Structural Design of AguaClara Plants

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Abstract

Our main objective we wish to accomplish this summer is to analyze the reinforcement configuration and structural strength of the sedimentation and flocculation tank walls. In the previous semester, the structural design team analyzed the structural capabilities of the columns and walls for the Alauca plant using various assumptions and load cases. The previous team analyzed the walls as closely spaced concrete columns. By modeling the walls as columns the flexural support provided by the horizontal reinforcement is unaccounted for, but it allowed for the use of the same tools and procedures that is used for beam analysis. We seek to attempt to validate the previous team's calculations as well as suggesting methods to analyze the horizontal reinforcement in order to reduce over-designing. This report is meant to augment the Spring 2011 report.

1 Introduction

AguaClara plants use a poured concrete slab and stacks of bricks (see Figure 1) with both horizontal and vertical rebar for reinforcement. The relatively shallow tanks are approximately 1.6 m to 2 m deep and thus the hydrostatic forces are relatively small. This has made it possible to construct walls that are only 15 cm thick when finished.

Unreinforced masonry built using stacked bonds provide no flexural support, and therefore no structural capabilities. In order to analyze this complex system involving reinforced masonry, key assumptions need to be made. Under tension, it is very likely that the mortar will separate from the bricks. Because of this, the mortar and the bricks provide no support in tension, and the columns must be analyzed as cracked. It is useful to imagine the bricks simply as spacers for the rebars, to hold the rebars in place but not provide any structural support. It is then the function of the plaster to prevent seepage into the wall even if the interior mortar has separated from the bricks, so the wall must be reanalyzed to ensure that the plaster is not cracked.

By imaging the bricks as spacers, the whole wall can indeed be analyzed using traditional concrete techniques, given that the compressive stress is changed to the compressive stress of clay masonry and the allowable tension in the "concrete" set to 0 or cracked. With these assumptions, the wall can then also be modeled with two-dimensional stresses using traditional concrete techniques.

Figure 1: Bricks with Both Horizontal and Vertical rebar at the Alauca water treatment plant.

2 Column and Wall Design

Following the Spring 2011 report, the column with the highest moment in the Alauca plant design was analyzed. The scenario is a free standing 1.74 m wall without rubble or back-fill contributing to the support.

The flocculation and sedimentation tank columns involves #3 rebars and #2 ties 20 cm apart (see Figure 2 and 4).

The flocculation and sedimentation tank wall layout involves bricks laid with running joints, inter-spaced with #3 rebars (see Figure 3 and 4) .

2.1 Design Shear and Moment Calculations

The first step in the analysis is to calculate the shear and moment loads which would act on the column. These loads were determined by the water depth and the tributary area which transferred a portion of the total water pressure against the wall to the column or wall element. For our analysis of the tank walls, we modeled each 0.28 m (two halves of a brick plus a 2 cm joint) width of wall as a column. This was to account for the vertical rebar in the wall which provides flexural support. Each stack of brick and vertical rebars could now be analyzed as a single column. In our analysis, we made several key assumptions:

- Columns and wall sections act as cantilever beams with tributary loads

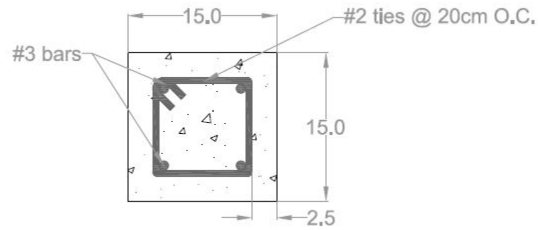


Figure 2: Column Rebar Configuration

- Columns will have tributary loads from half the span of the masonry walls on each side of the column
- Columns will act as t-beams, capitalizing on the reinforcement inside the wall section
- Wall sections will have a tributary load area of its own width (0.28 m)
- The water level reaches the top of the tank height
- The height of the tank (H_{Floc}) is 1.74 m without support from the back-fill, 0.8 m with support from the back-fill, and 1.17 m with support from the rubble-work

The maximum load at the base of the column (P_{Max}) with units (N/m) is a function of the density of water (ρ), the acceleration due to gravity (g), the height of the flocculator tank wall (H_{Floc}), and the tributary width of load (W_{Trib}) (Equation 1).

