

CE 225 STRUCTURAL DESIGN I

REINFORCED MASONRY SEISMIC SHEAR WALL DESIGN

This presentation covers the seismic design of special reinforced masonry shear walls without openings, and for Seismic Design Category C, D, E, and F. These also cover most of the requirements for the vertical segments of reinforced masonry shear walls with openings which is a prevalent form of construction in the Caribbean. To complete the design of reinforced masonry shear walls with openings, the additional requirements of the design include those for the reinforced masonry beams. Also, flanged walls are not considered in this presentation.

The following is based on the requirements of the IBC 2003 (i.e. ACI 530/ASCE 5/TMS 402 with the IBC modifications). Reference to code clauses will be via bold type in brackets beginning with an I for IBC clauses/equations, and A for ACI clauses/equations. Note that a shear wall is typically also a bearing wall so the same wall may have to be checked for its out-of-plane capacity.

This topic is of paramount importance in the design of masonry buildings in the Caribbean given its extensive usage and prevalent improper practice. It must be understood that when an R factor above 1.5 is used to get the required moment strengths, it is dangerous to simply add flexural rebar in order to get the moment strengths. This can result in extensive shear failure and possible collapse since the horizontal rebar is typically not increased to match. To respond elastically, much higher moment and shear strengths are required to ensure that the seismic energy demand on the building is the same as if there is proper inelastic action (which is assumed when a R factor above 1.5 is used).

a. Design Intention:

In general, the intention is to promote ductility via suppression of the brittle failure modes. Ductility is maximised by assuring that the failure mode is flexural. The brittle failure modes of shear failure and rebar buckling are suppressed by ensuring that the shear strength is sufficiently high, and the tensile strain in the extreme rebar is below a critical level, respectively. In the latter case, the maximum masonry compressive strain must be checked to ensure that it is below the maximum usable masonry compressive strain. If it is not, then either the masonry core must be confined (which is not yet constructable in the Caribbean), or the amount of vertical rebar must be reduced to ensure that the maximum masonry compressive strain is below the maximum usable masonry compressive strain.

b. Strength Reduction Factors and Load Combinations:-

Strength Reduction Factors:

Combinations of bending and axial load for reinforced masonry (**A3.1.4.1**):

$$\phi = 0.9$$

Shear for reinforced masonry (**A3.1.4.3**):

$$\phi = 0.8$$

Development and splices of reinforcement (**A3.1.4.5**):

$$\phi = 0.8$$

For the Caribbean it is strongly recommended that two-thirds of the above values be used, since it is typically the case that the level of QA/QC required by the IBC 2003 is not implemented.

Load Combinations:

$$1.2D + 1.0E + f_1 L$$

$$0.9D + 1.0E$$

where,

$f_1 = 1.0$ for floors in places of public assembly, or for live loads greater than 4.79 kN/m^2 , or for parking garage live load.

$f_1 = 0.5$ for all other live loads.

c. Stiffness Requirements:-

Under load combinations that include earthquake load, the drift must be within the allowable specified. To calculate the drift, the analysis must consider the cracked section flexural and shear stiffness properties which must not exceed half that of the gross section unless a cracked-section analysis is performed (**A3.1.5.3**).

d. Material Properties:-

The specified compressive strength of masonry shall equal or exceed 10.34 MPa (**A3.1.7.1.1**).

For concrete masonry the specified compressive strength of grout shall equal or exceed that for the masonry but shall be less than 34.47 MPa . For clay masonry, the grout specified compressive strength shall not exceed 41.37 MPa (**A3.1.7.2**).

Reinforcement tensile specified yield strength shall not exceed 413.7 MPa , and the actual yield strength shall not exceed 1.3 times the specified yield strength (**A3.1.7.3**).

