REINFORCED MASONRY SEISMIC SHEAR WALL DESIGN

This presentation covers the seismic design of special reinforced masonry shear walls without openings. 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 coupling beams. Also, flanged walls are not considered in this presentation.

The following is adapted from the International Building Code (IBC) 2006 and the Masonry Standards Joint Committee (MSJC) 2008 (a.k.a TMS 402-08/ACI 530-08/ASCE 5-08).

a. Design Intention:

In general, the intention is to promote ductility via suppression of the brittle failure modes. Ductility is maximized 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 amount of tensile rebar is below a critical amount. If it is not, then either the masonry core must be confined (which is not yet constructible in the Caribbean), or the amount of vertical rebar must be reduced.

b. Strength Reduction Factors and Load Combinations:-

Strength Reduction Factors:
Combinations of bending and axial load for reinforced masonry:
$\phi = 0.9$

Shear for reinforced masonry:
$\phi = 0.8$

Development and splices of reinforcement:
$\phi = 0.8$

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 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.

d. Material Properties:-

The specified compressive strength of masonry composite shall equal or exceed 10.34 MPa .

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.

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.

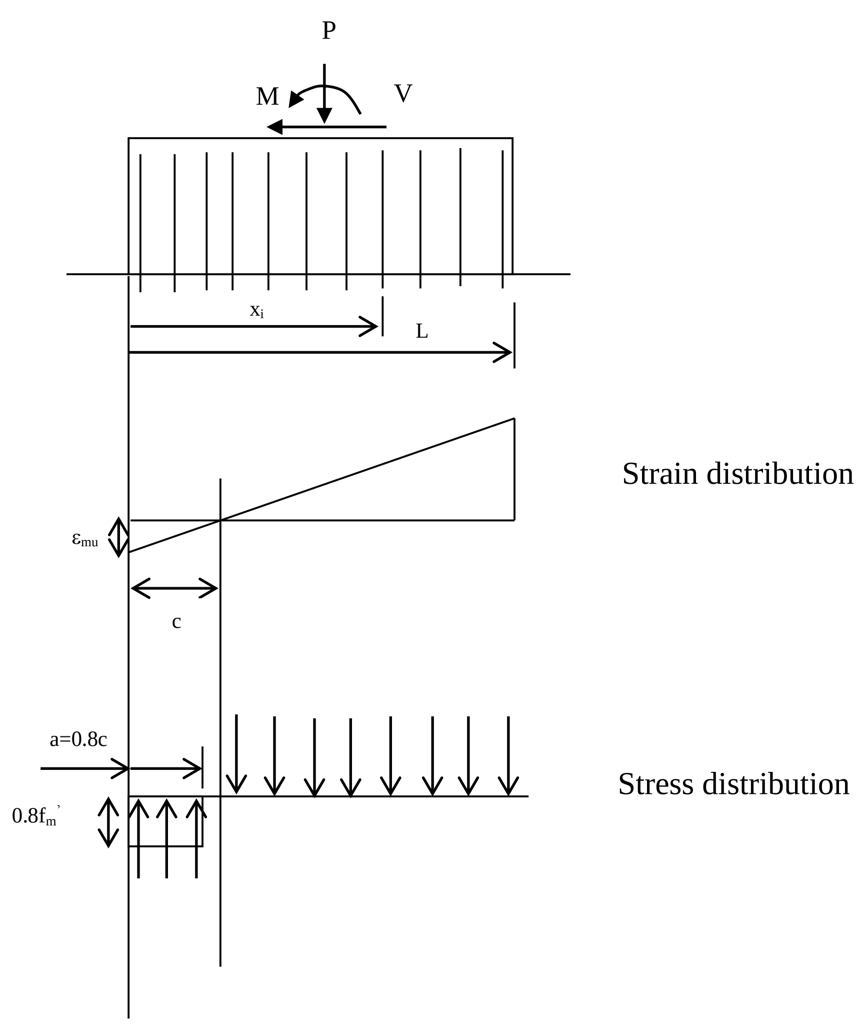
e. Nominal Moment Strength, M_n and The Strength Interaction Curve:-

PROCEDURE-1:

Main assumptions:

a. 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).
b. The maximum usable masonry compressive strain ε_{mu}, is 0.0035 for clay masonry and 0.0025 for concrete masonry.
c. The stress-strain curve for the steel rebar in tension is assumed to be elastic perfectly-plastic.
d. 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 fiber.

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) ignoring the compression resistance of steel reinforcement.

