Chapter 5
Vertical Loads

This chapter provides the design requirements for masonry subject to vertical loads resulting from superstructure weight and the vertical component of overturning loads.

It includes the load capacities of unreinforced masonry walls, grouted masonry walls and reinforced masonry lintels.
When vertical loads are applied to masonry walls, consideration must also be given to any bending moments induced by eccentricity of the loads.

The principal factors affecting the compressive capacity of a wall or pier are its slenderness ratio, its cross-sectional properties and compressive strength, the eccentricity of any load, rotational restraint and the buckling mode.

For the design of a cavity wall subjected to vertical loading, each leaf must be considered separately, therefore assuming that no mutual support is being supplied by the individual leaves.

In multi-storey buildings, the maximum vertical load on each loadbearing wall would result from the application of the maximum imposed load to all floors that contribute load to the wall, Figure 5.1. However this will probably not give the maximum eccentricity and may not necessarily lead to the worst case for design. It may be necessary to check several combinations of load, e.g. "chequer board" loading pattern, Figure 5.2.
5.2 DESIGN REQUIREMENTS

5.2.1 WALLS SUBJECT TO VERTICAL COMPRESSION FROM UNIFORM LOADS

Behaviour of Walls in Compression

When relatively squat masonry units (e.g., 76 mm x 110 mm x 230 mm bricks) are crushed in a compression testing machine, the platens tend to hold the unit together, inhibiting the formation of cracks giving an apparent increase in strength. AS 3700 requires that this apparent confined strength be modified by a reduction factor to yield an equivalent unconfined strength. This does not apply to hollow concrete blocks, since the thin face shells may be considered to be unconfined. (Initial failure is usually by splitting of the webs rather than by face-shell crushing).

When relatively squat masonry units (e.g., 76 mm high bricks) are built with mortar into a wall and subjected to a vertical load, the mortar expands laterally and tends to split the brick. For tall units (e.g., 190 mm concrete blocks) the widely spaced mortar joints have less effect. AS 3700 accounts for this phenomenon by means of a factor, $k_v$ (obtained from Table 3.2) which is used to modify the characteristic unconfined compressive strength of a masonry wall. The relationship between the characteristic unconfined compressive strength of the masonry units and characteristic unconfined compressive strength of a masonry wall is given by AS 3700 Clause 3.3.2.

The basic compressive capacity, $F_e$, of a stocky wall is given by multiplying the characteristic unconfined compressive strength of a masonry wall by the bedded area and the capacity reduction factor. If the cores are grout filled, further capacity can be included. The capacity of grouted masonry is substantially less than the sum of the strengths of the hollow masonry and the grout core, due to the interaction of the core and its surrounding shell. The mechanism of failure is influenced by the differing deformation characteristics of the materials, and possible shrinkage effects between the grout core and the tapered shells.

$$F_e = \frac{f_{m,eq} A_c}{\sqrt{1.3}}$$

This basic compressive capacity must be further modified to account for the eccentricity of any load, rotational restraint and the buckling mode. This is done by determining the appropriate value of the reduction factor, $k_r$.

For a wall that incorporates engaged piers, the thickness may be increased by a factor, $k_t$, from Table 7.2.

Lateral supports must be designed for any horizontal forces plus 25% of the vertical load, but not less than a pressure of 0.5 kPa as set out in Clause 2.6.3. If the thickness of an engaged pier is such that the deflection under design load is less than span/500 for unreinforced masonry or span/250 for reinforced masonry, it should be treated as a lateral support rather than an engaged pier. Many commonly constructed returns will not have sufficient length to be effective as lateral supports. See AS 3700 Clause 2.7.

For a wall that is supported along one or both of its vertical edges, two-way action will stiffen the wall provided there is sufficient shear capacity at the connection of the panel to its lateral supports. However, the consideration of this additional support is limited to the “Refined Design” method of AS 3600 Clause 7.3.4.

Control joints are not capable of effectively transmitting shear across the joint and must be treated as free ends. Vertical chases in masonry also diminish the shear transfer to adjacent lateral supports.

AS 3700 provides for two alternative approaches to the design for compression:

- Design by simple rules (Simplified Design), or
- Design by refined calculation (Refined Design) This allows for further choice between the assumption of eccentricities or their calculation by an equivalent-frame approach.

Simplified Design Method

The Simplified Design Method enables masonry loadbearing walls and piers to be designed using tabulated coefficients based on conservative values for the expected eccentricities and buckling mode of three particular applications, viz:

- A wall or pier supporting a concrete slab that bear on the top. This is the most common case for the loadbearing walls in medium-rise residential and commercial buildings. Charts for this case are plotted in this manual.
- A wall or pier supporting other systems that bear on the top. This case covers the situation where a steel or timber roof bears directly on the top of the wall. Generally, the magnitude of the loads will be relatively low and vertical load capacity will not represent a problem. No charts are provided for this case.
- A wall or pier supporting a load applied at the face of the masonry. This case often occurs in town-house construction where timber floors are fixed to the face of masonry party walls. Charts are provided for the case of 140 mm blockwork.

