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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 B: CONCRETE MASONRY FENCES BUILT ON CONCRETE STRIP FOOTINGS

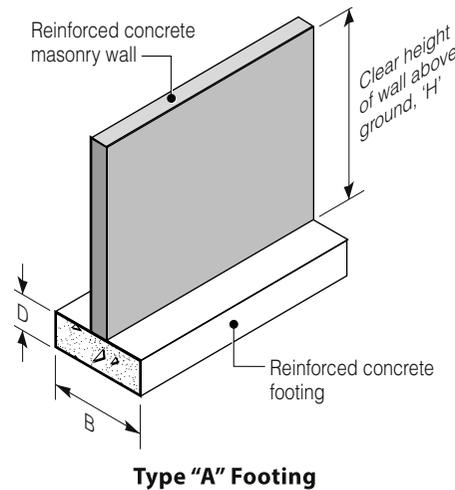
1 INTRODUCTION

Part B of this data sheet applies to 190mm wide partially reinforced concrete masonry walls located in the center of the footing or at the edge of the footing depending on the property boundary requirements.

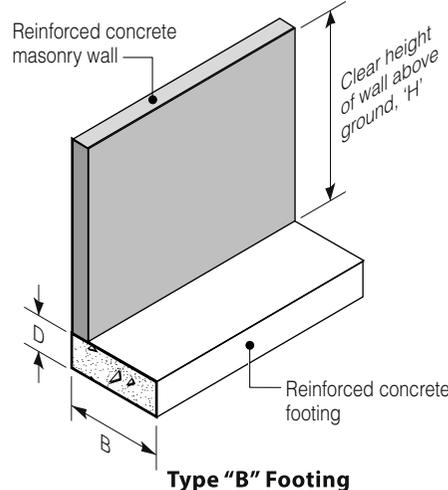
Free standing concrete masonry fences and boundary walls must be designed and constructed to withstand a range of loads, and in particular, wind loads. This manual provides guidance to qualified and experienced structural engineers on the selection of strip footing dimensions, wall steel spacing and masonry details for free standing reinforced concrete masonry fences and walls subject to a range of wind loads.

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

The vertical bars are N16 diameter and their spacing depends on the wall height and wind classification.



Type "A" Footing



Type "B" Footing

Figure 1 Reinforced Concrete Masonry Wall on Reinforced Concrete Strip Footings

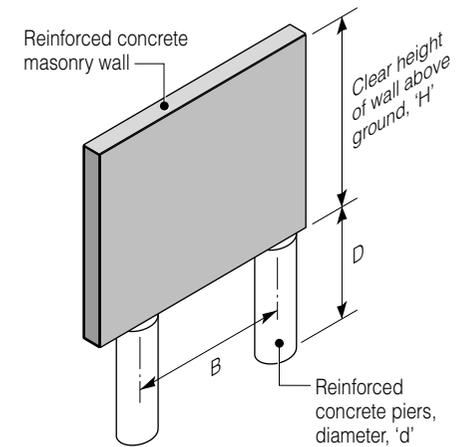


Figure 2 Reinforced Concrete Masonry Wall with Reinforced Concrete Piers

2 WALL CONSTRUCTION

The walls are built from 190 mm partially reinforced hollow concrete block work structurally tied to reinforced concrete strip footings at their base and with a reinforced bond beam at the top.

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.

Refer to Table 1

4 WALL RESISTANCE TO OVERTURNING

The resistance to overturning is provided by the combined weight of the wall 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. If the soil is firm with a high bearing capacity, the point of the 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. This conservative approach has been used in this Data Sheet and as such customary bearing failure analysis has not been performed, however, if it is considered bearing failure analysis is necessary eg. low friction angle or poor quality soil) Please refer to typical CMAA manual, MA 51 Reinforced Concrete Masonry Cantilever Retaining Walls for guidance.

