AUSTRALIAN DESIGN MANUAL FOR DIAPHRAGM WALLS

G. Simundic¹ and A.W. Page²

1. ABSTRACT

The use of diaphragm and other walls of geometric cross section was pioneered in the United Kingdom and has gained wide acceptance in that country. Diaphragm walls are particularly suited for applications where tall walls are required in structures such as sports halls and other light commercial and industrial buildings. There is scope in Australia for the use of diaphragm walls not only in the manner developed in the United Kingdom, but also for domestic construction in high wind areas. This paper gives an overview of a study which has investigated the potential for the use of diaphragm walls in Australia and the development of an industry design manual. Several areas of difference between Australia and the United Kingdom practice are identified and problems resolved. A computer program for the design of propped cantilever diaphragm walls has been developed and used to produce a series of design curves for wall selection and incorporation into the industry design manual.

2. INTRODUCTION

In recent years there has been a major shift away from the use of masonry for commercial and light industrial structures in Australia because of the impact of precast concrete and tilt-up construction. The disadvantage of conventional masonry walls in these structures is the need to provide additional support for the walls in the form of mullions or framing support

Keywords: Diaphragm; Design Manual; Masonry; Walls

¹ Laboratory Manager, Department of Civil, Surveying and Environmental Engineering, The University of Newcastle, NSW, 2308, Australia
² CBPI Professor in Structural Clay Brickwork, Department of Civil, Surveying and Environmental Engineering, The University of Newcastle, NSW, 2308, Australia
because of their height. The use of masonry diaphragm walls offers a viable alternative. Engineers in Australia have used the concept of diaphragm walls in isolated instances for many years, with the design being from first principles rather than from code provisions. Specific provisions for diaphragm walls were included in the SAA Masonry Code in 1988 (1), but their use has not been widespread due to the novel nature of the technique and the ignorance of designers of the potential of the system.

The Australian clay masonry industry has recently recognised the potential of walls of geometric section, and in collaboration with the University of Newcastle is producing a design guide for diaphragm walls for the use of structural engineers and architects. This guide will include both architectural aspects and structural design information with a series of design curves of lateral loading capacity being provided. A range of wall geometries have been considered, for use in both domestic or commercial construction.

This paper gives an overview of the design manual and describes a series of experiments which were carried out as a background to the study. In particular the tests related to the differences between Australian and British practice with regard to shear connector design and water penetration performance as described. Tests on shear connectors revealed major differences with previous British research. The reasons for these differences are identified and discussed. The water penetration properties of the diaphragm walls have been also investigated with the emphasis on thin diaphragm walls which are unique to Australia.

3. DESIGN OF DIAPHRAGM WALLS

A detailed review of the design of walls of geometric section for Australian conditions has been produced by Phipps and Page (2). The design of diaphragm walls will be rarely governed by the compressive capacity of the wall, with the flexural resistance usually being the controlling factor influencing the overall wall dimensions as well as the spacing of the diaphragms.

The Australian Masonry Standard AS3700 allows walls to be designed for the ultimate limit state utilising the flexural tensile strength of the masonry. For transient loads a value of 0.20 MPa can be assumed as the characteristic flexural tensile strength of masonry $f'_{mt}$, except at membrane type damp-proof courses and at the interface between masonry and other materials when it is taken as zero. Because of the possibility of low bond strength (or the loss of bond strength due to previous cracks) the British approach is to neglect bond strength and base ultimate strength calculation on the formation of a collapse mechanism with hinges forming at points of maximum bending moment. This is an inherently safe approach and will usually give a capacity which is lower than that predicted if the bond strength is mobilised (as is the case in this study).

3.1 Design for Compressive Forces

Diaphragm walls are usually used to enhance the lateral load capacity of the masonry. However diaphragm walls can also be successfully and economically used to support heavy axial loads, particularly for tall walls. The geometry of the cross section provides increased
resistance to buckling, with higher compressive capacities than would be applicable to an equivalent solid wall (3). Since the current Australian Masonry Code AS3700 does not have specific allowances for geometric sections in its compression provisions, diaphragm walls are usually conservatively designed by calculating the slenderness ratio using the overall depth of the wall. Alternatively a “first principles” approach can be used by calculating the equivalent thickness of a solid wall with the same radius of gyration as the hollow wall. If the simplified method is used, the wall is considered as an equivalent solid section to obtain the compressive reduction factor, with the actual cross-sectional area of the wall being used to calculate the compressive capacity. Overall limits on slenderness ratio are also applied to ensure a minimum level of “robustness”.

