th></th>
<th>Inst. mm</th>
<th>(\psi)</th>
<th>(k_{def})</th>
<th>Fin. mm</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Dead:</strong></td>
<td>0.25</td>
<td>0.90</td>
<td>1.00</td>
<td>0.60</td>
</tr>
<tr>
<td><strong>Live QH:</strong></td>
<td>0.13</td>
<td>0.47</td>
<td>0.00</td>
<td>0.60</td>
</tr>
<tr>
<td><strong>Shear deflection:</strong></td>
<td>0.18</td>
<td>0.47</td>
<td></td>
<td>0.25</td>
</tr>
<tr>
<td><strong>Total mm.</strong></td>
<td>1.54</td>
<td></td>
<td></td>
<td>2.15</td>
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</table>
Beam: GB2

<table>
<thead>
<tr>
<th>Load name</th>
<th>Loading w1</th>
<th>Start x1</th>
<th>Loading w2</th>
<th>End x2</th>
<th>R1comp</th>
<th>R2comp</th>
</tr>
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<tbody>
<tr>
<td>U G o.w.</td>
<td>0.25</td>
<td>0</td>
<td>L</td>
<td>0.52</td>
<td>0.52</td>
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<tr>
<td>U G New Roof</td>
<td>3.11</td>
<td>0</td>
<td>L</td>
<td>6.53</td>
<td>6.53</td>
<td></td>
</tr>
<tr>
<td>U QH New Roof</td>
<td>2.1</td>
<td>0</td>
<td>L</td>
<td>4.41</td>
<td>4.41</td>
<td></td>
</tr>
<tr>
<td>U G New Floor</td>
<td>95</td>
<td>0</td>
<td>L</td>
<td>1.99</td>
<td>1.99</td>
<td></td>
</tr>
<tr>
<td>U QA New Floor</td>
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<td>5.98</td>
<td>5.98</td>
<td></td>
</tr>
<tr>
<td>U G Partitions</td>
<td>96</td>
<td>0</td>
<td>L</td>
<td>2.02</td>
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<td></td>
</tr>
<tr>
<td>R G Wall Over</td>
<td>2.53</td>
<td>1</td>
<td>L</td>
<td>3.08</td>
<td>5.01</td>
<td></td>
</tr>
<tr>
<td>R G Wall Over - Inner</td>
<td>5.28</td>
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<td>1</td>
<td>4.65</td>
<td>0.63</td>
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</tr>
<tr>
<td>R G Wall Over - Outer</td>
<td>5.28</td>
<td>0</td>
<td>1</td>
<td>4.65</td>
<td>0.63</td>
<td></td>
</tr>
<tr>
<td>P G Beam: GB1 : R1</td>
<td>2.3</td>
<td>2.7</td>
<td>0.82</td>
<td>1.48</td>
<td></td>
<td></td>
</tr>
<tr>
<td>P QH Beam: GB1 : R1</td>
<td>1.2</td>
<td>2.7</td>
<td>0.43</td>
<td>0.77</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Total load (unfactored): 65.1 kN 35.10 (6.10) 91.2 kN 49.01 (42.15)

Load types: U:UDL; R:Part UDL; P:Point load; Load positions are measured in m. from R1
Load durations: G:Dead; Qx: Imposed; QA: Residential; QH: Roof

Maximum B.M. = 46.0 kNm (6.10) at 2.13 m. from R1
Maximum S.F. = 49.0 kN (6.10) at R1
Mid-span deflections: Dead: 39.0 x 10^8/EI (E in N/mm^2, I in cm^4)
Live: 21.7 x 10^8/EI
Total: 60.7 x 10^8/EI

Beam calculation to BS EN1993.1.1 using S275 steel

**SECTION SIZE : 203 x 133 x 25 UB S275** (Class 1, plastic)
D=203.2 mm B=133.2 mm t=5.7 mm T=7.8 mm I_x=2,340 cm^4 i_y=3.10 cm W_{pl,y}=258 cm^3 W_{el,y}=230 cm^3
Classification: Flange: c/t = 56.1/7.8 = 7.20 <= 9 (8.32): Class 1, plastic
Web: c/t = 172.4/5.7 = 30.2 <= 72 (66.6): Class 1, plastic