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

Table 2 *Strip Footing Width And Wall Reinforcement Spacing*

Wind Classification	Fence Height 'H'	Strip footing width Type A or B 'W'	Wall reinforcement maximum spacing 'S' (m)	190mm wall reinforcement size
N1 _f	1.80	1.1	2.0	N16
	1.60	1.0	2.0	N16
	1.40	0.9	2.0	N16
	1.20	0.8	2.0	N16
	1.0	0.7	2.0	N16
N2 _f	1.80	1.3	2.0	N16
	1.60	1.2	2.0	N16
	1.40	1.0	2.0	N16
	1.20	0.9	2.0	N16
	1.0	0.8	2.0	N16
N3 _f	1.8	1.7	1.2	N16
	1.6	1.5	1.6	N16
	1.4	1.4	2.0	N16
	1.2	1.2	2.0	N16
	1.0	1.0	2.0	N16
N4 _f	1.8	2.1	1.2	N16
	1.6	1.9	1.6	N16
	1.4	1.7	2.0	N16
	1.2	1.5	2.0	N16
	1.0	1.3	2.0	N16
N5 _f	1.8	2.7	1.0	N16
	1.6	2.4	1.2	N16
	1.4	2.1	1.6	N16
	1.2	1.9	2.0	N16
	1.0	1.6	2.0	N16
N6 _f	1.8	3.1	0.8	N16
	1.6	2.8	1.0	N16
	1.4	2.5	1.2	N16
	1.2	2.5	1.6	N16
	1.0	1.9	2.0	N16

