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CONCRETE MASONRY FENCES

This data sheet is applicable to any free-standing, cantilever fence or wall for residential or commercial applications.


PART A: CONCRETE MASONRY FENCES BUILT ON REINFORCED CONCRETE PIERS

1 INTRODUCTION

Free-standing concrete masonry fences and boundary walls must be designed and constructed to withstand a range of loads, and in particular, wind loads. This data sheet provides guidance to qualified and experienced structural engineers on the selection of pier dimensions and masonry details for free-standing reinforced concrete masonry fences and walls, subject to a range of wind loads and set in a variety of soils.

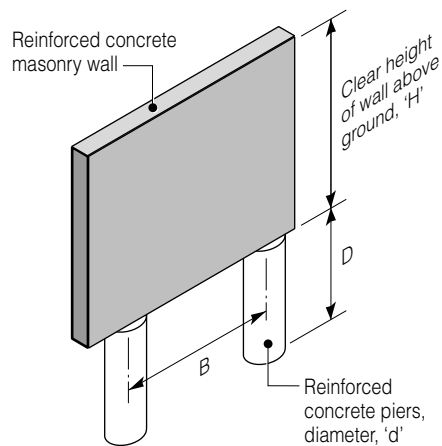
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2 STANDARD DESIGNS

There are many possible designs for concrete masonry fences and boundary walls. Two common arrangements are shown in **Figures 1** and **2**.



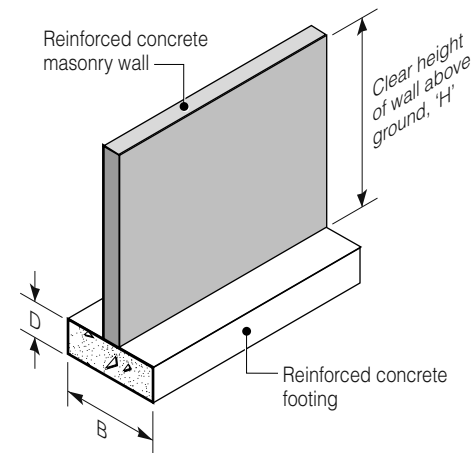
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Figure 1 Reinforced Concrete Masonry Wall with Reinforced Concrete Piers

Reinforced concrete masonry wall with reinforced concrete piers

In most circumstances, the most economical form of construction for free-standing concrete masonry fences and boundary walls is as follows (**Figure 1**):

- 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 N16



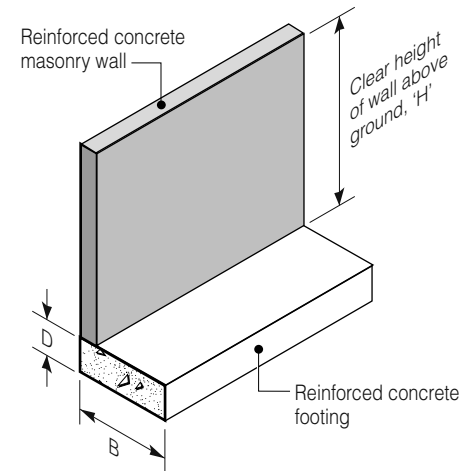
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Figure 2 Reinforced Concrete Masonry Wall on Reinforced Concrete Strip Footings

reinforcing bar, set in 20.20 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. **Table A** sets out the recommended

depths of 450mm piers, for various combinations of pier spacing, soil type (internal friction angle), wall heights and wind classifications. Each pier should include one (or more) reinforcing bar, which extends



to the top bond beam 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. **Table B** sets out the recommended vertical reinforcement for 190 mm reinforced concrete masonry.

3 WIND LOADS

Wind loads on free-standing concrete masonry fences and boundary 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, boundary walls and fences 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.

