Masonry Design

Masonry construction uses modular units:

- Brickwork (kiln dried clay bricks) – mainly for facades;
- Blockwork (concrete blocks) – mainly for structural use;
- Stonework (eg. stone arch bridges - not covered here) – usually ornamental.

Its form of construction may be:

- Unreinforced – the usual case;
- Reinforced – very useful for garden walls and piers;
- Prestressed – unusual, see Peter Rice’s Pavilion of the Future in Seville.

We will consider the structural design of unreinforced brick- and block-work.

Bricks

Mostly governed by aesthetic requirements; not normally structural.

- made of clay and kiln dried (hence they expand with moisture);
- very light and strong.

![Brick diagram]

Standard brick size

Format size

Brick Varieties

i. Common (for general building work)
ii. Facing - specially made for their appearance
iii. 'Engineering' - very dense and strong + defined absorption and strength limits
Blocks
The main structural element in masonry.

- Larger than bricks;
- Made from (lean) concrete - wet process (hence they shrink);
- Resistant to moisture.

A larger range of sizes is available, but it is usual in Ireland to use only the 215×100×440 solid block and the 215×215×440 hollow block.

'Work size' = size of block
'Co-ordinating size' = size of block + mortar (corresponds to format size in bricks).

Mortar joints are nominally 10 mm.

Movement joints
Used to allow for local effects of temperature and moisture content.

<table>
<thead>
<tr>
<th>Material</th>
<th>Spacing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay bricks</td>
<td>On plan: up to 12 m c/c (6 m from corners); Vertically: 9 m or every 3 storeys if ( h &gt; 12 ) m or 4 storeys</td>
</tr>
<tr>
<td>Concrete blocks</td>
<td>3 m – 7 m c/c</td>
</tr>
</tbody>
</table>
Masonry Design – Basis

Firstly, it is important to note that Irish masonry construction practice differs significantly from British practice and the Irish masonry design standard IS 325: Part 1: *Code of Practice for the Use of Masonry* is to be considered the superior design code and is recognized as such in the Irish Building Regulations, TGD A.

Partial Factors of Safety for design loads, $\gamma_f$, are:

a) Dead and imposed load
   i) design dead load = 0.9 $G_k$ or 1.4 $G_k$
   ii) design imposed load = 1.6 $Q_k$
   iii) design worst credible earth and water lateral load = 1.2 $E_u$

b) Dead and wind load
   i) design dead load = 0.9 $G_k$ or 1.4 $G_k$
   ii) design wind load = 1.4 $W_k$
   iii) design worst credible earth and water lateral load = 1.2 $E_u$

c) Dead, imposed and wind load
   i) design dead load = 1.2 $G_k$
   ii) design imposed load = 1.2 $Q_k$
   iii) design wind load = 1.2 $W_k$
   iv) design worst credible earth and water lateral load = 1.2 $E_u$

The partial factors of safety for material, $\gamma_m$, is given by the following:

<table>
<thead>
<tr>
<th>Category of manufacturing control of structural units</th>
<th>Special</th>
<th>Normal</th>
</tr>
</thead>
<tbody>
<tr>
<td>Special</td>
<td>2.5</td>
<td>3.1</td>
</tr>
<tr>
<td>Normal</td>
<td>2.8</td>
<td>3.5</td>
</tr>
</tbody>
</table>
**Characteristic Strength of Masonry, \( f_i \)**

The characteristic strength is found from Table 2 of I.S. 325 – next two pages.

This table requires:

1. the shape factor for the unit as laid, given by:

\[
\text{shape factor} = \frac{\text{height as laid}}{\text{thickness as laid}}
\]

The shape factors for usual cases are:

a) Block on flat: \((215\times100\times440)\) – 0.47;

b) Block on edge: \((100\times215\times440)\) – 2.15;

c) Hollow block: \((215\times215\times440)\) – 1.0.
Table 2. Characteristic compressive strength of masonry, $f_c$, in MPa.

(a) Constructed with standard format bricks

<table>
<thead>
<tr>
<th>Mortar designation</th>
<th>Compressive strength of brick determined as shown in relevant I.S. MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>5</td>
</tr>
<tr>
<td>(i)</td>
<td>2.5</td>
</tr>
<tr>
<td>(ii)</td>
<td>2.5</td>
</tr>
<tr>
<td>(iii)</td>
<td>2.5</td>
</tr>
<tr>
<td>(iv)</td>
<td>2.2</td>
</tr>
</tbody>
</table>

(b) Constructed with hollow blocks having an aspect ratio of 1.0.

