Structures Team Reflection Report

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AguaClara Reflection Report

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Abstract

Our main objective we wished to accomplish this semester was to create a means of automating the design of the columns and walls for the design tool. We began this work by analyzing the structural capabilities of the columns and walls for the Alauca plant.

There are three different load cases that guide the design of the columns and walls of the tank. The first case assumed that the tank walls were supported by the surrounding backfill. The second case assumed no support from this backfill. The third case analyzed the structural importance of the rubble which lies at the base of the sedimentation tank.

For our analysis of the walls, we modeled them as closely spaced columns. As seen in Figure 1, the vertical rebar that runs through these walls will add flexural support. By modeling the walls as columns we accounted for this vertical rebar. Modeling the walls as a combination of individual columns also allowed us to use the same tools and procedures that we used for the analysis of the columns. We set the moment at initial cracking as our first failure moment. We determined this moment by using the Transformed Moment of Inertia method. We also wanted to know the moment that would render the walls and columns to no longer be structurally intact. This would be the largest moment that these walls and columns could experience before ultimate failure. We believed that this value would be of importance for future analyses which would incorporate earthquake conditions. For this ultimate failure analysis, we used the Column Interaction Diagram. This method plots the area of all axial load and moment cases that a column would be able to support safely. All methods are explained in detail in the report.

Keywords: Structural capabilities, automated design, backfill, design moment, cracking moment, ultimate failure, moment capacity, transformed moment of inertia, column interaction diagram

Figure 1. Picture of the rebar pattern in the walls and columns of the AguaClara plant tanks.

Introduction

The Structures team is new to the AguaClara project. This team was created to provide structural analysis for the growing number of AguaClara water treatment plants. Currently, the automated design tool does not provide structural designs for the tanks. Our ultimate goal is to automate the design of the tanks including column spacing, column sizing, and rebar use when given the required tank height, width, and length.

To begin our analysis, we assumed three different cases which would affect the structural integrity of the tanks. In the first case, we assumed that the tank walls were supported by the backfill which enclosed the tanks. The second case assumed no support from this backfill. The third case analyzed how the rubble in the base of the sedimentation tank affected the stability of the wall which runs between the sedimentation tank and flocculator tank. For each case, we determined the moments applied to the columns and walls by the water pressure.. We then calculated the “Moment Capacity” of the columns and walls which determined the moment the columns and walls could experience before cracking as well as for the ultimate failure point.

For the cracking analysis, we used the Transformed Moment of Inertia method to determine the moment at cracking in the columns and walls. This analysis converted the steel into an equivalent concrete area which would yield the same strength and flexural properties. Using this transformed moment, we used relevant formulas to calculate the moment at cracking. The Column Interaction Diagram was used to determine the moment at ultimate failure. We designed a MathCAD file which would output the moment the column could experience before failure based on the dimensions and rebar properties given as inputs. Because the walls had vertical rebar running up between each brick, we modeled the walls as a collection of closely spaced columns. As a result, the MathCAD files we coded for the Transformed Moment of Inertia and the Column Interaction Diagram could be applied to the analysis of the walls. Using these tools, we summarized our results for the three different conditions in our report.

Both the Transformed Moment of Inertia and Column Interaction Diagram are tools that are currently used in the structural engineering field. These tools focus the analysis on the individual structural elements that provide support against the moment from the water pressure in the tanks. We did not use the Hardy Cross method that was used by the Structural Engineer in Honduras. The Hardy Cross method determines the moment in the structural elements factoring in all the members in the system. Although the Hardy Cross method is an excellent tool in determining the moment in statically indeterminate frames, it is no longer commonly used due to newer methods such as the finite element method and the direct stiffness method. Though our analysis approached the problem in a different manner, we hope that the results from our analysis will shed more light on this engineering venture.

Column and Wall Analysis

The Structures Team wanted to determine the structural integrity of the columns. We took the design specifications from the AguaClara plant to be built in Alauca (Figure 2 and Figure 3) and used these in our calculations to determine the applied load from the water in the tanks. Our analysis ran three scenarios, one where the backfill did not add support against the exterior wall, another scenario where the backfill did contribute support, and one where the rubble between the flocculator tank and sedimentation tank added support for the dividing wall. These three cases focused on determining the applied moments for the column in the middle of the wall separating the sedimentation tank and the flocculator tank as well as the column in the middle of the wall adjacent to the backfill. These columns would experience the largest moments due to the water pressure and will guide the design for the tanks. For the analysis of the walls, the wall that lies between the sedimentation tank and the flocculator tank as well as the outer flocculator tank wall are the walls which will guide the design of the plant. The calculations for these "Design Moments" are shown below.