The nomenclature used in this Data Sheet for the “Wind Classification for Free-Standing Fences and Walls” (N1_f to C4_f) has been adopted to differentiate it from the corresponding nomenclature, “Wind Classification for Housing” (N1 to C4), which is set out in AS 4055 for houses. Although the resulting ultimate free-stream gust dynamic wind pressures, designated q_{zu} , are the same, their derivation is different. The worked example below demonstrates the derivation for a “Wind Classification for Free-Standing Fences and Walls” of N1_f.

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

4 SOIL PROPERTIES

Soil properties used to determine the resistance to overturning of the piers for free-standing concrete masonry fences and boundary walls should be based on reduction factors given in AS 4678 and “cautious estimates of the mean” density, internal friction angle and cohesion as defined in AS 4678.

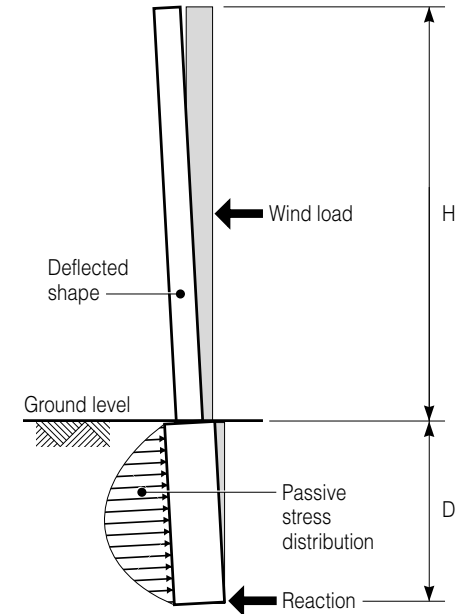
5 PIER RESISTANCE

The resistance of piers of free-standing concrete masonry fences and boundary walls to overturning of the piers will be based on the principles for laterally-loaded piles set out in AS 2159. In particular, the resistance of a single isolated pier will be taken as three times the calculated passive resistance based on the factored mean internal angle of friction.

Figure 3 shows the distribution of pressures resisting the overturning moments and Table 2 the assumed forward movement of the pier.

Table 2 Assumed Forward Movement of Piers

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.



Taking moments about pier base of the stress distribution:
 $M = KK_p \rho b D^3 / 6$

Figure 3 Distribution of Pressures Resisting the Overturning Moments

6 DESIGN TABLES

Tables A and B set out the required depths of 450 mm diameter piers for free-standing fences and walls and required connection reinforcement between piers and 190-mm reinforced concrete masonry respectively.

Table A Required Depth of 450 mm Diameter Piers for Free-Standing Fences and Walls