<table>
<thead>
<tr>
<th>Mortar designation</th>
<th>Compressive strength of block determined on edge per I.S. 20 MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>3.0</td>
</tr>
<tr>
<td>(i)</td>
<td>1.9</td>
</tr>
<tr>
<td>(ii)</td>
<td>1.9</td>
</tr>
<tr>
<td>(iii)</td>
<td>1.9</td>
</tr>
<tr>
<td>(iv)</td>
<td>1.9</td>
</tr>
</tbody>
</table>

(c) Constructed with hollow blocks having an aspect ratio of between 2.0 and 4.0.

<table>
<thead>
<tr>
<th>Mortar designation</th>
<th>Compressive strength of block determined on edge per I.S. 20 MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>3.0</td>
</tr>
<tr>
<td>(i)</td>
<td>3.0</td>
</tr>
<tr>
<td>(ii)</td>
<td>3.0</td>
</tr>
<tr>
<td>(iii)</td>
<td>3.0</td>
</tr>
<tr>
<td>(iv)</td>
<td>3.0</td>
</tr>
</tbody>
</table>
(d) Constructed with solid blocks having an aspect ratio of 1.0.

<table>
<thead>
<tr>
<th>Mortar designation</th>
<th>Compressive strength of block determined on edge per I.S. 20</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>MPa</td>
</tr>
<tr>
<td>(i)</td>
<td>3.2</td>
</tr>
<tr>
<td>(ii)</td>
<td>3.2</td>
</tr>
<tr>
<td>(iii)</td>
<td>3.1</td>
</tr>
<tr>
<td>(iv)</td>
<td>2.8</td>
</tr>
</tbody>
</table>

(e) Constructed from solid concrete blocks having an aspect ratio of between 2.0 and 4.0.

<table>
<thead>
<tr>
<th>Mortar designation</th>
<th>Compressive strength of block determined on edge per I.S. 20</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>MPa</td>
</tr>
<tr>
<td>(i)</td>
<td>5.0</td>
</tr>
<tr>
<td>(ii)</td>
<td>5.0</td>
</tr>
<tr>
<td>(iii)</td>
<td>5.0</td>
</tr>
<tr>
<td>(iv)</td>
<td>4.4</td>
</tr>
</tbody>
</table>

(f) Constructed from solid concrete blocks having an aspect ratio of between 0.4 and 0.5.

<table>
<thead>
<tr>
<th>Mortar designation</th>
<th>Compressive strength of block determined on edge per I.S. 20</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>MPa</td>
</tr>
<tr>
<td>(i)</td>
<td>3.9</td>
</tr>
<tr>
<td>(ii)</td>
<td>3.7</td>
</tr>
<tr>
<td>(iii)</td>
<td>3.6</td>
</tr>
<tr>
<td>(iv)</td>
<td>3.1</td>
</tr>
</tbody>
</table>
2. The appropriate mortar designation: usually taken as (iii):
From these tables we derive a “quick-use” table for $f_k$ in N/mm$^2$ assuming Mortar Designation (iii):

<table>
<thead>
<tr>
<th>Designation (N/mm$^2$)</th>
<th>Solid Block</th>
<th>Hollow Block (SF = 1.0)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>On edge (SF = 2.15)</td>
<td>On flat (SF = 0.47)</td>
</tr>
<tr>
<td>5 (grey)</td>
<td>5.0</td>
<td>3.6</td>
</tr>
<tr>
<td>10 (red)</td>
<td>8.2</td>
<td>5.4</td>
</tr>
<tr>
<td>15 (green)</td>
<td>10</td>
<td>6.6</td>
</tr>
<tr>
<td>20 (black)</td>
<td>11.6</td>
<td>7.9</td>
</tr>
</tbody>
</table>

The colours of the blocks are used to identify different strength blocks on site.
Masonry Design – Axial Capacity

The axial capacity is given by the equation:

\[ N = \beta \cdot \frac{f_k t b}{\gamma_m} \]