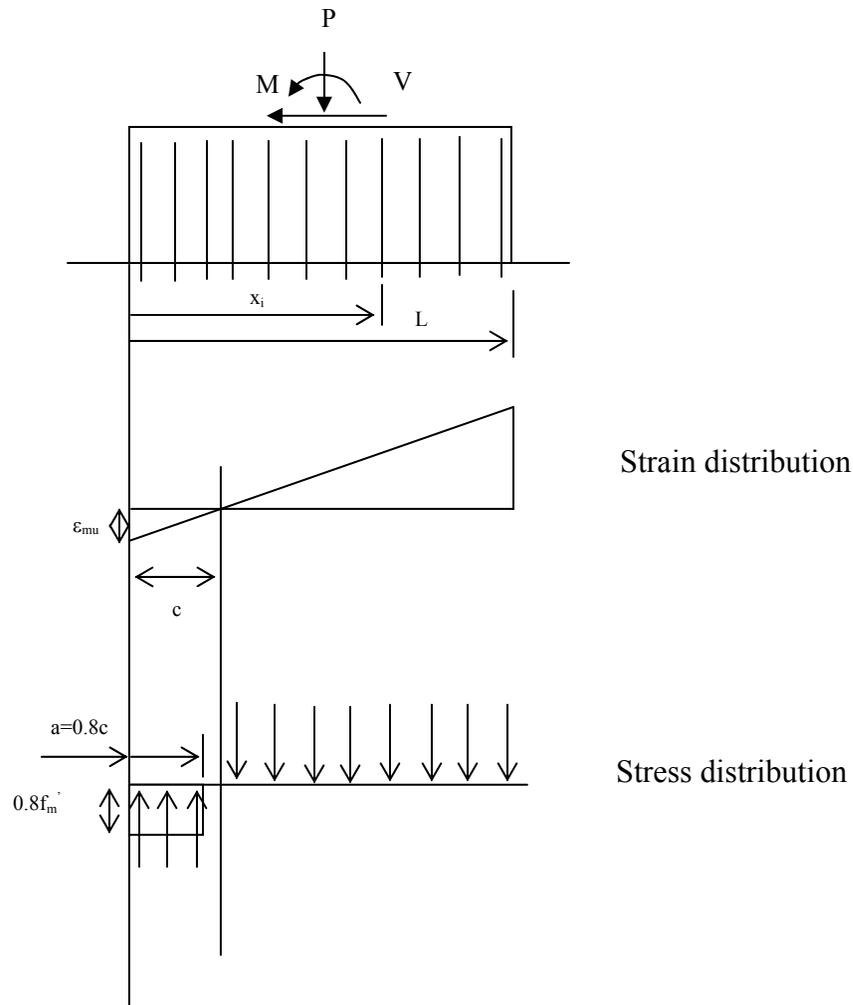
e. Nominal Moment Strength, M_n :-

PROCEDURE-1:

Main assumptions:

- Plane sections remain plane regardless of the load level or dimensions of the wall (i.e. the strain distribution from top to bottom of the section is linear).
- The maximum usable masonry compressive strain ϵ_{mu} , is 0.0035 for clay masonry and 0.0025 for concrete masonry.
- The stress-strain curve for the steel rebar in tension is assumed to be elastic perfectly-plastic.
- The stress distribution for the masonry in compression is assumed to be rectangular at a value of $0.8 f'_m$ acting over an area 0.8 times the neutral axis depth (i.e. $a = 0.8c$) from the extreme compression fibre.

Applied to the wall we get:



Step 1. Set an initial estimate for c

Step 2. Determine masonry compression force, $C_m = 0.8f'_m ta$ (note $a=0.8c$)
ACI Code clause 3.1.7.3 requires that compression resistance of steel reinforcement be ignored.

Step 3. Determine the steel tensile force (for those bars in tension):

$$T = \sum A_{s_{j+1}} f_y$$

Step 4. Determine c such that $C_m - T - P = 0$

Step 5. Take moments of all forces about the neutral axis. That is,

$$M_n = C_m (c - a/2) + \sum f_y A_{s_i} (c - x_i) + P (L/2 - c)$$

This procedure can be readily implemented using a spreadsheet (and the “goal-seek” procedure for step 4). Implementation by spreadsheet is also very convenient for checking the maximum vertical reinforcement as well (i.e. moment required to cause rebar buckling) as discussed later.

The nominal flexural strength at any section along a member shall not be less than one-fourth of the maximum nominal flexural strength at the critical section (A3.2.4.1.1).

