FREE-STANDING MASONRY PRIVACY WALLS

Rod Johnston

Principal of Electronic Blueprint
Principal of Quasar Management Services Pty Ltd
(Consultant to the Concrete Masonry Association of Australia)

SUMMARY

Free-standing masonry privacy walls must be designed and constructed to withstand a range of loads, and in particular, wind and earthquake loads. This paper provides a detailed description of the design process and the determination of:

- Wind loads for various locations and exposures
- Earthquake loads
- Active and passive soil pressures that affect the stability of the system
- Pier dimensions to provide stability, including the relevant structure/soil interaction
- Pier and masonry reinforcement design
- Detailing of masonry privacy walls.

BACKGROUND

The collapse of a number of free-standing masonry privacy walls under extreme wind has prompted the Queensland government consider regulating their design and construction. Free-standing masonry privacy walls must be designed and constructed to withstand a range of loads, and in particular, wind loads. There are several possible designs for masonry privacy walls, two of which are shown below. The diagrams and tables herein are from *Concrete Masonry Fences*, Data Sheet 5, Concrete Masonry Association of Australia, May 2007.

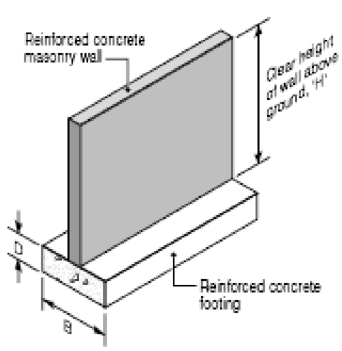

Figure 1 *Reinforced Concrete Masonry Wall on Reinforced Concrete Strip Footings*

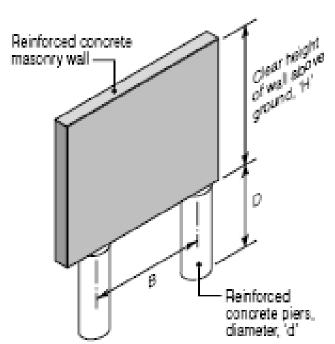

Figure 2 *Reinforced Concrete Masonry Wall with Reinforced Concrete Piers*

REINFORCED MASONRY WALL ON CONCRETE STRIP FOOTINGS

The type of retaining wall shown in Figure 1 may be designed using the principles set out below for a wall with reinforced concrete piers (as per Figure 2), except that the resistance to overturning is provided by the combined weight of the wall and footing acting about an assumed point of rotation close to the toe of the footing. The distance from the toe to the point of rotation depends on the bearing capacity of the foundation soil, including its compaction. It the soil is firm with a high bearing capacity, the point of rotation will be close to the toe. If the soil is soft with a low bearing capacity, the point of rotation will move closer to the centre of the footing. A reasonably conservative assumption is that the point about which the footing rotates is approximately B/3 from the toe of the footing, where B is the total footing width.

REINFORCED MASONRY WALL WITH REINFORCED CONCRETE PIERS

In most circumstances, the economical form of construction for free-standing masonry privacy walls is as shown in Figure 2. The wall consists of 190 mm hollow concrete blockwork, with a reinforced bond beam and capping block at the top and a reinforced bond beam at the bottom. The bond beams should include a single horizontal 16 mm diameter reinforcing bar, set in 190 mm knock-out bond beam blocks. The wall is supported, at centres ranging from 1.8 m to 3.0 m, by 450 mm diameter reinforced concrete piers, constructed in holes bored to the required depths and spacings. The depths of 450 mm piers, for various combinations of pier spacing, soil type (internal friction angle), wall heights and wind classifications, may be calculated by the methodology shown below. Each pier should include one (or more) reinforcing bar, which extends up and is grouted into the 190 mm concrete blockwork. The required number of vertical bars depends on the spacing of the piers, the wall height and wind classification.

WIND LOADS

Wind loads on free-standing masonry privacy walls should be calculated using AS/NZS 1170.2. However, designers often associate these structures with the design of houses to AS 4055. Strictly speaking, masonry privacy walls are outside the scope of AS 4055, although the nomenclature used therein is useful in classifying the wind exposure of housing sites for wind loads on such structures.