The slenderess coefficient ($a_s$) allows for restraint at the top and bottom of the masonry. For a member laterally supported at both top and bottom, $a_s = 1.0$ and for a member laterally supported and rotationally restrained at only its bottom, $a_s = 2.5$. One of the consequences of simplification has been that, despite appearances to the contrary, these values for $a_s$ do not relate to the values used for the Refined Design Method. The value of $a_s$ used in Equations 7.3.4.3(1), 7.3.4.3(2) and 7.3.4.3(4) for Refined Design is not the same as the value $a_s = 1.0$ used in Equations 7.3.3.4(1) for Simplified Design.

For a wall that is supported along one or both of its vertical edges, allowance is made for the strengthening effect of two-way action in compression.
Refined Design Method

The Refined Design Method permits the assessment of compressive strength based on eccentricities and fixity determined by either of two methods, i.e., the assumption of eccentricities or their calculation by an equivalent-frame approach. The results are generally not as conservative as the Simplified Design Method, but involve more rigorous calculation.

The value of $a_v$ used in Equations 7.3.4.3(1), 7.3.4.3(2) and 7.3.4.3(4) for Refined Design (Figure 5.3) is not the same as the value of $a_v = 1.0$ used in Equation 7.3.3.4(1) for Simplified Design.

The Refined Design Method permits the assessment of eccentricities and fixity by either of two methods, i.e., the assumption of eccentricities (Figure 5.4) or their calculation by an equivalent-frame approach (Figure 5.5). Both methods are described in the worked example.

The bending moments at the top and bottom of the wall, i.e., the effective load eccentricities, are influenced by a number of factors. These include the relative rotation between the floor and the wall, local crushing in the wall/beam joint, changing wall and slab stiffness with load level, non-linear and time-dependent material characteristics, two-way slab action, and the construction and loading sequence of the floor slab and walls.
A rigid frame analysis can be carried out to calculate the theoretical moment transferred from deflecting slabs into the walls above and below if the connection was rigid. In this analysis, the far ends of the members (floors and walls) may conservatively be assumed to be pinned. Depending on the relative stiffnesses of the floors and walls and the amount of precompression in the walls, a joint fixity factor may be determined (Figure 5.7). Once the joint fixity is found, the wall moment can be calculated by multiplying the rigid frame moment by the joint fixity factor.

In most cases, eccentricities on internal walls will be small, and eccentricities on external walls will be larger (see Design Chart of Calculated Eccentricities). In this case the wall will usually be bent in double curvature, and have a higher buckling resistance (Figures 5.5 and 5.6).

Where the wall compression is low, such as occurs at the top of a building, excessive eccentricities will be indicated by a rigid frame analysis. The eccentricity can be minimised by the use of flexible packing near the loaded edge of the wall.

The reduction factor for slenderness and eccentricity \( k \), is given by the following equation:

\[
k = 0.5 \left[ 1 + \frac{e_2}{e_1} \right] \left[ 1 - 0.025 \left( \frac{e_2}{e_1} \right)^{1.33} \left( S_r - 8 \right) \right] + 0.5 \left( 1 - 0.037 \left( \frac{e_1}{e_2} \right)^{1.18} \left( 0.03 S_r \right) \right) + 0.5 \left( 1 - 0.6 \left( \frac{e_1}{e_2} \right) \left( 1.18 - 0.03 S_r \right) \right)
\]

Values of \( k \) are also tabulated in Table 7.3. Crushing failure, independent of member slenderness, is not covered in Table 7.3.

In long-span heavily-loaded, one-way floor systems where large floor deflections are expected, the eccentricity of load can be controlled by the use of compressible packing between the floor and its supporting wall adjacent to the most highly-stressed face.

Figure 5.6 Typical Wall Buckling Modes

Figure 5.7 Joint Fixity Factor for use in Refined Design Method
5.2.3 REINFORCED MASONRY LINTELS SUBJECT TO VERTICAL BENDING

Reinforced concrete masonry lintels can be used to span over door, window or other openings and carry roof, floor or masonry loads. If several courses of masonry are constructed above the lintel, the vertical load may be calculated assuming a triangular area of masonry acting vertically on the lintel and a substantial load transferred by arching action back to the supports.

This chapter includes values for the shear and bending moment capacity of 140 mm and 190 mm reinforced concrete masonry lintels. Part B: Chapter 6 of this manual includes a worked example showing the derivation of the shear and moment capacities of reinforced masonry.

5.2.4 STEEL LINTELS AND ARCH BARS

Loads on Lintels

Masonry constructed in stretcher bond will arch over an opening, provided there are sufficient number of masonry courses above and sufficient strength at the supports. AS 3700 Commentary suggests that the load exerted on the lintel can be assumed to be exerted by a triangle of masonry above the opening.

End Support

AS 3700 Clause 4.12 requires that lintels be supported on the masonry abutments for a distance of at least 100 mm.

Corrosion Resistance

Steel lintels and arch bars must comply with the durability requirements of AS 3700 Table 5.1 for the particular exposure classification. AS 3700 Clause 5.4.1 makes it clear that lintels or arch bars supporting the external leaf of a cavity wall or veneer wall are considered to be in an “exterior environment”.