Nominal Axial Load Strength, P_n :-

The nominal axial load strength of the wall in Newtons (i.e. the axial load capacity in the absence of in-plane moment) is given by (A3.2.4.1.1),

$$P_n = 0.80[0.80 f'_m (A_n - A_s) + f_y A_s](1 - (h/140r)^2) \quad \text{for } h/r < 99 \quad (\text{A3-16})$$

$$P_n = 0.80[0.80 f'_m (A_n - A_s) + f_y A_s](70r/h)^2 \quad \text{for } h/r > 99 \quad (\text{A3-17})$$

A_n = net cross-sectional area in mm^2
 A_s = area of steel in mm^2
 f_y = steel yield strength in MPa
 f_m = specified masonry compressive strength in MPa
 h = wall effective height in mm
 r = wall radius of gyration in mm

Given this value and PROCEDURE-1 above, interaction curves (i.e. P-M) can be easily prepared. Such curves are very practical design aids, especially when expressed in non-dimensional form. The following table presents interaction curve data for a rectangular section with mild steel rebar.

Moment Coefficients m_i for Rectangular Masonry Walls with Uniformly Distributed Reinforcement; $f_y = 275$ MPa

$\rho(f_y/f_m)$	Axial Load Ratio, $P/(f_m' L t)$								
	0	0.05	0.10	0.15	0.20	0.25	0.30	0.35	0.40
0.0100	0.0052	0.0279	0.0480	0.0652	0.0795	0.0910	0.0995	0.1052	0.1080
0.0200	0.0101	0.0322	0.0519	0.0687	0.0826	0.0938	0.1021	0.1076	0.1102
0.0400	0.0194	0.0406	0.0593	0.0754	0.0887	0.0993	0.1072	0.1123	0.1147
0.0600	0.0284	0.0487	0.0666	0.0819	0.0946	0.1047	0.1122	0.1170	0.1193
0.0800	0.0370	0.0565	0.0737	0.0883	0.1005	0.1101	0.1172	0.1218	0.1238
0.1000	0.0454	0.0641	0.0805	0.0946	0.1062	0.1154	0.1221	0.1265	0.1284
0.1200	0.0535	0.0714	0.0873	0.1007	0.1119	0.1207	0.1271	0.1312	0.1329
0.1400	0.0613	0.0786	0.0938	0.1068	0.1175	0.1259	0.1320	0.1359	0.1375
0.1600	0.0690	0.0856	0.1003	0.1127	0.1230	0.1311	0.1369	0.1406	0.1421
0.1800	0.0764	0.0925	0.1066	0.1186	0.1285	0.1362	0.1418	0.1453	0.1466
0.2000	0.0837	0.0992	0.1128	0.1244	0.1339	0.1413	0.1467	0.1500	0.1512

$$\rho = A_s/Lt; M_n = m_i f_m' L^2 t$$

TABLE 1. In-Plane Interaction Curves for the Moment Strength of Rectangular Walls

Rapid Estimation of Required Vertical Rebar:

PROCEDURE-2:

The following procedure can be used for a quick estimate of the vertical rebar required as input to PROCEDURE-1 above. It is an approximation therefore also very useful for preliminary design.

Step 1. Determine $a_1 = P/(0.85 f_m' t)$, (P must be the factored axial load)

Step 2. Determine $M_p = P(L/2 - a_1/2)$, (M_p =moment resisted by the axial load)

Step 3. Determine $M_s =$ Factored applied moment - M_p , (M_s =moment resisted by the reinforcement).

Step 4. Determine $a_2 = a_1 M_s/M_p$

Step 5. Determine $A_s = M_s / (f_y (L/2 - a_1 - a_2/2))$

e. Nominal Shear Strength, V_n :-

The IBC 2003 requires that for shear walls in Seismic Design Category D, E, and F, the shear strength of the wall must be checked in 2 regions – within the hinging region (which may be at both the base and top of the wall), and outside this region. Also, given the importance of suppressing the brittle shear failure mode, excess shear capacity in relation to flexural capacity must be provided. This is done by requiring that the shear strength is at least 1.25 times the shear force associated with the moment strength.