Pier spacing, B (m)	Soil friction angle	Wall height, H (m)	Required depth of piers, D (m), for wind loads						Pier spacing, B (m)	Soil friction angle	Wall height, H (m)	Required depth of piers, D (m), for wind loads					
			N1 _f	N2 _f	N3 _f	N4 _f	N5 _f	N6 _f				N1 _f	N2 _f	N3 _f	N4 _f	N5 _f	N6 _f
3.00	25°	1.80	1.01	1.20	1.50	1.80	2.13	2.40	1.80	25°	1.80	0.80	0.95	1.19	1.42	1.68	1.90
		1.60	0.92	1.11	1.39	1.67	1.98	2.24			1.60	0.73	0.87	1.10	1.32	1.57	1.77
		1.40	0.84	1.01	1.27	1.54	1.83	2.08			1.40	0.66	0.80	1.00	1.21	1.45	1.64
		1.20	0.75	0.91	1.15	1.39	1.67	1.90			1.20	0.59	0.72	0.91	1.10	1.32	1.51
		1.00	0.65	0.80	1.02	1.24	1.50	1.72			1.00	0.51	0.63	0.80	0.98	1.18	1.36
	30°	1.80	0.94	1.11	1.39	1.67	1.97	2.22		30°	1.80	0.74	0.88	1.10	1.32	1.56	1.76
		1.60	0.86	1.02	1.29	1.55	1.84	2.08			1.60	0.68	0.81	1.02	1.22	1.45	1.64
		1.40	0.77	0.93	1.18	1.42	1.70	1.93			1.40	0.61	0.74	0.93	1.12	1.34	1.52
		1.20	0.69	0.84	1.06	1.29	1.55	1.76			1.20	0.55	0.66	0.84	1.02	1.22	1.39
		1.00	0.60	0.74	0.94	1.15	1.39	1.59			1.00	0.48	0.59	0.74	0.91	1.10	1.26
	35°	1.80	0.87	1.03	1.29	1.54	1.83	2.06		35°	1.80	0.68	0.81	1.02	1.22	1.44	1.63
		1.60	0.79	0.95	1.19	1.43	1.70	1.92			1.60	0.63	0.75	0.94	1.13	1.34	1.52
		1.40	0.72	0.87	1.09	1.32	1.57	1.78			1.40	0.57	0.68	0.86	1.04	1.24	1.41
		1.20	0.64	0.78	0.98	1.20	1.43	1.63			1.20	0.50	0.61	0.78	0.94	1.13	1.29
		1.00	0.56	0.69	0.87	1.07	1.28	1.47			1.00	0.44	0.54	0.69	0.84	1.01	1.16
2.40	25°	1.80	0.91	1.08	1.35	1.62	1.92	2.16	Notes:	25°	1.80	0.80	0.95	1.19	1.42	1.68	1.90
		1.60	0.83	1.00	1.25	1.51	1.79	2.02			1.60	0.73	0.87	1.10	1.32	1.57	1.77
		1.40	0.75	0.91	1.14	1.38	1.65	1.87			1.40	0.66	0.80	1.00	1.21	1.45	1.64
		1.20	0.67	0.82	1.03	1.26	1.50	1.72			1.20	0.59	0.72	0.91	1.10	1.32	1.51
		1.00	0.59	0.72	0.92	1.12	1.35	1.55			1.00	0.51	0.63	0.80	0.98	1.18	1.36
	30°	1.80	0.84	1.00	1.25	1.50	1.78	2.00		30°	1.80	0.74	0.88	1.10	1.32	1.56	1.76
		1.60	0.77	0.92	1.16	1.39	1.66	1.87			1.60	0.68	0.81	1.02	1.22	1.45	1.64
		1.40	0.70	0.84	1.06	1.28	1.53	1.73			1.40	0.61	0.74	0.93	1.12	1.34	1.52
		1.20	0.62	0.76	0.96	1.16	1.39	1.59			1.20	0.55	0.66	0.84	1.02	1.22	1.39
		1.00	0.54	0.67	0.85	1.04	1.25	1.43			1.00	0.48	0.59	0.74	0.91	1.10	1.26
	35°	1.80	0.78	0.93	1.16	1.39	1.65	1.85		35°	1.80	0.68	0.81	1.02	1.22	1.44	1.63
		1.60	0.71	0.85	1.07	1.29	1.53	1.73			1.60	0.63	0.75	0.94	1.13	1.34	1.52
		1.40	0.65	0.78	0.98	1.19	1.41	1.61			1.40	0.57	0.68	0.86	1.04	1.24	1.41
		1.20	0.58	0.70	0.89	1.08	1.29	1.47			1.20	0.50	0.61	0.78	0.94	1.13	1.29
		1.00	0.50	0.62	0.79	0.96	1.16	1.33			1.00	0.44	0.54	0.69	0.84	1.01	1.16

Notes:
 Calculations are based on 190 mm reinforced concrete masonry
 Wall height is the clear height of the wall above ground surface.
 Design pressure is based on an aerodynamic shape factor, C_{fig}, of 1.20

Table B

Required Vertical Reinforcement for Piers and 190-mm Reinforced Concrete Masonry for Free-Standing Fences and Walls