Corrosion protection requirements in AS/NZS 2699.3 are as follows:

R0, R1, R2, R3 – For all applications except as listed for R4 and R5.

- Steel lintels, hot dip galvanised in accordance with AS/NZS 4680 or AS/NZS 4791, except that the minimum coating mass shall be 300 g/m² for R0, R1 and R2 and be 600 g/m² for R3.
- Steel lintels with an inorganic zinc silicate coating, abrasive blast cleaned to a minimum of AS 1627.4 Class 2.5, and a coating of at least 75 microns of inorganic zinc silicate in compliance with AS/NZS 3750.15 Type 3 or Type 4, except that for R3 the average coating thickness shall be not less than 100 microns in compliance with AS/NZS 3750.15 Type 3.
- Steel lintels with a duplex coating, hot-dip galvanised in accordance with AS/NZS 4680 with a coating mass of 600 g/m² and a coating of at least 50 microns of two-pack non-inhibitive epoxy primer to AS/NZS 3750.13 and at least 200 microns of two-pack high-build epoxy mastic to AS 3750.11.
- Steel lintels with a duplex coating, hot-dip galvanised in accordance with AS/NZS 4680 with a coating mass of 600 g/m² and a coating of at least 50 microns of two-pack non-inhibitive epoxy primer to AS/NZS 3750.13 and at least 200 microns of two-pack high-build epoxy mastic to AS 3750.14.

R5 – For applications in saline or contaminated water including tidal splash zones and within 1 km of an industry producing chemical pollutants.

Detailing and Capacities

Detailing information is given in Part C: Chapter 3, while permissible spans for certain applications, in accordance with NCC–BCA Volume Two, are given in Steel Lintels and Arch Bars – Permissible Spans.
5.3.1 GENERAL

Design and Detailing
All design and detailing shall comply with the requirements of AS 3700 and, where appropriate, AS/NZS 1170.

All charts in this chapter (except the Calculated Eccentricities Table) are based on the Simplified Design approach set out in AS 3700 Clause 7.3.3.

The basic compressive capacity, $F_o$, representing the compressive strength of the masonry before consideration of slenderness or eccentricity is noted with each chart.

It is the designer’s responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties. Control joints and openings must be treated as free ends as specified by AS 3700.

Masonry Properties
The standard designs in this chapter are based on minimum masonry properties complying with the General Specification set out in Part C:Chapter 2, modified as noted on the standard design chart and as noted below.

Hollow concrete blocks
Width 90 mm, 110 mm, 140 mm and 190 mm
Height 190 mm
Length 390 mm

Face-shell bedded
Minimum face-shell thickness,
- $t_s = 25$ mm for 90 mm, 110 mm and 140 mm units
- $t_s = 40$ mm for some 140 mm units
- $t_s = 30$ mm for 190 mm units

Minimum characteristic compressive strength,
- $f'_c = 15$ MPa

Minimum characteristic lateral modulus of rupture,
- $f'_{ru} = 0.8$ MPa

Solid or cored concrete bricks
Width 110 mm
Height 76 mm
Length 230 mm

Fully bedded
Minimum characteristic compressive strength,
- $f'_c = 10$ MPa

Minimum characteristic lateral modulus of rupture,
- $f'_{ru} = 0.8$ MPa

Mortar joints
Mortar type M3 (or M4)
Joint thickness 10 mm

Concrete grout
Minimum characteristic compressive strength,
- $f'_c = 20$ MPa

Minimum cement content 300 kg/m$^3$

5.3.2 STANDARD DESIGN CHARTS

How to Read
The general procedure with most charts is as follows:
- Select the required wall thickness and material details.
- Select the appropriate support conditions (eg, supported on four sides).
- Project the length of the wall between vertical supports and the height of wall between horizontal supports to determine the design point.
- Select a curve which is above or to the right of the design point. Read off the load capacity corresponding to the selected curve. If necessary, interpolate between curves.
- Check that the masonry wall is adequate for other loadings, design requirements and construction requirements. Some charts have superimposed the robustness requirements for the same conditions.

5.3.3 INDEX OF CHARTS

Vertical Moment and Shear Capacities,
Reinforced Masonry Lintels:
- 140 mm and 190 mm leaves

Permissible Spans,
Steel Lintels and Arch Bars
Simplified Design Method, Compressive Capacity, Unreinforced Masonry,
Supporting a Concrete Slab,
- 190 mm High, Face-Shell Bedded,
  - Ungrouted:
    - 90 mm leaf
    - 110 mm leaf
    - 140 mm leaf
    - 190 mm leaf
  - Grouted:
    - 140 mm leaf
    - 190 mm leaf
    - 76 mm high, 110 mm leaf, fully bedded
    - 162 mm high, 110 mm leaf, face-shell bedded

Supporting a Load Applied at the Face:
- 140 mm leaf, 25 mm face-shell bedded
- 140 mm leaf, 40 mm face-shell bedded