In Caribbean masonry construction practice, the industry is not as yet accustomed to the use of structural horizontal steel. The typical usage is of thin-wire joint mesh reinforcement for minor settlement and shrinkage cracking. There is also a common but very inappropriate practice of using reinforcement rods in the masonry joint. This is improper because there is not enough mortar around the bar for proper bond strength. Attempts to increase the mortar thickness will weaken the wall's compressive strength significantly. The proper thing to do is to use special blocks that can accept the bar. This block is of 2 types – the depressed web block, and the bond beam block. If these types of block are not yet available, the webs of the standard block must be saw cut.

The design shear strength ϕV_n shall exceed the shear corresponding to the development of 1.25 times the nominal flexural strength M_n of the member, except that V_n need not exceed 2.5 times the required shear strength V_u (A3.1.3)

For shear walls in SDC D, E, or F:-

In the hinging region:

A hinging region is of vertical dimension L_w . Design actions are to be checked in the middle, vertically, of this region (I2106.5.2).

$$V_n = A_n \rho_n f_y \tag{I21-1}$$

Where ρ_n is the amount of horizontal rebar ($= A_s/L_w$; L_w is the length of the wall = height of the hinging region, as the shear crack is at 45 deg).

Outside the hinging region:

Use the procedure for shear walls in SDC A, B, or C as presented below.

For shear walls in SDC A, B, or C:-

The nominal shear strength consists of 2 components: the strength due to the masonry and that due to the horizontal reinforcement.

Shear Strength due to Masonry, V_m ¹:

$$V_m = 0.83(4.0 - 1.75(M/Vd_v))A_n \sqrt{f'_m} + 0.25P \tag{A3-21}$$

Shear Strength due to Reinforcement, V_s :

$$V_s = 0.5 (A_v/s) f_y d_v \tag{A3-22}$$

Where M = unfactored moment at the wall section

V = unfactored shear at the wall section

P = unfactored shear at the wall section

A_n = net cross-sectional area

A_v = horizontal steel

s = vertical spacing of horizontal steel

d_v = actual dimension of wall in direction of applied shear

Arrangement	A_n for 150mm Clay Block (mm ² /m)
@200mm c/c (i.e. fully grouted)	132690
@400mm c/c	104955
@600mm c/c	99580
@800mm c/c	88185

TABLE 2. Effective In-Plane Shear Area for Common Reinforcement Arrangements for 150mm Clay Block.

Nominal Wall Shear Strength:

$$V_n = V_m + V_s$$

The maximum allowable nominal shear strength is given by:

$$\text{For } M/(Vd_v) < 0.25: V_n = 6 \sqrt{f'_m} A_n \quad (\text{A3-19})$$

$$\text{For } M/(Vd_v) > 1.00: V_n = 4 \sqrt{f'_m} A_n \quad (\text{A3-20})$$

Interpolate for intermediate values of $M/(Vd_v)$.

f. Detailing Rules

Development length for End Anchorage and Splices (Tension or Compression) (3.2.3.3) :-

$$l_d = l_{de}/\phi \quad (\text{A3-13})$$

$$l_{de} = 1.5d_b^2 f_y \gamma / (K \sqrt{f'_m}) \quad (\text{A3-14})$$

where d_b = bar diameter in mm
 l_d = required length in mm
 l_{de} = embedment length in mm
 f'_m, f_y = in MPa

γ = 1.0 for 10mm to 16mm rebars
 = 1.4 for 19mm to 22 mm rebars
 = 1.5 for 25 to 29mm rebars

K is the lesser of:
 the masonry cover, and
 the clear spacing between adjacent reinforcement, and
 $5d_b$.

$$l_d > 300\text{mm}$$

The minimum length of lap for bars shall be 305mm or the length determined by eqn (A3-13) above (A3.2.3.4).

When standard hooks are used, its equivalent embedment length in tension is $13d_b$. A standard hook has an extension of $4d_b$ if it has a 180-deg bend, and an extension of $12d_b$ if it has a 90-deg bend. A standard hook has an internal radius of $5d_b$ for mild steel rods from 10mm to 25mm diameter, but $6d_b$ for high tensile rods.