3.2 Design for Lateral Loads

The analysis of a wall spanning vertically between a footing and a roof and subjected to a lateral load has been previously presented by Phipps and Page (2), and is described briefly here. As shown in Figure 1 the peak moments and crack locations are at the wall base and near mid-height. If the dead load effects of roof are ignored, assuming for the worst case that the roof uplift force is balanced by the mass of the capping beam and transmitted directly to the footings by a tie down system, the axial load on the wall is its self weight (P). The value of the moment of resistance at the base $M_A$, will be directly related to the weight of the wall, and given by (see Figure 1):

$$M_A = P \times \frac{T}{2}$$

where $T =$ overall depth of the wall. \[1\]

If the flexural strength of the masonry is utilised (the “strength method”), as permitted by the Australian Code, the lateral load capacity is given by:

$$w = \frac{2}{H^2} \left( M_A + 2M_{\text{max}} + 2\sqrt{(M_{\text{max}} + M_A)M_{\text{max}}} \right) \[2\]

where: $M_{\text{max}} = C_m f_{m} Z_d + f_{d,b} Z_d$, $f_{d,n} = P/A_m$, $C_m =$ capacity reduction factor

and $Z_d =$ section modulus.

If the flexural strength of the masonry is ignored (the "mechanism method"), as used in the
British approach, the lateral load capacity is given by (see Figure 1):

\[ w = \frac{1.6 \bar{x} A_m T}{H (1 - x_i)} \]

where: \( x_i = \frac{h_i}{H} \).

\[ \bar{x} \text{ = density of masonry, } A_m = \text{cross-sectional area of diaphragm wall.} \]

The spacing of diaphragms will be governed by ability of the flange to span between them. The horizontal bending capacity is enhanced by partial two-action in some cases (when the flange is also supported along its bottom horizontal edge), and also by in-plane arching between the diaphragms. This enhancement is usually ignored in the design. Any discontinuity such as vertical control joints must also be considered. Note that if the diaphragms are widely spaced, not all of the flange is included in the cross section considered for vertical bending due to shear lag effects.

### 3.3. Design for Shear

Walls of geometric section can fail in shear by sliding at the base, by web failure (diagonal tension cracking) or by failure at the flange-web interface. Consideration of these aspects have been previously presented (4). One area which has not been fully explained is the behavior of metal connectors across an interface when they are being used to transfer shear forces (as, for example, between the flange and the web). Figure 2 shows a cross section through a wall in which the leaves are connected to the webs with steel connectors.

![Figure 2. Shear Transfer in Diaphragm Walls](image)

**Figure 2. Shear Transfer in Diaphragm Walls**

The shear force \( V_c \) acting on an individual connector is:

\[ V_c = t_s x f_v x s_v \]

where: \( s_v = \text{vertical spacing of connectors, } t_d = \text{diaphragm thickness and } f_v = \text{shear stress.} \)

It is assumed in equation [4] that the connectors transfer all the shear force across the web/flange interface with the contribution of the vertical mortar joint being ignored.

British research (5) has established the mechanism of shear transfer across the interface. The mechanism is as illustrated in Figure 2, with the connectors deforming elastically until two plastic hinges are formed a distance \( j \) apart. British tests have indicated that the two plastic hinges can be taken to be at a distance apart of six times the connector thickness. One of the aims of the Australian investigation was to confirm the validity of this relationship.
4. EXPERIMENTAL PROGRAM

In conjunction with the preparation of the design manual, two sets of tests were carried out to confirm overseas design recommendations for wall profiles designed specifically for masonry housing in high wind areas and unique to Australian conditions. These walls typically consist of two skins of conventional 110 mm brickwork connected by a 50 mm thick diaphragm (resulting in an overall wall thickness of 270 mm, the same as that for a conventional cavity wall). The major advantage of this configuration is that composite action is achieved between each leaf, with a consequent significant increase in wall strength and stiffness. The diaphragm consists of 30 mm thick paving units, 230 mm wide and 160 mm high, with a 10 mm mortar collar joint between the paver and the inner and outer skins. The geometry of the paver is such that the top of the paver coincides with every second mortar joint, allowing the inclusion of metal shear connectors (usually conventional cavity ties) at these locations. These act as both shear connectors and as ties holding the two leaves together. This form of construction is unique to Australia, and is particularly suitable to masonry housing in cyclonic areas. The two aspects of design which required clarification were the effectiveness of the ties acting as shear connectors, and the effectiveness of this type of section (and similar) to resist water penetration.