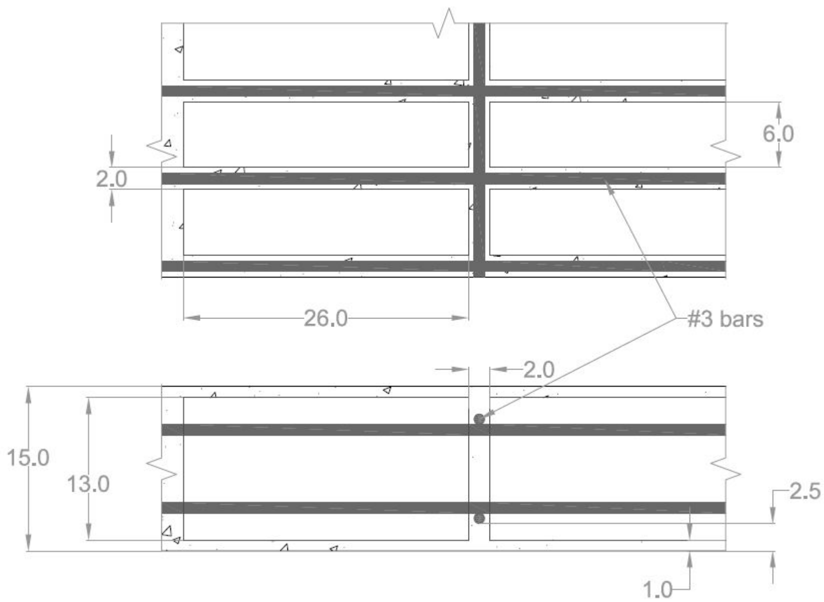
$$P_{Max} = \rho \cdot g \cdot H_{Floc} \cdot W_{Trib} \tag{1}$$

For the entire wall and column system, the tributary width of load (W_{Trib}) was calculated by taking the length of the wall ($L_{FlocWithWalls}$) and dividing by two (Equation 2). The column with the largest tributary width and greatest height would experience the worst case moment and shear. This value will change depending on the dimension for the wall, but for the Alauca plant, the tributary width for the column was 1.99 m. For the wall sections, the tributary width was taken to be 0.28 m. These walls would only have to support the moment from the water pressure affecting the section.

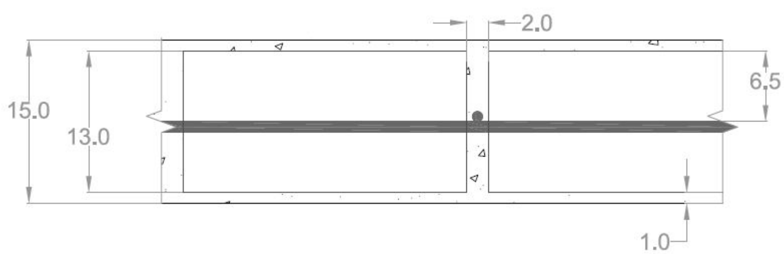
$$W_{Trib} = \frac{L_{FlocWithWalls}}{2} \tag{2}$$

The shear in the column (V_y) is a function of the maximum load at the base of the column (P_{Max}), the height of the flocculator tank wall (H_{Floc}), and the distance from the base of the column (y) (Equation 3).

$$V_y = \frac{1}{2}\left(P_{Max} - \frac{P_{max} \cdot y}{H_{Floc}}\right) \cdot (H_{Floc} - y) \tag{3}$$



(a) Wall Rebar Configuration up to 1m (in cm)



(b) Wall Rebar Configuration from 1m to 1.74m (in cm)

Figure 3: Wall Rebar Configuration

Figure 4: Wall and Column Rebar Configuration at the Alauca Water Treatment Plant

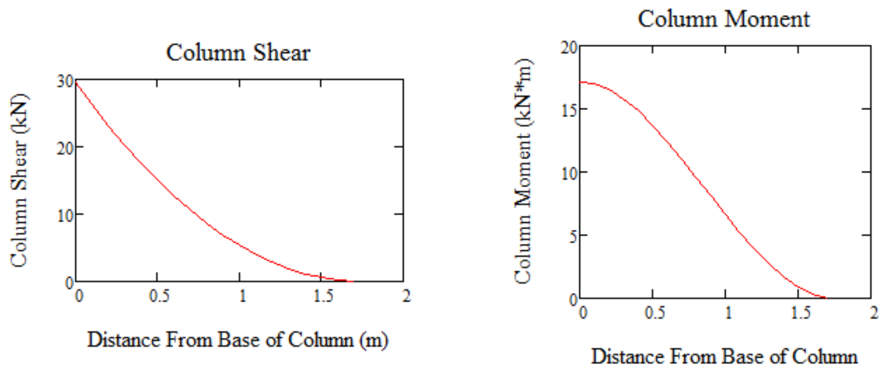


Figure 5: Shear and Moment in the Column with respect to Distance from the Base

The moment in the column (M_y) is a function of the maximum load at the base of the column (P_{Max}), the height of the flocculator tank wall (H_{Floc}), and the distance from the base of the column (y) (Equation 4).