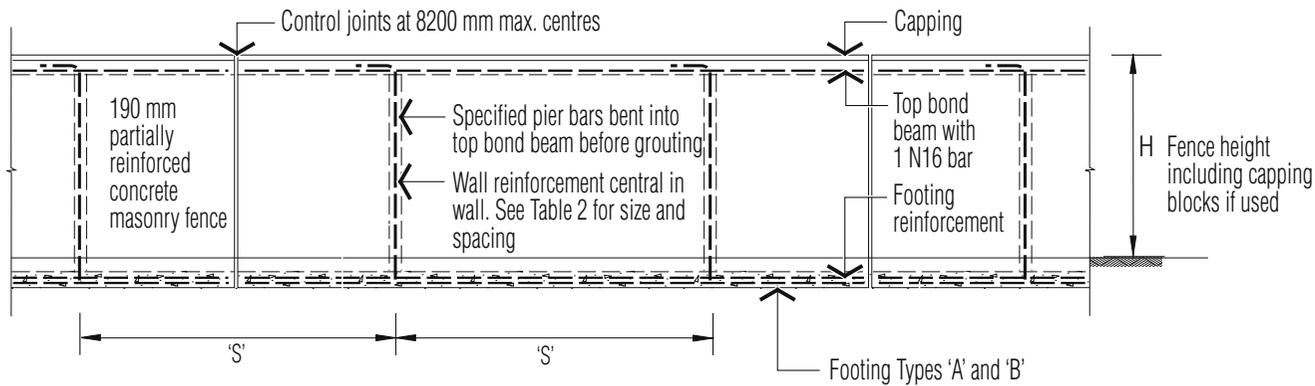


Figure 3 Typical Reinforcement Details and Control Joint Locations

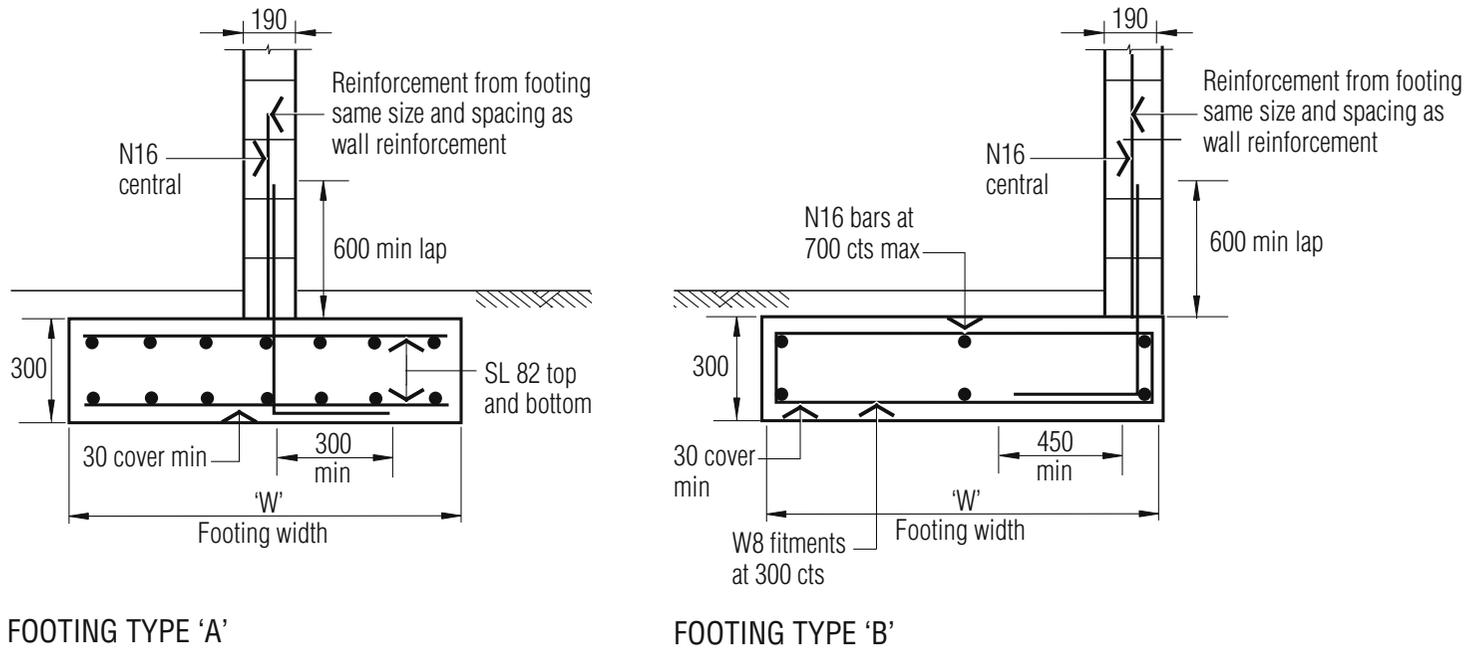
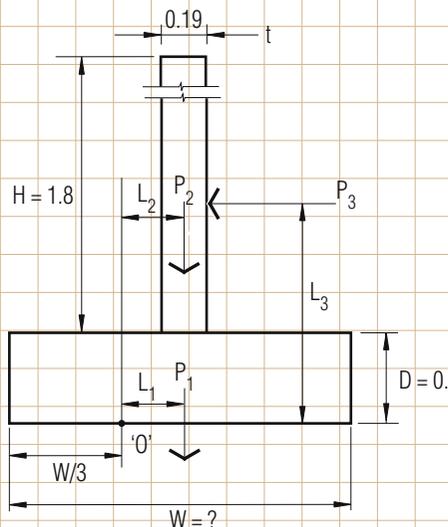


Figure 4 Typical 'A' And 'B' Footing Details

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 wall is to be partially reinforced with N16 reinforcing steel at S = 2.0 m vertical centres. The footing is to be strip footing Type A. i.e., with the wall stem located in the centre of the footing width B.		Terrain category multiplier $M_{z,cat} = 0.91$ For $h < 3.0$ m <i>AS/NZS 1170.2 Table 4.1(A)</i>	Shielding multiplier $M_s = 0.830$ <i>Interpolated from AS/NZS 1170.2 Table 4.3</i>	Notes: <i>This pressure is taken to represent a Wind Classification for Free-Standing Fences and Walls of NI_f.</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	The corresponding Wind Loads for Housing (on the same site) can be derived using AS 4055
		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	<i>Region A AS 4055 Fig 2.1 For Sydney</i>
		Average spacing of shielding buildings $l_s = h(10/n_s + 5)$ $= 1.8([10/2] + 5)$ $= 18.0$ m	Windward slope $H/2L_u = 4.0/(2 \times 60.0)$ $= 0.033 < 0.05$	<i>Terrain Category TC 3 AS 4055 Clause 2.3 For numerous closely spaced obstructions the size of houses</i>
WIND LOAD USING AS/NZS 1170.2:2002				
Region	A		Topography multiplier $M_t = 1.00$ <i>AS/NZS 1170.2 Clause 4.4.2</i>	<i>Average slope $\phi_s = 4 : 60 = 1 : 15$</i>
Degree of hazard	2			
Location	Non-cyclonic	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	<i>Topography TI For $\phi_s < 1 : 10$ AS 4055 Clause 2.4, Table 2.3</i>
Design event for safety	1 in 500	Shielding parameter $s = l_s / (h_s b_s)^{0.5}$ <i>AS/NZS 1170.2 Clause 4.3.3</i> $= 18.0 / (3.0 \times 7.0)^{0.5}$ $= 3.93$	Ultimate free stream gust dynamic wind pressure $q_{zu} = 0.0006 V_{zu}^2$ <i>AS/NZS 1170.2 Clause 2.4.1</i> $= 0.0006 \times 34.0^2$ $= 0.694$ kPa	<i>Shielding Partial Shielding (PS) AS 4055 Clause 2.5</i>
Regional wind speed $V_R = 45$ m/s	<i>AS/NZS 1170.2 Table 3.1</i>			<i>Classification NI AS 4055 Clause 2.2, Table 2.2</i>
Regional wind multiplier $M_d = 1.0$	<i>AS/NZS 1170.2 Clause 3.3.1</i>			