1. The minimum total wall reinforcement shall be 0.2%bt (vertical plus horizontal; gross area).
2. The minimum vertical or horizontal reinforcement shall be 0.07%bt.
3. The amount of vertical rebar shall be greater than one half the amount of horizontal rebar (A3.2.6.2).
4. All reinforcement shall be uniformly distributed.
5. Maximum Reinforcement (A3.2.6.2 and A3.2.3.5):

Increasing the tensile reinforcement decreases the wall's ductility capacity and promotes shear failure. Rebar buckling also reduces the ductility. The following ensures that ductility is not reduced below levels for adequate performance:

- a. If the seismic forces are calculated with an $R \leq 1.5$:

The tensile reinforcement at the end of the wall can elongate beyond its yield strain. When the wall's lateral displacement reverses direction, this rebar can buckle and greatly reduce the wall's ductility. To prevent this occurrence, the rebar strain must not exceed 2 times the strain at yield of

the rebar, and the maximum masonry compressive strain must not exceed the maximum usable masonry compressive strain. That is, the moment strength of the wall (ϕM_n), calculated with this maximum steel strain, must be greater than the applied factored bending moment at the base of the pier or wall (M_u). For the calculation of the moment strength, the stress in the tension reinforcement shall be assumed to be $1.25 f_y$ and the compressive stress-strain relationship shall be: a uniform stress of $0.8 f'_m$ acting over an area of 0.8 times the neutral axis depth. The unfactored gravity axial load shall be used in the calculation. Note that the amount of tensile reinforcement must not be increased to increase the moment strength, since inadequate ductility will result.

- b. If the seismic forces are calculated with an $R > 1.5$:

The rebar strain must not exceed 5 times the strain at yield of the rebar, and the maximum masonry compressive strain must not exceed the maximum usable masonry compressive strain.. That is, the moment strength of the wall (ϕM_n), calculated with this maximum steel strain, must be greater than the applied factored bending moment at the base of the pier or wall (M_u). For the calculation of the moment strength, the stress in the tension reinforcement shall be assumed to be $1.25 f_y$ and the compressive stress-strain relationship shall be: a uniform stress of $0.8 f'_m$ acting over an area of 0.8 times the neutral axis depth. The unfactored gravity axial load shall be used in the calculation.
6. The maximum spacing of the vertical reinforcement shall be the smallest of: 1/3 the height of the wall; 1/3 the length of the wall, or 1219mm.
7. For all cross-sections lying within the region bounded by the base of the wall and a plane located at a distance L above the base, the maximum spacing of the horizontal reinforcement must be less than the smaller of 600mm or 3 times the nominal wall thickness.
8. In the region defined by item 8 above, the masonry contribution to the shear strength shall be ignored.
9. Except at wall intersections, the end of a horizontal reinforcing bar needed to satisfy shear strength requirements, shall be bent around the end vertical reinforcing bar with a 180-deg hook.
10. At wall intersections, the end of a horizontal reinforcing bar needed to satisfy shear strength requirements, shall be bent around the end vertical reinforcing bar with a 90-deg hook and shall extend horizontally into the intersecting wall.
11. Vertical reinforcement of at least 129mm^2 shall be provided at corners, within 400mm of each side of openings, within 200mm of each side of movement joints, and within 200mm of the ends of walls.
12. Horizontal reinforcement shall be placed at the top and bottom of wall openings and shall extend not less than 600mm nor less than 40 bar diameters past the opening.
13. Horizontal reinforcement shall be provided within 400mm from the top of walls.

g. Design Example:

A 3.0m long x 4.8m high pier at the ground floor of a shear wall with openings of SDC D is comprised of 150mm clay hollow brick units. The wall has the following unfactored design actions at the base and top of the pier.