4.1 Shear Connector Tests

The shear tests were aimed at checking the British relationships for design of connectors across a mortar joint between a diaphragm and a flange. To check this relationship two sets of specimens were constructed and tested in shear, one to simulate the normal connection of a flange to a web (Figure 3, specimens type A, B, C and W), the other to simulate the thin wall housing detail described earlier (Figure 3, specimens type W). In both cases a symmetric double shear arrangement was used for the tests. Initially 15 specimens were built and tested (referred to as Stage 1). To more clearly define the shear mechanism a further tests were subsequently performed (referred to as Stage 2).

All specimens were of “I” cross section and were built in the two different configurations. For the wide wall specimens, a vertical damp proof membrane was incorporated between the web and the flange to eliminate any shear contribution from the mortar joint, thus ensuring that all shear forces were transferred by the shear connectors. For the narrow wall
specimens, no membrane was included, as the performance of the composite section (as built) was to be checked (in this case the wall ties possibly act as both connectors and tension ties preventing the two skins from spreading apart). To stabilise the arrangement a preload of 2.6 kN was applied to each flange representing a stress of 0.05 MPa (typical of that which may be present in practice).

Each I section was loaded symmetrically in shear by applying a uniform vertical load to the web of the specimen using a hydraulic jack. The effects of flange prestress on the shear behaviour has been reported by Phipps (6), and this was considered in the assessment of the results. During the test the relative displacement between the flange and the web was continuously monitored at four locations by potentiometric transducers. The load was applied until a large shear displacement (in the order of 30 mm) was obtained. For each test, load-deflection graphs was plotted for every position and the average of all four positions obtained.

A typical load-deflection curve is shown in Figure 4. Loads corresponding to the change in slope of the curve (corresponding to the start of hinge formation) and the ultimate load where the curve reaches a plateau (corresponding to the final hinge mechanism) are summarised in Table 1. For purposes of comparison with the British shear results, the distance between the nominal plastic hinges \( j \) was back-calculated from the equations:

\[
j = \frac{f_n \times r \times u^2}{2 \times V_c} \quad \text{for a connector of rectangular cross section,} \quad [5]
\]

\[
j = \frac{1.33 \times f_n \times d^3}{V_c} \quad \text{for a connector of circular cross section.} \quad [6]
\]

It can be seen from Table 1 that the calculated values for \( j \) are significantly different to the suggested British value of six times the connector thickness, with the value varying from 1.2 to 2.5 for the rectangular cross section connectors, and 0.5 to 0.9 times the diameter for the circular cross section wire ties. The actual distance between the plastic hinges in the tests could not be determined with any accuracy because of the large shear displacement that was imposed on the specimens during each test. The reason for this large difference between the obtained value and that used in British design practice was unclear and a second series of tests were performed to more clearly define this mechanism (Stage 2).
Stage 2 Tests

A smaller second series of tests were performed in an attempt to more clearly define the distance between the plastic hinges on the shear connectors. Five specimens were constructed for the second stage, one specimen for each wall type. The specimens were made and cured in the same way as the specimens tested in the first stage. The testing set up and the testing procedures were the same as in the first stage except that the relative displacement between the flange and the web was carefully monitored and the test stopped at a shear deflection of about 10 mm. Stopping the test at this smaller displacement allowed the distance between plastic hinges on the shear connectors $j$ to be measured on each connector after the completion of the test.

The failure loads for each of the tests are summarised in Table 1. The design distance between the plastic hinges $j$ was back-calculated from the equations 5 and 6 and the actual distance $j$ was measured from the shear connectors. It can be seen from Table 1 that the measured values for $j$ are very close to the suggested British value of six times the connector thickness. If the distance $j$ is back-calculated from the British Design Equations (5 and 6) using the observed ultimate load, the calculated values are significantly different, with the value varying from 0.4 to 2.2 (similar to the first stage results). Conversely this means that the predicted ultimate load using $6u$ rather than a smaller value is conservative.