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 $p_{nu} = C_{fig} q_{zu}$ $= 1.20 \times 0.695$ $= 0.834 \text{ kPa}$	AS/NZS 1170.2 Clause 2.4.1
	Porosity reduction factor $K_p = 1 - (1 - \delta)^2$ $= 1 - (1 - 1)^2$ $= 1.0$		
		LOAD FACTORS AND CAPACITY REDUCTION FACTORS	
	Length of wall used for calculations $B' = 1.0 \text{ m}$	Load factor on overturning wind pressure $G_w = 1.0$	AS 1170.0 2002 Clause 4.2.1(b)(iv)
		Load factor on restoring forces $G_r = 0.8$	AS 4678 2002 Clause J3(c)

<p>SHEAR FORCE AND BENDING MOMENTS AT THE BASE OF WALL</p>	<p>METHOD TO FIND STRIP FOOTING WIDTH B</p>	<p>Density of concrete footing $\gamma_f = 23.5 \text{ kN/m}^3$</p>	<p>Wall restoring moment about point 'O' $M_w = P_2 \times L_2$ $= 4.92 \times W/6$ $= 0.82 W \text{ kNm/m}$ P_3 factored</p>
<p>Shear force at base support of exposed wall $V_b = G_w p_{nu} B' h$ $= 1.0 \times 0.834 \times 1.0 \times 1.80$ $= 1.50 \text{ kN/m}$</p>	<p>To find the required base width 'W' for any given wind pressure and known 300 mm base depth D. Set up either a quadratic equation or an iterative process to solve for 'W'.</p>	<p>P_1 factored weight of base $= G_r \gamma_f D W$ $= 0.9 \times 23.5 \times 0.3 \times W$ $P_1 = 6.35 W$</p>	<p>wind force per meter run of wall $= G_w p_{nu} H 1.0$ $= 1.0 \times 0.83 \times 1.8 \times 1.0$ $P_3 = 1.49 \text{ kN/m}$</p>
<p>Bending moment at base of support of exposed wall $M_b = 0.5 G_w p_{nu} B' h^2$ $= 0.5 \times 1.0 \times 0.834 \times 1.0 \times 1.80^2$ $= 1.35 \text{ kN/m}$</p>	<p>STRIP FOOTING DETAILS</p> 	<p>L_1 lever arm base $= \frac{W - W}{2 \quad 3}$ $L_1 = \frac{W}{6}$</p>	<p>L_3 lever arm of wind force about point 'O' $= H/2 + D$ $= 1.8/2 + 0.3$ $L_3 = 1.2 \text{ m}$</p>
<p>OVERTURNING ANALYSIS As the horizontal force increases, (i.e., normally from wind) the wall will rotate about its base. The resistance to this movement is provided by the weight and width of base and wall stem providing restoring moments about a point assumed to be one third along the base from either end (toe) depending on which side of wall the wind is blowing. Note one third base location is conservative and will provide adequate bearing capacity for most average strength soils.</p>	<p>N_{1f} wind pressure $p_{nu} = 0.83 \text{ kN/m}^2$</p> <p>Density of Partially reinforced wall $\rho_w = 16.0 \text{ kN/m}^3$</p>	<p>Base restoring moment about point 'O' $M_B = P_1 \times L_1$ $= 6.35 W \times W/6$ $= 1.06 W^2 \text{ kNm/m}$</p> <p>P_2 factored weight of wall $= G_r \gamma_w H t$ $= 0.9 \times 16.0 \times 1.8 \times 0.19$ $P_2 = 4.92 \text{ kN/m}$</p> <p>L_2 lever arm of wall weight about point 'o' $= \frac{W - W}{2 \quad 3}$ $L_2 = \frac{W}{6}$</p>	<p>Wind force overturning moment about point 'O' $M_w = P_3 \times L_3$ $= 1.49 \times 1.2$ $= 1.79 \text{ kNm/m}$</p> <p>Sum of moments about point 'O' = 0 $M_B + M_w - M_w = 0$ $1.06 W^2 + 0.82W - 1.79 = 0$</p>

