UNIVERSITY OF CALIFORNIA, SAN DIEGO

SE140B Professional Issues and Design of Civil Structures II

PROFESSOR

Alessandro Palermo

TEACHING ASSISTANTS

Erick Zavala

Rupesh Shrestha

Project 02: Cantilever Retaining Wall and Combined Spread Footing Design

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AUTHORS:

Liu, Chris

Kim, Isaac

Ahn, Hannah

Tran, Brian

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1. ANALYSIS AND DESIGN OF CANTILEVER RETAINING WALL

The objective of this project is to design two geotechnical-structural systems for the 8-story building being designed for SLOTH Architects. A cantilever reinforced concrete retaining wall and the connected combined spread footing between Columns F7 and F8. It was designed according to ACI 318-19 code and ASCE 7-22 load combinations.

1.1.1 Description, Materials and Preliminary Dimensions

This section introduces the essential materials and preliminary geometric parameters required for designing the cantilever retaining wall. It defines the physical and mechanical properties of concrete and soil, which serve as the foundation for all subsequent structural and geotechnical calculations. The compressive strength of concrete was calculated using k, the last digit of our student PID number. After obtaining the compression resistance, it was used in order to calculate the rest of the reinforced concrete material properties. Similarly, the unit weight of concrete was determined by adding k to 120 pcf and the internal friction angle was calculated by adding the minimum of the last three PID digits to 30 degrees.

Table 1.1 presents the material properties used in the analysis, including the compressive strength, Young's modulus, and unit weight of reinforced concrete. For the backfill soil, it includes the internal friction angle, unit weight, and allowable bearing stress. These parameters are fundamental for both stability analysis and structural design.

Table 1.1 Material Properties

Description	Variable	Value
Reinforced Concrete		
Compression Resistance	f_c	5.9 [ksi]
Young's Modulus	E_c	4378.3 [ksi]
Weight per unit volume	γ_c	0.15 [kip/ft^3]
Backfill		
Internal friction angle	$\emptyset$	32 [°]
Weight per unit volume	γ_s	0.129 [kip/ft^3]
Allowable soil stress	q_t	4 [kip/ft^2]

Table 1.2 defines the geometric configuration of the retaining wall, including the height and thickness of the stem, the thickness of the base slab, and the dimensions of the toe and heel. These dimensions form the basis for weight estimation, pressure calculations, and structural modeling.

Table 1.2 Dimensions of cantilever retaining wall

Description	Variable	Value
Stem (vertical wall that holds back the soil)	H	12 [ft]
Bottom wall thickness	G_b	12 [in]
Top wall thickness	G_t	12 [in]
Slab depth	F	2 [ft]
Toe (portion of slab that extends on the exposed side)	B	2 [ft]
Heel (portion of slab that extends under the retaining soil)	C	4.5 [ft]

1.1.2 Wall stability calculations

This section evaluates the global stability of the retaining wall under static loading. It includes assessments of sliding and overturning resistance, ensuring that the wall meets minimum safety factors according to design standards. The self weight of the slab was calculated by multiplying its total volume by the unit weight of the reinforced concrete. After finding the weight of the stem, the wall weight was calculated by summing the slab and stem weight. In order to calculate the backfill weight, the volume of the backfill zone was multiplied by the unit weight of the soil. Lastly, the surcharge load was found by multiplying the weight of the surcharge by the length of the retaining wall heel.

Table 1.3 calculates the self-weight of the wall components and the weight of the retained soil and surcharge. These loads are essential for determining base reactions and evaluating stability.

Table 1.3 Weight of the structure per unit length

Description	Variable	Value
Wall weight	W_{wall}	4.05 [kip]
Slab weight	W_{slab}	2.25 [kip]
Backfill weight	W_s	6.97 [kip]
Surcharge load	W_{sc}	1.62 [kip]

Both the active and passive lateral earth pressure coefficients are calculated using Rankine and Coulomb earth pressure theories, based on the soil's internal friction angle. After the foundation depths are determined, they are used along with the lateral earth pressure coefficient in order to calculate the Earth forces due to the various weights and loads.