- \( b \) is the length, normally taken per metre, so \( b = 1000 \text{ mm} \);
- \( t \) is the thickness of the load-bearing leaf;
- \( f_k \) is the characteristic compressive strength of masonry.
- \( \gamma_m \) is the partial factor of safety for material:
  Unless in exceptional circumstances, \( \gamma_m = 3.5 \).
- \( \beta \) is the capacity reduction factor:
  \[ \beta = 1.1 \left[ 1 - 2 \cdot \frac{e_m}{t} \right] \]

where \( \frac{e_m}{t} \) is the maximum eccentricity ratio of the wall which is a function of

- \( \frac{e_g}{t} \) – the eccentricity ratio due to gravity loads;
- \( \frac{e_w}{t} \) – the eccentricity ratio due to wind/lateral loads;
- \( \frac{e_a}{t} \) – the additional eccentricity ratio due to slenderness effects.

The eccentricities are the most awkward inputs to calculate and are explained in the following.
Eccentricity due to gravity loads

Eccentricity of gravity loads depends on the way the load is delivered to the wall. We assume that the introduced eccentricity vanishes at the bottom, giving an eccentricity distribution diagram.

The different types of construction give different eccentricities as:

**Case (a):** Occurs if the floor is concrete and \( \frac{L}{t} \leq 30 \). The eccentricity is:

\[
e_s = \frac{t}{2} - \frac{x}{2}
\]

**Case (b):** Most cases besides (a) and (c); the eccentricity is:

\[
e_s = \frac{t}{2} - \frac{x}{3}
\]

**Case (c):** Joist hangers and the like; the eccentricity is taken as:

\[
e_s = \frac{L}{2} + 25 \text{ mm}
\]
If there are loads from above; \( W_g > 0 \), then the net eccentricity of both the loads from above \( W_g \) and the loads from the current floor \( W_f \) is:

\[
e_e = \frac{W_f \cdot e}{W_g + W_f}
\]

and if other floors frame in at this level, we have in general \( e_e = \frac{M_{net}}{\sum W} \).

Having calculated \( e_e \) we change back to an eccentricity ratio, \( \frac{e_e}{t} \).
\( e_w \) – Eccentricity due to wind loads

\[
\frac{e_w}{t} = \text{Eccentricity due to wind loads}
\]

\[
M_w = \frac{W_w h^2}{8}
\]

\[
M_w = \frac{W_w h^2}{16}
\]

The eccentricity and bending moment are related as:

\[
e_w = \frac{M_w}{W_g}
\]

Having calculated \( e_w \), we change back to an eccentricity ratio, \( \frac{e_w}{t} \).

Note that simply-supported is usually conservative.

For cases in between simple and fixed supports, we define the degree of fixity as:

\[
\phi = \frac{g_d}{f_k \gamma_m}
\]

where \( g_d \) is the design vertical stress based on the 0.9\( G_k \) load case. Based, on \( \phi \) we can interpolate between the simple and fixed cases.

If \( \phi > 1 \) the wall is fully restrained.

C. Caprani
$e_a \over t$ – Additional eccentricity due to slenderness

This is given by:

$$\frac{e_a}{t} = \frac{\lambda^2}{2400} - 0.015$$

in which $\lambda$ is the Slenderness Ratio (SR), and:

$$\lambda = \frac{h_{ef}}{t_{ef}} \leq 27$$

- Effective thickness, $t_{ef}$:

  \[t_{ef} = t\]

  \[
t_{ef} = \max \left\{ \frac{2}{3}(t_1 + t_2), t_1, t_2 \right\}
  \]

  There are more considerations for the effective thickness, such as piers, which we are not examining.

- Effective height, $h_{ef}$:

  There are two cases:
  
  1. Enhanced Restraint, where $h_{ef} = 0.75h$:
     
     a. The floor passes over the top of the wall;
     
     b. The floor is a concrete floor and has a bearing length $> t/2$.
  
  2. All other cases, where $h_{ef} = h$.  

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Once again, there is an assumed distribution of $e_a$ over the height of the wall.

These diagrams can be thought of as inaccuracies in construction. For example, the wall goes most out of plumb in the middle, where it is furthest from its supports.

Interpolation for intermediate degrees of fixity is used for other restraint conditions.
The total eccentricity is determined by adding together the eccentricity diagrams.

Ignoring the wind load for the moment, we have, for example:

\[ e_{w} = \text{Total eccentricity} \]

The support wind eccentricity (if any) is also taken into account.