**Shear**
Design shear force, V_{Ed} = 49.0 kN
Shear resistance, V_{pl,Rd} = (A_v f_v/3) / γ_M = (1,285 x 275/1000) / 1.0 = 204 kN (>49.0) OK [EC3 6.2.6]
Shear buckling: h_x/I_x = 187.6/5.7 = 32.91 <= 72_x (66.6): check not required [EC3 6.2.6(6)]

**Moment resistance**
Design moment, M_{Ed} = 45.99 kNm
Moment resistance, M_{pl,Rd} = f_v W_{pl,y} = 275 x 258/1000 = 70.95 kNm OK
Beam has full lateral restraint: check for lateral-torsional buckling not required

**Combined bending and shear**
V_{Ed} = 0.5 V_{c,Rd} : Check for bending/shear interaction not required [EC3 6.2.8(2)]
Web capacity at bearings

Resistance of web to transverse forces, $F_{Rd} = f_{yw} \cdot L_{eff} \cdot t_y / \gamma_M$

$$f_{yw} = 275 \text{ N/mm}^2$$

$$\lambda_F = \sqrt{\frac{L_{eff} \cdot f_{yw}}{2 \cdot t_y}}$$

$$F_c = 0.9 \cdot K \cdot (t_y / h_w)$$

Type (c) load application assumed:

$$I_{y} = \min \left( S_{b} + 2l_y(1 + x(m_1 + m_2)), l_y + t_y(m_1/2 + (l_y/l)^2 + m_2) \right)$$

$$k_F = 2 + 6((S_{b} + c)/h_w)^2$$

$$m_1 = f_{yw} \cdot b / (f_{yw} \cdot t_y)$$

$$m_2 = 0.02(h_w / t_y)^2$$

$$\lambda_F > 0.5$$ else 0.0

Reaction R1: 49.01 kN

Required minimum stiff bearing length, $S_{b} = 10$ mm

$\lambda_F = 0.373$

$$L_{eff} = 38.48$$

$$F_{R,d} = 275 \times 38.48 \times 5.7 / (1000 \times 1.0) = 60.3 \text{ kN OK}$$

Reaction R2: 42.15 kN

Required minimum stiff bearing length, $S_{b} = 10$ mm

$\lambda_F = 0.373$

$$L_{eff} = 38.48$$

$$F_{R,d} = 275 \times 38.48 \times 5.7 / (1000 \times 1.0) = 60.3 \text{ kN OK}$$

If the stiff bearing length(s) above cannot be provided, stiffeners will be required

Deflection

LL deflection = 21.70 x 1e8 / (210,000 x 2,340) = 4.4 mm (L/951) OK

TL deflection = 60.72 x 1e8 / (210,000 x 2,340) = 12.4 mm (L/340)

Bearing details

203x133x25 UB stiff bearing length, $b_1 = l + 1.6r + 2T = 33.5$ mm

R1: 300 x 275 x 150h mm padstone

Masonry: 5N/mm² brick, M2 mortar, category II mfr, class 2 execution,

Shape factor, $\delta = 0.850$  
Normalised compressive strength, $f_c = 5.00 \times 0.850 = 4.25$

Characteristic compressive strength, $f_{ck} = 0.50f_{ck}^{0.70}, f_{ck}^{0.30} = 1.69$ N/mm²

Design compressive strength, $f_d = f_{ck} / \gamma_m = 1.69 / 3.0 = 0.565$ N/mm² x 1.200 = 0.678 N/mm²

Stress under padstone = 49.01 x 1000/300 x 275 = 0.59 N/mm² OK

R2: 300 x 100 x 150h mm padstone

Masonry: 7.3N/mm² 100mm block, M4 mortar, category II mfr, class 2 execution,

Shape factor, $\delta = 1.38$  
Normalised compressive strength, $f_c = 7.30 \times 1.38 = 10.1$

Characteristic compressive strength, $f_{ck} = 0.55f_{ck}^{0.70}, f_{ck}^{0.30} = 4.20$ N/mm²

Design compressive strength, $f_d = f_{ck} / \gamma_m = 4.20 / 3.0 = 1.40$ N/mm² x 1.200 = 1.68 N/mm²