Table 1.4 presents the earth pressure forces acting on the retaining wall, based on Coulomb theory. It includes active and passive pressure coefficients, force magnitudes, and the respective locations of their points of application. Surcharge effects on earth pressure are also considered.

Table 1.4 Earth pressure on retaining wall

Description	Variable	Value
Coulomb Active Pressure Coefficient	K_a	0.307
Active Earth Pressure Force	H_a	3.88 [kip]
Point of Application H_a relative to the bottom of the footing	h_a	4.67 [ft]
Active Earth Pressure Force from Surcharge (H_{sc}) on the backfill	H_{sc}	1.55 [kip]
Point of Application H_{sc} relative to the bottom of the footing	h_{sc}	7 [ft]
Coulomb Passive Pressure Coefficient	K_p	3.25
Passive Earth Pressure Force	H_p	-3.36 [kip]
Point of Application H_p relative to the bottom of the footing	h_p	1.33 [ft]

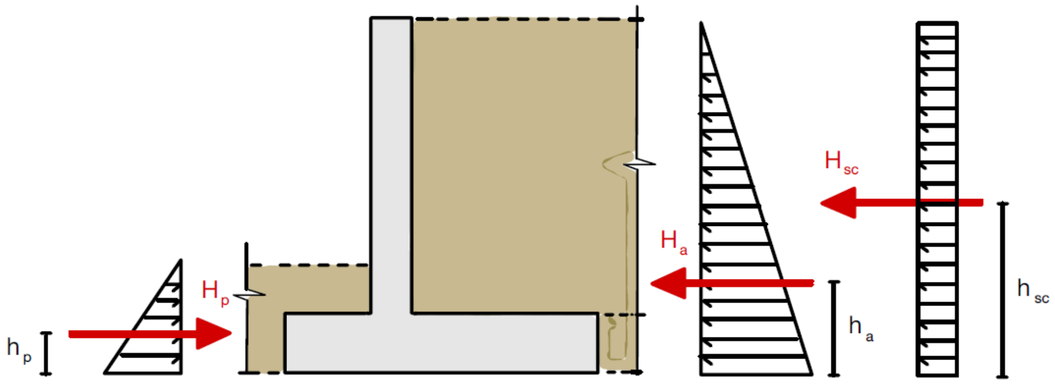


Figure 1.1 : Earth pressure system on retaining wall

The first step in order to calculate the sliding factor of safety is to determine the stabilizing sliding force by summing the vertical forces and multiplying the total by the sliding coefficient and then adding the passive earth pressure resistance. Similarly, the total unstabilizing force was found by summing the horizontal components of the lateral earth pressures. After obtaining both values the stabilizing force was divided by the unstabilizing force in order to verify that the factor of safety against sliding is greater than or equal to the minimum required design value, thereby verifying the structural stability of the retaining wall against lateral displacement. In order to determine the moment factor of safety, the sums of the stabilizing and overturning moments are totaled respectively. The stabilizing moments are generated by the vertical weights of the concrete wall components and the soil backfill acting about the toe of the footing, while the overturning moments are produced by the lateral forces of the active earth pressure and surcharge pushing against the back of the wall. Finally, dividing the total stabilizing moment by the total overturning moment, ensuring the ratio meets or exceeds the design threshold.

Table 1.5 summarizes the safety factors against sliding and overturning under static loading. These values confirm whether the wall maintains equilibrium and complies with code requirements.