Table 1 *Wind Classification for Free-Standing Fences and Walls*

Wind Classification	Design gust wind speed at height 'h' V_{zu} (m/s)	Ultimate free-stream gust dynamic wind pressure q_{zu} (kPa)	Ultimate net wind pressure on free-standing wall p_{nu} (kPa)
$N1_f$	34	0.69	0.83
$N2_f$	40	0.96	1.15
$N3_f$ $C1_f$	50	1.50	1.80
$N4_f$ $C2_f$	61	2.23	2.68
$N5_f$ $C3_f$	74	3.29	3.94
$N6_f$ $C4_f$	86	4.44	5.33

Note: Design pressure is based on an aerodynamic shape factor, C_{fig}, of 1.20

EARTHQUAKE LOADS

Earthquake loads should be calculated using AS 1170.4. Method EDC I. For simple structures in most Australian applications, this permits the lateral earthquake inertia load to be assumed to be 10% of the seismic weight. For a typical 190 mm hollow block privacy wall, the average blockwork weight is 245 kg/m^2 or 2.40 kN/m^2, and the resulting horizontal earthquake inertia force is 0.24 kPa. This is significantly less that the expected wind loads shown above.

SOIL PROPERTIES

Soil properties used to determine the resistance to overturning of the piers should be based on reduction factors given in AS 4678 and “cautious estimates of the mean” density, internal and external friction angles and cohesion.

PIER RESISTANCE

The overturning resistance of piers supporting free-standing masonry privacy walls may be based on the principles for laterally loaded “short” piles set out in AS 2159.

The method of determining the soil lateral resistance, employed in the worked example below, is based on *Lateral Resistance on Piles in Cohesionless Soils*, by B.B. Broms (May 1964). For a single short pier in cohesionless soil, this paper suggests that the resistance be determined from the design passive resistance multiplied by a factor of 3.0 (designated k_{pier} in the example). A similar paper, *Lateral Resistance on Piles in Cohesive Soils*, by B.B. Broms (March 1964) covers cohesive soils. There are other more recent papers describing methods of varying complexity, based on tests and/or theory. However, the selected method has been chosen for its simplicity, in the context of the fact that there is relatively low risk associated with privacy walls with a maximum height of approximately 1.8 m.

The assumed distribution of pressures resisting the overturning moments are as follows.

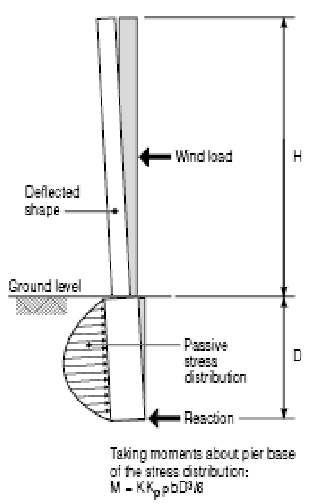

Figure 3 *Distribution of Pressures Resisting the Overturning Moments*

Position	Movement
Bottom of pier	Zero movement. There is a reactive force against the base “kicking back” into the soil. The magnitude of the force is relatively large, and a function of the passive pressure at the base of the pier. Although spread over a small increment of depth, it is assumed to be a point reaction.
Mid-height of pier	Movement assumed to be same as that which would occur under a uniformly-distributed horizontal force, equal in magnitude to the total passive resistance.
Top of pier	Movement assumed to be twice the movement at mid-height.

WORKED EXAMPLE

Set out in the following pages is a worked example, the purpose of which is to demonstrate the method by which free-standing masonry privacy walls may be designed for a particular wind and earthquake loads, and soil type. It also may serve as a test for any software developed for designing masonry privacy walls.

Design Brief

The objective is to design a 1.8 m high free-standing masonry privacy wall located in a Sydney suburb, on a gentle slope (with 60 metres upwind distance to the crest of a 4.0 m hill) and shielded by houses of 3.0 m roof height and 7.0 m width. The piers will be set in "in-situ" sandy-clay material (with cautious estimates of the means of density 20 kN/m^3, internal angle of friction 30^o and cohesion 5.0 kPa).