Pier spacing, B (m)	Wall height, H (m)	Required number and size of vertical bars, per pier, for wind loads					
		N1 _f	N2 _f	N3 _f	N4 _f	N5 _f	N6 _f
3.00	1.80	1-N16	1-N16	1-N20	2-N20	3-N20	4-N20
	1.60	1-N16	1-N16	1-N20	2-N20	3-N20	4-N20
	1.40	1-N16	1-N16	1-N16	2-N20	2-N20	3-N20
	1.20	1-N16	1-N16	1-N16	2-N20	2-N20	3-N20
	1.00	1-N16	1-N16	1-N16	1-N20	2-N20	2-N20
2.40	1.80	1-N16	1-N16	1-N20	2-N20	2-N20	4-N20
	1.60	1-N16	1-N16	1-N16	2-N20	2-N20	3-N20
	1.40	1-N16	1-N16	1-N16	1-N20	2-N20	2-N20
	1.20	1-N16	1-N16	1-N16	1-N20	2-N20	2-N20
	1.00	1-N16	1-N16	1-N16	1-N20	2-N20	2-N20
1.80	1.80	1-N16	1-N16	1-N16	1-N20	2-N20	2-N20
	1.60	1-N16	1-N16	1-N16	1-N20	2-N20	2-N20
	1.40	1-N16	1-N16	1-N16	1-N16	2-N20	2-N20
	1.20	1-N16	1-N16	1-N16	1-N16	1-N20	2-N20
	1.00	1-N16	1-N16	1-N16	1-N16	1-N20	2-N20

Notes:

Calculations are based on 190 mm reinforced concrete masonry

Height is the clear height of the wall above ground surface.

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

Where more than two bars are specified, it may be preferable to use external 'posts' rather than maintaining the 'post' within the 190-mm thickness of the wall.

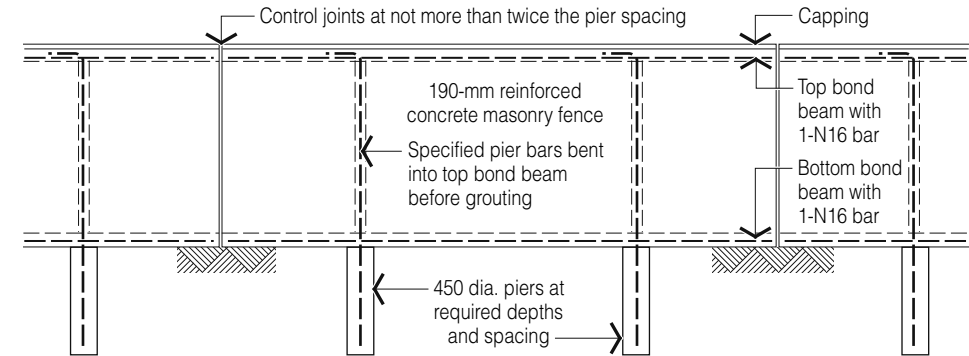


Figure 4 Typical Reinforcement Details and Control Joint locations

7 TYPICAL DETAILS

The vertical bars between the piers and 190-mm reinforced concrete masonry fence should be carried up to the top bond beam and bent down before grouting (**Figure 4**).

Where more than two bars are specified, it may be preferable to use external 'posts' rather than maintaining the 'post' within the 190 mm thickness of the wall (**Figure 5**).

It is recommended a control joint be placed centrally between piers at not more than twice the pier spacing (**Figure 4**).

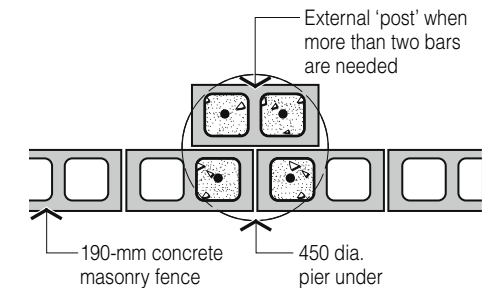


Figure 5 External 'Post' Detail

8 WORKED EXAMPLE

Set out following is a worked example, the purpose of which is to:

- Demonstrate the method by which free-standing concrete masonry fences and cantilever walls may be designed for a particular wind and soil; and
- Serve as a test for any software developed for designing concrete masonry fences and cantilever walls.