Table 1.5 Stability Control of Cantilever Retaining Wall

Description	Variable	Value
Static Condition		
Safety Factor against Sliding (≥1.50)	SF_s	2.131
Safety Factor against Overturning (≥2.00)	SF_o	2.175

After calculating the lever arm arm of each component from the center of gravity as well as the moment of each component about the center of gravity, they are all summed together in order to obtain the stabilizing moment. Once the stabilizing moment is found, it is added along with the unstabilizing moment so that the resultant moment about the center of gravity can be determined. Then all of the vertical forces are combined to find the resultant force along with the area and moment of inertia of the footing. These values are then used to calculate the bearing stress on the left and right side of the footing in order to determine the minimum and maximum stresses. Finally, verify that the maximum demand pressure does not exceed the allowable limit.

Table 1.6 presents the maximum and minimum bearing pressures transmitted to the soil from the retaining wall. These values ensure that the bearing stress remains within the allowable soil capacity to prevent foundation failure.

Table 1.6 Stresses on the Bearing Soil

Description	Variable	Value
Maximum applied stress ($\sigma_{max} < q_t$)	σ_{max}	3.31 [kip/ft^2]
Minimum applied stress ($\sigma_{min} < q_t$)	σ_{min}	0.055 [kip/ft^2]

1.1.3 Structural Analysis

This section provides the internal forces (bending moments and shear forces) acting on the structural elements of the wall (stem, heel, and toe) resulting from applied loads and soil pressure. These values are critical for structural design. For the stem, the lateral load due to active pressure and surcharge are calculated, then used to find the moment and shear force. In order to find the moment and shear for the toe, the vertical load due to the weight at the toe and soil pressure are determined along with the pressure at the toe near the fixed end. These values are then used to calculate the moment and shear demand at the toe. Similarly for the heel, all the vertical loads and pressure at the heel are used to find the moment and shear demand.

Table 1.7 details the bending moments and shear forces acting on the stem, heel, and toe. These are derived from the combined effects of self-weight, backfill pressure, and surcharge loads, and are used in the design of flexural and shear reinforcement.

Table 1.7 External Loads on the Cantilever Retaining Wall

Description	Variable	Value
Stem		
Bending Moment	M_{stem}	31.01 [kip.ft]
Shear Force	V_{stem}	6.69 [kip]
Heel		
Bending Moment	M_{heel}	17.96 [kip.ft]
Shear Force	V_{heel}	8.45 [kip]
Toe		
Bending Moment	M_{toe}	-4.93 [kip.ft]
Shear Force	V_{toe}	-6.23 [kip]

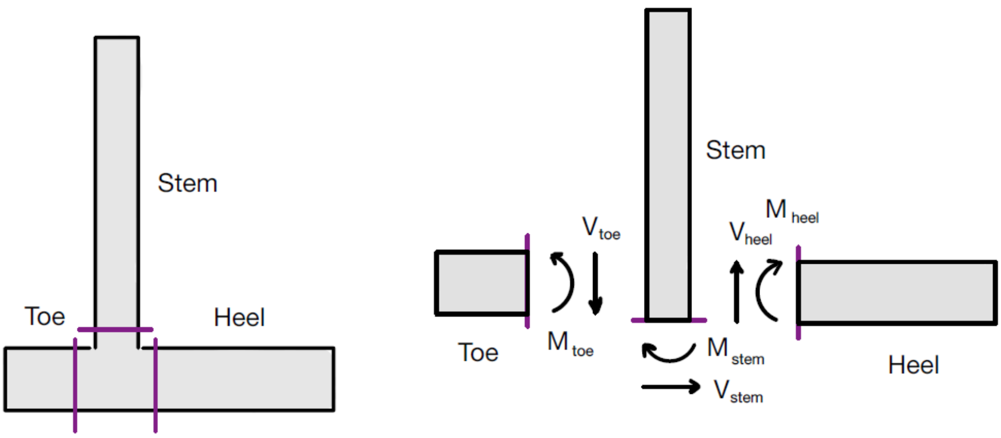


Figure 1.2 : Internal forces per element (toe, heel and stem)

1.1.4 Stem, Heel and Toe Flexure Design Calculations

Given the demand moment values calculated for the stem, heel, and toe, the next step is to begin flexural design. To begin, the assumed depth of each component was taken to be 8.5” for the stem and 20.5” for the toe and heel. This is to account for the 3” soil cover requirement and the 0.5” assumed radius of a #8 steel rebar. If a smaller value is used, the current depth values can still be used, since it would be more conservative to assume a smaller d value.