Hence, the total eccentricity is calculated as:

\[
e_m = \max \left\{ \frac{e_s}{t}, 0.6 \frac{e_s}{t} + \frac{e_w}{t} + \frac{e_a}{t} \right\}
\]

This is so as wind eccentricity at the top of the wall is beneficial, whereas the wind eccentricity at 0.6h is not. Usually, for simplicity, the maximum wind eccentricity at mid span is added to the 0.6e_s + e_a value. The support wind eccentricity (if any) is also taken into account.

Remember! It is usually ok to assume that the governing case is when the wind is blowing, but this may not always be the case. We’ll see in an example.
Capacity Reduction Factor

The total eccentricity determined above is used in the expression for the capacity reduction factor:

\[ \beta = 1.1 \left[ 1 - 2 \cdot \frac{e_m}{t} \right] \]

Often there is no wind load present and this simplifies the calculation as once \( \frac{e_c}{t} \) has been calculated, the following table may be used to determine \( \beta \):

<table>
<thead>
<tr>
<th>Slenderness ratio ( h_u/t_u )</th>
<th>Eccentricity at top of wall, ( e_a )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Up to 0.5t (see Note 1) 0.1t 0.2t 0.3t</td>
</tr>
<tr>
<td>0</td>
<td>1.00 0.88 0.66 0.44</td>
</tr>
<tr>
<td>6</td>
<td>1.00 0.88 0.66 0.44</td>
</tr>
<tr>
<td>8</td>
<td>1.00 0.88 0.66 0.44</td>
</tr>
<tr>
<td>10</td>
<td>0.97 0.88 0.66 0.44</td>
</tr>
<tr>
<td>12</td>
<td>0.93 0.87 0.66 0.44</td>
</tr>
<tr>
<td>14</td>
<td>0.89 0.83 0.66 0.44</td>
</tr>
<tr>
<td>16</td>
<td>0.83 0.77 0.64 0.44</td>
</tr>
<tr>
<td>18</td>
<td>0.77 0.70 0.57 0.44</td>
</tr>
<tr>
<td>20</td>
<td>0.70 0.64 0.51 0.37</td>
</tr>
<tr>
<td>22</td>
<td>0.62 0.56 0.43 0.30</td>
</tr>
<tr>
<td>24</td>
<td>0.53 0.47 0.34</td>
</tr>
<tr>
<td>26</td>
<td>0.45 0.38</td>
</tr>
<tr>
<td>27</td>
<td>0.40 0.33</td>
</tr>
</tbody>
</table>

**NOTE 1** It is not necessary to consider the effects of eccentricities up to and including 0.05t.

**NOTE 2** Linear interpolation between eccentricities and slenderness ratios is permitted.

**NOTE 3** The derivation of \( \beta \) is given in Annex B.
Design the inner leaf as the main load-bearing element.

\[ h = 2850 - 200 - 75 = 2575 \text{ mm} \]

\[ \frac{L}{\epsilon} = \frac{9800}{2.15} = 46.6 \approx 30 : \text{ Use Case (b)} \]

Load from the floor, \( W_g \):

\[ W_g = (1.4 \times 6 + 1.6 \times 2.5) \times 9 = 55.8 \text{ kN/m} \]

With wind:

\[ W_g = (1.2 \times 6 + 1.2 \times 2.5) \times 9 = 45.9 \text{ kN/m} \]

\[ e = \frac{2L - \frac{L}{2}}{3} = \frac{2L - \frac{L}{2}}{3} \]

\[ \frac{e}{\epsilon} = \left( \frac{2}{3} - \frac{1}{2} \right) = 0.1666 \]
Ew:

Conservatively, use pinned case:

\[ M_w = 1.2 \times 0.6 \times 2.575^2 / 8 = 0.6kNm \]

\[ \therefore \frac{Ew}{t} = \frac{M_w}{t \cdot Wg} = \frac{0.6 \times 10^3}{215 \times 45.9} = 0.061 \]

\[ e_a = \frac{h_{ef}}{t_{ef}} \quad h_{ef} = 1.0h = 2575mm \]

\[ t_{ef} = \frac{2}{3} (100 + 215) = 210 \neq 215 \]

\[ \therefore \frac{e_a}{t} = 215 \]

\[ \therefore d = \frac{2575}{215} = 11.9 < 27 \text{ Ver} \]

\[ \therefore \frac{e_a}{t} = 11.9^2 \div 2400 - 0.015 = 0.054y \]

\[ \frac{Em}{t} = \max \left\{ \begin{array}{c} 0.1666 \\ 0.6(0.1666) + 0.061 + 0.44 \\ = 0.205 \end{array} \right. \]

\[ = 0.205 \]