DESIGN BRIEF			
Design a 1.8 m high free-standing concrete masonry boundary 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 “insitu” sandy-clay material, with cautious estimates of the means of density 20 kN/m ³ , internal angle of friction 30° and cohesion 5.0 kPa.		Terrain category multiplier $M_{z,cat} = 0.91$ For $h < 3.0$ m AS/NZS 1170.2 Table 4.1(A)	Shielding multiplier $M_s = 0.830$ <i>Interpolated from AS/NZS 1170.2 Table 4.3</i>
		Number of upwind shielding buildings within a 45° sector of 20 h radius $n_s = 2$	Height of the hill, ridge or escarpment $H = 4.0$ m
		Average roof height of shielding buildings $h_s = 3.0$ m	Horizontal distance upwind from the crest of a hill, ridge or escarpment to a level half the height below the crest $L_u = 60.0$ m
WIND LOAD USING AS/NZS 1170.2:2002			Windward slope $H/2L_u = 4.0/(2 \times 60.0) = 0.033 < 0.05$
Region	A	Average spacing of shielding buildings $l_s = h(10/n_s + 5) = 1.8([10/2] + 5) = 18.0$ m	Topography multiplier $M_t = 1.00$ AS/NZS 1170.2 Clause 4.4.2
Degree of hazard	2		<i>Average slope</i> $\phi_s = 4 : 60 = 1 : 15$
Location	Non-cyclonic		
Design event for safety	1 in 500	Average breadth of shielding buildings $b_s = 7.0$ m	Ultimate design gust wind speed $V_{zu} = V_R M_d (M_{z,cat} M_s M_t) = 45.0 \times 1.0 \times 0.91 \times 0.830 \times 1.0 = 34.0$ m/s
Regional wind speed $V_R = 45$ m/s	AS/NZS 1170.2 Table 3.1	Shielding parameter $s = l_s / (h_s b_s)^{0.5}$ AS/NZS 1170.2 Clause 4.3.3	<i>Topography</i> T1 For $\phi_s < 1 : 10$ AS 4055 Clause 2.4, Table 2.3
Regional wind multiplier $M_d = 1.0$	AS/NZS 1170.2 Clause 3.3.1	$= 18.0 / (3.0 \times 7.0)^{0.5} = 3.93$	<i>Shielding</i> Partial Shielding (PS) AS 4055 Clause 2.5
			<i>Classification</i> N1 AS 4055 Clause 2.2, Table 2.2
			Ultimate free stream gust dynamic wind pressure $q_{zu} = 0.0006 V_{zu}^2 = 0.0006 \times 34.0^2 = 0.694$ kPa

Ultimate design gust wind speed $V_{hu} = 34.0 \text{ m/s}$ AS 4055 Clause 2.1, Table 2.1	Structure Geometry Height of wall $h = 1.8 \text{ m}$	Wind loads 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)
Ultimate free stream gust dynamic wind pressure $q_{zu} = 0.0006 V_{zu}^2$ $= 0.0006 \times 34.0^2$ $= 0.694 \text{ kPa}$	Solid height of wall $c = 1.8 \text{ m}$	Aerodynamic shape factor $C_{fig} = C_{pn} K_p$ $= 1.20 \times 1.0$ $= 1.20$	AS/NZS 1170.2 D2.1
Note For convenience, design tables will be prepared using the the ultimate design gust wind speed, V_{hu} , and the resulting ultimate free-stream gust dynamic wind pressure, q_{zu} , determined using AS 4055. This will enable the use of a wind classification nomenclature similar to that used in AS 4055. As indicated above, this may lead to small errors in the determination of pressure, but these are not considered significant.	Total length of wall $b = 9.0 \text{ m}$	Note For convenience, design tables will be prepared using the aerodynamic shape factor, C_{fig} , of 1.20 This may lead to small errors in the determination of pressure, but these are not considered significant.	
	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) $\Phi = 0$	Ultimate net wind pressure on free-standing wall $p_{nu} = C_{fig} q_{zu}$ $= 1.20 \times 0.695$ $= 0.833 \text{ kPa}$	AS/NZS 1170.2 Clause 2.4.1
	Porosity reduction factor $K_p = 1 - (1 - \delta)^2$ $= 1 - (1 - 1)^2$ $= 1.0$ AS/NZS 1170.2 D2.1		
	Length of wall between vertical supports $B' = 2.4 \text{ m}$	LOAD FACTORS AND CAPACITY REDUCTION FACTORS Load factor on overturning wind pressure $G_w = 1.0$ Load factor on restoring forces $G_r = 0.8$	AS 1170.0 2002 Clause 4.2.1(b)(iv) AS 4678 2002 Clause J3(c)