The design moment capacities for each component must exceed the respective moment demands. The stem had a moment demand of 31.01 k-ft, the heel with a moment demand of 17.96 k-ft, and the toe with a moment demand of -4.93 k-ft. For simplicity’s sake, the heel and toe flexural design were considered in tandem, taking the maximum between them into account—17.96 k-ft. This is to eliminate the need for splicing, which is undesirable and unnecessary for a slab that is just 7.5 ft wide.

Below, Table 1.8 details the final design moment capacity values calculated for the stem, heel, and toe, which safely exceed the demand moments. Note that the slab (heel and toe) bending moment capacity far exceeds the moment capacity of the stem due to a larger depth to steel value.

Table 1.8 Resistant Moment Capacity for the elements

Description	Variable	Value
Stem		
Bending Moment Capacity	ϕMr_{stem}	43.6 [kip.ft]
Heel		
Bending Moment Capacity	ϕMr_{heel}	110.17 [kip.ft]
Toe		
Bending Moment Capacity	ϕMr_{toe}	110.17 [kip.ft]

Beginning with the stem design, the initial philosophy was to use the smallest possible number and diameter of bars that would accommodate the flexural loading. Since the required steel area value was 0.853 in^2 for the stem, the idea was to use (2) #6 bars for every 12” width of the base, which amounts to #6 @ 6” O.C. Although this technically worked for the calculations, this caused the flexural DCR for the stem to be 0.97, which was a little risky. Our group decided to increase the bar size to #7 while maintaining the spacing, amounting to #6 @ 6” O.C. This lowered the DCR to 0.71 and satisfied the rest of the flexural design requirements.

Moving on to the slab (toe and heel) design, our group began with the same approach: finding the required steel area based on the demands and designing for that. However, the section required only 0.196 in^2 since the moment demand for the slab was lower— just 17.96 k-ft. To meet this area requirement, one could simply use a #4 bar. However, this failed the minimum area check. Since the area is larger, the minimum reinforcement requirement of 0.827 in^2 exceeded the flexural reinforcement requirement of 0.196 in^2. The 0.827 in^2 requirement could have been met with a #6 @ 6” O.C., but for the sake of using just one bar size, #7 @ 6” O.C. was eventually chosen. This conveniently met the minimum area requirement check as well as the rest of the moment design checks. This confirmed that combining the toe and heel design into a singular “slab” design was the right choice. Since the toe and heel have the same relevant depth, they have the same minimum area requirements, so #7 @ 6” conveniently meets the needs of both.

Table 1.9 below shows the longitudinal reinforcement for each element of the retaining wall. For simplicity, there is no difference in bar size and spacing between the reinforcement used in the slab. Only in the stem was the spacing changed. The interior tension side uses a spacing of 6” O.C. while the exterior side uses a spacing of 12” O.C.

Table 1.9 Longitudinal Reinforcement Steel Bars

Description	Variable	Value
Stem		
Exterior side reinforcement bars	le_{stem}	#7 @ 12” O.C.
Interior side (retaining soil) reinforcement bars	li_{stem}	#7 @ 6” O.C.
Heel		
Top reinforcement bars	lt_{heel}	#7 @ 6” O.C.
Bottom reinforcement bars	lb_{heel}	#7 @ 6” O.C.
Toe		
Top reinforcement bars	lt_{toe}	#7 @ 6” O.C.
Bottom reinforcement bars	lb_{toe}	#7 @ 6” O.C.