$$M_y = \frac{1}{2}\left(P_{Max} - \frac{P_{Max} \cdot y}{H_{Floc}}\right)(H_{Floc} - y)\left(Y + \frac{H_{Floc} - y}{3}\right) \quad (4)$$

Figure 5 shows the shear and moment from the wall tributary section with respect to the distance from the base of the column when no support from the back-fill is assumed in the flocculator tank column.

The maximum shear and moment would occur at the base. We set y=0 and calculated the maximum shear and moment using equations 3 and 4 respectively. The results for this example are as shown below in Table 1.

2.2 Flexural Analysis

Since the columns and walls are loaded in flexure, a concrete beam analysis was used to calculated the flexural strength and shear strength. Three components of the tank walls were analyzed, the composite flexural strength of the concrete column with the walls, acting as a T-beam (henceforth referred to as T-beam), under the load of the entire tributary area (1.99 m); the flexural strength of the concrete columns under the load of just its exposed area (0.15 m); and the flexural strength of the masonry wall per analyzed column (0.28 m).

The first step of the analysis is to determine whether the thickness of the wall is adequate to sustain deflections. This is done using table 9.5(a) from ACI Building code 318 (see Figure 6). The wall slab is considered to be one end continuous since it is a two span slab, so the thickness of the slab must be greater than

	Tributary Width (m)	Load, P_{Max} (kN/m)	Shear (kN)	Moment (kN m)
Concrete Column	0.15	2.56	2.23	1.29
Masonry Wall Sections	0.28	4.78	4.16	2.41
Composite Concrete Column and Masonry Wall	1.99	33.96	29.54	17.13

(a) Max Shear and Moment at Bottom of Tank

	Shear (kN)	Moment (kN m)
Masonry Wall Sections	0.75	0.94
Composite Concrete Column and Masonry Wall	5.34	6.66

(b) Max Shear and Moment at 1m from Bottom of Tank

Table 1: Maximum Shear and Moment in Tank Walls and Columns

$$l/24 = 1.99m/24 = 0.083m$$

The thickness of the wall slab is 15 cm.

For the T-beam analysis, the amount of wall reinforcement that can be included in the column reinforcement must be calculated. This distance is called the effective width, b_{eff}. The effective width is a function of the span length, l, beam width, bw, span thickness, t and the beam spacing (Equation 5).

$$b_{eff} = \min \begin{cases} 0.25 \cdot l \\ bw + 16 \cdot t \\ 12 \cdot [beamspacing] \end{cases} \tag{5}$$

The effective width of this column-masonry wall system is 0.5 m. Because the effective width of this column does not include any additional reinforcement from the masonry wall, the T-beam analysis is the same as the concrete column analysis except for a much greater shear and moment from the increased tributary width.

To calculate the moment resistance offered by the reinforcement you first determine the tension force, T, by multiplying steel area, A_s and the strength of the steel, f_y (Equation 6).

$$T = A_s \cdot f_y \tag{6}$$

TABLE 9.5(a) — MINIMUM THICKNESS OF NONPRESTRESSED BEAMS OR ONE-WAY SLABS UNLESS DEFLECTIONS ARE CALCULATED

	Minimum thickness, h			
	Simply supported	One end continuous	Both ends continuous	Cantilever
Member	Members not supporting or attached to partitions or other construction likely to be damaged by large deflections			
Solid one-way slabs	$\ell/20$	$\ell/24$	$\ell/28$	$\ell/10$
Beams or ribbed one-way slabs	$\ell/16$	$\ell/18.5$	$\ell/21$	$\ell/8$

Notes:
Values given shall be used directly for members with normalweight concrete and Grade 60 reinforcement. For other conditions, the values shall be modified as follows:
a) For lightweight concrete having equilibrium density, w_c, in the range of 90 to 115 lb/ft^3, the values shall be multiplied by $(1.65 - 0.005w_c)$ but not less than 1.09.
b) For f_y other than 60,000 psi, the values shall be multiplied by $(0.4 + f_y/100{,}000)$.