Without wind:

\[ \frac{Em}{t} = \max \left\{ \begin{array}{c} 0.1666 \\ 0.6(0.1666) + 0.44 \\ = 0.144 \end{array} \right. \]

\[ = 0.1666 \]
Thus,
\[ \beta_{\text{wind}} = 1.1 \left[ 1 - 2 \times 0.205 \right] = 0.65 \]
\[ \beta_{\text{no wind}} = 1.1 \left[ 1 - 2 \times 0.144 \right] = 0.78 \]

Try a SW block, SF = 0.47
\[ \Rightarrow f_k = 3.6 \text{ N/mm}^2 \]

\[ N = \beta \frac{f_k t b}{8m} \]

\[ N_{\text{wind}} = 0.65 \times \frac{3.6 \times 215 \times 1000}{3.5} \div 10^3 \]
\[ = 143.7 \text{ kN/m} \]
\[ > 55.8 \text{ kN/m} \ldots \text{un} \]
\[ > 45.9 \text{ kN/m} \ldots \text{on} \]

\[ N_{\text{no wind}} \Rightarrow \text{not necessary to check} \]
Example

Design the masonry walls indicated as Wall 1 and Wall 2.

Use the following information:

- normal manufacturing and construction controls;
- mortar designation (iii);
- use solid concrete blocks (100 × 215 × 440) in your design for both walls;
- The density of brickwork & blockwork may be taken as 21.2 kN/m²;
- The roof plant has been allowed for in the roof loading given;
- The joists have a bearing length of 100 mm on the inner leaf of Wall 1.
WALL 0

- Contours:
  Roof:
  \( 6 \times 7.2 / 2 \)
  \( 7.5 \times 7.2 / 2 \)

  Walls:
  \( 21.2 \times 0.315 \times (1.325 + 1.945) \)
  \( 28.52 \)

\[ \varepsilon = \frac{28.52}{50.12} = 0.57 \text{ kN/m} \]

\[ W_f = 1.4 \times 50.12 + 1.6 \times 2.7 = 113.4 \text{ kN/m} \]

First floor:

  \( 1.8 \times 3.6 / 2 \)
  \( 1.5 \times 3.6 / 2 \)

\[ \varepsilon = \frac{3.24}{2.7} = 1.2 \]

\[ W_f = 1.4 \times 3.24 + 1.6 \times 2.7 = 8.9 \text{ kN/m} \]

- Gravity Eccentricity:

  Triangular stress distribution:

  \[ \varepsilon = \frac{1}{2} - \frac{1}{3} \]

  \[ = \frac{4}{3} - \frac{1.00}{3} \]

  \[ = 74.17 \text{ mm} \]

  \[ \varepsilon = \frac{1}{2} = 0.345 \]

  \[ \varepsilon = \frac{8.9 \times 74.17}{113.4 + 8.9} / 2.15 = 0.025 \]

C. Caprani
Eccentricity due to slenderness:

\[ \epsilon_f = \frac{P}{3(EI)} \left( \frac{b + h}{2} \right) + \epsilon_0 \]
\[ = \frac{210}{215} + 0 \]
\[ = 210 < 215 \Rightarrow \epsilon_f = 215 \text{mm} \]

\[ h \epsilon_f = h \left( \text{Assume no restraint} \right) \]
\[ = 3250 \text{mm} \]
\[ \Rightarrow \lambda = \frac{3250}{215} = 15.1 < 27 \text{ VCK} \]

\[ \epsilon_a/\epsilon = \frac{15.1^2}{2400} - 0.015 = 0.08 \]

- Total eccentricity:

\[ \epsilon_x = \epsilon_a + \epsilon_c \]

\[ \epsilon_{max}/\epsilon = 0.6 \frac{\epsilon_x}{\epsilon} + 1.0 \frac{\epsilon_c}{\epsilon} \]
\[ = 0.6 \times 0.025 + 0.08 \]
\[ = 0.095 \]

- Design:

\[ \beta = 1.1 \left[ 1 - 2 \times 0.095 \right] = 0.89 \]