Dead:

Base:	Top:
$P_D = 62.1 \text{ kN}$	$P_D = 56.2 \text{ kN}$
$M_D = 93.3 \text{ kNm}$	$M_D = 82.1 \text{ kNm}$
$V_D = 9.8 \text{ kN}$	$V_D = 8.5 \text{ kN}$

Live:

Base:	Top:
$P_L = 38.6 \text{ kN}$	$P_L = 33.7 \text{ kN}$
$M_L = 51.4 \text{ kNm}$	$M_L = 43.7 \text{ kNm}$
$V_L = 5.1 \text{ kN}$	$V_L = 4.3 \text{ kN}$

Seismic:

Base:	Top:
$P_E = 18.6 \text{ kN}$	$P_E = 14.7 \text{ kN}$
$M_E = 175.5 \text{ kNm}$	$M_E = 106.8 \text{ kNm}$
$V_E = 54.8 \text{ kN}$	$V_E = 47.8 \text{ kN}$

Wall properties:

Masonry - $f'_m = 12 \text{ MPa}$ Rebar - $f_y = 410 \text{ MPa}$

(1) Estimate Vertical Rebar Required:-

Using PROCEDURE-2 above, consider dead plus live plus earthquake at the wall's base:
Assuming live load $< 4.79 \text{ kN/m}^2$

$$P_u = (1.2 \times 62.1) + (1.0 \times 18.6) + (0.5 \times 38.6) = 112.4 \text{ kN}$$

$$M_u = (1.2 \times 93.3) + (1.0 \times 175.5) + (0.5 \times 51.4) = 313.2 \text{ kNm}$$

$$V_u = (1.2 \times 9.8) + (1.0 \times 54.8) + (0.5 \times 5.1) = 69.1 \text{ kN}$$

$\phi = 0.9$ for combined axial load and flexure,
For Caribbean region use $0.67 \times 0.9 = 0.6$

The following is excerpted from the spreadsheet program.

1. Input data for D+L+E:-

Input wall length (m) =	3
Input wall height (m) =	4.8
Input factored earthquake moment (kNm) =	313.2
Input factored axial load (kN) =	112.4
Input wall thickness (mm) =	140
Input f_m (MPa) =	12
Input vertical rebar f_y (MPa) =	410
Input axial/flexure phi factor =	0.6

2. Determine initial estimate of vertical steel:-

(from PROCEDURE-2)

a_1 (m) =	0.078711485
M_p (kNm) =	164.1764146
M_s (kNm) =	357.8235854
a_2 (m) =	0.171552203
A_s (mm ²) =	653.4873381

Input reinforcement to try:

Input number of vertical rebars =	8		
Input vertical rebar diameter (mm) =	12	OK	(must be < 19 (=150/8))
Input vertical rebar spacing (mm) =	400	OK	(must be < h/3;L/3;1219)
Hence A_s provided (mm ²) =	904.7786976		
Check vertical steel ratio =	0.002154235	OK	(must be > 0.0007;<0.04xcell area)

As indicated, try T12@400mm.

(2) For the selected rebar, calculate the actual moment strength:

Two load combinations typically control: A. D+L+E and, and B. D+E:

Case A:-

The design actions were previously determined.

Using the spreadsheet we get,

3. Nominal Moment Strength :-

A. Under Maximum Gravity Loads:

(from PROCEDURE-1)

Input the x_i in meter units from the compression edge. The following caters for up to 15 rebars.

Input max. usable masonry comp. strain = 0.0035

		Tensile Strain	Stress (MPa)	Force (kN)	Rebars Moment
x1 (m) =	0.1	0	0	0.0	0.0
x2 (m) =	0.5	0.001468375	293.6749021	33.2	4.9
x3 (m) =	0.9	0.005443074	410	46.3	25.4
x4 (m) =	1.3	0.009417774	410	46.3	43.9
x5 (m) =	1.7	0.013392473	410	46.3	62.5
x6 (m) =	2.1	0.017367173	410	46.3	81.0
x7 (m) =	2.5	0.021341873	410	46.3	99.5
x8 (m) =	2.9	0.025316572	410	46.3	118.1
x9 (m) =	0	0	0	0.0	0.0
x10 (m) =	0	0	0	0.0	0.0
x11 (m) =	0	0	0	0.0	0.0
x12 (m) =	0	0	0	0.0	0.0
x13 (m) =	0	0	0	0.0	0.0
x14 (m) =	0	0	0	0.0	0.0
x15 (m) =	0	0	0	0.0	0.0
					435.3