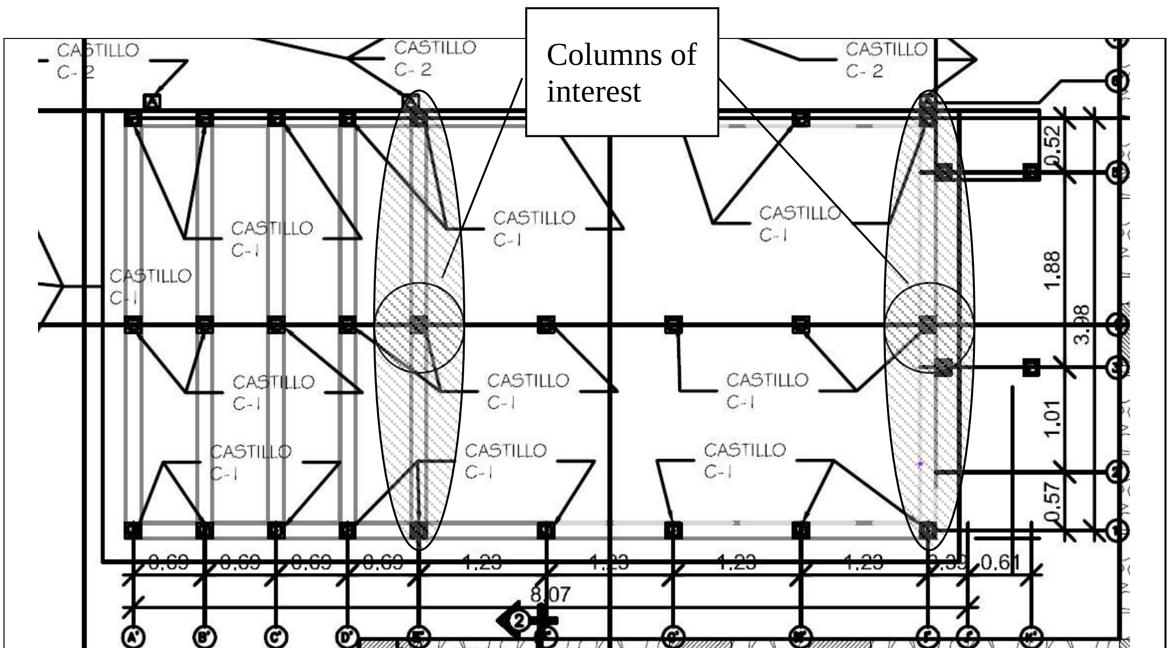

Figure 2. Plan view of the sedimentation (left side) and flocculator (right side) tank in the Alauca plant. The results from the analysis of the columns and walls marked in the figure will guide the design of the tanks.

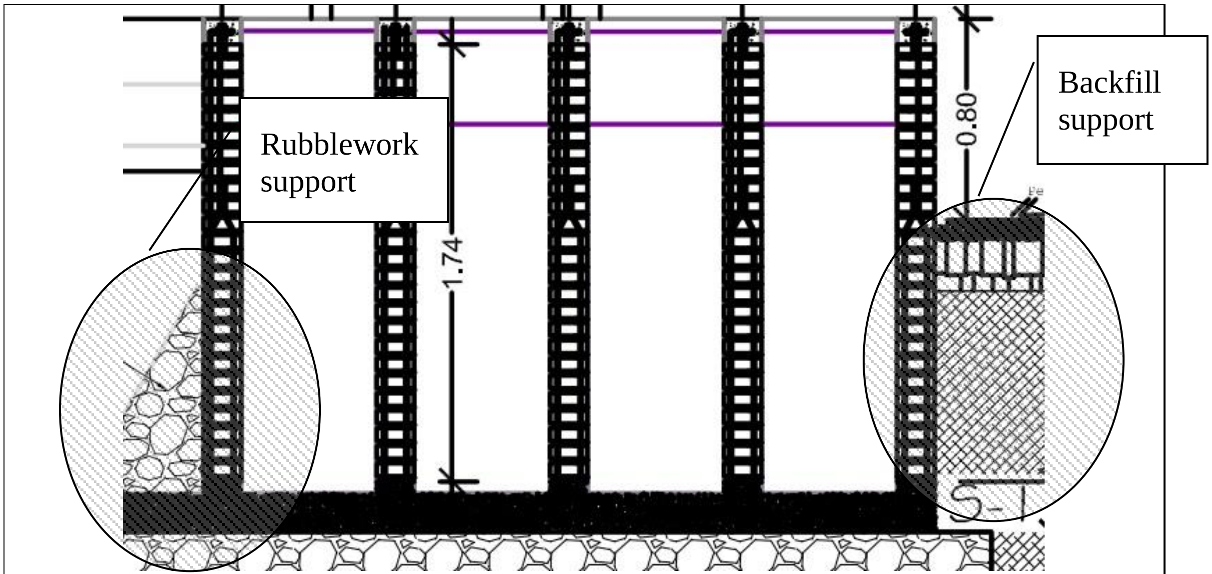

Figure 3. Elevation view of the Flocculator tank in the Alauca plant. The rubblework and backfill which affect the structural analysis of the walls and columns are marked. The figure also shows the change in the number of rebar in the wall 0.80 m down from the top of the wall. We discuss changing the number of rebar in the wall in the Results and Discussion section of the report.

Design Shear and Moment Calculations

The first step in our analysis was 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 0.305 m (1 foot) width of wall as a column. This was to account for the vertical rebar in the wall which provides flexural support. 0.305 m was used as the width because each brick that is used in the construction of the plants are one foot long. Each stack of brick and vertical rebar 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 4)
- Columns will have tributary loads from half the span of the masonry walls on each side of the column
- Wall sections will have a tributary load area of its own width (0.305 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 backfill, 0.8 m with support from the backfill, and 1.17 m with support from the rubblework

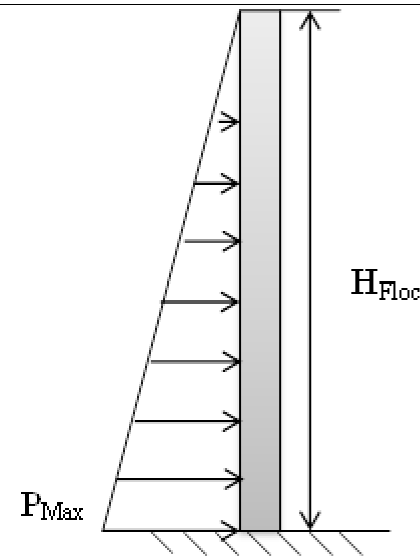

Figure 4. Representative diagram showing the transposition of the columns and wall sections into a cantilever beam. P_{Max} is the maximum force due to water pressure at the base of the wall and H_{Floc} is the height of the tank column.

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 columns, 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. In the Alauca plant, the columns shown in Figure 2 are the ones with the highest moment. For the wall sections, the tributary width was taken to be 0.305 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).

$$V_y = \frac{\rho \cdot g \cdot W_{Trib}(H_{Floc} - y)^2}{2} \tag{3}$$

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).

$$M_y = \frac{\rho \cdot g \cdot W_{Trib}(H_{Floc} - y)^3}{6} \quad (4)$$

The team then plotted the shear and moment with respect to the distance from the base of the column. Figure 5 shows an example of these shear and moment diagrams when no support from the backfill is assumed in the flocculator tank column.