Try 5N rod: \( A = 0.47 \Rightarrow f_{12} = 3.6 \text{N/mm}^2 \)

\[ N = 0.89 \times \frac{3.6 \times 215}{3.5} = 176.8 \text{ kN/m} \]
\[ > 118.4 + 8.9 = 127.3 \text{ kN/m} \text{ VCK} \]

C. Caprani
...
Masonry Design – Flexural Capacity

Firstly, we introduce some general information:

1. **Edge support conditions** are identified as:

   ![Diagram showing edge support conditions]

   - **Free edge**
   - **Fixed edge**
   - **Simply Supported Edge**

2. **Limiting dimensions of panels**:
   - **a. Free Standing wall**: \( h \leq 12 \cdot t_{sf} \)
   - **b. Top and bottom supports only**: \( h \leq 40 \cdot t_{sf} \)
   - **c. 3 supported edges**:
     - 2 or more edges fixed: \( hl \leq 1500 \cdot t_{sf}^2 \)
     - All other cases: \( hl \leq 1350 \cdot t_{sf}^2 \)
   - **d. 4 supported edges**:
     - 3 or more edges fixed: \( hl \leq 2250 \cdot t_{sf}^2 \)
     - All other cases: \( hl \leq 2025 \cdot t_{sf}^2 \)

   Luckily, it is unusual for these limiting dimensions to be problematic.

3. **For irregular shapes** we convert to equivalent area rectangles, noting the support conditions.

C. Caprani
Design Applied Moments

\[
m_\perp = \alpha W_k \gamma_f L^2 \quad \text{and} \quad m_\parallel = \mu \alpha W_k \gamma_f L^2 = \mu m_\perp
\]

In which:
- \( \perp \) – perpendicular to bed joints;
- \( \parallel \) – parallel to bed joints;
- \( \alpha \) – coefficient from Table 9 of IS 325;
- \( W_k \) – characteristic wind load;
- \( \gamma_f \) – partial factor of safety for load:
  - for panels not providing lateral stability: \( \gamma_f = 1.2 \);
  - for panels providing structural stability: \( \gamma_f = 1.4 \);
- \( L \) – the span of the panel, usually horizontal;
- \( \mu \) – the orthogonal ratio of strength:
  \[
  \mu = \frac{f_{k\parallel}}{f_{k\perp}}
  \]
- \( f_{k\parallel} \) – \( \parallel \) characteristic strength from Table 3 of IS 325;
- \( f_{k\perp} \) – \( \perp \) characteristic strength from Table 3 of IS 325.

Table 3 of the code is given next. Note that \( \mu \) is always less than unity.

C. Caprani
### Table 3. Characteristic flexural strength of masonry, \( f_{lx} \), MPa

<table>
<thead>
<tr>
<th>Mortar designation</th>
<th>Plane of failure parallel to bed joints</th>
<th>Plane of failure perpendicular to bed joints</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Clay bricks having a water absorption</td>
<td>(i)</td>
<td>(ii) and (iii)</td>
</tr>
<tr>
<td>less than 7%</td>
<td>0.7</td>
<td>0.5</td>
</tr>
<tr>
<td>between 7% and 12%</td>
<td>0.5</td>
<td>0.4</td>
</tr>
<tr>
<td>over 12%</td>
<td>0.4</td>
<td>0.3</td>
</tr>
<tr>
<td>Calcium silicate bricks</td>
<td>0.3</td>
<td>0.2</td>
</tr>
<tr>
<td>Concrete bricks</td>
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<td>0.2</td>
</tr>
<tr>
<td>Concrete blocks of compressive strength in MPa</td>
<td></td>
<td></td>
</tr>
<tr>
<td>aspect ratio</td>
<td>3</td>
<td>5</td>
</tr>
<tr>
<td>as laid (&lt;1.0^*)</td>
<td>0.5</td>
<td>0.60</td>
</tr>
<tr>
<td>and over</td>
<td>0.15</td>
<td>0.45</td>
</tr>
<tr>
<td>Concrete blocks of compressive strength in MPa</td>
<td></td>
<td></td>
</tr>
<tr>
<td>aspect ratio</td>
<td>5</td>
<td>7</td>
</tr>
<tr>
<td>as laid of 0.4 to 0.5</td>
<td>0.8</td>
<td>0.5</td>
</tr>
<tr>
<td>and over</td>
<td>0.10</td>
<td></td>
</tr>
</tbody>
</table>

*When used with flexural strength in parallel direction, assume the orthogonal ratio \( \mu = 0.3 \).