Stress under padstone = 42.15 x 1000/300 x 100 = 1.40 N/mm² OK
Beam: GB3

<table>
<thead>
<tr>
<th>Load name</th>
<th>Loading w1</th>
<th>Start x1</th>
<th>Loading w2</th>
<th>End x2</th>
<th>R1comp</th>
<th>R2comp</th>
</tr>
</thead>
<tbody>
<tr>
<td>U G o.w.</td>
<td>0.25</td>
<td>0</td>
<td>L</td>
<td>0.21</td>
<td>0.21</td>
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</tr>
<tr>
<td>U G Existing Floor</td>
<td>0.75</td>
<td>0</td>
<td>L</td>
<td>0.64</td>
<td>0.64</td>
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</tr>
<tr>
<td>U QA Existing Floor</td>
<td>2.25</td>
<td>0</td>
<td>L</td>
<td>1.91</td>
<td>1.91</td>
<td></td>
</tr>
<tr>
<td>U G Partitions</td>
<td>0.96</td>
<td>0</td>
<td>L</td>
<td>0.82</td>
<td>0.82</td>
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<tr>
<td>R G Wall Over</td>
<td>2.53</td>
<td>1</td>
<td>L</td>
<td>0.36</td>
<td>1.41</td>
<td></td>
</tr>
</tbody>
</table>

Total load (unfactored): 8.93 kN

Load types: U:UDL; R:Part UDL; Load positions are measured in m. from R1

Load durations: G:Dead; Qx: Imposed; QA: Residential

Maximum B.M. = 2.61 kNm (6.10) at 0.94 m. from R1
Maximum S.F. = -7.02 kN (6.10) at R2

Mid-span deflections: Dead: 0.31 x 10^8/EI (E in N/mm^2, I in cm^4)
Live: 0.24 x 10^8/EI
Total: 0.56 x 10^8/EI

Timber beam calculation to BS EN1995-1-1 using C24 timber

Use 2no 47 x 147 C24 5.8 kg/m approx

W_{el,y} = 338.5 cm^3  I_y = 2,488 cm^4  Depth factor, k_{h,y} = (150/147)^0.2 = 1.00  [EC5 eq.(3.1)]

Timber grade: C24

Grade bending stress, f_{m,k} = 24.0 N/mm^2
Grade shear stress, f_{v,k} = 2.5 N/mm^2

Material partial factor, γ_M = 1.3  [EC5 UK Table NA.3]

E_{0,05} = 7,400 N/mm^2;  E_{0,mean} = 11,000 N/mm^2

Loading modification factor, k_{mod} = 0.6 (Service class 1; Live load duration: Permanent)  [EC5 Tables 2.2/3.1]

Load sharing factor, k_{sys} = 1.0
Deflection modification factor, k_{def} = 0.60  [EC5 Table 3.2]

Bending

Design bending strength, f_{m,y,d} = f_{m,k}k_{mod}k_{sys}k_{def}/γ_m = 24.0 x 0.60 x 1.0 x 1.0/1.30 = 11.12 N/mm^2
Design bending stress, σ_{m,y,d} = 2.61 x 1000/338.5 = 7.72 N/mm^2 OK

Bending resistance = 11.12 x 338.5/1000 = 3.77 kNm

Shear

Design shear stress, f_{v,d} = f_{v,k}k_{mod}k_{sys}k_{def}/γ_m = 2.50 x 0.60 x 1.0/1.30 = 1.15 N/mm^2
Design shear stress, σ_{v,y,d} = 7.02 x 1000 x 3/(2 x 94 x 147) = 0.76 N/mm^2 OK

Shear resistance = 94 x 147 x 1.15 x 2/(3 x 1000) = 10.6 kN
Deflection

Final deflection limit = 0.003L = 5.10 mm

Instantaneous mid-span shear deflection = \(1.2 \times 1.83 \times 10^6/((E/16) \times 94 \times 147)\) = 0.23 mm

Final shear deflection is assumed to increase in proportion to total bending deflection

<table>
<thead>
<tr>
<th>Mid-span deflections: x 1e8/EI</th>
<th>Inst. mm</th>
<th>(\psi_2)</th>
<th>(k_{def})</th>
<th>Fin. mm</th>
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</thead>
<tbody>
<tr>
<td>Dead</td>
<td>0.31</td>
<td>1.14</td>
<td>1.00</td>
<td>0.60</td>
</tr>
<tr>
<td>Live QA</td>
<td>0.24</td>
<td>0.89</td>
<td>0.30</td>
<td>0.60</td>
</tr>
<tr>
<td>Shear deflection</td>
<td>0.23</td>
<td>0.33</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total mm.</td>
<td>2.27</td>
<td>3.21</td>
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</table>
Beam: GB4