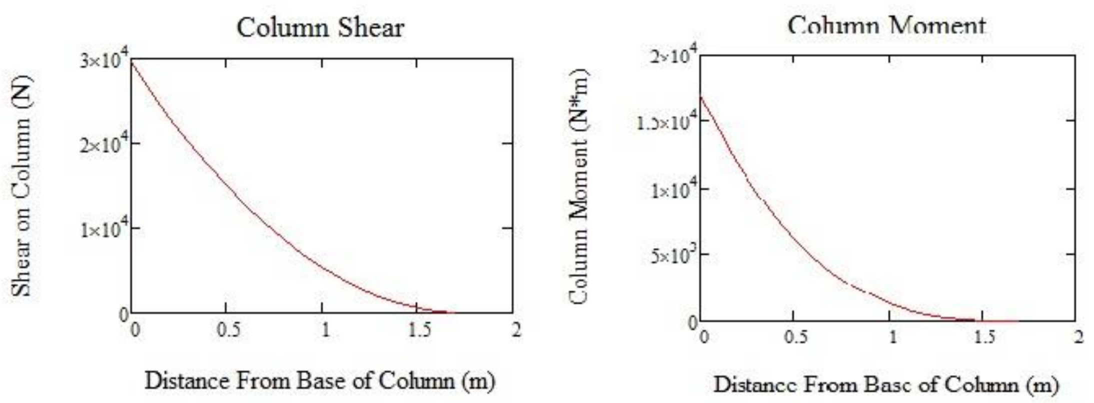

Figure 5. Shear and Moment in the column with respect to distance from the base

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. These set of calculations were performed for all three support cases on the columns and 0.305 m wall sections; the results are summarized in Table 6.

Table 1. Maximum Shear and Moment in Flocculator Tank Columns

Support Case	*Water Height*	*Maximum Shear*	*Maximum Moment*
No support from backfill	1.74 m	29.5 kN	17.1 kN·m
Support from backfill	0.8 m	6.25 kN	1.67 kN·m
Rubblework support	1.17 m	14.1 kN	5.21 kN·m

Failure at Cracking Analysis

After determining the maximum moment the columns and walls would experience for each load case, we needed to calculate the moment which would cause these columns and walls to experience their first crack. We could then compare these values to conclude whether the columns and walls would crack under current design conditions. We would set this value to be our main failure criterion because any major crack would lead to further crack propagation and possible exposure of rebar. We note that this method only tells us the moment when cracks would form due to the concrete being in tension. We have not performed an analysis on the size of the cracks to determine whether these initial cracks would be of significance. Future team members should run calculations to determine at what crack size the walls and columns would begin to lose structural capabilities. This would increase the failure threshold for the design of the walls and columns.

For the crack analysis we used the Transformed Moment of Inertia Method. This method replaces the steel with an equivalent amount of concrete that will give the same structural response as the steel. This gives us one uniform value for the moment of inertia. We can then use this new moment of inertia to determine the moment at cracking in the concrete using a formula that was developed empirically and is widely accepted in industry. We go over these calculations in greater depth below.

The first step to transform the moment of inertia is to determine the strength ratio (n) between the Young's Modulus of steel (E_s) to the Young's Modulus of concrete (E_c) (Equation 5).

$$n = \frac{E_s}{E_c} \tag{5}$$

The second step is to theoretically replace all the steel in the column with concrete. The area of concrete (A_{sc}) that would yield the same compression strength properties as the steel which is replaced is the product of this strength ratio (n) and the area of the steel on the compression side of the column (A_{steel}). This is shown in Equation 6 below.

$$A_{sc} = A_{steel} \cdot n \tag{6}$$

For the tension side, the area of concrete (A_{st}) which would replace the steel is a product of the original area of the steel (A_{steel}) on the tension side and the ratio minus one (Equation 7).

$$A_{st} = A_{steel} \cdot (n-1) \tag{7}$$

The shifted neutral axis (Y_{NA}) is then a function of the width of the column (W_{col}), the height of the column (h_{col}), the additional area of concrete (A_{sc} and A_{st}), as well as the distance of the rebar from the datum line (d_{col} and d_{prime}) (Equation 8).

$$Y_{NA} = \frac{W_{col} \cdot h_{col} \cdot \frac{h_{col}}{2} + 2A_{sc} \cdot d_{prime} + 2A_{st} \cdot d_{col}}{W_{col} \cdot h_{col} + 2A_{sc} + 2A_{st}} \tag{8}$$

The new moment of inertia (I_T) is then found using the parallel axis theorem. It is also a function of the same variables from the calculation for the new neutral axis (Equation 9).

$$I_T = \frac{W_{col} \cdot h_{col}^{\ 3}}{12} + A_{sc} \cdot \left(Y_{NA} - d_{prime}\right)^2 + A_{st} \cdot \left(h_{col} - d_{prime} - Y_{NA}\right)^2 \tag{9}$$

Equation 10 shows how to determine the moment at cracking (M_{crack}). This is a function of the strength of concrete (fc), the transformed moment of inertia (I_T), and the shifted neutral axis (Y_{NA}). Note that this is a formula based on empirical data that is commonly used in concrete design. The strength of the concrete (fc) must be in psi; the unit is then divided out within the square root and then added to the result of the square root for this equation function appropriately.

$$M_{crack} = \frac{7.5\sqrt{\frac{fc}{1psi}} \cdot 1psi \cdot I_T}{Y_{NA}} \tag{10}$$

Ultimate Failure Analysis

We analyzed the maximum moment the walls and columns could resist before total failure. We used the Column Interaction Diagram which plots the total moment and axial load combination a column can withstand before failure. Ultimate failure of the column results when the concrete on the compression face fails in compression.

A brief description of the calculations used for the interaction diagram follows. A more complete description can be found in Appendix A.

Figure 6 shows a general cross section of a column showing the dimensions of the column along with the positioning of the rebar. The variable “h” refers to the column depth, “w” refers to the column width, “d” refers to the distance from the compression face of the column to the center of the tension rebar, “ d’ ” refers to the distance from the tension face of the column to the center of the tension rebar. This is equivalent to the concrete cover depth plus the diameter of the tie bar plus the radius of the reinforcing steel.