Refined Design Method – Reduction Factors for Eccentricity and Slenderness Using Calculated Eccentricities:
- Various Wall Locations, Heights and Thicknesses
### 140-mm leaf

<table>
<thead>
<tr>
<th>BARS</th>
</tr>
</thead>
<tbody>
<tr>
<td>N12</td>
</tr>
<tr>
<td>N16</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Vc</th>
<th>Mc</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.1</td>
<td>2.6</td>
</tr>
<tr>
<td>6.3</td>
<td>2.6</td>
</tr>
</tbody>
</table>

### 190-mm leaf

<table>
<thead>
<tr>
<th>BARS</th>
</tr>
</thead>
<tbody>
<tr>
<td>N12</td>
</tr>
<tr>
<td>N16</td>
</tr>
<tr>
<td>N20</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Vc</th>
<th>Mc</th>
</tr>
</thead>
<tbody>
<tr>
<td>12.5</td>
<td>11.4</td>
</tr>
<tr>
<td>13.7</td>
<td>19.4</td>
</tr>
<tr>
<td>6.4</td>
<td>3.4</td>
</tr>
<tr>
<td>7.6</td>
<td>3.6</td>
</tr>
<tr>
<td>9.1</td>
<td>3.6</td>
</tr>
</tbody>
</table>

**NOTES**

- $V_c$ = Shear capacity (kN)
- $M_c$ = Moment capacity (kN.m)
- Mortar type, M3
- Block characteristic compressive strength, $f'_{uc}$ = 15 MPa
- Grout compressive strength, $f'_c$ = 20 MPa
### STEEL LINTELS AND ARCH BARS – Permissible Spans

<table>
<thead>
<tr>
<th>Section and Arrangement</th>
<th>End support</th>
<th>Brick Veneer</th>
<th>Cavity Brick</th>
<th>Either leaf</th>
<th>Single-leaf Wall</th>
<th>Single-leaf Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td>75 x 8 flat (4.71 kg/m)</td>
<td>100 mm</td>
<td>490</td>
<td>250</td>
<td>NA</td>
<td>NA</td>
<td></td>
</tr>
<tr>
<td>75 x 10 flat (5.89 kg/m)</td>
<td>100 mm</td>
<td>610</td>
<td>250</td>
<td>250</td>
<td>250</td>
<td></td>
</tr>
<tr>
<td>90 x 90 x 6 angle (8.22 kg/m)</td>
<td>100 mm for spans &lt; 1 m</td>
<td>3010</td>
<td>2050</td>
<td>2050</td>
<td>1570</td>
<td></td>
</tr>
<tr>
<td>90 x 90 x 8 angle (10.6 kg/m)</td>
<td>100 mm for spans &lt; 1 m</td>
<td>3010</td>
<td>2170</td>
<td>2170</td>
<td>1810</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Section and Arrangement</th>
<th>End support</th>
<th>Brick Veneer</th>
<th>Cavity Brick</th>
<th>Either leaf</th>
<th>Single-leaf Wall</th>
<th>Single-leaf Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td>100 x 100 x 6 angle (9.16 kg/m)</td>
<td>100 mm for spans &lt; 1 m</td>
<td>3130</td>
<td>2290</td>
<td>2290</td>
<td>1810</td>
<td></td>
</tr>
<tr>
<td>100 x 100 x 8 angle (11.8 kg/m)</td>
<td>100 mm for spans &lt; 1 m</td>
<td>3370</td>
<td>2410</td>
<td>2410</td>
<td>1930</td>
<td></td>
</tr>
<tr>
<td>150 x 90 x 8 angle (14.3 kg/m)</td>
<td>150 mm for spans &gt; 1 m</td>
<td>4210</td>
<td>3370</td>
<td>3370</td>
<td>2770</td>
<td></td>
</tr>
<tr>
<td>150 x 100 x 10 angle (18.0 kg/m)</td>
<td>150 mm for spans &gt; 1 m</td>
<td>4330</td>
<td>3490</td>
<td>3610</td>
<td>3010</td>
<td></td>
</tr>
</tbody>
</table>

* Maximum span 10 m, N3 wind

---

**Maximum clear span (mm) for following loading cases:**

* Maximum span 10 m, N3 wind
NOTE: It is the designer’s responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties.
NOTE: It is the designer's responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties.
NOTE: It is the designer’s responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties.
NOTE: It is the designer's responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties.
NOTE: It is the designer’s responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties.
NOTE: It is the designer’s responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties.