Step 3. Determine the steel tensile force (for those bars in tension):
$T = \sum A_{si} f_s$

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

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

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

This procedure can be readily implemented using a spreadsheet (and the "goal-seek" procedure for step 4).

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.

<u>Nominal Axial Load Strength, P_n:-</u>

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,

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

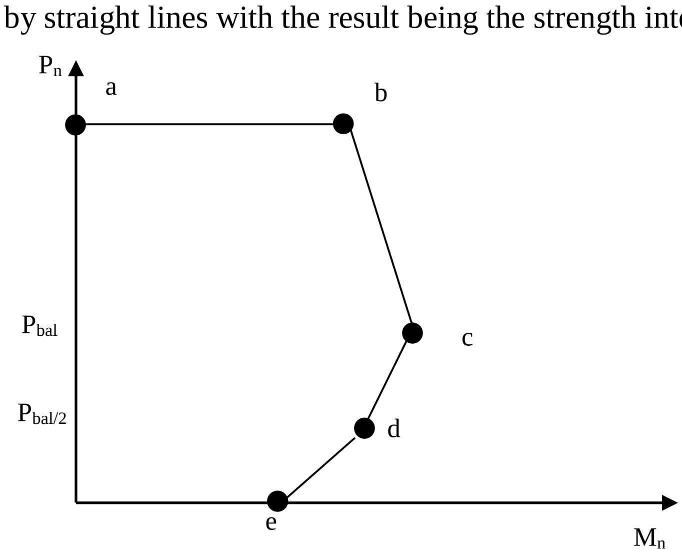
$$P_n = 0.80[0.80\, f^{'}_m (A_n - A_s) + f_yA_s](70r/h)^2) \quad \text{for } h/r > 99 \qquad (2)$$

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, strength interaction curves (i.e. P-M) can be easily prepared. A typical way of doing this for a given arrangement of the vertical rebar is to use the spreadsheet to determine the P_n , M_n points for the following 5 points:

a. P_n for $M_n = 0$ (i.e. pure compression; equ (1) or (2))
b. M_n for $P_n = P_0$
c. M_n for $P_n = P_{bal}$
d. M_n for $P_n = P_{bal/2}$
e. M_n for $P_n = 0$ (i.e. pure bending)

When the coordinates of these points are determined, they are plotted graphically and the points connected by straight lines with the result being the strength interaction curve.



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 $M_s = M_u/\phi - M_p$, (M_s=moment resisted by the reinforcement)

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

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

To determine if the selected vertical rebar is suitable, the P-M curve is modified to give the ϕP_n, ϕM_n curve then the P_u, M_u points for each load case is superimposed on the plot. The vertical rebar is OK if all the points fit within the region to the left of the curve.

f. **Boundary Elements:-**

Boundary elements are not required if:
For $M_u/(V_u L_w) \leq 1$:
$0.1 A_g f_m' > P_u$

For $M_u/(V_u L_w) \leq 3$:
$0.25 A_n \sqrt{f_m'} > V_u$ (A_n in mm^2; f_m' in MPa), and
$0.1 A_g f_m' > P_u$

If these conditions are not met, boundary elements are only required if:
For single-curvature bending:
$c \geq l_w / [600(C_d \delta_{ne}/h_w]$ where c is the neutral axis depth; l_w is the wall's length; δ_{ne} is the wall's displacement under code-prescribed loads for U = 1.2D + L ± E, and h_w is the wall's height.
For double-curvature bending:
The maximum extreme fiber compressive stress is greater than 0.2 f_m' calculated using elastic modeling and gross section properties.

However, if these checks indicate that boundary elements are needed, it is strongly recommended that for the Caribbean, the structural system be redesigned so that they are not needed. This is because the code requires that the wall must be lab tested. This is not practical at this time in the Caribbean so is considered a constructability constraint.

g. **Nominal Shear Strength, V_n:-**

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.

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.

$V_n = A_n \rho_n f_y$ (3)

Where ρ_n is the amount of horizontal rebar (= $A_s/L_w t$; 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 (metric):

$V_m = 0.083(4.0 – 1.75(M/Vd_v))A_n \sqrt{(f_m')} + 0.25P$ (4)

Shear Strength due to Reinforcement, V_s:

$V_s = 0.5 (A_v/s) f_y d_v$ (5)

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:

For $M/(Vd_v) < 0.25$: $V_n = 6 \sqrt{(f_m')} A_n$ (6)

For $M/(Vd_v) > 1.00$: $V_n = 4 \sqrt{(f_m')} A_n$ (7)

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

h. Detailing Rules

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

$l_d = l_{de}/ \phi$ (8)

$l_{de} = 1.5 d_b^2 f_y \gamma / (K \sqrt{(f_m')})$ (9)

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
$5d_b$.