Figure 6: ACI 318-08 Table 9.5(a)

	T-beam	Concrete Column	Masonry Wall Sections up to 1 m	Wall Sections from 1 m to 1.74 m
Thickness, h (cm)	15	15	15	15
Width, bw (cm)	15	15	28	28
Compressive Strength f'_c (psi)	3000	3000	8250	8250
Reinforcing Bar number	3	3	3	3
Reinforcing Stirrups number	2	2	N/A	N/A
Cover (cm)	2.5	2.5	2.5	6.5
Number of Tension-side Bars	2	2	1	1
Yield Strength of Rebar, f_y (ksi)	40	40	40	40

Table 2: Flexural Analysis Inputs

Because the internal moment is radially symmetric along the neutral axis, the compression force, C, is equal to the tension force, T (Equation 7).

$$C = T \tag{7}$$

The stress experienced by the concrete in compression is calculated using the "Whitney" Stress Block. The "Whitney" Stress Block translates a non-linear stress distribution into an equivalent constant stress distribution. The height of the stress block, a, is a function of the concrete compressive stress, f'_c, the compression force, C, and the width of the member (Equation 8).

$$a = \frac{C}{0.85 \cdot f_c' \cdot bw} \tag{8}$$

The distance from the compression face to the location of the steel is the thickness of the beam, h, minus the cover, minus the stirrup diameter, minus the radius of the reinforcing bar (Equation 9).

$$d = h - \{cover\} - \{Diameter_{stirrup}\} - \{Radius_{rebar}\} \tag{9}$$

The internal moment, M_n, is then calculated by multiplying the tension force, T, by the internal moment arm (Equation 10).

$$M_n = T \cdot (d - \frac{a}{2}) \tag{10}$$

Table 2 shows the various inputs that were used to calculate the internal moment, M_n.

The internal moment, M_n must then be multiplied by a strength reduction factor, Φ. The Strength reduction factor, Φ, is a function of the strain in the tension steel. The factor is used to dissuade a design that has a compression controlled failure because compression failures are much more dramatic than tension failures (Equation 11).

	T-beam	Concrete Column	Masonry Wall Sections up to 1 m	Wall Sections from 1 m to 1.74 m
Internal Moment, M_n (KN m)	4.37	4.17	2.34	1.55
Internal Moment with Strength Reduction Factor of 0.9 (KN m)	3.93	3.75	2.11	1.40

Table 3: Flexural Analysis Results

$$\Phi = \begin{cases} 0.65 & \varepsilon_s \leq \varepsilon_y \\ 0.65 + (\varepsilon_t - \varepsilon_y)\left(\frac{0.250}{(-0.005-\varepsilon_y)}\right) & \varepsilon_y \leq \varepsilon_s \leq 0.005 \\ 0.90 & \varepsilon_s > 0.005 \end{cases} \tag{11}$$

Table 3 shows the calculated internal moments M_n for the various members. By comparing it to Table 1, we can see that the wall cannot function as a T-beam, but instead the masonry wall does contribute significantly to the strength of the wall. However, we can also see that the masonry wall itself is not enough to withstand the force.

3 Suggestions and Future Work

An alternate construction technique can be implemented to increase the flexural capability of the wall. Using perforated bricks instead of solid bricks would allow additional reinforcement to be placed in the wall, running bonds, and also mortar to be grouted into the bricks. This would provide increased flexural support from additional steel reinforcement and a more continuous wall. The moment capacity, Mn*Φ, from adding one additional rebar to the masonry wall is 4.18 KN m,. This is sufficient to withstand the hydrostatic force and may even negate the need for the concrete columns. Perforated bricks would also allow for a more adaptable design, depending on the tank depth and maximum moment.

In the following semester, the beam analysis calculations should be translated into a Mathcad document and laboratory testing for the strength of the wall should be performed.

The wall should be around 1 m high by 0.8 m wide. The clay bricks should be of comparable to those used in the water treatment plants. The mortar shall also be of comparable strength to that used in the water treatment plants. Type N masonry mortar can be made with:

1. 2 cylinders (3 by 6 inch) Portland Cement

2. 4 cylinders hydrated lime
3. 24 cylinders sand
4. 11 cylinders water
    (a) Add water incrementally until the desired consistency is achieved. State with no more that 50% of the target water content. More water may be needed to give the mortar the proper consistency, but adding water will lower the strength of the mortar.