### Simplified Design Method

**190-mm leaf** (Strength grade 15 MPa, Height 190 mm, \( F_0 = 594 \text{ kN/m} \))

<table>
<thead>
<tr>
<th>Height of Wall Between Supports (m)</th>
<th>Design compressive force, ( F_d \text{ (kN/m)} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>120</td>
<td>0.5 kPa</td>
</tr>
<tr>
<td>160</td>
<td>Robustness governs in shaded area</td>
</tr>
<tr>
<td>200</td>
<td></td>
</tr>
<tr>
<td>240</td>
<td></td>
</tr>
<tr>
<td>280</td>
<td></td>
</tr>
<tr>
<td>320</td>
<td></td>
</tr>
<tr>
<td>360</td>
<td></td>
</tr>
<tr>
<td>400</td>
<td></td>
</tr>
<tr>
<td>440</td>
<td></td>
</tr>
<tr>
<td>480</td>
<td>( 480 - F_d )</td>
</tr>
</tbody>
</table>

**Face-shell bedded, 25 mm face shell**

\[ a_v = 1.0, \varphi = 0.5 \]
NOTE: It is the designer's responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties.
PART B: CHAPTER 5
Vertical Loads

NOTE: It is the designer's responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties.

UNGROUTED UNREINFORCED HOLLOW MASONRY

110-mm leaf (Strength grade 15 MPa, Height 162 mm, \( F_0 = 192 \, \text{kN/m} \))

Simplified Design Method

- Laterally-supported at top and supporting a concrete slab
- Design compressive force, \( F_d \) (kN/m)

Face-shell bedded, 25 mm face shell

- \( a_v = 1.0 \), \( \sigma = 0.5 \)

0.5 kPa
Robustness governs in shaded area

Length of Wall Between Supports (m)

Height of Wall Between Supports (m)

0 1.0 2.0 3.0 4.0 5.0 6.0 7.0 8.0 9.0

15 30 45 60 75 90 105 120 135 150 – \( F_d \)
NOTE: It is the designer’s responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties.
NOTE: It is the designer's responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties.
REFINED DESIGN METHOD – Reduction Factors for Eccentricity and Slenderness Using Calculated Eccentricities

BASIS OF TABLE
The following table sets out some typical eccentricity to thickness ratios and reduction factors for masonry walls in medium-rise buildings. They are based on calculations of moments and loads for:
- the upper floors slabs ranging from 3.0 to 4.0 metres span;
- slab span to thickness ratios of approximately 20:1;
- imposed loads of 3 kPa;
- wall heights from 2.4 metres to 3.6 metres;
- wall thicknesses of 90, 110, 140 and 190 mm;
- joint fixity factor given in the Figure 5.7;
- $a_v = 0.85$ above the slab (for the top storey);
- $a_v = 0.75$ below the slab (for the second-top storey).

The values have been reproduced here to give an indication of the accuracy of the assumed eccentricities and to assist designers in determining the appropriate analysis method to use. These values of eccentricity to thickness ratios and reduction factors should not be used for design without confirmation by the analysis method described in this manual.

### REDUCTION FACTORS (K) USING CALCULATED ECCENTRICITIES

<table>
<thead>
<tr>
<th>Wall details</th>
<th>Slenderness ratios</th>
<th>Eccentricity ratios</th>
<th>Reduction factors</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Above slab $S_r$</td>
<td>Below slab $S_r$</td>
<td>Above slab $e_1/t_w$</td>
</tr>
<tr>
<td>Location</td>
<td>Height $H$ (mm)</td>
<td>Thickness $t_w$ (mm)</td>
<td></td>
</tr>
<tr>
<td>Internal</td>
<td>2400</td>
<td>90</td>
<td>22.7</td>
</tr>
<tr>
<td></td>
<td></td>
<td>110</td>
<td>18.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>140</td>
<td>14.6</td>
</tr>
<tr>
<td></td>
<td>3000</td>
<td>140</td>
<td>18.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>190</td>
<td>13.4</td>
</tr>
<tr>
<td></td>
<td>3600</td>
<td>190</td>
<td>16.1</td>
</tr>
<tr>
<td>External</td>
<td>2400</td>
<td>90</td>
<td>22.7</td>
</tr>
<tr>
<td></td>
<td></td>
<td>110</td>
<td>18.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>140</td>
<td>14.6</td>
</tr>
<tr>
<td></td>
<td>3000</td>
<td>140</td>
<td>18.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>190</td>
<td>13.4</td>
</tr>
<tr>
<td></td>
<td>3600</td>
<td>190</td>
<td>16.1</td>
</tr>
</tbody>
</table>

* The calculated eccentricity ratios for external walls above the slab exceed 0.33. In these situations the actual support conditions must be different from the mechanism implicit in the moment distribution and joint-fixity factor used to calculate these values. Use of the assumed values is considered reasonable in these cases.
5.4 WORKED EXAMPLE

5.4.1 GENERAL

Purpose of the worked example
The purpose of the following worked example is to demonstrate the steps to be followed when performing manual calculations or when preparing computer software for the analysis and design of masonry. The worked example also serves the purpose of demonstrating the origin of the Standard Designs which are based on similar masonry capacity considerations. Although comprehensive in its treatment of AS 3700, the worked example is not intended to analyze or design all parts of the particular structure. It deals only with enough to demonstrate the design method.

Design and detailing
All design and detailing shall comply with the requirements of AS 3700 and, where appropriate, AS/NZS 1170.

It is the designer’s responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties. Control joints and openings must be treated as free ends as specified by AS 3700.