*See 24.2.

---

**C. Caprani**
Design Moments of Resistance

Resistance parallel to the bed joints, \( M_{R//} \):

\[
M_{R//} = \left[ \frac{f_{kx//}}{\gamma_m} + g_d \right] Z
\]

where:

- \( g_d \) – the design vertical axial stress, based on \( 0.9G_k \) only;
- \( Z \) – the section modulus, \( \frac{bt^2}{6} \), but usually taken per metre, hence \( b = 1000 \text{ mm} \)

Resistance perpendicular to the bed joints, \( M_{R\perp} \):

\[
M_{R\perp} = \frac{f_{kx\perp}}{\gamma_m} \cdot Z
\]

where:

- \( Z \) – the section modulus, \( \frac{ht^2}{6} \), but usually taken per metre, hence \( h = 1000 \text{ mm} \)

The required wall thickness is the larger of:

\[
t_{//} \geq \sqrt{\frac{6m_1\gamma_m}{f_{kx//}h}} \quad t_{\perp} \geq \sqrt{\frac{6m_1\gamma_m}{f_{kx\perp}b}}
\]

where again as usual; \( h, b = 1000 \text{ mm} \).
NOTE 1. Linear interpolation of $\mu$ and $h/L$ is permitted.

NOTE 2. When the dimensions of a wall are outside the range of $h/L$ given in this table, it will usually be sufficient to calculate the moments on the basis of a single span. For example, a panel of type A having $h/L$ less than 0.3 will tend to act as a free-standing wall, whilst the same panel having $h/L$ greater than 1.75 will tend to span horizontally.

**Key to support conditions**

---

**Values of $\mu$**

<table>
<thead>
<tr>
<th>$h/L$</th>
<th>0.3</th>
<th>0.5</th>
<th>0.75</th>
<th>1.00</th>
<th>1.25</th>
<th>1.5</th>
<th>1.75</th>
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<td>1.00</td>
<td>0.031</td>
<td>0.045</td>
<td>0.058</td>
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<td>0.090</td>
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<td>0.90</td>
<td>0.032</td>
<td>0.047</td>
<td>0.061</td>
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<td>0.081</td>
<td>0.087</td>
<td>0.092</td>
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<tr>
<td>0.80</td>
<td>0.034</td>
<td>0.049</td>
<td>0.064</td>
<td>0.075</td>
<td>0.083</td>
<td>0.089</td>
<td>0.093</td>
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<tr>
<td>0.70</td>
<td>0.036</td>
<td>0.051</td>
<td>0.066</td>
<td>0.077</td>
<td>0.085</td>
<td>0.091</td>
<td>0.095</td>
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<tr>
<td>0.60</td>
<td>0.038</td>
<td>0.053</td>
<td>0.069</td>
<td>0.080</td>
<td>0.088</td>
<td>0.093</td>
<td>0.097</td>
</tr>
<tr>
<td>0.50</td>
<td>0.040</td>
<td>0.056</td>
<td>0.073</td>
<td>0.083</td>
<td>0.090</td>
<td>0.095</td>
<td>0.099</td>
</tr>
<tr>
<td>0.40</td>
<td>0.043</td>
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<td>0.064</td>
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<td>0.095</td>
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<td>0.086</td>
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<td>0.104</td>
<td>0.109</td>
<td>0.111</td>
</tr>
</tbody>
</table>

---

C. Caprani
<table>
<thead>
<tr>
<th>$\mu$</th>
<th>Values of $\alpha$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$h/L$</td>
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<tr>
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<td>0.30</td>
<td>0.020</td>
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</table>

C. Caprani
NOTE 1 Linear interpolation of $\mu$ and $h/L$ is permitted.

NOTE 2 When the dimensions of a wall are outside the range of $h/L$ given in this table, it will usually be sufficient to calculate the moments on the basis of a simple span. For example, a panel of type A having $h/L$ less than 0.3 will tend to act as a freestanding wall, whilst the same panel having $h/L$ greater than 1.75 will tend to span horizontally.