The first wall should be built as the current design with stacked bonds interspaced with rebar (see Figure 7).

The second wall should be built with perforated bricks and running bonds (use half bricks where necessary), the rebar shall be interspaced and mortar shall be grouted into the perforations (see Figure 8).

The testing procedure shall be as follows:

1. Cut plastic sheet 4 feet wide and about 12 feet long
2. Place test frame on plastic
3. Lay coursing marks to correspond to the top surface of the brick for each brick, allowing 2 cm for joints and rebar
4. Build wall with rebars and mortar
5. Perform one mortar flow test as soon as mortar is ready to use, and one more flow test as soon as the wall is finished
    (a) Wipe brass table and mold clean
    (b) Place mold on center of table
    (c) fill half depth with mortar
    (d) Tamp with rectangular prism 20 times
    (e) Fill full depth
    (f) Tamp 20 times
    (g) Smooth top with trowel using a sawing motion across the top of the mold
    (h) Wipe table top clean and dry
    (i) Remove mold and immediately turn handle 25 times in 15 seconds
    (j) Measure the major and minor diameters of the resulting "pancake"
6. Wait 28 days
7. Perform strength test using a suction device on a fistula in the plastic
8. Measure:

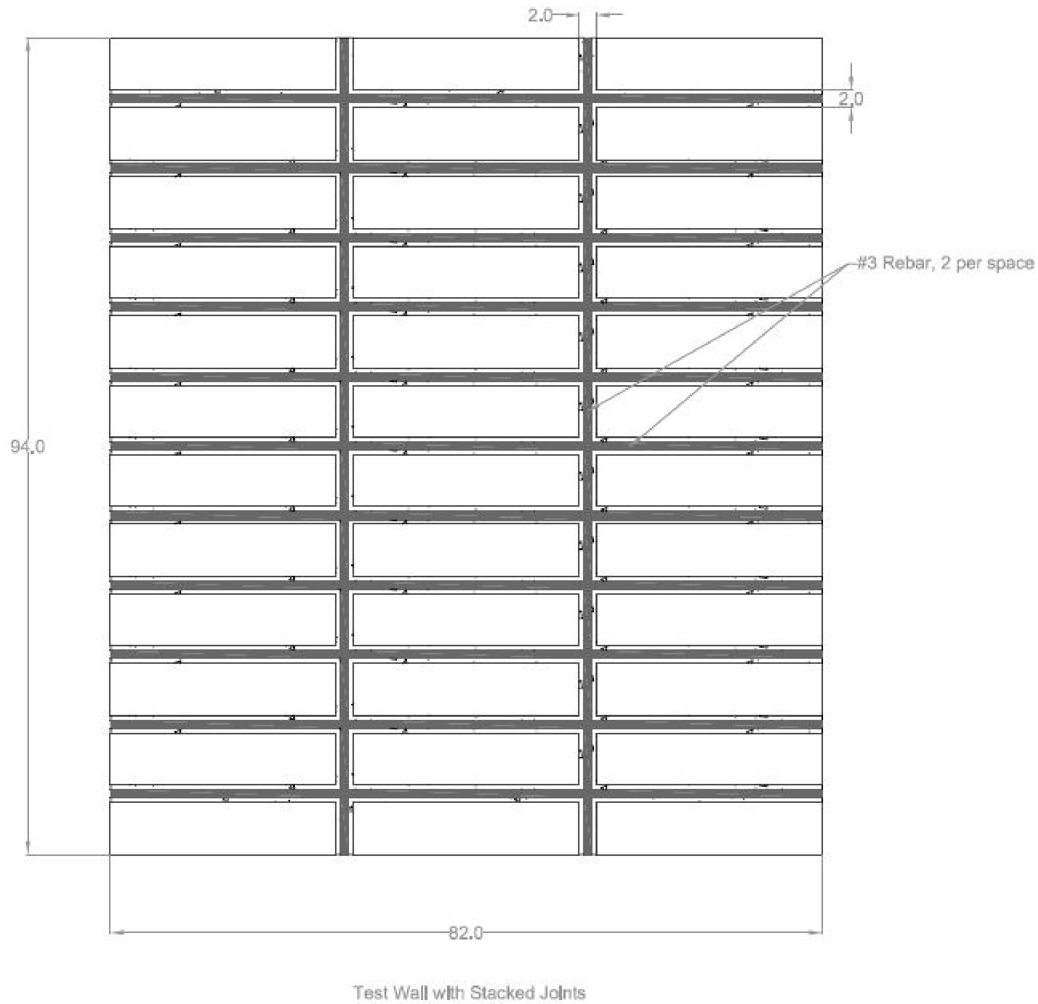