<table>
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<th>Load name</th>
<th>Loading w1</th>
<th>Start x1</th>
<th>Loading w2</th>
<th>End x2</th>
<th>R1comp</th>
<th>R2comp</th>
</tr>
</thead>
<tbody>
<tr>
<td>U</td>
<td>0.15</td>
<td>0</td>
<td>L</td>
<td>0.14</td>
<td>0.14</td>
<td></td>
</tr>
<tr>
<td>U</td>
<td>3.11</td>
<td>0</td>
<td>L</td>
<td>2.80</td>
<td>2.80</td>
<td></td>
</tr>
<tr>
<td>P G</td>
<td>Beam: RB1 : R1</td>
<td>1.03</td>
<td>L/2</td>
<td>0.51</td>
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<tr>
<td>P QH</td>
<td>Beam: RB1 : R1</td>
<td>0.61</td>
<td>L/2</td>
<td>0.31</td>
<td>0.31</td>
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</tr>
</tbody>
</table>

Total load (unfactored): 7.51 kN

Load types: U:UDL; P:Point load; Load positions are measured in m. from R1

Maximum B.M. = 2.82 kNm (6.10) at 0.90 m. from R1
Maximum S.F. = 5.11 kN (6.10) at R1
Mid-span deflections: Dead: 0.57 x 10^8/EI (E in N/mm^2, I in cm^4)
Live: 0.07 x 10^8/EI
Total: 0.64 x 10^8/EI

Timber beam calculation to BS EN1995-1-1 using C24 timber

Use 2no 47 x 147 C24 5.8 kg/m approx

\[W_{d,y} = 338.5 \text{ cm}^3, I_y = 2488 \text{ cm}^4\] Depth factor, \(k_{y,y} = (150/147)^{0.2} = 1.00\) [EC5 eq.(3.1)]

Timber grade: C24

Grade bending stress, \(f_m,k = 24.0 \text{ N/mm}^2\)
Grade shear stress, \(f_v,k = 2.5 \text{ N/mm}^2\)
Material partial factor, \(\gamma_M = 1.3\) [EC5 UK Table NA.3]
\(E_{0.05} = 7400 \text{ N/mm}^2; E_{0,\text{mean}} = 11000 \text{ N/mm}^2\)
Loading modification factor, \(k_{mod} = 0.6\) (Service class 1; Live load duration: Permanent) [EC5 Tables 2.2/3.1]
Load sharing factor, \(k_{sys} = 1.0\)
Deflection modification factor, \(k_{def} = 0.60\) [EC5 Table 3.2]

Bending

Design bending strength, \(f_{m,y,d} = f_m,k \cdot k_{mod} \cdot k_{y,y} \cdot k_{sys}/\gamma_m = 24.0 \times 0.60 \times 1.00 \times 1.0/1.30 = 11.12 \text{ N/mm}^2\)
Design bending stress, \(\sigma_{m,y,d} = 2.82 \times 1000/338.5 = 8.33 \text{ N/mm}^2\) OK
Bending resistance = 11.12 x 338.5/1000 = 3.77 kNm

Shear

Design shear strength, \(f_v,d = f_v,k \cdot k_{mod} \cdot k_{sys}/\gamma_m = 2.50 \times 0.60 \times 1.0/1.30 = 1.15 \text{ N/mm}^2\)
Design shear stress, \(\sigma_{v,y,d} = 5.11 \times 1000 \times 3/(2 \times 94 \times 147) = 0.56 \text{ N/mm}^2\) OK
Shear resistance = 94 x 147 x 1.15 x 2/(3 x 1000) = 10.6 kN

Deflection
Final deflection limit = 0.003L = 5.40 mm

Instantaneous mid-span shear deflection = $1.2 \times 2.06 \times 10^6 / ((E/16) \times 94 \times 147) = 0.26$ mm

*Final shear deflection is assumed to increase in proportion to total bending deflection*