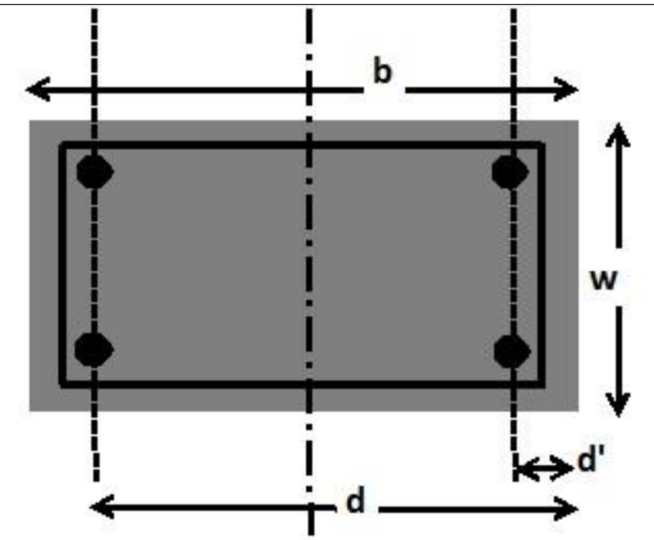

Figure 6. Typical cross section of a column.

The interaction diagram works by using similar triangles of known distances and strains to calculate the strain anywhere in the column. The neutral axis, or the value of "c" in Figure 7 is set at zero initially and increased incrementally to a maximum value of "h." The failure strain of concrete in compression ($\varepsilon_{failure}$) is known to be .003. Knowing this failure strain, the position of reinforcing steel, and the fact that the strain profile is linear within the linear elastic limits of the concrete, the strain is in the reinforcing steel and the compression concrete is calculated for each increment of "c." The strain profile for an arbitrary neutral axis position is shown in Figure 7.

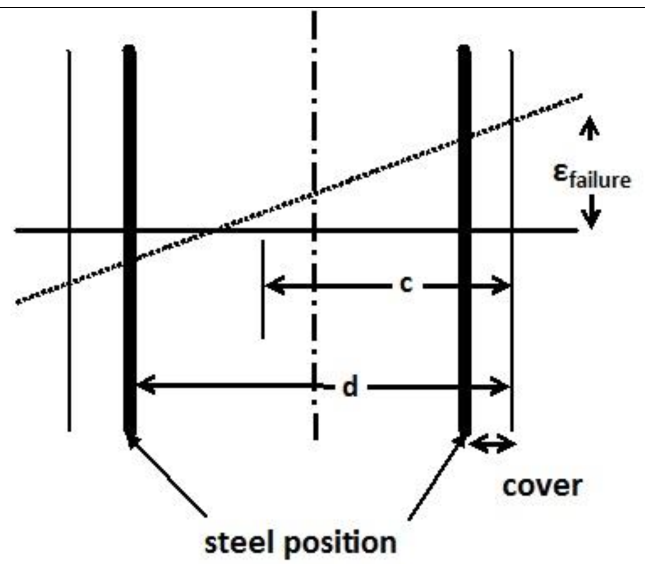

Figure 7. Strain profile through column for an arbitrary neutral axis position.

Figure 8 shows the strain in the reinforcing steel for all increments of the neutral axis location. A negative strain value means the steel is in tension. The graph is non-intuitive because it represents strains as the location of the neutral axis is changed. The compression steel is shown as partially in tension because until the neutral axis location reaches the centerline of the compression steel, it is not yet in compression. The reverse is true for the tension steel. Tension steel and compression steel refer to the steel location in this case, rather than whether they experience tension or compression at any given point.

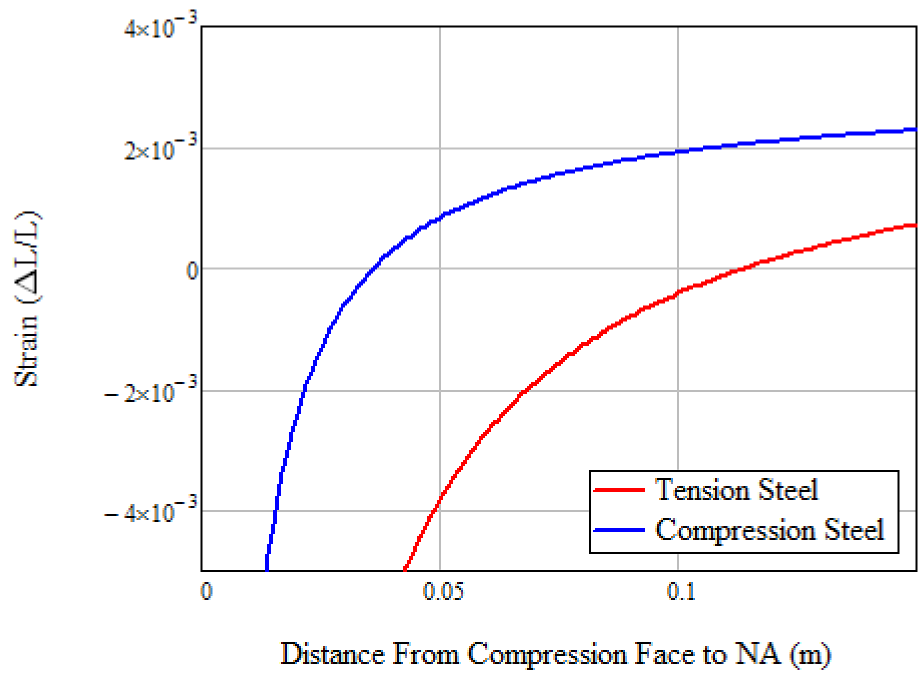

Figure 8. Strain in steel for all increments of the neutral axis

Using these strain values, we determined the forces and moments in the steel and column and superimposed these results to calculate the total force and moment in the column at failure. Our final product was the Column Interaction Diagram which shows the allowable axial load and moment cases the column can withstand until ultimate failure (Figure 9). The calculations mentioned above to create the Column Interaction Diagram can be found in greater detail in Appendix A.

The calculated strains are used to determine forces in the reinforcing steel and the compression concrete. These forces are superimposed to determine the maximum moment and axial force in the column at failure, shown in the column interaction diagram (Figure 9). The diagram shows the maximum combinations of moment and axial force (red line) the column can withstand before failure. Additional axial force on the column results in a greater maximum moment up to a limit, at which point maximum moment decreases with additional axial force. The blue line represents the maximum usable capacity of the column: the maximum capacity multiplied by a phi value factor of safety.

The phi value is based on yielding in the tension steel. Yielding in the steel before failure in the compression concrete results in a slow failure of the column that will show warning signs. The discontinuity in usable capacity results from changes in steel yielding at certain moment and axial force combinations.