Masonry Properties
The worked examples in this chapter are based on masonry properties complying with the General Specification set out in Part C: Chapter 2, modified as noted in the calculations and as noted below.

Hollow concrete blocks
Width 90 mm, 110 mm, 140 mm and 190 mm
Height 190 mm
Length 390 mm
Face-shell bedded
Minimum face-shell thickness,
\( t_s = 25 \text{ mm for 90 mm, 110 mm and 140 mm units} \)
\( t_s = 30 \text{ mm for 190 mm units} \)
Minimum characteristic compressive strength,
\( f'_{uc} = 15 \text{ MPa} \)
Minimum characteristic lateral modulus of rupture,
\( f'_{ut} = 0.8 \text{ MPa} \)

Solid or cored concrete bricks
Width 110 mm
Height 76 mm
Length 230 mm
Fully bedded
Minimum characteristic compressive strength,
\( f'_{uc} = 10 \text{ MPa} \)
Minimum characteristic lateral modulus of rupture,
\( f'_{ut} = 0.8 \text{ MPa} \)

Mortar joints
Mortar type M3 (or M4)
Joint thickness 10 mm

Concrete grout
Minimum characteristic compressive strength,
\( f_c = 20 \text{ MPa} \)
Minimum cement content 300 kg/m³

5.4.2 INDEX TO WORKED EXAMPLE

Worked example includes both Design by Simplified Rules and Design by Refined Calculation, as follows:

PART 1 Design brief, vertical loads and masonry properties.
PART 2 Design by Simplified Rules.
PART 3 Design by Refined Calculation.
PART 1:

DESIGN BRIEF

Design loadbearing wall indicated in the accompanying drawings.

VERTICAL LOADS

Length of wall under consideration
$L = 3.70$ m

Roof
Area of roof acting on wall
$A_R = \frac{7.5 \times 5.33}{2} = 20.0$ m$^2$

Permanent load of roof structure
$g_R = 1.2$ kPa

Imposed load of roof
$q_R = 0.25$ kPa

Factored roof loads
$F_{dR} = \frac{1.2 g_R A_R + 1.5 q_R A_R}{L} = \frac{(1.2 \times 2.7 \times 20.0) + (1.5 \times 0.25 \times 20.0)}{3.70} = 9.8$ kN/m

Floor 1 Walls
Only the internal leaf is loadbearing

Height of wall
$H_{W1} = 2.7$ m

Length of loadbearing leaf
$L_{W1} = 3.7$ m

Use 90-mm denseweight hollow blockwork
Density 2180 kg/m$^3$ 80% solid
1 face with plasterboard 10 mm thick

Surface density
$g_{W1} = \frac{(90 \times 0.8 \times 2180 \times 9.81)}{1,000,000} + \frac{(1 \times 10 \times 800 \times 9.81)}{1,000,000} = 1.62$ kN/m$^2$

Permanent load
$G_{W1} = \frac{g_{W1} H_{W1} L_{W1}}{L_{W1}} = \frac{1.62 \times 2.70 \times 3.70}{3.70} = 4.37$ kN/m

Factored wall loads
$F_{dW1} = 1.2 G_{W1} = 1.2 \times 4.37 = 5.2$ kN/m

Cont...
### Floor 2 Walls

- **$F_{dW2}$** = 5.2 kN/m (similar to $F_{dW1}$)

### Floor 3 Walls

- **$F_{dW3}$** = 5.2 kN/m (similar to $F_{dW1}$)

### Slab 1

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area</td>
<td>8.0 m²</td>
</tr>
<tr>
<td>Slab thickness</td>
<td>175 mm</td>
</tr>
<tr>
<td>Permanent load</td>
<td>25.0 x 0.175 = 4.38 kPa</td>
</tr>
<tr>
<td>Imposed load</td>
<td>2.0 kPa</td>
</tr>
</tbody>
</table>

### Slab 2

- **$F_{dS2}$** = 17.9 kN/m (similar to $F_{dS1}$)

### Total Factored Loads on Subject Leaf

- **$F_d$** = $F_{dR} + F_{dW1} + F_{dW2} + F_{dW3} + F_{dS1} + F_{dS2}$
- **$F_{dR}$** = 9.8 + 5.2 + 5.2 + 5.2 + 17.9 + 17.9 = 61.2 kN/m

### Height ratio

- **$h_u$** = 190 mm
- **$t_j$** = 10 mm

### Masonry Properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Width of masonry unit</td>
<td>90 mm</td>
</tr>
<tr>
<td>Face-shell thickness</td>
<td>25 mm</td>
</tr>
<tr>
<td>Bedded area</td>
<td>50,000 mm²/m</td>
</tr>
<tr>
<td>Mortar joint thickness</td>
<td>10 mm</td>
</tr>
<tr>
<td>Mortar type</td>
<td>M3 (1:5 + water thickener)</td>
</tr>
<tr>
<td>Compressive strength factor</td>
<td>$k_h$ = 1.3</td>
</tr>
</tbody>
</table>