<table>
<thead>
<tr>
<th>Mid-span deflections: x 1e8/EI</th>
<th>Inst. mm</th>
<th>ψ₀</th>
<th>k_{def}</th>
<th>Fin. mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dead:</td>
<td>0.57</td>
<td>2.09</td>
<td>1.00</td>
<td>0.60</td>
</tr>
<tr>
<td>Live OH:</td>
<td>0.07</td>
<td>0.27</td>
<td>0.00</td>
<td>0.60</td>
</tr>
<tr>
<td>Shear deflection:</td>
<td>0.26</td>
<td></td>
<td>0.00</td>
<td>0.60</td>
</tr>
<tr>
<td>Total mm.</td>
<td>2.62</td>
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<td>4.01</td>
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Beam: FB1

<table>
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<tr>
<th>Load name</th>
<th>Loading w1</th>
<th>Start x1</th>
<th>Loading w2</th>
<th>End x2</th>
<th>R1comp</th>
<th>R2comp</th>
</tr>
</thead>
<tbody>
<tr>
<td>U G o.w.</td>
<td>0.4</td>
<td>0</td>
<td>L</td>
<td>0.76</td>
<td>0.76</td>
<td></td>
</tr>
<tr>
<td>U G 1st Floor</td>
<td>.95</td>
<td>0</td>
<td>L</td>
<td>1.80</td>
<td>1.80</td>
<td></td>
</tr>
<tr>
<td>U QA 1st Floor</td>
<td>2.85</td>
<td>0</td>
<td>L</td>
<td>5.41</td>
<td>5.41</td>
<td></td>
</tr>
<tr>
<td>U G Gnd Floor</td>
<td>.95</td>
<td>0</td>
<td>L</td>
<td>1.80</td>
<td>1.80</td>
<td></td>
</tr>
<tr>
<td>U QA Gnd Floor</td>
<td>2.85</td>
<td>0</td>
<td>L</td>
<td>5.41</td>
<td>5.41</td>
<td></td>
</tr>
<tr>
<td>U G Gnd Wall Over - Inner</td>
<td>3.24</td>
<td>0</td>
<td>L</td>
<td>6.16</td>
<td>6.16</td>
<td></td>
</tr>
<tr>
<td>U G Gnd Wall Over - Outer</td>
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<td>L</td>
<td>5.24</td>
<td>5.24</td>
<td></td>
</tr>
<tr>
<td>U G New Roof</td>
<td>3.11</td>
<td>0</td>
<td>L</td>
<td>5.91</td>
<td>5.91</td>
<td></td>
</tr>
<tr>
<td>U QH New Roof</td>
<td>2.1</td>
<td>0</td>
<td>L</td>
<td>3.99</td>
<td>3.99</td>
<td></td>
</tr>
</tbody>
</table>

For spans: 3.8 m.

Total load (unfactored): 97.6 kN
(6.10): 136.2 kN

Load types: U:UDL; Load positions are measured in m. from R1
Load durations: G:Dead; Q: Imposed; QA: Residential; QH: Roof

Maximum B.M. = 64.7 kNm (6.10) at 1.90 m. from R1
Maximum S.F. = 68.1 kN (6.10) at R1
Mid-span deflections: Dead: 48.6 x 10^8 / EI (E in N/mm^2, I in cm^4)
Live: 21.2 x 10^8 / EI
Total: 69.7 x 10^8 / EI

Beam calculation to BS EN1993.1.1 using S275 steel

**SECTION SIZE**: 152 x 152 x 37 UC S275 (Class 1, plastic)

D=161.8 mm  B=154.4 mm  t=8.0 mm  I_y=2,210 cm^4  I_z=3.87 cm  W_{pl,y}=309 cm^3  W_{el,y}=273 cm^3

Classification: Flange: c/t = 65.6/11.5 = 5.70 <= 9  ε(8.32): Class 1, plastic
Web: c/t = 123.6/8.0 = 15.4 <= 72: (66.6): Class 1, plastic

**Shear**

Design shear force, V_{Ed} = 68.1 kN
Shear resistance, V_{pl,Rd} = (A_v * f_y) / 3 / γ_m = ((1,426 x 275 / (3 x 1000)) / 1.0 = 226 kN (>=68.1) OK [EC3 6.2.6]
Shear buckling: h_w/t_w = 138.8/8.0 = 17.35 <= 72: (66.6): check not required [EC3 6.2.6(6)]