Table 1. Shear Connector Test Results - Stages 1 and 2

<table>
<thead>
<tr>
<th>SPECIMEN</th>
<th>DIMENSIONS OF SHEAR CONNECTORS (SC) OR WALL TIES (WT) (width x thickness x length) (mm)</th>
<th>FIRST HINGE LOAD (kN)</th>
<th>DISTANCE $j$ (back calculated) (mm)</th>
<th>ULTIMATE LOAD (kN)</th>
<th>DISTANCE $j$ (back calculated) (mm)</th>
<th>DISTANCE $j$ (measured) (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average A:</td>
<td>SC 25x3x120</td>
<td>37.3</td>
<td>6.2 = 2.1 u</td>
<td>43.3</td>
<td>5.4 = 1.8 u</td>
<td>-</td>
</tr>
<tr>
<td>Average B:</td>
<td>SC 20x3x120</td>
<td>25.3</td>
<td>7.35 = 2.5 u</td>
<td>31</td>
<td>5.9 = 2.0 u</td>
<td>-</td>
</tr>
<tr>
<td>Average C:</td>
<td>SC 10x3x120</td>
<td>20</td>
<td>4.68 = 1.6 u</td>
<td>25.3</td>
<td>3.7 = 1.2 u</td>
<td>-</td>
</tr>
<tr>
<td>Average W:</td>
<td>WT d = 3.85 mm</td>
<td>16.7</td>
<td>3.40 = 0.9 d</td>
<td>31.3</td>
<td>1.83 = 0.5 d</td>
<td>-</td>
</tr>
<tr>
<td>Average D:</td>
<td>WT d = 3.85 mm</td>
<td>180</td>
<td>*</td>
<td>180</td>
<td>*</td>
<td>-</td>
</tr>
<tr>
<td>A1</td>
<td>SC 25x3x120</td>
<td>35</td>
<td>6.6 = 2.2 u</td>
<td>65</td>
<td>3.6 = 1.2 u</td>
<td>18.2 = 6.1 u</td>
</tr>
<tr>
<td>B1</td>
<td>SC 20x3x120</td>
<td>30</td>
<td>6.2 = 2.1 u</td>
<td>70</td>
<td>2.6 = 0.9 u</td>
<td>17 = 5.7 u</td>
</tr>
<tr>
<td>C1</td>
<td>SC 10x3x120</td>
<td>15</td>
<td>6.2 = 2.1 u</td>
<td>35</td>
<td>2.7 = 0.9 u</td>
<td>18 = 6 u</td>
</tr>
<tr>
<td>W1</td>
<td>WT d = 3.85 mm</td>
<td>17</td>
<td>3.3 = 0.9 u</td>
<td>34</td>
<td>1.7 = 0.4 u</td>
<td>21 = 5.5 u</td>
</tr>
</tbody>
</table>

(*) Could not be calculated because of influence of mortar joints on the shear strength.

$d$ = diameter of connector

$u$ = thickness of connector

The factors affecting the location of the plastic hinges have been reported by Phipps and Montague (6) who indicate that the design equation ignores the influence of factors such as prestress (prestress enhances connector strength), the mortar around the connector (the mortar contributes to shear strength), and the unit strength (the higher the unit strength the greater the shear strength). The design equation is thus conservative as it only considers the contribution of the connector itself, and ignores the above effects. With this conservative approach, the value of $j$ is $6u$. If the other factors are considered, the value of $j$ approaches the observed value of (0.4 to 2.2) $u$. The connector carries the shear force across the joint in bending, making local contact in the masonry joint, and deforming elastically until two
plastic hinges are formed (at the position of maximum moments) \( j \) apart. The connector bears on portions of the surrounding masonry in a “prying” mechanism, producing bending in the connector. This results in the maximum moment in the connector being produced at a point inside the shear interface. The distance between two plastic hinges \( j \) is thus greater than the thickness of the vertical mortar joint, with the distance being influenced by the factors mentioned above.

4.2 Water Penetration Tests

The water penetration tests were aimed at checking the water penetration characteristic of the diaphragm walls. Five typical wall configurations were selected from the list of the walls to be included in the Design Manual. The results of these tests have been reported elsewhere (7). From the test results for thin walls in a severe exposure conditions it would be advisable to use a membrane on the interface of the flange and the web. For other exposures, and for thicker walls, water penetration appears not to be a problem. The results further highlighted the dependence of water penetration on workmanship and the shear connector detail.