$l_d > 300$mm

The minimum length of lap for bars shall be 305mm or the length determined by eqn (8) above.

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.

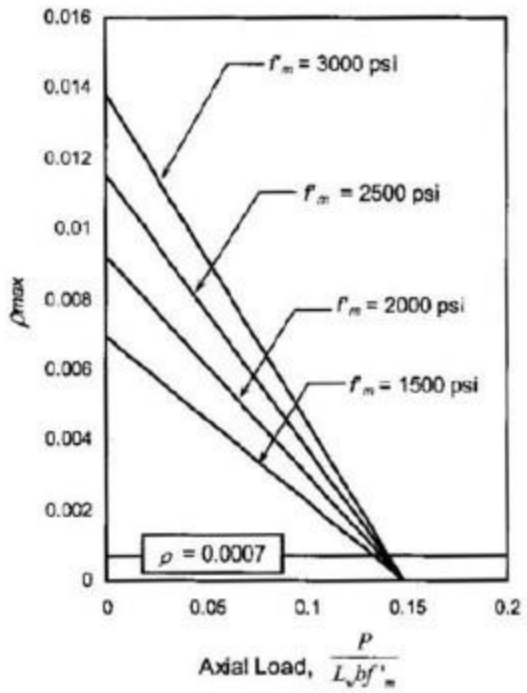
1. The minimum total wall reinforcement shall be 0.2% of the gross cross-sectional areas.
2. The minimum vertical or horizontal reinforcement shall be 0.07% of the respective gross cross-sectional area
3. The amount of vertical rebar shall be greater than one half the amount of horizontal rebar.
4. All reinforcement shall be uniformly distributed.
5. Maximum Reinforcement:
   This check is not required if:
   For $M_u/(V_u L_w) \leq 1$:
   $0.1A_g f_m' > P_u$

   For $M_u/(V_u L_w) \leq 3$:
   $0.25A_n\sqrt{f_m'} > V_u$, and
   $0.1A_g f_m' > P_u$

   This check is also not required if boundary elements are provided. However if the above checks fail yet boundary elements are not required, then the maximum reinforcement check must be performed. The maximum reinforcement is the vertical reinforcement in the wall for the following condition:

   The strain diagram comprises a maximum rebar strain 4 times the strain at yield, and the maximum usable masonry compressive strain and the compressive stress is a uniform stress of $0.8f'_m$ acting over an area of 0.8 times the neutral axis depth.

   A spreadsheet program similar to the one used to calculate the P-M strength interaction curve can be used to determine the maximum reinforcement ratio. This is typically done in such a manner that the check to be performed is if the sum of the compressive forces is greater than the sum of the rebar tensile and axial forces. If so, the rebar is adequate. Fortunately, a design aid can also be prepared using the spreadsheet for a series of calculations involving varying the wall's properties. A useful chart (credit: S.K. Ghosh Associates) is shown below; "b" is the wall's thickness.



Maximum Reinforcement Ratio in Special Reinforced Masonry Shear Walls (f_y = 60 ksi)

6. Bars larger than 29 mm diameter must not be used; the bar diameter must not exceed the wall nominal thickness/8; the bar diameter must not exceed the smaller cell dimension/4; the total area of rebars placed in a cell must not exceed 4% of the cell's area.
7. 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.
8. 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.
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.

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 kN	P_D = 56.2 kN
	M_D = 93.3 kNm	M_D = 82.1 kNm
	V_D = 9.8 kN	V_D = 8.5 kN

Live:

	Base:	Top:
	P_L = 38.6 kN	P_L = 33.7 kN
	M_L = 51.4 kNm	M_L = 43.7 kNm
	V_L = 5.1 kN	V_L = 4.3 kN

Seismic:

	Base:	Top:
	P_E = 18.6 kN	P_E = 14.7 kN
	M_E = 175.5 kNm	M_E = 106.8 kNm
	V_E = 54.8 kN	V_E = 47.8 kN