**Moment resistance**

Design moment, M_{Ed} = 64.71 kNm
Moment resistance, M_{q,y,Rd} = f_y * W_{pl,y} = 275 x 309/1000 = 84.97 kNm OK
Beam is laterally restrained at supports only: effective length = 1.0L

**Lateral-torsional buckling check**

Design buckling resistance moment, M_{b,Rd} = χ_{LT,mod} * M_{c,Rd}

χ_{LT,mod} = χ_{LT} / L (but <=1/χ_{LT}^2 and <=1.0) [Eq.6.58]
\[
f = 1 - 0.5(1 - k_c)[1 - 2(\lambda_{LT} - 0.8)^2]
\]

6.3.2.3(2) \hspace{1cm} k_c = 1 / C_1 \quad \text{[NA2.18]}

Use buckling curve b: \(\alpha = 0.340\) \hspace{1cm} \text{[EC3 Tables 6.3/6.4 NA 2.17]}

\[
\phi_{LT} = 0.5[1 + \alpha_{LT}(\lambda_{LT} - \lambda_{LT,0}) + \beta\lambda_{LT}^2]
\]

\(\lambda_{LT,0} = 0.4\) \hspace{1cm} \beta = 0.75 \hspace{1cm} \text{[EC3 UK NA 2.17]}

\[
\chi_{LT} = 1 / [\phi_{LT} + \lambda_{LT}^2]
\]

\(\lambda_{LT} = \frac{f_y W_{pl,y} / M_{cr}}{C_1 \left(\pi^2 E I / L^2\right) (I_w / I_z + L^2 G I_T / \pi^2 E I_z)}\)

\(W_{pl,y} = 309.0 \text{ cm}^3\) \hspace{1cm} \(I_w = 0.040 \text{ dm}^6\) \hspace{1cm} \(I_T = 19.2 \text{ cm}^4\) \hspace{1cm} \(G = 81,000 \text{N/mm}^2\)

<table>
<thead>
<tr>
<th>Segment</th>
<th>(M_{Max})</th>
<th>(k)</th>
<th>(C_1)</th>
<th>(M_{cr})</th>
<th>(\chi_{LT})</th>
<th>(\phi_{LT})</th>
<th>(\chi_{LT,mod})</th>
<th>(M_{d,Rd})</th>
<th>(M_{b,Rd})</th>
<th>OK</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00-3.80</td>
<td>64.7</td>
<td>1.00</td>
<td>1.00d</td>
<td>146.7</td>
<td>0.761</td>
<td>0.779</td>
<td>0.838</td>
<td>85.0</td>
<td>71.2</td>
<td>OK</td>
</tr>
</tbody>
</table>

C1 derivation: \(d\) taken as 1.0 (conservative)

Beam is laterally restrained at supports only: effective length = 1.0L

**Combined bending and shear**

\(V_{Ed} \leq 0.5 V_{c,Rd}\) : Check for bending/shear interaction not required \hspace{1cm} \text{[EC3 6.2.8(2)]}

Web capacity at bearings

Resistance of web to transverse forces, \(F_{Rd} = f_{yw} L_{eff} t_w / \gamma_M 1\)

\(f_{yw} = 275 \text{ N/mm}^2\) \hspace{1cm} \(L_{eff} = \chi_F l_y\) \hspace{1cm} \(\chi_F = 0.5 \lambda_{F} \leq 1.0\)

\(F_{cr} = 0.9 k_h E (t_w f_{yw})\) \hspace{1cm} \(k_h = 2 + 6((S_s + c)/h_w)^2 \leq 6\)

Type (c) load application assumed:

\(l_y = \min S_s + 2t_f (1 + (m_1 + m_2)), l_y + t_w (m_1/2 + (l_y/t_f)^2 + m_2)\) or \(l_y + t_w (m_1 + m_2)\)

\(l_y = k_h E t_w^2 / (2 f_{yw} h_w) \leq s_s + c\)

\(m_1 = f_{yw} b_c (f_{yw} t_w)\) \hspace{1cm} \(m_2 = 0.02(h_w t_f)\) \hspace{1cm} \text{if } \lambda_F > 0.5 \text{ else } 0.0\)