5. DESIGN MANUAL

In order to encourage the use of diaphragm walls by designers, the Australian clay masonry industry is in the process of preparing a "user friendly" design manual. The research described in this paper provides background to the manual.

5.1 Overview of the Manual

The Design Manual will be used by structural engineers and architects, and will therefore cover both architectural and structural aspects. The Design Manual will contain information on the following:

- ARCHITECTURAL DESIGN
  Facade Options; Brick Type and Bond Pattern; Web Detailing; Openings - Window and Door Detailing; Roof / Wall Junction; Control Joints - Movement Gaps, Articulation Joints

- CONSTRUCTION ASPECTS
  Foundations and Footings; Material Selection; Construction Sequence, Propping, Weather Protection; Bond Beam and Roof Tie-Down; Roofing; Finishes and Cleaning

- PERFORMANCE ASPECTS
  Rain Resistance; Fire Resistance Levels; Acoustical Properties; Thermal Performance

- STRUCTURAL ASPECTS
  Conceptual Behaviour; Cross-Sectional Properties; Design for Compression; Design for Lateral Loads; Design for Shear

- DESIGN CHARTS
  Outline of Wall Types; Design Charts; Worked Examples
There are ten suggested wall types, with the wall thickness varying from 230 mm to 590 mm. The walls are made of standard or modular bricks with the diaphragms engaged, ladder (every second course engaged with no units in the courses between), or tied (see Figure 5).

The design curves are presented for a range of wall types. For every wall type four curves are presented: one based on the mechanism method; and three on the strength method (for different values of $f_{nt}$ of 0.2, 0.3 and 0.4 MPa). Each graph includes three or four curves corresponding to different distances between diaphragms. This allows the designer some flexibility in choosing the bonding pattern.

![Figure 5. Typical Example of Wall Types](image)

The designer is left to choose the type of the wall from a list of suggested wall geometries. Once the wall geometry has been chosen, either the Mechanism Method or the Strength Method for calculating the lateral load capacity must be nominated. If the Strength Method is chosen, an appropriate value for $f_{nt}$ must also be nominated, depending on the expected level of supervision on the designed building and the confidence in achieving the required bond strength. The designer can then select the appropriate chart, and for the required height of the diaphragm wall determine the lateral load capacity of the wall (in kPa) and the corresponding value of the clear distance between diaphragms $d$.

5.2 Design Curves

The design curves for lateral load capacity were generated from a computer program written for the design of propped cantilever diaphragm walls using the provisions of the Australian Masonry Code, AS3700 (1). Both the "strength" and "mechanism" methods of determining the lateral load capacity were included in the program, with design curves being generated for both methods. The choice of method in the Design Manual is left to the designer. The design curves for lateral load capacity have been plotted from the results of the analyses of each wall type. The intention of presenting design curves is to allow the designer to easily choose the lateral load capacity for a given wall geometry from the list of the suggested diaphragm wall geometries presented. These curves cover wall heights from 2 to 8 metres. Some of the curves exhibit discontinuities in slope in some locations because
of a change in the failure mechanism governing the design. Usually for low height walls shear resistance is the governing factor in assessing the lateral load capacity. For taller walls lateral load capacity governs with the capacity being governed by the vertical bending requirement. This is in turn influenced by the effective flange width which is a direct function of the height of the wall.

6. SUMMARY AND CONCLUSIONS

Diaphragm walls are a viable alternative to precast or tilt-up construction for tall walls or walls subjected to high lateral loads. Although this type of wall has been used in the United Kingdom for some time, the concept has not gained wide acceptance in Australia, mainly due to the ignorance on the part of engineers and architects of the effectiveness of the system. To overcome this ignorance and encourage greater use of masonry, an industry based design manual for diaphragm walls has been prepared. This paper has given an overview of the design manual and described some background research aimed at assessing the viability of the concept for Australian applications.

7. ACKNOWLEDGMENTS

The research described in this paper was partially funded by the Clay Brick and Paver Institute of New South Wales (Australia). Their support, and donation of material by its member companies (particularly Boral Bricks NSW Pty Ltd and PGH Bricks NSW), is gratefully acknowledged.

8. REFERENCES