Wind Load using AS/NZS 1170.2-2002

Region A
Degree of hazard 2
Location Non-cyclonic
Design event for safety 1 in 500
Regional wind speed, V_R = 45 m/s AS/NZS 1170.2 Table 3.1
Regional wind multiplier, M_d = 1.0 AS/NZS 1170.2 Clause 3.3.1
Terrain category multiplier, h < 3.0 m $M_{z, cat}$ = 0.91 AS/NZS 1170.2 Table 4.1(A)
Number of upwind shielding buildings within a 45^o sector of 20 h radius, n_s = 2
Average roof height of shielding, h_s = 3.0 m
Average spacing of shielding buildings, $l_s = h (10 / n_s + 5)$
$= 1.8 ([10 / 2] + 5)$
= 18.0 m
Average breadth of shielding buildings, b_s = 7.0 m
Shielding parameter, $s = l_s / (h_s b_s)^{0.5}$
$= 18.0 / (3.0 \text{ x } 7.0)^{0.5}$
= 3.93 AS/NZS 1170.2 Table 4.3.3
Shielding multiplier, M_s = 0.830 Interpolated AS/NZS 1170.2 Table 4.3
Height of the hill, ridge or escarpment, H = 4.0 m
Horizontal distance upwind from crest L_u = 60.0 m
Windward slope $H/2L_u$ = 4.0 / (2 x 60.0)
= 0.033 < 0.05
Topography multiplier M_t = 1.00 AS/NZS 1170.2 Clause 4.4.2
Ultimate design gust wind speed, $V_{z\,u} = V_R M_d (M_{z, cat} M_s M_t)$
= 45.0 x 1.0 x 0.91 x 0.830 x 1.0
= 34.0 m/s
Ult free stream gust dynamic pressure, $q_{zu} = 0.0006 V_{z\,u}^2$ AS/NZS 1170.2 2.4.1
$= 0.0006 \text{ x } 34.0^2$
= 0.69 kPa

Structure Geometry

Height of wall, h = 1.8 m
Solid height of wall, c = 1.8 m
Total length of wall, b = 9.0 m
Length/solid height, b/c = 9.0 / 1.8
= 5.0
Solid height/total height, c/h = 1.8 / 1.8

$= 1.0$

Angle of incident wind (Normal = 0), $\theta = 0$

Porosity reduction factor, $K_p = 1 - (1 - \delta)^2$ AS/NZS 1170.2 D2.1

$= 1 - (1 - 1)^2$

$= 1.0$

Length of wall between vertical supports, $B' = 2.4$ m

Wind Load

Net pressure coefficient, $C_{pn} = 1.3 + 0.5\,(0.3 + \log_{10}(b/c))\,(0.8 - c/h)$

$= 1.3 + 0.5\,(0.3 + \log_{10}(5.0))\,(0.8 - 1.0)$

$= 1.20$ AS/NZS 1170.2 Table D2(A)

Aerodynamic shape factor, $C_{fig} = C_{pn} K_p$

$= 1.20 \times 1.0$

$= 1.20$ AS/NZS 1170.2 D1.4

Ultimate net wind pressure, $p_{nu} = C_{fig}\, q_{zu}$

$= 1.20 \times 0.695$

$= 0.834$ kPa AS/NZS 1170.2 2.4.1

Earthquake Load

Hazard factor for Sydney $z = 0.08$

Subsoil classification C

Seismic weight $W = 2.4$ kN/m^2

Lateral earthquake load $p_e = 0.10\, W$

$= 0.10 \times 2.40$

$= 0.240$ kPa < 0.834 kPa Base design on wind

Load factors and capacity reduction factors

Load factor on overturning wind $G_w = 1.0$

Load factor on restoring forces $G_r = 0.8$

Shear force and bending moments at the base of wall

Shear force at base $V_b = \gamma\, p_{n\,u}\, B'\, h$

$= 1.0 \times 0.834 \times 2.4 \times 1.80$

$= 3.60$ kN

Bending moment at base $M_b = 0.5\, G_w\, p_{n\,u}\, B'\, h^2$

$= 0.5 \times 1.0 \times 0.834 \times 2.4 \times 1.80^2$

$= 3.22$ kN.m

Foundation soil

The piers will be set in “in-situ” sandy-clay material with the following properties.
Any over-excavation should be filled with compacted cement-stabilised road base.
Design will be to the principles set out in AS 4678.