### Worked Example

- **$F_{dW1}$** = 5.2 kN/m (similar to $F_{dW1}$)
- **$F_{dS1}$** = (4.5 x 2.0) - (2.0 x 1.0) / 2 = 8.0 m²
- **$F_{dS2}$** = 17.9 kN/m (similar to $F_{dS1}$)

### Characteristic unconfined masonry strength

- **$f'_{uc}$** = 15 MPa
- **$f'_{imb}$** = $k_m f''_{uc}$
- **$f''_{uc}$** = 19.0
- **$k_m$** = 1.6

- **$f''_{mb}$** = $k_h f''_{imb}$
- **$k_h$** = 1.3

- **$f''_{mb}$** = 6.20 MPa
- **$f''_{imb}$** = 15.3 MPa

- **$f''_{imb}$** = 6.20 MPa
- **$f''_{imb}$** = 8.06 MPa
NOTE: This wall is not grouted. Where grout is used elsewhere, it is specified as:

**Characteristic grout cylinder strength**

\[ f'c = 20 \text{ MPa} \]

**NOTE:** This wall is not grouted. Where grout is used elsewhere, it is specified as:

\[ > 12 \text{ MPa} \]

3.5

**Design characteristic grout strength**

\[ f'cg = 1.3 f'uc \]

\[ = 1.3 \times 15 \]

\[ = 19.5 \text{ MPa} \]

< 20 MPa

\[ 3.5 \]

**Basic compressive capacity**

\[ F_0 = \phi \left[ f'm A_b + k_c \left( \frac{f'cg}{1.5} \right) A_c \right] \]

\[ = 0.5 \left( \frac{8.06 \times 50,000}{1000} + 1.4 \times \sqrt{\frac{19.5}{1.5}} \right) \]

\[ = 201 \text{ kN/m} \]

**Density factor**

\[ k_c = 1.4 \text{ for density 2180} \]

> 2000 kg/m³

\[ 7.3.2 \]

**Slenderness ratio**

\[ S_{rs} = \frac{a_v H}{k_t t} \]

\[ = 1.0 \times 2700 \]

\[ = 1.0 \times 90 \]

\[ = 30.0 \]

**Clear height**

\[ H = 2.70 \text{ m} \]

**Clear length**

\[ L = 3.70 \text{ m} \]

**Capacity reduction factor**

\[ \phi = 0.5 \]

**Thickness coefficient**

\[ k_t = 1.0 \]

**Slenderness and eccentricity factor**

\[ k = 0.67 - 0.02 (S_{rs} - 14) \]

\[ = 0.67 - 0.02 (30.0 - 14) \]

\[ = 0.35 \]

**Slenderness and eccentricity factor**

\[ k = 0.67 - 0.02 (S_{rs} - 14) \]

\[ = 0.67 - 0.02 (30.0 - 14) \]

\[ = 0.35 \]

**Density factor**

\[ k_c = 1.4 \text{ for density 2180} \]

> 2000 kg/m³

\[ 7.3.2 \]

**Slenderness ratio**

\[ S_{rs} = \frac{a_v H}{k_t t} \]

\[ = 1.0 \times 2700 \]

\[ = 1.0 \times 90 \]

\[ = 30.0 \]

**Clear height**

\[ H = 2.70 \text{ m} \]

**Clear length**

\[ L = 3.70 \text{ m} \]

**Capacity reduction factor**

\[ \phi = 0.5 \]

**Thickness coefficient**

\[ k_t = 1.0 \]

**Design capacity**

\[ f_u = k F_0 \]

\[ = 0.35 \times 201 \]

\[ = 70.4 \text{ kN/m} \]

As this result is close, design by refined method
**PART 3:** Slab moment

\[ M_{ab} = F_{dS1} \varepsilon_{S1} \]

\[ = \frac{18.3 \times 15}{1000} = 0.275 \text{ kNm/m} \]

Vertical coefficient (supports slab)

\[ a_v = 0.75 \]

7.3.4.3

Horizontal coefficient (one support)

\[ a_h = 2.5 \]

7.3.4.3

Slenderness ratio

\[ a_h = 0.75 x 2700 \]

\[ 1.0 x 90 \]

= 22.5

\[ < \frac{0.7}{\sqrt{a_h a_h L}} \]

= \[ \frac{0.7}{90} \sqrt{0.75 x 2700 x 2.5 x 3700} \]

= 33.7 \text{ OK} \]

Effective eccentricity at top of wall

\[ e_1 = \frac{M_{ab}}{F_{dA}} = \frac{0.275 x 1000}{57.7} = 4.8 \]

Slenderness and eccentricity factor

\[ k = 0.5 \left( 1 + \frac{e_1}{e_2} \right) \left( 1 - 2.083 \frac{e_1}{t_w} \right) - \frac{(0.025 - 0.037 \frac{e_1}{t_w} \cdot 1.33 S_r - 8)}{2 t_w} \text{ - } (1 - 2 e_1) \]

= 0.437

Design capacity

\[ F_{cap} = k F_o \]

\[ = 0.437 \times 201 \]

\[ = 87.8 \text{ kN/m} \]