COHESION	Cohesion (cautious estimate of mean)	PIER DETAILS	Multiplier to account for lateral resistance of piers pushing into a body of soil
SHEAR FORCE AND BENDING MOMENTS AT THE BASE OF WALL	$c_f = 5.0 \text{ kPa}$	Total depth of pier	$k_{\text{pier}} = 3.0$
Shear force at base support of exposed wall	Design properties of soil	$D = 0.900 \text{ m}$	
$V_b = G_w p_{\text{nu}} B' h$ $= 1.0 \times 0.834 \times 2.4 \times 1.80$ $= 3.60 \text{ kN}$	Foundation soil partial factor on $\tan(\phi_f)$	Pier diameter	
	$\Phi_{\tan(\phi_f)} = 0.85$	$d_{\text{pier}} = 0.450 \text{ m}$	OVERTURNING ANALYSIS
		<i>Note</i>	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.
Bending moment at base of support of exposed wall	Foundation soil partial factor on cohesion	<i>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.</i>	
$M_b = 0.5 G_w p_{\text{nu}} B' h^2$ $= 0.5 \times 1.0 \times 0.834 \times 2.4 \times 1.80^2$ $= 3.22 \text{ kN.m}$	$\Phi_{c_f}^* = 0.70$		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.
	Foundation soil design internal friction angle	Effective pier thickness perpendicular to the wall	
FOUNDATION SOIL	$\phi_f^* = \tan^{-1}[\Phi_{\tan(\phi_f)} \tan(\phi_f)]$ $= \tan^{-1}[0.85 \tan(30^\circ)]$ $= 26.1^\circ$	$T_p = (3.1416/4)^{0.5} d_{\text{pier}}$ $= (3.1416/4)^{0.5} \times 0.450$ $= 0.399 \text{ m}$	Passive force over the total depth, D $P_p = G_r k_{\text{pier}} K_p \rho L_p D^2/3$ $= 0.8 \times 3.0 \times 2.58 \times 19.6 \times 0.399 \times 0.900^2/3$ $= 13.1 \text{ kN.m}$
The piers will be set in “insitu” sandy-clay material with the following properties.			
Any over-excavation should be filled with compacted cement-stabilised road base.	Foundation soil design cohesion	Effective pier length along the wall	Lever arm of passive force
Design will be to the principles set out in AS 4678.	$c_f^* = \Phi_{c_f} c_f$ $= 0.70 \times 5.0$ $= 3.5 \text{ kPa}$	$L_p = (3.1416/4)^{0.5} d_{\text{pier}}$ $= (3.1416/4)^{0.5} \times 0.450$ $= 0.399 \text{ m}$	$y_p = D/2$ $= 0.900/2$ $= 0.450 \text{ m}$
	Passive pressure coefficient of foundation soil		
Density (cautious estimate of mean)	$K_p = \frac{1 + \sin(\phi_f^*)}{1 - \sin(\phi_f^*)}$ $= \frac{1 + \sin(26.1^\circ)}{1 - \sin(26.1^\circ)}$ $= 2.58$		
$\rho_f = 20 \text{ kN/m}^3$			
Internal angle of friction (cautious estimate of the mean)			
$\phi_f = 30^\circ$			