Figure 9 below shows the interaction diagram for columns in the Alauca plant. The only axial load the column would face would be the self-weight of the column. Equation 11 shows the formula used to determine the weight of the column ($Weight_{Column}$) where w_{column} and b_{column} are the width and length of the column, H_{Floc} is the height of the tank, ρ_c is the density of the concrete, and g is the acceleration due to gravity.

$$Weight_{Column} = w_{column} \cdot b_{column} \cdot H_{Floc} \cdot \rho_c \cdot g \tag{11}$$

This resulted in a self-weight of 0.9 kN which is negligible in determining the moment capacity of the column. Thus, to determine the moment capacity, we set the force applied to be 0 kN in our calculations. This resulted in a moment capacity of 4.13 kN*m.

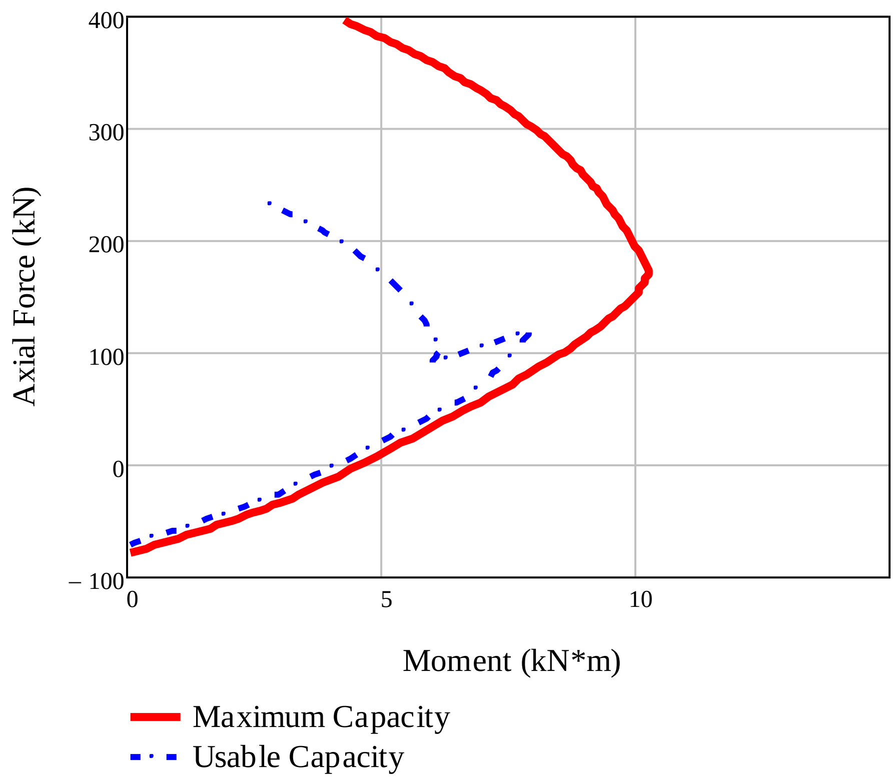

Figure 9. Column Interaction Diagram for column with properties from Table 2

Table 2 lists the properties of the rebar and concrete used in the analysis of the columns.

Table 2. Column Properties

Column width	15 cm
Column length	15 cm
Cover	2.5 cm
Strength of concrete	3000 psi
Steel grade	40 ksi
Rebar used	#3
# of rebar on each face	2

Results and Discussion

We summarize the properties of the column, wall section, and materials used in our analysis in Tables 3, 4, and 5 below. W_{col} refers to the width of the column and wall used in our analysis. D_{rebar} is the diameter of the vertical rebar while D_{tie} is the diameter of the rebar which wraps around the vertical rebar to hold it in place. N_{rebar} refers to the number of rebar on each side of the column. This value is half of the total number of rebar found in the column. Cover refers to the distance from the outer surface to the outer edge of the closest rebar. The values for the rebar are in U.S. units because the rebar number directly correlates to the diameter of the rebar by an 1/8 inch. For example, #2 rebar means the diameter of the rebar is 2/8 inches and #3 rebar means the diameter of the rebar is 3/8 inches. The material properties are also kept U.S. because the strength of the concrete (fc) needs to be in psi for the Transformed Moment of Inertia analysis. The yield strength of steel (fy) is given by its grade in U.S. units. For example, 40 grade steel means the steel yields at 40 ksi.

Table 3. Column Properties Used

W_{col}	15 cm
D_{rebar}	3/8 in
D_{tie}	1/4 in
N_{rebar}	2
Cover	2.5 cm

Table 4. Wall Properties Used

W_{col}	15 cm
D_{rebar}	3/8 in
D_{tie}	0 in
N_{rebar}	1
Cover	2.5 cm

Table 5. Material Properties Used

fc	3000 psi
fy	40 ksi
E_s	29000 ksi

E_c	3122 ksi

Figures 10, 11, and 12 illustrate the column analysis using the full tributary width, the analysis where the tributary width is equivalent to the width of the column, and the analysis of the one foot section of the wall.

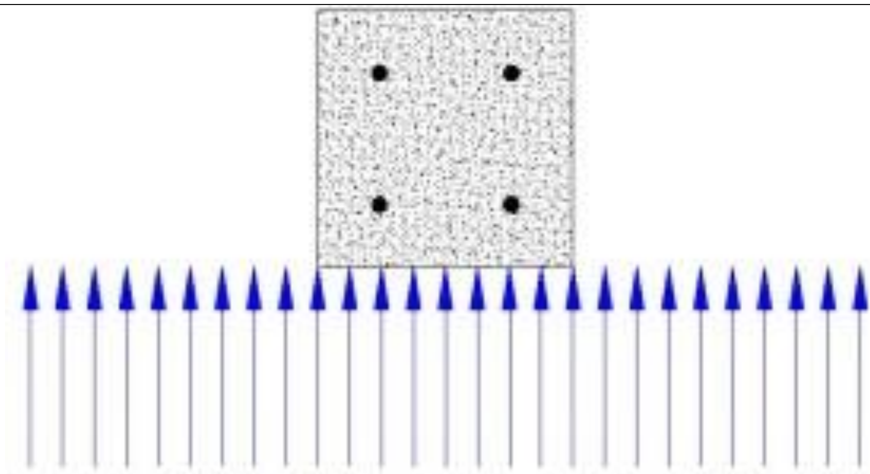

Figure 10. Column with moment with full tributary width from both sides of the wall