Density $\rho_f = 19.6$ kN/m^3 (Cautious estimate of mean)

Internal angle of friction $\varphi_f = 30^o$ (Cautious estimate of mean)

Cohesion, $c_f = 5.0$ kPa (Cautious estimate of mean)

Design properties of soil

Foundation soil factor on $\tan(\varphi_f)$ $\Phi_{\tan(\varphi f)} = 0.85$

Foundation soil factor on cohesion $\Phi_{cf} = 0.70$

Foundation soil design internal friction $\varphi^*_f = \tan^{-1}[\Phi_{\tan(\varphi f)}.\ \tan(\varphi_f)]$

$= \tan^{-1}[0.85.\ \tan(30^o)]$

$= 26.1^o$

Foundation soil design cohesion, $c^*_f = \Phi_{cf} \cdot c_f$
$= 0.70 \times 5.0$
$= 3.5$ kPa

Passive pressure coefficient $K_p = \dfrac{1 + \sin(\varphi^*_f)}{1 - \sin(\varphi^*_f)}$
$= \dfrac{1 + \sin(26.1^o)}{1 - \sin(26.1^o)}$
$= 2.58$

Pier details

Total depth of pier, $D = 0.900$ m This value will be checked

Pier diameter, $d_{pier} = 0.450$ m

The following calculations convert a circular pier to an equivalent square pier of the same overall cross-sectional area. By using this effective square section, the designer can have confidence in the calculated weight of pier, and the effective horizontal lever arms from an assumed point of rotation.

Effective pier thickness, $T_p = (3.1416 / 4)^{0.5}\ d_{pier}$
$= (3.1416 / 4)^{0.5}\ 0.450$
$= 0.399$ m

Effective pier length along the wall $L_p = 0.399$ m

Overturning Analysis

When piers push into a soil, the resistance is significantly greater than the passive resistance normally associated with the cross section of the pier. The multiplier to account for this increased lateral resistance of piers pushing into a body of soil is assumed to be $k_{pier} = 3.0$

As the horizontal force increases, the wall support will rotate about its base, pushing forward into the soil. The movement will vary linearly from the maximum at the ground surface to zero at the bottom of the support.

The resistance to this movement is provided by the passive resistance of the soil in front of the support. Under uniform movement, the passive pressure varies uniformly from zero at the surface to a maximum at the base of the support.

Passive force over the total depth, $P_p = G_r\ k_{pier}\ K_p\ \rho\ L_p\ D^2 / 3$
$= 0.8 \times 3.0 \times 2.58 \times 19.6 \times 0.399 \times 0.900^2 / 3$
$= 13.1$ kN.m

Lever arm of passive force, $y_p = D / 2$
$= 0.900 / 2$
$= 0.450$ m

Restoring moment due to passive force $M_p = P_p\ y_p$
$= 13.1 \times 0.450$
$= 5.87$ kN.m

Factored weight of wall , $P_{vw} = G_r\ \rho_w\ h\ t\ b$
$= 0.8 \times 16.0 \times 1.800 \times 0.19 \times 2.4$
$= 10.5$ kN

Lever arm of wall weight, $y_w = T_p\ (0.5 - 0.167)$
$= 0.399\ (0.5 - 0.167)$
$= 0.133$ m

Restoring moment due to wall weight, $M_w = P_{vw}\ y_w$
$= 10.5 \times 0.133$
$= 1.40$ kN.m