Restoring moment about the base of passive force $M_p = P_p y_p$ $= 13.1 \times 0.450$ $= 5.87 \text{ kN.m}$	Restoring moment about centroid due to pier/footing weight $M_f = P_{vf} y_f$ $= 2.69 \times 0.133$ $= 0.36 \text{ kN.m}$	REINFORCED MASONRY 'POSTS' <ul style="list-style-type: none"> 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 at each end. Main reinforcement, 1-N16 bar in the centre of the pier 	Masonry unit height $h_b = 190 \text{ mm}$
Factored weight of wall $P_{vw} = G_r \rho_w h t b$ $= 0.8 \times 16.0 \times 1.8 \times 0.19 \times 2.4$ $= 10.5 \text{ kN}$	Total restoring moment $M_R = M_p + M_w + M_f$ $= 5.87 + 1.40 + 0.36$ $= 7.63 \text{ kN.m}$		Ratio of block to joint thickness $h_b/h_j = 190/10$ $= 19.0$
Lever arm of wall weight $y_w = T_p(0.5 - 0.167)$ $= 0.399(0.5 - 0.167)$ $= 0.133 \text{ m}$	Bending moment at base of pier from wind $M_b = G_w p_{nu} B' h (h/2 + D)$ $= 1.0 \times 0.828 \times 2.40 \times 1.80 \times (1.80 / 2 + 0.900)$ $= 6.48 \text{ kN.m}$	Masonry Properties Masonry unit characteristic unconfined compressive strength $f_{uc} = 15.0 \text{ MPa}$	Block height factor $k_h = 1.3$
Restoring moment about centroid due to wall weight $M_w = P_{vw} y_w$ $= 10.5 \times 0.133$ $= 1.40 \text{ kN.m}$	$< 7.63 \text{ kN.m}$ OK ie, wall is stable	Units are hollow	Characteristic masonry strength $f_m = k_h f_{mb}$ $= 1.3 \times 6.20$ $= 8.06 \text{ MPa}$
Factored weight of pier/footing $P_{vf} = G_r \rho_f T_f L_f D$ $= 0.8 \times 23.5 \times 0.399 \times 0.399 \times 0.9$ $= 2.69 \text{ kN}$		Block type factor $k_m = 1.6$	Concrete Grout Properties <i>Concrete grout specification:</i> <i>Concrete grout shall comply with AS 3700 and have:</i> <ul style="list-style-type: none"> minimum portland cement content of 300 kg/cubic metre; 10 mm maximum aggregate size; sufficient slump to completely fill the cores; and minimum compressive cylinder strength of 20 MPa.
Lever arm of pier/footing $y_f = T_p(0.5 - 0.167)$ $= 0.399(0.5 - 0.167)$ $= 0.133 \text{ m}$		Equivalent brickwork strength $f_{mb} = k_m(f_{uc})^{0.5}$ $= 1.6(15.0)^{0.5}$ $= 6.20 \text{ MPa}$	
		Mortar joint height $h_j = 10 \text{ mm}$	