Total factored load at top of wall

\[ F_{dA} = 63.2 - 5.5 \]

\[ = 57.7 \text{ kN/m} \]

Effective eccentricity at top of wall

\[ e_1 = \frac{M_{ab}}{F_{dA}} \]

\[ = \frac{0.275 x 1000}{57.7} = 4.8 \]

Option 2 – Modified Rigid Frame Analysis

Consider members at top of wall

Slab

Thickness

\[ t_s = 175 \text{ mm} \]

Effective width (continuous)

\[ b_s = 1000 \text{ mm} \]

Elastic modulus

\[ E_s = 25,000 \text{ MPa} \]

Stiffness factor

\[ R = 0.75 \]

\[ < \frac{1 - t_s}{t_w} + 2 \frac{e_1}{t_w} \]

\[ \frac{1 - t_s}{t_w} = 0.872 \text{ OK} \]

\[ \frac{e_1}{t_w} = 0.5 (1 - 0.6) \left( 1 - \frac{e_1}{t_w} \right) (1.18 - 0.03 S_r) \]

\[ \frac{e_1}{t_w} = 0.437 \text{ OK} \]

\[ \varepsilon_{S1} = \frac{t}{6} \]

\[ = \frac{90}{6} = 15 \text{ mm} \]

\[ F_{dS1} = 18.3 \text{ kN/m} \]

\[ F_{dS1} = 39.4 \]

\[ \gamma = 15 \]

\[ F_{dA} = 5.5 \text{ kN/m} \]
### Worked Example

**Moment of inertia**

\[
I_s = \frac{b_s t_s^3}{12} = \frac{1000 \times 175^3}{12} = 447 \times 10^6 \text{ mm}^4
\]

**Walls**

- **Thickness**
  \[t_w = 90 \text{ mm}\]
- **Allowance for openings**
  \[p = 0.7\]

**Length**

\[L_s = 3800 \text{ mm (approximate)}\]

**Stiffness**

\[R(E_s L_s) = \frac{0.75 \times 25,000 \times 447 \times 10^6}{3800} = 2206 \times 10^6 \text{ Nm/m}^2\]

**Permanent load**

\[g_{s1} = 4.38 \text{ kPa}\]

**Imposed load**

\[q_{s1} = 2.0 \text{ kPa}\]

**Fixed end moment**

\[FEM = (1.2 g_{s1} + 1.5 q_{s1}) \frac{L_s^2}{8} = \left[1.2 \times 4.38 + (1.5 \times 2.0)\right] \frac{3.8^2}{8} = 14.9 \text{ kNm/m}\]

**Stiffness**

\[
R \left( \frac{E_w I_w}{H_w} \right) = \frac{0.75 \times 8060 \times 42.5 \times 10^6}{2875} = 89 \times 10^6 \text{ Nm/m}^2
\]

**Elastic modulus**

\[E_w = 1000 \times 8.06 = 8060 \text{ MPa}\]

**Effective width (allowing for openings)**

\[b_w = p \times b_s = 0.7 \times 1000 = 700 \text{ mm}\]

**Height (centre to centre)**

\[H_w = 2700 + 175 = 2875 \text{ mm}\]

**Allowance for openings**

\[p = 0.7\]

**Fixed end moment of inertia**

\[
I_w = \frac{b_w t_w^3}{12} = \frac{700 \times 90^3}{12} = 42.5 \times 10^6 \text{ mm}^4
\]

**Moment distributed to wall under slab**

\[M_A G = DF \times FEM = 0.0373 \times 14.9 = 0.56 \text{ kNm/m}\]

**Axial load on wall**

\[F_{dA} = 61.2 - 5.2 = 56.0 \text{ kN/m}\]

**Compressive stress at top of wall**

\[f_A = \frac{F_{dA}}{A_b \times p} = \frac{56.0 \times 1000}{50,000 \times 0.7} = 1.6 \text{ MPa} > 0.25 \text{ MPa} \text{ OK}\]

Cont…
### Ratio slab stiffness to wall stiffness

\[
\frac{2206}{89 + 89} = 12.3
\]

### Slenderness and eccentricity factor

\[
k = 0.5 \left(1 + \frac{e_2}{e_1}\right) \left(1 - 2.083 \frac{e_1}{t_w}\right) - 0.025 \left(1 - 0.037 \frac{e_1}{t_w}\right) (1.33S_t - 8)
\]

### Joint fixity factor

Conservative extrapolation of AS 3700 Commentary or FIG 5.7 this manual

\[
J = 0.4
\]

### Modified moment at top of wall

\[
M_{ABm} = 0.4 \times 0.56 = 0.22 \text{ kNm/m}
\]

### Eccentricity ratio at top of wall

\[
e_1 = 4.0 \text{ mm}
\]

### Eccentricity ratio at top of wall

\[
e_1 = \frac{4.0}{90} = 0.044
\]

\[
< 0.05 \quad \text{Almost concentric}
\]

### Design capacity

\[
F_{cap} = kF_o = 0.505 \times 201 = 101.5 \text{ kN/m} > 61.2 \text{ kN/m} \quad \text{OK}
\]