Factored weight of pier/footing, $P_{vf} = G_r \rho_f T_f L_f D$
$= 0.8 \times 23.5 \times 0.399 \times 0.399 \times 0.900$
$= 2.69$ kN

Lever arm of pier $y_f = T_p (0.5 – 0.167)$
$= 0.399 (0.5 - 0.167)$
$= 0.133$ m

Restoring moment due to pier $M_f = P_{vf} y_f$
$= 2.69 \times 0.133$
$= 0.36$ kN.m

Total restoring moment, $M_R = M_{p} + M_{w} + M_f$
$= 5.87 + 1.40 + 0.36$
$= 7.63$ kN.m

Bending moment due to wind $M_b = G_w p_{n\,u} B h (h/2 + D)$
$= 1.0 \times 0.834 \times 2.40 \times 1.80 \times (1.80 / 2 + 0.900)$
$= 6.48$ kN.m < 7.63 kN.m OK

Reinforced Masonry Pier Design

- Concrete blocks: Width 190 mm, Strength grade 15 MPa
- Blockwork will be built continuous for a length of 2.4 m, with a pier located at the centre and articulation joints (joints permitting relative movement due to soil expansion and contraction) at each end.
- Main reinforcement 1-16 mm diameter bar in the centre of the pier (500 MPa yield)

Masonry Properties

Block characteristic compressive strength $f'_{uc} = 15.0$ MPa

Block type factor, $k_m = 1.6$ (Hollow blocks)

Equivalent brickwork strength, $f'_{mb} = k_m (f'_{uc})^{0.5}$
$= 1.6 (15.0)^{0.5}$
$= 6.20$ MPa

Mortar joint height, $h_j = 10$ mm

Masonry unit height, $h_b = 190$ mm

Ratio of block to joint thickness, $h_b/h_j = 190/10$
$= 19.0$

Block height factor $k_h = 1.3$

Characteristic masonry strength, $f'_m = k_h f'_{mb}$
$= 1.3 \times 6.20$
$= 8.06$ MPa

Concrete Grout Properties

Concrete grout shall comply with AS 3700 and have:

- a minimum portland cement content of 300 kg/cubic metre;
- a maximum aggregate size of 10 mm;
- sufficient slump to completely fill the cores; and
- a minimum compressive cylinder strength of 20 MPa.

Specified characteristic grout strength $f'_c = 20$ MPa > 12 MPa OK AS 3700 Clause 5.6

Design characteristic grout strength, $f'_{cg} = \min [(1.3 \times f'_{uc}), 20.0]$ AS 3700 Clause 3.5
$= \min [(1.3 \times 15), 20.0]$
$= \min [19.5, 20.0]$
$= 19.5$ MPa

Dimensions and Properties of Reinforced Concrete Masonry Pier

The most adverse loading is on the pier near the middle of the wall

Width of pier (along the wall), $B = 390$ mm

Depth of pier (through the wall), $D = 190$ mm

Density of reinforced concrete masonry, $\rho_{mas} = 2{,}200$ kg/m^3

Modulus of elasticity, $E = 1{,}000\ f'_m$
$= 1{,}000 \times 8.06$
$= 8{,}060$ MPa

Second moment of area, $I = B\, D^3 / 12$
$= 390 \times 190^3 / 12$
$= 222.9 \times 10^6$ mm^4

Main Reinforcement

Main reinforcement yield strength, $f_{sy} = 500$ MPa

Main reinforcement shear strength (dowel), $f_{sv} = 17.5$ MPa

Number of main tensile reinforcing bars, $N_t = 1$

Diameter of main tensile reinforcing bars, $D_{dia\ t} = 16$ mm

Area of main reinforcement, $A_{st} = 200$ mm^2

Effective depth of reinforcement, $d = D/2$ (Centrally located reinforcement)
$= 190/2$
$= 95$ mm

Effective width of reinforced section, $b = \min$ (4D or 2D + length to end)
$= 4 \times 190$
$= 760$ mm