Key to support conditions:

- ______ denotes free edge
- ____________ simply supported edge
- XXXXXXX an edge over which full continuity exists

<table>
<thead>
<tr>
<th>$\mu$</th>
<th>$h/L$</th>
<th>0.3</th>
<th>0.5</th>
<th>0.75</th>
<th>1.00</th>
<th>1.25</th>
<th>1.50</th>
<th>1.75</th>
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<td>0.023</td>
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<td>0.030</td>
<td>0.060</td>
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<td>0.121</td>
<td>0.166</td>
<td>0.201</td>
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<tr>
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<td>0.113</td>
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<td>0.062</td>
<td>0.108</td>
<td>0.169</td>
<td>0.214</td>
<td>0.269</td>
<td>0.325</td>
</tr>
</tbody>
</table>

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Example:

Briar: White Absorption D12%

Concrete: 4"

Check face under load at 0.42 kN/m²:

\[ f_{ck1} = 0.3 \text{ kN/mm}^2 \quad f_{ck2} = 0.7 \text{ kN/mm}^2 \]

\[ \mu = \frac{0.7}{0.3} = 2.33 \]

\[ \frac{W}{L} = \frac{3}{4} = 0.75 \Rightarrow T_9 (E) \quad x = 0.053 \]

\[ W_L = x \times L = 0.053 \times 1.2 \times 0.42 \times 4^2 = 0.427 \text{ kN/m} \]

\[ Z = 1000 \times 10^2 \times 0.52^6 = 1.75 \times 10^6 \text{ mm}^3 \]

\[ M_{av} = 0.9 \times 1.75 \times 10^8 / 3.5 = 0.45 \text{ kN/mm per m} \]

\[ 0.45 > 0.427 \Rightarrow OK \]
Example:

Design the brick & mortar combination for:

Brick - 102.5mm; $w_k = 0.65 \text{ kN/m}^2$
Use normal control & construction.

Looking at Table 3 for bricks & mortar, assume $\mu = 0.35$, $w_r = \frac{2.475}{4} = 0.62$, $T = 24$ C

$\therefore \mu = 0.0425$

$M_\mu = \mu w_k L^2 = 0.0425 \times 0.65 \times 1.2 \times 4.5^2$

$= 0.67 \text{ kN/m/m}\text{m}$

$w_{rl} \geq M_\mu \therefore \frac{f_{ck}L}{3.5} \times \frac{1000 \times 102.5}{6} \geq 0.67 \times 10^6$

$\therefore f_{ck} \geq 1.34 \text{ N/mm}^2$

Choose bricks with $< 7\%$ moisture & mortar type (iii)

Check, $M = \frac{0.5}{1.5} = 0.33 \sim 0.35 \therefore OK\!$
Gable Wall Example:

\[ A_k = \frac{2bh^2}{3} \]
\[ q_2 = \frac{0.6kh^2}{f_2} \]

\[ V_{k} \leq \frac{V_{ts}}{2} \text{ kips} \]

- supported by purlins

2.8

\[ 6.5 \]

suspended on this line by dormer floor;

load width cut to wall = 3.5 m.

Check SW 215 solid blocks.

\[ \Rightarrow \text{equivalent solid rectangle height:} \]

\[ 6.5h = \frac{1}{2} \times 6.5 \times 2.8 \]
\[ = 1.4 \text{ m.} \]

C. Caprani
SW Breezes aspect = 0.47

\[ f_{kz} = 0.25 \text{ kN/m}^2 \quad f_{\text{ex}} = 0.65 \text{ kN/m}^2 \]

\[ M = \frac{0.25}{0.65} = 0.39 \]

Panel type = Fancy & conservative

**Gal:**

\[ a = 3.5 \times 0.8 \times 3 = 9.45 \text{ kN/m} \]

\[ Gal = \frac{9.45 \times 10^3}{1000 \times 215} \text{ kN/m} \]

\[ = 0.044 \text{ kN/m}^2 \]

\[ M_L = k L \frac{1}{2} L^2 \]

\[ k = \frac{h}{L} = 1.4\frac{1}{6.5} = 0.22 \]

\[ k \approx 0.011 \]

\[ M_L = 0.011 \times 0.75 \times 1.2 \times 6.5^2 = 0.43 \text{ kN.m} \]

\[ M_L = M_M = 0.168 \text{ kN.m} \]

\[ M_{\text{ev}} = \left[ \frac{0.25}{3.5} + 0.044 \right] \left( \frac{10^3 \times 215^2}{6} \right) = 0.89 \text{ kN.m} \]

\[ 0.89 > 0.43 \Rightarrow OK \]