Figure 7: Test Wall with Stacked Joints

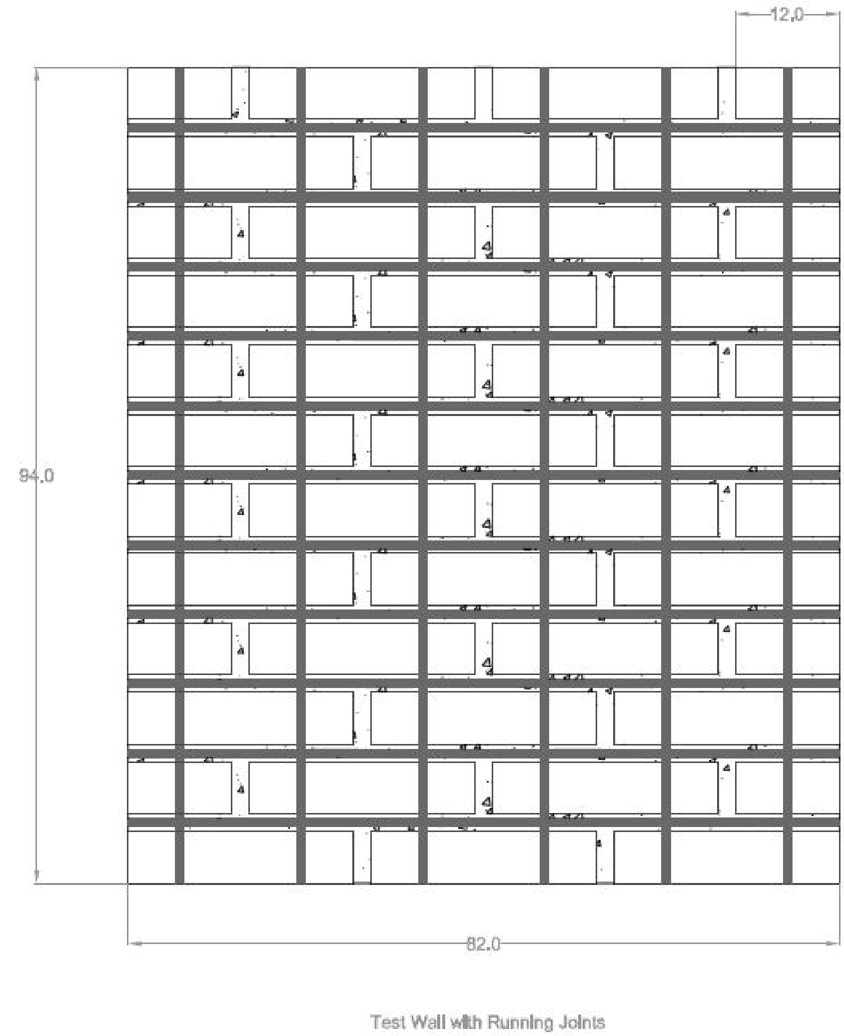


Figure 8: Test Wall with Running Joints

(a) Height difference in water column at instant of flexural failure of wall

(b) Air pressure difference

(c) Average width of wall (around 82 cm)

(d) Space of wall from center of lower support to center of upper support, l (around 94 cm)

(e) Distance from center of lower support to plane of failure, x

(f) Bending moment at plane of failure

$$M = \frac{wl}{2}x - \frac{wx^2}{2}$$

where:
w is the Air pressure multiplied by the average width of the wall
l is the height of the wall (94 cm)
x is the distance from center of lower support to plane of failure

(g) Width of wall at failure plane, b (around 82 cm)

(h) Thickness of wall at failure plane, t (around 6 cm)

(i) Area of wall at failure plane, A (b*t)

(j) Moment of inertia at failure plane

$$I = \frac{1}{12}bt^3$$

(k) Flexural stress at failure plane

$$\sigma_{flexural} = \frac{M \cdot \frac{t}{2}}{I}$$

(l) Weight of upper portion of failed wall, P

(m) Axial compression stress due to self-weight at failure plane

$$\sigma_{axial} = \frac{P}{A}$$

(n) Net tension stress at failure plane

$$\sigma_{axial} - \sigma_{flexural}$$

(o) Net compression stress at failure plane

$$\sigma_{axial} + \sigma_{flexural}$$