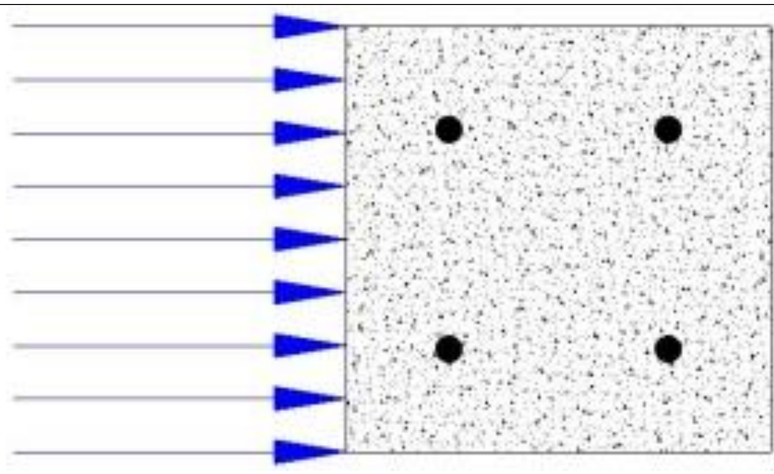

Figure 11. Column resisting moment from water pressure acting along just its own width

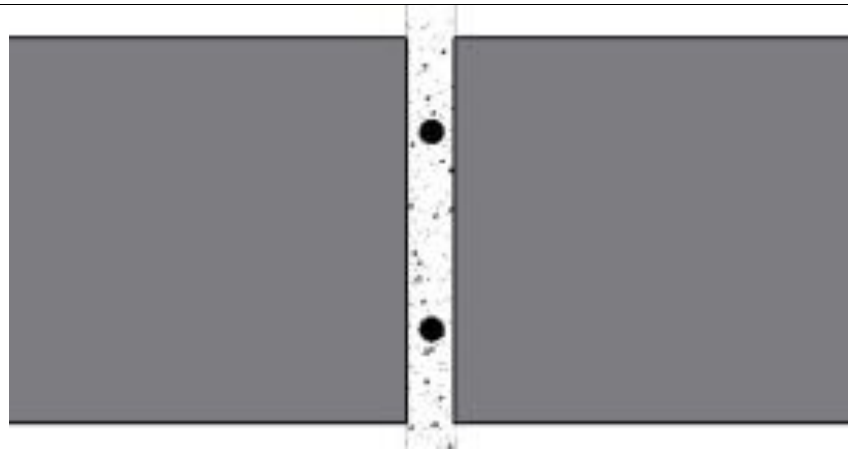

Figure 12. One foot section of the wall modeled as a column. The vertical rebar is taken to be the rebar located at the center of the column. Half of a brick is taken on each side and modeled as concrete.

Figure 13 also shows the three support cases which set the height of the water for our analysis. Scenario 3 shows the case when support from the backfill is assumed. This set the tank height to be 1.74 m in our analysis of the column and wall. Scenario 1 assumed that the backfill did provide support setting the height of the water to be 0.8 m. Scenario 2 assumed the backfill in the sedimentation tank also provided support setting the effective tank height to be 1.17 m.

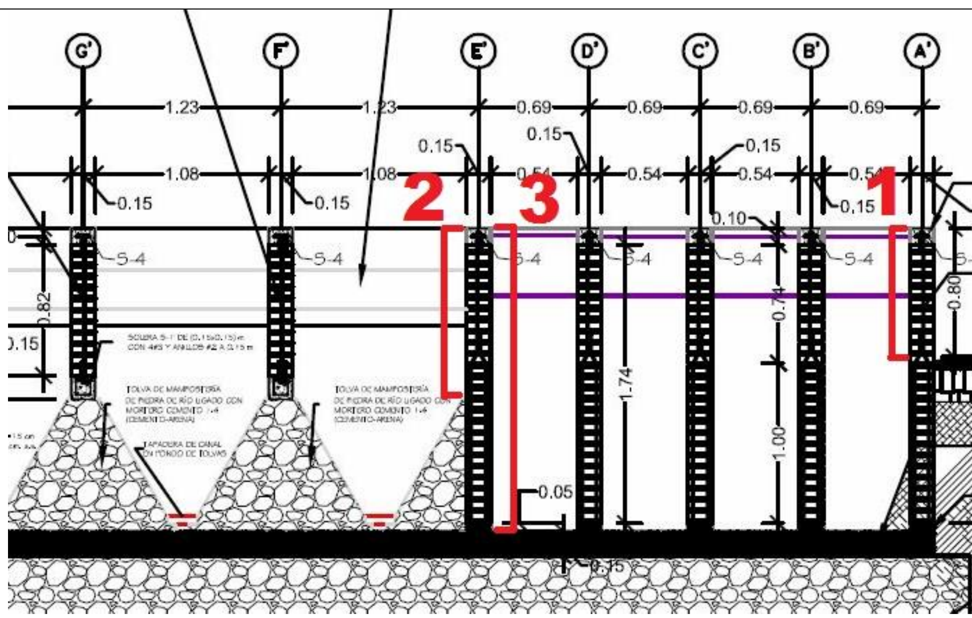

Figure 13. View of the Alauca plant which shows the three support cases. These cases determined the height of the water used in our analysis.

We provide a table summarizing the results of all of our calculations in Table 6 below.