Shear width of reinforced section, $b_v = 200$ mm Only one core is grouted

Design area of main tensile reinforcement, $A_{sd} = \min [\ 0.29\ (1.3\ f'_m)\ b\ d / f_{sy}\ ,\ A_{st}\ ]$
$= \min [(0.29 \times 1.3 \times 8.06 \times 760 \times 95/500), 200]$
$= \min [\ 462\ ,\ 110\ ]$
$= 200$ mm^2

Reinforced Masonry Shear and Moment Capacity

Shear capacity, $\varphi V = \varphi\ (f'_{vm}\, b_w\, d + f_{vs}\, A_{st} + f_{sy\ f}\, A_{sv}\, d / s)$
$= 0.75\ [(0.35 \times 200 \times 95) + (17.5 \times 200) + 0] / 1000$
$= 0.75\ (6.65 + 3.50 + 0)$
$= 7.61$ kN

Bending capacity, $\varphi M = \varphi\ f_{sy}\, A_{sd}\ d\ [1 - 0.6\ f_{sy}\, A_{sd}\, d / (1.3\ f'_m\, b\, d)]$
$= 0.75 \times 500 \times 200 \times 95[1 - (0.6 \times 500 \times 200)/(1.3 \times 8.06 \times 760 \times 95)]/!0^6$
$= 6.56$ kN.m

Reinforced Masonry Lateral Load Capacity

Height of cantilever above piers, $L_c = 1.800$ m

Load capacity (limited by shear) $W_{vu} = 1.0\ \varphi V / B\, L_c$
$= 1.0 \times 7.61 / (2.400 \times 1.800)$
$= 1.76$ kPa

Load capacity (limited by bending) $W_{mu} = 2\ \varphi M / B\, L_c^2$
$= 2 \times 6.56 / (2.400 \times 1.800^2)$
$= 1.69$ kPa

Load capacity (considering shear & bending), $W_{lu} = \min (W_{vu}\ , W_{mu})$
$= \min (1.76, 1.69)$
$= 1.69$ kPa > 0.834 kPa OK

CONCLUSIONS AND RECOMMENDATIONS

1. The worked example herein describes a methodology for the design of free-standing masonry privacy walls for various combinations of wind, earthquake and soil conditions.

2. Wind loads on free-standing masonry privacy walls should be calculated using AS/NZS 1170.2-2002.

3. Although masonry privacy walls are outside the scope of AS 4055-2006, it is recommended that a system of suitable wind load classifications, using nomenclature compatible with AS 4055 be adopted.

4. It is recommended that earthquake loads be calculated using AS 1170.4-2007, Method EDC I, which permits the lateral earthquake inertia load to be assumed to be 10% of the seismic weight.

5. Although masonry privacy walls are outside the scope of both AS 4678 and AS 2159, it is recommended that the determination of design soil properties be carried out in accordance with AS 4678-2002 and the pier analysis be in accordance with AS 2159-1995. A method of analysis based on the recommendations of Broms (1964) has been described, although the choice of analysis remains at the discretion of the design engineer.

REFERENCES

AS/NZS 1170.2-2002 *Wind loads*, Standards Australia

AS 1170.4-2007 *Earthquake actions in Australia,* Standards Australia

AS 2159-1995 *Piling – Design and installation,* Standards Australia

AS 3700-2001 *Masonry structures,* Standards Australia

AS 4055-2006 *Wind loads for housing*, Standards Australia

AS 4678-2002 *Earth retaining structures,* Standards Australia

Broms, B.B., *Lateral Resistance on Piles in Cohesive Soils*, Journal of the Soil Mechanical and Foundations Division, Proceedings of the American Society of Civil Engineers, pp 27-63, March 1964.

Broms, B.B., *Lateral Resistance on Piles in Cohesionless Soils*, Journal of the Soil Mechanical and Foundations Division, Proceedings of the American Society of Civil Engineers, pp 123-156, May 1964.

Concrete Masonry Fences, Data Sheet 5, Concrete Masonry Association of Australia, May 2007

Johnston, R.K, *Design of Masonry Structures for Earthquake in Australia,* 14th International Brick and Block Masonry Conference, Australia, 2008.