Reactions R1 & 2: 68.12 kN

Required minimum stiff bearing length, \(S_s = 0 \text{ mm}\)

(c end of beam to stiff bearing) taken as 0.0

\(m_1 = 19.3 \text{ mm}\) \hspace{1cm} \(m_2 = 0.00 \text{ F}_{cr} = 1,394 \text{ kN}\) \hspace{1cm} \(k_h = 2.00 \text{ l_y} = 0.00 \text{ I}_y = 35.72 \text{ }\lambda_F = 0.237 \text{ }\chi_F = 1.00 \text{ }L_{eff} = 35.72\)

Resistance of web to transverse forces, \(F_{Rd} = 275 \times 35.72 \times 8.0(1000 \times 1.0) = 78.6 \text{ kN}\) \hspace{1cm} \text{OK}

**Deflection**

LL deflection = \(21.18 \times 1 e8/(210,000 \times 2,210) = 4.6 \text{ mm (L/833)}\) \hspace{1cm} \text{OK}

TL deflection = \(69.75 \times 1 e8/(210,000 \times 2,210) = 15.0 \text{ mm (L/253)}\)
Beam: FB2

<table>
<thead>
<tr>
<th>Load name</th>
<th>Loading w1</th>
<th>Start x1</th>
<th>Loading w2</th>
<th>End x2</th>
<th>R1comp</th>
<th>R2comp</th>
</tr>
</thead>
<tbody>
<tr>
<td>U</td>
<td>0.3</td>
<td>0</td>
<td>L</td>
<td>0.57</td>
<td>0.57</td>
<td></td>
</tr>
<tr>
<td>U G</td>
<td>3.24</td>
<td>0</td>
<td>L</td>
<td>6.16</td>
<td>6.16</td>
<td></td>
</tr>
<tr>
<td>U G Gnd Wall Over - Inner</td>
<td>6.48</td>
<td>0</td>
<td>L</td>
<td>12.31</td>
<td>12.31</td>
<td></td>
</tr>
<tr>
<td>U G Gnd Wall Over - Outer</td>
<td>3.57</td>
<td>0</td>
<td>L</td>
<td>6.78</td>
<td>6.78</td>
<td></td>
</tr>
</tbody>
</table>

Total load (unfactored): 51.6 kN

(6.10): 69.7 kN

Maximum B.M. = 33.1 kNm (6.10) at 1.90 m. from R1

Maximum S.F. = 34.9 kN (6.10) at R1

Mid-span deflections: Dead: 36.9 x 10^-8 /EI (E in N/mm^2, I in cm^4)

Live: 0.0 x 10^-8 /EI

Total: 36.9 x 10^-8 /EI

Beam calculation to BS EN1993.1.1 using S275 steel

SECTION SIZE: 152 x 152 x 23 UC S275 (Class 3, semi-compact)

D=152.4 mm B=152.2 mm t=5.8 mm I_y=1,250 cm^4 I_z=3.70 cm W_{pl,y}=182 cm^3 W_{el,y}=164 cm^3

Classification: Flange: c/t = 65.6/6.8 = 9.65 <= 14 Class 3, semi-compact

Table 5.2 Web: c/t = 123.6/5.8 = 21.3 <= 72: Class 1, plastic

Shear

Design shear force, V_{Ed} = 34.9 kN

Shear resistance, V_{pl,Rd} = (A_c f_y / 3) / f_0 = (992.9 x 275 / (6.3 x 1000)) / 1.0 = 158 kN (>=34.9) OK [EC3 6.2.6]

Shear buckling: h_w/t_w = 138.8/5.8 = 23.93 <= 72: (66.56) check not required

Moment resistance

Design moment, M_{Ed} = 33.12 kNm

Moment resistance, M_{pl,Rd} = f_y W_{pl,y} = 275 x 164/1000 = 45.10 kNm OK

Beam is laterally restrained at supports only: effective length = 1.0L

Lateral-torsional buckling check

Design buckling resistance moment, M_{b,Rd} = \chi_{LT,Rd} M_c,Rd

\chi_{LT,Rd} = \chi_{LT,mod} (but <= 1/f_{LT}^2 and <=1.0) [Eq.6.58]

f = 1 - 0.5(1-k_c) [1 - 2(\chi_{LT} - 0.8)] \geq 6.3.2.3(2) k_c = 1/C_1 [NA2.18]