Table 6. Summary of Results

		3	1	2
		Flocculator center exterior (column no backfill)	Flocculator center exterior (column with backfill)	Sed / Floc Boundary (with rubble work)
	Water Height (m)	1.74	0.8	1.17
Full Tributary Width	Tributary Width (m)	1.99	1.99	1.99
	Max Failure Moment (kN*m)	4.13	4.13	4.13
	Max Cracking Moment (kN*m)	1.75	1.75	1.75
	Calculated Moment (kN*m)	17.14	1.67	5.21
12" Wall Section	Max Failure Moment (kN*m)	2.61	2.61	2.61
	Max Cracking Moment (kN*m)	3.34	3.34	3.34
	Calculated Moment (kN*m)	2.67	0.26	0.8
Column Only	Tributary Width (m)	0.15	0.15	0.15
	Max Failure Moment (kN*m)	4.13	4.13	4.13
	Max Cracking Moment (kN*m)	1.75	1.75	1.75
	Calculated Moment (kN*m)	1.29	0.13	0.39

The Structures team's analysis of the flocculator and sedimentation tank walls and columns yielded some valuable results. The assumptions used in the design of the tank walls have large implications for the adequacy of the columns and walls given three worst case scenarios. Using the assumption that the walls act as beams to support all of the hydrostatic loads to the columns in the center of the tanks, we find that the columns would crack and fail in all cases except for the case where backfill supports the exterior tank walls. Walls and columns in existing AguaClara plants have not yet fallen over, so these assumptions do not adequately model the tank wall systems.

Given the presence of vertical steel reinforcement in the tank walls, the walls were modeled as a series of columns 0.305 meters wide and 15 cm deep. Our results show that these "wall columns" are sufficient to resist the calculated moment without cracking or failing. This indicates that the center columns may not be required. An interesting result we ran into, however, is that the cracking moment capacity of the "wall columns" exceeds the failure moment capacity. This result is possible since our analysis for failure in the walls contains implicit assumptions that the tension reinforcement has already yielded. This result indicates that once substantial cracks form in the walls, they will fail. The design moment only comes close to the cracking moment capacity during the worst case when the external flocculator walls are fully loaded without the presence of backfill. In future designs, this one wall may be made thicker to resist the higher moment value.

Comparing the capacities of the 0.305 meter wall sections and the columns we find that the columns are quicker to crack, but have a higher overall failure moment capacity. This may be a result of the smaller cover value in the columns where as in the wall sections the cover is about 0.15 m. Although cover is usually in the range of 1.5 cm to 7 cm, the unusually large 0.15 m cover is explained by the model we used to analyze our walls. We positioned the vertical rebar in the middle of this 0.305 m wide wall section and placed half a brick on each side of this rebar creating the “column” that was analyzed. Since cover is the distance from the outer surface of the column to the circumference of the rebar, the cover value for this wall section which is modeled as a column is approximately half the width of the brick. This wall column behaves slightly differently due to the unusually large cover. This additional concrete may help prevent initial cracking, but does not utilize the tensile characteristics of steel to maximize the ultimate capacity. Although the columns may indeed add extra structural support to the tanks, the AguaClara team must consider the importance of cracking as a failure criteria and consider removing these columns entirely. Small scale cracking in the columns in walls between tanks may not be of large concern, but cracking in exterior walls will lead to tank leakage.

It is important to note unknowns in our analysis of the tank walls. First to note is that our analysis of cracking in the walls is based on cracking in the mortar, equivalent to concrete in a column cracking. Actual failure in the walls would occur as separation of the mortar from the brick face. We know that the failure stress for this mode is of the order of 25 MPa, but this mode of failure is currently not included in our calculations. This failure mode would yield smaller acceptable moment capacities for the wall sections. Second to note is the unknown characteristics of moment sharing between wall sections and columns in the walls. We have modeled the walls using simplified assumptions that individual sections of wall resist all hydrostatic moment acting on them. In reality, horizontal reinforcing steel transmits some of this moment laterally between wall sections, central concrete columns, and to tank corners, which add substantial rigidity and strength. Inclusion of this moment sharing would increase moment capacity of the walls.

Our team also used the Transformed Moment of Inertia method to determine whether the walls truly required two bars of rebar. Our analysis calculated that the moment at incipient cracking with two rebar is 3.34 kN while the moment is 3.29 kN with just one rebar. The water height that would result in a moment of 3.29 kN is 1.85 m. Because the second rebar provides minimal additional moment capacity to the walls, we suggest using only one vertical rebar in the walls. With the current dimensions of the tank, one vertical rebar will be enough to resist the moment from the water pressure of the walls. This 1.85 m does not factor in support from the backfill and rubblework. If these elements do provide additional moment capacity for the walls, then additional rebar may be removed to reduce costs of construction.

To maximize the use of steel in the walls of the tanks, we suggest removing the excess rebar and placing the remaining rebar closer to the tension side of the walls instead of placing them in the middle of the wall. The steel would then be able to resist the tension forces in the walls more effectively. Wire meshing with minimal cover from mortar should also be considered. This method of construction may provide adequate tensile support while reducing costs significantly.

Future Work

By analyzing the tank walls and columns, the Structures Team has gained a better understanding of the problems posed as well as how to tackle these problems. We give the following suggestions as possible future projects subsequent Structures Team members can work on:

- Write a MathCAD file that will produce column sizing and spacing given key inputs such as the dimensions of the tank and height of the water; this will include rebar size and amount of rebar used. This is the ultimate goal of our team
- Replace the code for the Column Interaction Diagram method with the equations that determine moment capacity at zero axial force (http://www.strunet.com/concrete_column/ccol10.html)
- Continue with the cracked analysis to determine the size of the cracks at initial cracking and the size of the cracks which make the columns and walls become vulnerable
- Determine the proportion of the moment that is applied to the walls and to the columns. Currently, the moment from the water pressure is supported by both the columns and walls. Further analysis would result in a lower failure criterion for the walls and columns. Our current "Design Loads" are too high and will result in a very conservative design
- Explore the use of steel meshing as opposed to rebar
- Account for the structural support provided by the horizontal rebar
- Begin analysis of the stacked rapid sand filter walls
- Create a materials property team to determine actual strength properties of the material used in Honduras as well as maintain quality control of construction; this team will have to design prototype-able methods to test materials in Honduras.

Team Reflections

Because the Structures team is a newly formed team, we had a hard time hitting the ground running. A good portion of our time was spent on collecting resources, talking to professors, and figuring out exactly which tasks could viably be completed given our engineering background and time constraints. However, after we narrowed down our goals, we have steadily made progress in our analysis of key structural elements in the Alauca plant and will soon be able to automate an efficient design of tank columns and walls.

The team has definitely been challenged intellectually, given our limited structural engineering experience, as well as organizationally, as this is everyone's first time working with each other. However, we have utilized our resources to acquire the necessary knowledge required to accomplish our tasks. Also, we have learned each other's strengths and weaknesses and are now able to work efficiently to ensure all of our assignments are completed on time and to the best of our abilities.