Specified characteristic grout cylinder strength $f_c = 20 \text{ MPa}$ $> 12 \text{ MPa}$ OK <i>AS 3700 Clause 11.7.3</i>	Area of main reinforcement $A_{st} = N_t(3.1416 D_{dia,t}^2/4)$ (approx) $= 1 \times 3.1416 \times 16^2/4$ $= 200 \text{ mm}^2$	Main Reinforcement Effective depth of reinforcement For centrally located reinforcement: $d = D/2$ For reinforcement near one face shell: $d = D - d_1 + D_{dia,t}/2$ $= 190/2$ $= 95 \text{ mm}$	Fitments There are no shear reinforcement fitments required in this type of construction, which incorporates a single vertical reinforcing bar Fitment yield strength $f_{sy,f} = \text{NA}$
Design characteristic grout strength $f_{cg} = \min[(1.3 \times f_{uc}), 20.0]$ <i>AS 3700 Clause 3.5</i> $= \min[(1.3 \times 15), 20.0]$ $= \min[19.5, 20.0]$ $= 19.5 \text{ MPa}$	Dimensions The most adverse loading is on the pier near the middle of the wall Width of pier (along the wall) $B = 390 \text{ mm}$ Depth of pier (through the wall) $D = 190 \text{ mm}$	Effective width of reinforced section $b = \min(4D \text{ or } 2D + \text{length to structural end})$ $= 4 \times 190$ $= 760 \text{ mm}$ <i>AS 3700 Clause 8.5</i>	Fitment area $A_f = \text{NA}$ Fitment spacing $s = \text{NA}$
Main Reinforcement Properties Main reinforcement yield strength $f_{sy} = 500 \text{ MPa}$	Density of reinforced concrete masonry $\rho_{mas} = 2,200 \text{ kg/m}^3$	Shear width of reinforced section $b_v = 200 \text{ mm}$ <i>Note: Only one core is grouted</i>	
Main reinforcement shear strength (dowel action) $f_{sv} = 17.5 \text{ MPa}$	Modulus of elasticity $E = 1,000 f'_m$ $= 1,000 \times 8.06$ $= 8,060 \text{ MPa}$	Design area of main tensile reinforcement $A_{sd} = \min[0.29(1.3f'_m)bd/f_{sy}, A_{st}]$ $= \min[(0.29 \times 1.3 \times 8.06 \times 760 \times 95 / 500), 200]$ $= \min[462, 200]$ $= 200 \text{ mm}^2$	
Number of main tensile reinforcing bars $N_t = 1$	Second moment of area of reinforced concrete masonry pier $I = B D^3/12$ $= 390 \times 190^3/12$ $= 222.9 \times 10^6 \text{ mm}^4$		
Diameter of main tensile reinforcing bars $D_{dia,t} = 16 \text{ mm}$			

Reinforced Masonry Capacity			
Shear capacity	AS 3700 Clause 8.8	Load capacity (limited by deflection)	
$\phi V = \phi(f_{vm} b_w d + f_{vs} A_{st} + f_{sv} A_{sv} d/s)$		$W_{\Delta u} = \Delta_a E I / 48 L_c^4 B'$	
$= 0.75[(0.35 \times 200 \times 95) + (17.5 \times 200) + 0] / 1000$		$= [36 \times 8,060 \times 222.9 \times 10^6 / (48 \times 1.800^4 \times 2.400)] 10^{-9}$	
$= 0.75(6.65 + 3.50 + 0)$		$= 53.5 \text{ kPa}$	
$= 7.61 \text{ kN}$		Load capacity (limited by shear, bending moment or deflection)	
Bending Moment Capacity	AS 3700 Clause 8.6	$W_{lu} = \min(W_{vu}, W_{mu}, W_{\Delta u})$	
$\phi M = \phi f_{sy} A_{sd} d [1 - 0.6 f_{sy} A_{sd} d / (1.3 f'_m b d)]$		$= \min(1.76, 1.69, 53.5)$	
$= 0.75 \times 500 \times 200 \times 95 [1 - (0.6 \times 500 \times 200) / (1.3 \times 8.06 \times 760 \times 95)] / 10^6$		$= 1.69 \text{ kPa}$	
$= 6.56 \text{ kN.m}$		$> 0.834 \text{ kPa}$	OK
Height of cantilever wall above the piers			
$L_c = 1.800 \text{ m}$			
Limiting deflection			
$\Delta_a = L_c / 50$			
$= 1,800 / 50$			
$= 36 \text{ mm}$			
Load capacity (limited by shear)			
$W_{vu} = 1.0 \phi V / (B' L_c)$			
$= 1.0 \times 7.61 / (2.400 \times 1.800)$			
$= 1.76 \text{ kPa}$			
Load capacity (limited by bending moment)			
$W_{mu} = 2 \phi M / B' L_c^2$			
$= 2 \times 6.56 / (2.400 \times 1.800^2)$			
$= 1.69 \text{ kPa}$			