Tension
(mm)= 2547.8

Phi x P (kN) = 67.44

Input an estimate for "c" (m) = 0.352227876

a (m) = 0.281782301

Cm (kN) = 378.715412

T (kN) = 311.2754109

Equilibrium check: Cm - T - P = 0 1.10061E-06 Goal-Seek here

Nominal moment strength, Mn (kNm) = 592.7

Design strength (phi x nominal, kNm) = 355.6 STRENGTH OK

$$\phi M_n = 355.6 > M_u = 313.2 \text{ kNm} : \text{OK}$$

Case B :

$$P_u = (0.9 \times 62.1) + (1.0 \times 18.6) = 74.5 \text{ kN}$$

$$M_u = (0.9 \times 93.3) + (1.0 \times 175.5) = 259.5 \text{ kNm}$$

$$V_u = (0.9 \times 9.8) + (1.0 \times 54.8) = 63.6 \text{ kN}$$

B. Under Minimum Gravity Loads:

Max. usable masonry comp. strain = 0.0035

		Tensile Strain	Stress (MPa)	Force (kN)	Rebars Moment
x1 (m) =	0.1	0	0	0.0	0.0
x2 (m) =	0.5	0.001706885	341.3770657	38.6	6.3
x3 (m) =	0.9	0.005872394	410	46.3	26.1
x4 (m) =	1.3	0.010037902	410	46.3	44.7
x5 (m) =	1.7	0.01420341	410	46.3	63.2
x6 (m) =	2.1	0.018368918	410	46.3	81.8
x7 (m) =	2.5	0.022534427	410	46.3	100.3
x8 (m) =	2.9	0.026699935	410	46.3	118.8
x9 (m) =	0	0	0	0.0	0.0
x10 (m) =	0	0	0	0.0	0.0
x11 (m) =	0	0	0	0.0	0.0
x12 (m) =	0	0	0	0.0	0.0
x13 (m) =	0	0	0	0.0	0.0
x14 (m) =	0	0	0	0.0	0.0
x15 (m) =	0	0	0	0.0	0.0
					441.2

Tension (mm)= 2563.9

Input Phi x P (kN) = 44.7

Input an estimate for "c" (m) = 0.33609344

a (m) = 0.268874752

Cm (kN) = 361.3676663

T (kN) = 316.6676635

Equilibrium check: Cm - T - P = 0 2.83671E-06 Goal-Seek here

Input Mu (kNm) = 259.5

Nominal moment strength, Mn (kNm) = 566.1

Design strength (phi x nominal, kNm) = 339.7 **STRENGTH OK**

$$\phi M_n = 339.7 > M_u = 259.5 \text{ kNm} : \text{OK}$$

(3) Check Maximum Reinforcement:

4. Check Maximum Reinforcement:-

With all tensile rebar at 1.25fy and extreme tensile rebar at critical strain.

Use the "gravity beneficial" load combination factors.

Input R factor = 4.5

Critical tensile strain = 0.01025

		Tensile Strain	Stress (MPa)	Force (kN)	Rebars Moment
x1 (m) =	0.1	0	0	0.0	0.0
x2 (m) =	0.5	0.000223681	512.5	57.9	3.1
x3 (m) =	0.9	0.001894734	512.5	57.9	26.3
x4 (m) =	1.3	0.003565788	512.5	57.9	49.4

x5 (m) =	1.7	0.005236841	512.5	57.9	72.6
x6 (m) =	2.1	0.006907894	512.5	57.9	95.8
x7 (m) =	2.5	0.008578947	512.5	57.9	119.0
x8 (m) =	2.9	0.01025	512.5	57.9	142.1
x9 (m) =	0	0	0	0.0	0.0
x10 (m) =	0	0	0	0.0	0.0
x11 (m) =	0	0	0	0.0	0.0
x12 (m) =	0	0	0	0.0	0.0
x13 (m) =	0	0	0	0.0	0.0
x14 (m) =	0	0	0	0.0	0.0
x15 (m) =	0	0	0	0.0	0.0
					508.4

Tension
(mm)= 2453.5
Strain grade
= 0.004178

Input unfactored P (kN) (0.9D + 1.0E) = 74.5
Input an estimate for "c" (m) = 0.446457403
a (m) = 0.357165923
Cm (kN) = 480.031
T (kN) = 405.531

Equilibrium check: Cm - T - P = 0 0 Goal-
Seek
here

Max masonry compressive strain = 0.0019 OK

As indicated, the maximum masonry compressive strain is 0.0019 < 0.0035 (maximum usable masonry compressive strain for clay masonry). If not, then the vertical rebar would have to be reduced, or the grout in the end cells confined.