<p>Quadratic equation</p> $= \frac{-B \pm \sqrt{B^2 - 4ac}}{2a}$ $= \frac{-0.82 \pm \sqrt{0.82^2 - 4 \times 1.06 \times (-1.79)}}{2 \times 1.06}$ $= \frac{-0.82 + 2.87}{2.12}$ <p>∴ Base width W required = 0.96 m But Say = 1.0 m</p> <p>Check: $1.06 \times 0.96^2 + 0.82 \times 0.96 - 1.79 = 0$ OK</p> <p>Notes :</p> <ol style="list-style-type: none"> A similar approach can be used to determined footing width W for strip footings with the wall stem located at the edge of the footing. (See Figure 1 Type B footing) The footing Type B width values shown in Table 2 are the same as footing Type A width values and hence are conservative 	<p>SPACING OF REINFORCED MASONRY 'POSTS'</p> <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 <p>Masonry Properties</p> <p>Masonry unit characteristic unconfined compressive strength $f_{uc} = 15.0 \text{ MPa}$</p> <p>Units are hollow</p> <p>Block type factor $k_m = 1.6$</p> <p>Equivalent brickwork strength $f_{mb} = k_m (f_{uc})^{0.5}$ $= 1.6 (15.0)^{0.5}$ $= 6.20 \text{ MPa}$</p> <p>Mortar joint height $h_j = 10 \text{ mm}$</p> <p>Masonry unit height $h_b = 190 \text{ mm}$</p>	<p>Ratio of block to joint thickness $h_b/h_j = 190/10$ $= 19.0$</p> <p>Block height factor $k_h = 1.3$</p> <p>Characteristic masonry strength $f'_m = k_h f_{mb}$ $= 1.3 \times 6.20$ $= 8.06 \text{ MPa}$</p> <p>Concrete Grout Properties</p> <p>Concrete grout specification: Concrete grout shall comply with AS 3700 and have:</p> <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. <p>Specified characteristic grout cylinder strength $f'_c = 20 \text{ MPa}$</p> <p>> 12 MPa OK AS 3700 Clause 5.6</p>	<p>Design characteristic grout strength $f'_{cg} = \min[(1.3 \times f_{uc}), 20.0]$ AS 3700 Clause 3.5 $= \min[(1.3 \times 15), 20.0]$ $= \min[19.5, 20.0]$ $= 19.5 \text{ MPa}$</p> <p>Main Reinforcement Properties</p> <p>Main reinforcement yield strength $f_{sy} = 500 \text{ MPa}$</p> <p>Main reinforcement shear strength (dowel action) $f_{sv} = 17.5 \text{ MPa}$</p> <p>Number of main tensile reinforcing bars $N_t = 1$</p> <p>Diameter of main tensile reinforcing bars $D_{dia,t} = 16 \text{ mm}$</p> <p>Area of main reinforcement $A_{st} = N_t (3.1416 D_{dia,t}^2/4)$ (approx) $= 1 \times 3.1416 \times 16^2/4$ $= 200 \text{ mm}^2$</p>
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Dimensions	Main Reinforcement	Fitments
The most adverse loading is on the pier near the middle of the wall	Effective depth of reinforcement For centrally located reinforcement: $d = D/2$	There are no shear reinforcement fitments required in this type of construction, which incorporates a single vertical reinforcing bar
Width of pier (along the wall) $B = 390 \text{ mm}$	For reinforcement near one face shell: $d = D - d_1 + D_{\text{dia.t}}/2$ $= 190/2$ $= 95 \text{ mm}$	Fitment yield strength $f_{\text{sy.f}} = \text{NA}$
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 $A_f = \text{NA}$
Density of reinforced concrete masonry $\rho_{\text{mas}} = 2,200 \text{ kg/m}^3$	Shear width of reinforced section $b_v = 200 \text{ mm}$ <i>Note: Only one core is grouted</i>	Fitment spacing $s = \text{NA}$
Modulus of elasticity $E = 1,000 f'_m$ $= 1,000 \times 8.06$ $= 8,060 \text{ MPa}$	Design area of main tensile reinforcement $A_{\text{sd}} = \min[0.29(1.3 f'_m)bd/f_{\text{sy}}, A_{\text{st}}]$ $= \min[(0.29 \times 1.3 \times 8.06 \times 760 \times 95 / 500), 200]$ $= \min[462, 200]$ $= 200 \text{ mm}^2$	
Second moment of area of reinforced concrete masonry pier $I = B D^3/12$ $= 390 \times 190^3/12$ $= 222.9 \times 10^6 \text{ mm}^4$		