Although our results are not conclusive, we feel we have made solid strides and have created a good foundation for future progress. We have done a thorough analysis of the walls and columns and have annotated all of our files allowing future members to continue our work. We hope our efforts have helped AguaClara take a step in the right direction in its mission to make clean water more accessible to the communities that need it the most.

Appendix A

Calculations for Forces and Moments for the Column Interaction Diagram

Tension Steel:

The stress in steel (f_{st}) is a function of the strain (ε_{st}). fy is the yield strength of the steel and E_s is the Young's Modulus for the steel.

if $|\varepsilon_{st} \cdot E_s| \leq fy$

$$f_{st} = \varepsilon_{st} \cdot E_s$$

if $|\varepsilon_{st} \cdot E_s| > fy$ or $\varepsilon_{st} \cdot E_s < 0$

$$f_{st} = -fy$$

otherwise

$$f_{st} = fy$$

The force in the steel (F_{st}) can be found by multiplying the stress in the steel (f_{st}) and the area of the steel (A_{steel}).

$$F_{st} = f_{st} \cdot A_{steel}$$

The distance from the neutral axis to the tension steel ($Dist_{st}$) is a function of the length of the column (b_{col}) and the location of the center of the rebar from the compression face of the column (d_{prime}).

$$Dist_{st} = -\left(\frac{b_{col}}{2} - d_{prime}\right)$$

The moment in the tension steel ($Moment_{st}$) is the product of the force in the steel (F_{st}) and the distance from the neutral axis ($Dist_{st}$).

$$Moment_{st} = F_{st} \cdot Dist_{st}$$

Compression Steel:

The stress in the compression steel (f_{sc}) is a function of the strain (ε_{sc}).

if $|\varepsilon_{sc} \cdot E_s| \leq fy$

$$f_{sc} = \varepsilon_{sc} \cdot E_s$$

if $|\varepsilon_{sc} \cdot E_s| > fy$ or $\varepsilon_{sc} \cdot E_s < 0$

$$f_{sc} = -fy$$

otherwise

$$f_{sc} = fy$$

The force in the steel (F_{sc}) can be found by multiplying the stress in the steel (f_{sc}) and the area of the steel (A_{steel}).

$$F_{sc} = f_{sc} \cdot A_{steel}$$

The distance from the neutral axis to the compression steel ($Dist_{sc}$) is a function of the length of the column (b_{col}) and the location of the center of the compression steel from the compression face of the column (d_{prime}).

$$Dist_{sc} = \frac{b_{col}}{2} - d_{prime}$$

The moment in the compression steel ($Moment_{sc}$) is the product of the force in the steel (F_{sc}) and the distance from the neutral axis ($Dist_{sc}$).

$$Moment_{sc} = F_{sc} \cdot Dist_{sc}$$

Compression Concrete:

The compression force in the concrete (F_c) is a function of the alpha value, the width of the column (W_{col}) and the average strength of the concrete (fc_{Ave}).

$$F_c = alpha \cdot W_{col} \cdot fc_{Ave}$$

The alpha value is a function of the varying location of the neutral axis (c_{col}) and the beta1 value (β_1).

$$alpha = c_{col} \cdot \beta_1$$

The beta1 value is a function of the strength of the concrete (fc) where:

if fc ≤ 4000 psi

$$\beta_1 = 0.85$$

if 4000 psi < fc < 8000 psi

$$\beta_1 = \left[0.85 - \left(5 \cdot 10^{-5}\right) \cdot \left(fc - 4000\right)\right]$$

if fc ≥ 8000 psi

$$\beta_1 = 0.65$$

The average strength of the concrete (fc_{Ave}) can be calculated as follows.

$$fc_{Ave} = 0.85 \cdot fc$$

The distance from the neutral axis to the compression concrete ($Dist_{cc}$) is a function of the length of the column (b_{col}) and the alpha value.

$$Dist_{cc} = \frac{b_{col}}{2} - \frac{alpha}{2}$$

The moment in the compression concrete ($Moment_{cc}$) is the product of the force in the concrete (F_{cc}) and the distance from the neutral axis ($Dist_{cc}$).

$$Moment_{cc} = F_{cc} \cdot Dist_{cc}$$

Concrete Compression Force Correction:

Because the steel in the rebar replaces the area of the concrete used in the analysis for the compression concrete, a correction must be made to account for the extra capacity in the calculations. This concrete compression force correction reduces the moment and force in the capacity calculations by removing the concrete that is replaced by the steel in the analysis. The reduction in the force (F_{ccc}) is a function of the strength of the concrete (fc) and the area of the steel (A_{steel}).

$$F_{ccc} = -0.85 \cdot fc \cdot A_{steel}$$

The moment correction (M_{ccc}) can then be calculated by multiplying the reduction in force (F_{ccc}) by the distance of the compression steel from the neutral axis ($Dist_{sc}$).

$$Moment_{ccc} = F_{ccc} \cdot Dist_{sc}$$

Total and Useable Capacity:

We can superimpose the forces and moments such that the moment and axial capacity of the columns are the sum of each part of the column. This results in the axial capacity equation of:

$$Force_{col} = F_{st} + F_{sc} + F_{cc} + F_{ccc}$$

And a moment capacity equation of:

$$Moment_{col} = Moment_{st} + Moment_{sc} + Moment_{cc} + Moment_{ccc}$$

The factor of safety (Phi_{col}) is a function of the strain in the tension steel.

if $\varepsilon_{st} \leq -0.005$

$$Phi_{col} = 0.9$$

if $\varepsilon_{st} \geq -0.002$

$$Phi_{col} = 0.6$$

otherwise

$$Phi_{col} = \left[0.65 + (\varepsilon_{st} - 0.002) \cdot \left(\frac{0.25}{-0.005 - 0.002}\right)\right]$$

Thus the useable capacity before failure can be determined as:

$$Force_{useable} = Phi_{col} \cdot Force_{col}$$

$$Moment_{useable} = Phi_{col} \cdot Moment_{col}$$

References

Course notes from CEE 6730 Design of Concrete Structures. Lecturer Professor Kenneth Hover.