Wall properties:
Masonry - f_m' = 12 MPa   Rebar - f_y = 410 MPa

(1) Estimate Vertical Rebar Required:-

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

P_u = (1.2 x 62.1) + (1.0 x 18.6) + (0.5 x 38.6) = 112.4 kN
M_u = (1.2 x 93.3) + (1.0 x 175.5) + (0.5 x 51.4) = 313.2 kNm
V_u = (1.2 x 9.8) + (1.0 x 54.8) + (0.5 x 5.1) = 69.1 kN

ϕ= 0.9 for combined axial load and flexure,
For Caribbean region use 0.67 x 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 fm (MPa) =	12
Input vertical rebar fy (MPa) =	410
Input axial/flexure phi factor =	0.6

2. Determine initial estimate of vertical steel:-
(from PROCEDURE-2)

a1 (m) =	0.078711485
Mp (kNm) =	164.1764146
Ms (kNm) =	357.8235854
a2 (m) =	0.171552203
As (mm2) =	653.4873381

Input reinforcement to try:			
Input umber 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 As provided (mm2) =	904.7786976		
Check vertical steel ratio =	0.002154235	OK	(must be > 0.0007;<0.04xcell area)

As indicated, try T12@400mm with unreinforced cells ungrouted. Therefore the radius of gyration, r, is 150 mm/m length.

Check for if boundary elements are needed:
$0.1A_g f_m' = 0.1x3x0.14x12000 = 504$ kN > $P_u = 112.4$ kN
$0.25A_n\sqrt{f_m'} = 0.25 \times 104955 \times 3 \times \sqrt{(12)} = 272681$ N = 272.7 kN > $V_u = 69.1$ kN
$M_u/(V_u L_w) = 313.2/(69.1 \times 3) = 1.51 < 3.0$
Therefore boundary elements are not needed. [Recall that if they were required, Caribbean constructability indicates that the structural system must be re-designed].

(2) <u>For the selected rebar, calculate the ϕP_n, ϕM_n strength interaction curve:</u>

For point "a":
$h/r = 4800/(150 \times 3) = 10.7$, therefore from eq (1)
$P_n = 0.80[0.80 f'_m (A_n - A_s) + f_y A_s](1 - (h/140r)^2)$
$= 0.8 [0.8 \times 12000 \times (0.105x3 - 904.8 \times 10^{-6}) + 410000 \times 904.8 \times 10^{-6}] \times (1 - 4800/(140x450)^2) = 2703.6$ kN
$\phi P_n = 0.6 \times 2703.6 = 1622.2$ kN
Hence point "a" = 0, 1622.2

For point "b":
By using the spreadsheet and setting P_n as the value for point "a", we get $M_n = 1343.6$ kNm.
ϕP_n, $\phi M_n = 0.6 \times 2703.6, 0.6 \times 1343.6 = 1622.2$ kN, 806.2 kNm
Hence point "b" = 806.2, 1622.2

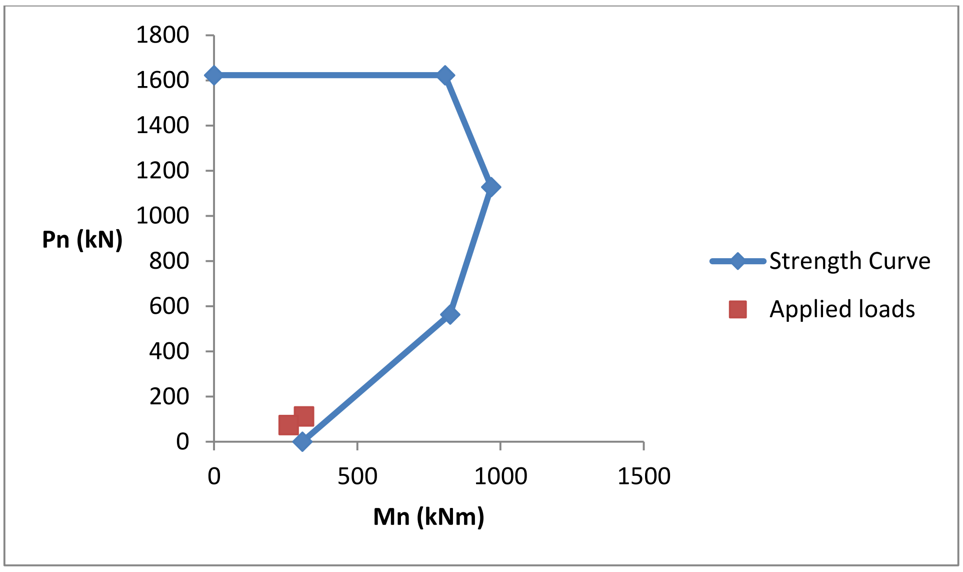
For point "c":
By setting the extreme tensile fiber stress as f_y, the strain distribution equation indicates c = 1.828m and by using the spreadsheet, equilibrium indicates that P_{bal} (= C - $\sum$T) = 1879.1 kN. We then get $M_n = 1612.1$ kNm.
ϕP_n, $\phi M_n = 0.6 \times 1879.1, 0.6 \times 1612.1 = 1127.5$ kN, 967.3 kNm
Hence point "c" = 967.3, 1127.5