Reinforced Masonry Capacity		
From Table 2 maximum spacing of the N16 reinforcing steel is 2.0m		Limiting deflection
	<i>Clause 8.6 (a) AS 3700</i>	$\Delta_a = H_c/50$
$\phi = 0.75$	<i>AS 3700 Clause 4.4</i>	$= 1,800/50$
$f_{vm} = 0.35$	<i>AS 3700 Clause 8.8</i>	$= 36 \text{ mm}$
Shear capacity		Load capacity (limited by shear)
$\phi V = \phi(f_{vm} b_w d + f_{vs} A_{st} + f_{sy.f} A_{sv} d/s)$		$W_{vu} = 1.0 \phi V / (B' H_c)$
$= 0.75[(0.35 \times 200 \times 95) + (17.5 \times 200) + 0]/1000$		$= 1.0 \times 7.61 / (1.0 \times 1.800)$
$= 0.75(6.65 + 3.50 + 0)$		$= 4.22 \text{ kPa}$
$= 7.61 \text{ kN}$		
Bending Moment Capacity		Load capacity (limited by bending moment)
$\phi M = \phi f_{sy} A_{sd} d [1 - 0.6 f_{sy} A_{sd} d / (1.3 f_m b d)]$		$W_{mu} = 2 \phi M / B' H_c^2$
$= 0.75 \times 500 \times 200 \times 95 [1 - (0.6 \times 500 \times 200) / (1.3 \times 8.06 \times 760 \times 95)] / 10^6$		$= 2 \times 6.56 / (1.0 \times 1.800^2)$
$= 6.56 \text{ kN.m}$		$= 4.04 \text{ kPa}$
Check actual theoretical spacing of reinforcing steel		Load capacity (limited by deflection)
$\phi M = p_{nu} S H H/2$		$W_{\Delta u} = \Delta_a E I / 48 H_c^4 B'$
$S = \frac{2\phi M}{p_{nu} H^2}$		$= [36 \times 8,060 \times 222.9 \times 10^6 / (48 \times 1.800^4 \times 1.0)] 10^{-9}$
$= \frac{2 \times 6.56}{0.83 \times 1.8^2}$		$= 128 \text{ kPa}$
$= 4.9 \text{ m}$		Load capacity (limited by shear, bending moment or deflection)
$\therefore > 2.0 \text{ allowed}$		$W_{lu} = \min(W_{vu}, W_{mu}, W_{\Delta u})$
$S = 2.0 \text{ m OK}$		$= \min(4.22, 4.04, 128)$
		$= 4.04 \text{ kPa}$
		$> 0.834 \text{ kPa OK}$
Height of cantilever wall above the strip footing base		
$H_c = 1.800 \text{ m}$		