Use buckling curve b: \alpha = 0.340 [EC3 Tables 6.3/6.4 NA 2.17]

\chi_{LT} = 1/[(\phi_L + \gamma(\phi_T^2 - \beta \chi_{LT}^2))] [EC3 (6.56)]

\phi_L = 0.5 [1 + \alpha_L (\chi_{LT} - \chi_{LT,0}) + \beta \chi_{LT}^2]

\chi_{LT,0} = 0.4 \ \beta = 0.75 [EC3 UK NA 2.17]
\[ \lambda_{LT} = \frac{f_y W_{pl,y}}{M_{cr}} \]

\[ M_{cr} = C_1 \left( \frac{\pi^2 E I_z}{L^2} \right) + \left( I_w/I_z + \frac{L^2 G I_T}{\pi^2 E I_z} \right) \]  
SN003

\[ W_f = 164.0 \text{ cm}^3 \quad I_w = 0.021 \text{ dm}^6 \quad I_T = 4.63 \text{ cm}^4 \quad G = 81,000 \text{ N/mm}^2 \]

<table>
<thead>
<tr>
<th>Segment</th>
<th>( M_{Max} )</th>
<th>( k )</th>
<th>( C_t )</th>
<th>( M_{cr} )</th>
<th>( \lambda_{LT} )</th>
<th>( \phi_{LT} )</th>
<th>( \chi_{LT} )</th>
<th>( \chi_{LT-mod} )</th>
<th>( M_{cr,d} )</th>
<th>( M_{cr,d} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00-3.80</td>
<td>33.1</td>
<td>1.00</td>
<td>1.00d</td>
<td>62.3</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>48.8</td>
<td>1.00</td>
<td>36.6</td>
</tr>
</tbody>
</table>

Beam is laterally restrained at supports only: effective length = 1.0L

**Combined bending and shear**

\[ V_{Ed} \leq 0.5 V_{c,Rd} : \text{Check for bending/shear interaction not required} \]  
[EC3 6.2.8(2)]

**Web capacity at bearings**

Resistance of web to transverse forces, \( F_{Rd} = f_{yw} L_{eff} t_w/h_w \)

\[ f_{yw} = 275 \text{ N/mm}^2 \]

\[ \chi_F = 0.5/\lambda_F \leq 1.0 \]

\[ \lambda_F = \sqrt{l_y t_w f_{yw}/F_{cr}} \]

\[ F_{cr} = 0.9 k_F E (t_w^3/h_w) \]

\[ k_F = 2 + 6 ((S_s + c)/h_w)^2 \leq 6 \]

Type (c) load application assumed:

\[ l_y = \min S_s + 2(l_y + t_y(m_1 + m_2)) \]
\[ l_y + t_y(m_1/2 + (l_y/t_y)^2 + m_2) \] or \( l_y + t_y(m_1 + m_2) \)

\[ l_y = k_s E t_y^2/(2 f_{yw} h_w) \leq s_c + c \]

\[ m_1 = f_{yw} b_y / f_w \quad m_2 = 0.02 (h_w / t_y)^2 \] if \( \lambda_F > 0.5 \)

Reactions R1 & 2: 34.86 kN

Required minimum stiff bearing length, \( S_s = 0 \) mm

\( c \) (end of beam to stiff bearing) taken as 0.0

\( m_1 = 28.2 \quad m_2 = 0.00 \quad F_{cr} = 531 \text{ kN} \quad k_F = 2.00 \quad l_y = 0.00 \quad l_y = 24.63 \quad \lambda_c = 0.272 \quad \chi_c = 1.00 \quad L_{eff} = 24.63 \)

Resistance of web to transverse forces, \( F_{R,d} = 275 \times 24.63 \times 5.8/(1000 \times 1.0) = 39.3 \text{ kN} \) OK

**Deflection**

\[ LL \text{ deflection} = 0.000 \times 1e8/(210,000 \times 1,250) = 0.0 \text{ mm} \] OK

\[ TL \text{ deflection} = 36.90 \times 1e8/(210,000 \times 1,250) = 14.1 \text{ mm} \] (L/270)