Therefore for the vertical rebar, **use T12-400.**

(4) Shear Reinforcement:

Given the wall's dimensions, most the entire height is within the 45 deg shear failure zone. Given SDC D, design for the hinging region only and apply the result to the remaining 1.8m.

V:-

Consider one hinge at the base of the pier.

For equilibrium, the maximum shear corresponding to the nominal moment,

$$V = (M_n + M_{top})/h$$

ACI clause 3.1.3 requires that a factor of 1.25 be used with M_n .

$$M_{top} = (1.2 \times 82.1) + (1.0 \times 106.8) + (0.5 \times 43.7) = 227.2 \text{ kNm}$$

$$\text{Therefore } V = (1.25 \times 592.7 + 227.2) / 4.8 = 201.7 \text{ kN}$$

$$V_u = (1.2 \times 9.8) + (1.0 \times 54.8) + (0.5 \times 5.1) = 69.1 \text{ kN}$$

$2.5V_u = 2.5 \times 69.1 = 172.8 < 201.7 \text{ kN}$. Therefore use the 172.8 kN as the required shear strength.

$$\phi V_n = \phi A_n \rho_n f_y = V \quad ; \quad \phi = 0.8 \times 0.67 = 0.54$$

$$\rho_n = 172.8 / (0.54 \times 0.105 \times 3.0 \times 410000) = 0.00248$$

$$\text{Minimum shear rebar} = 0.0007 \times 1000 \times 150 = 105 \text{ mm}^2/\text{m}$$

$$A_v = \rho_n L_w t = 0.00248 \times 3000 \times 150 = 1116 \text{ mm}^2$$

Over the 3.0m height this is $1116 / 3.0 = 372 \text{ mm}^2/\text{m} (>105)$

Use T16-400 (502 mm²/m).

Check Other Reinforcement Content Rules:

Shear rebar spacing must be less than $600 > 400$:OK
 $3 \times \text{thickness} = 3 \times 150 = 450 > 400$: OK

Total rebar must be $> 0.002 \times 1000 \times 150 = 300\text{mm}^2/\text{m}$
 Total rebar used = $(904.8/3) + 502 = 803.6 > 300\text{mm}^2/\text{m}$: OK
 Vertical rebar must be $>$ half horizontal rebar:
 $904.8/3 = 301.6 > 502/2 = 251\text{mm}^2/\text{m}$: OK

(5) Anchorage and Splices:

$$l_{de} = 1.5d_b^2 f_y \gamma / (K \sqrt{f_m})$$

For straight vertical rebar in tension/compression (i.e. the 12mm bars):

K = smallest of: cover = $150/2 - 12/6 = 69\text{mm}$
 Clear spacing = 400mm
 $5d_b = 5 \times 12 = 60\text{mm}$

$= 1.5 \times 12^2 \times 410 \times 1 / (60 \times \sqrt{410}) = 73\text{mm}$
 $l_d = 73 / (0.8 \times 0.67) = 136\text{mm} < 305\text{mm}$.
 Hence use 300mm anchorage and splices.

For straight horizontal rebar (i.e. the 16mm bars):

K = smallest of: cover = $150/2 - 16/6 = 67\text{mm}$
 Clear spacing = 400mm
 $5d_b = 5 \times 16 = 80\text{mm}$

$= 1.5 \times 16^2 \times 410 \times 1 / (67 \times \sqrt{410}) = 116\text{mm}$
 $l_d = 116 / (0.8 \times 0.67) = 216\text{mm} < 305\text{mm}$.
 Hence use 300mm anchorage and splices.