For point "d":
By using the spreadsheet and setting as $P_{bal}/2 = 1879.1/2 = 939.55$ kN we get M_n = 1101.7 kNm.
ϕP_n, $\phi M_n = 0.6 \times 939.55, 0.6 \times 1373.1 = 563.7$ kN, 823.9 kNm
Hence point "d" = 823.9, 563.7

For point "e":
By using the spreadsheet and setting as $P_n = 0$, we get $M_n = 512.6$ kNm.
ϕP_n, $\phi M_n = 0, 0.6 \times 512.6 = 0$, 307.6 kNm
Hence point "e" = 307.6, 0.0

For the load case 0.9D + E ,
$P_u = (0.9 \times 62.1) + (1.0 \times 18.6) = 74.5$ kN
$M_u = (0.9 \times 93.3) + (1.0 \times 175.5) = 259.5$ kNm
$V_u = (0.9 \times 9.8) + (1.0 \times 54.8) = 63.6$ kN

Hence the P_u, M_u data points for the two load cases are: 112.4 kN, 313.2 kNm, and 74.5 kN, 259.5 kNm. The strength interaction plot is therefore as shown below, including the P_u, M_u data points for the two load cases. As the applied loads are to the left of the strength curve, the vertical rebar is OK.



(3) Check Maximum Reinforcement:

$0.1A_g f_m' = 0.1x3x0.14x12000 = 504$ kN > $P_u = 112.4$ kN
$0.25A_n\sqrt{f_m'} = 0.25 \times 104955 \times 3 \times \sqrt{(12)} = 272681$ N = 272.7 kN > $V_u = 69.1$ kN
$M_u/(V_u L_w) = 313.2/(69.1 \times 3) = 1.51 < 3.0$
Therefore the maximum reinforcement check is not needed.

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

A factor of 1.25 must be used with M_n.

$M_{top} = (1.2x82.1) + (1.0x106.8) + (0.5x43.7) = 227.2$ kNm

Therefore V = (1.25x 592.7+227.2)/4.8 = 201.7 kN

$V_u = (1.2x9.8) + (1.0x54.8) + (0.5x5.1) = 69.1$ kN
$2.5V_u = 2.5x69.1 = 172.8 < 201.7$ kN. Therefore use the 172.8 kN as the required shear strength.

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

$\rho_n = 172.8/(0.54x0.105x3.0x410000) = 0.00248$
Minimum shear rebar = 0.0007 x 1000 x 150 = 105 mm^2/m
$A_v = \rho_n L_w t = 0.00248 \times 3000 \times 150 = 1116 mm^2$
Over the 3.0m height this is 1116/3.0 = 372 mm^2/m (>105)
Use T16-400 (502 mm^2/m).

Check Other Reinforcement Content Rules:

Shear rebar spacing must be less than	600 > 400 :OK
	3x thickness = 3 x 150 = 450 > 400: OK

Total rebar must be > 0.002 x 1000 x 150 = 300mm^2/m
Total rebar used = (904.8/3) + 502 = 803.6 > 300mm^2/m: OK

Vertical rebar must be > half horizontal rebar:
904.8/3 = 301.6 > 502/2 = 251mm^2/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. the12mm bars):
K = smallest of: cover = 150/2-12/2 = 69mm
Clear spacing = 400mm
$5d_b$ = 5x12 = 60mm

= 1.5x12^{2}x410x1/(60x√(12) = 426.1mm
l_d = 426.1/(0.8x0.67) = 795mm > 305mm.
Hence use 800mm anchorage and splices.

For straight horizontal rebar (i.e. the16mm bars):
K = smallest of: cover = 150/2-16/2 = 67mm
Clear spacing = 400mm
$5d_b$ = 5x16 = 80mm

= 1.5x16^{2}x410x1/(67x√(12) = 678.3mm
l_d = 678.3/(0.8x0.67) = 1265.6mm > 305mm.
Hence use 1300mm anchorage and splices.