1.1.5 Stem, Heel and Toe Shear Design Calculations

This portion of the report address the shear design calculations for the retaining wall. As stated previously, the components considered were the stem and the slab (toe and heel). The shear demand of the stem was 6.69 kips, while the shear demand of the slab was taken to be the maximum of the toe and heel demand: 8.45 kips. After computing the relevant factors, the nominal concrete strength was computed to be 10.69 k-ft for the stem and 15.57 k-ft for the slab. The shear capacities safely exceed the shear demands by a wide margin, yielding DCR values of 0.63 and 0.54 respectively. As such, additional steel shear reinforcement was unnecessary. Table 1.10 below shows the shear capacities of the stem and slab.

Table 1.10 Shear Resistant for the elements

Description	Variable	Value
Stem		
Maximum Shear Capacity	ϕVr_{stem}	10.691[kip]
Slab		
Maximum Shear Capacity	ϕVr_{slab}	15.57 [kip]

1.1.6 Other Miscellaneous Calculations

In addition to regular moment and shear design, some additional calculations included development length calculations for the flexural reinforcement and shrinkage/temperature reinforcement design. Development length calculations were only required for the stem, since the longitudinal stem reinforcement is the only steel that needed to be properly embedded in another component. The development length for the stem was found to be 42” on both sides. However, since the thickness of the slab is just 24”, it was necessary to hook the #7 bars. The extension length of the bars was found to be 8.5”, which was rounded up to 9”. The hook development length was found to be 12”. The inner bar turn diameter of the hook was calculated to be 5.3”, which was rounded up to 5.5”. These values were then used to create the drawings. The hook embedment length exceeded 12”, going all the way to the bottom of the slab until it reached the 3” cover limit, turning at 5.5” diameter, then extending an additional 9”, in accordance with the extension length calculations. The reason for using hooked bars instead of headed bars is simple: low cost.

In addition, for each retaining wall component, the spacing values were checked to be within the minimum and maximum spacing requirements. See the spacing values for flexural design tabulated below in Table 1.12.

Table 1.12 Serviceability Criteria - Longitudinal Reinforcement Steel Bars

Description	Variable	Value
Stem		
User-defined spacing	S_{stem}	6 [in]
Maximum Spacing	Smx_{stem}	18 [in]
Minimum Spacing	Smn_{stem}	1 [in]
Development Length	L_p	42 [in]
Heel		
User-defined spacing	S_{heel}	6 [in]
Maximum Spacing	Smx_{heel}	18 [in]
Minimum Spacing	Smn_{heel}	1 [in]
Toe		
User-defined spacing	S_{toe}	6 [in]
Maximum Spacing	Smx_{toe}	18 [in]
Minimum Spacing	Smn_{toe}	1 [in]

An additional calculation performed was the shrinkage/temperature steel design. The maximum spacing value was calculated to be the minimum of 5*(Thickness of Component) and 18 inches. 18 inches governed for both the stem and the slab. However, the spacing of the stem shrinkage/temperature steel was decreased to 14” O.C., bringing the required steel area of the stem down to 0.565 in^2 from 0.726 in^2 and the required steel area of the slab down to 1.129 in^2 from 1.492 in^2. Since the required area of the stem is so much smaller, (2) #5 bars were used rather than (2) #7 bars. However, for the slab reinforcement, (2) #7 bars were used to meet the minimum area requirement.

Initially, the spacing was set to be much wider. But after actually spacing the bars in AutoCAD, the shrinkage bars were found to run through other longitudinal reinforcement, which doesn’t make sense. The spacing had to be adjusted to accommodate the realistic conditions of the section.

Table 1.13 below shows the design temperature/shrinkage steel and spacing values.

Table 1.13 Reinforcement steel bars placed perpendicular to the direction of the applied moment

Description	Variable	Value
Stem		
Exterior side reinforcement bars	pe_{stem}	#5 @ 14” O.C.
Interior side (retaining soil) reinforcement bars	pi_{stem}	#5 @ 14” O.C.
Heel		

Description	Variable	Value
Top reinforcement bars Bottom reinforcement bars	pe_{heel} pi_{heel}	#7 @ 14” O.C. #7 @ 14” O.C.
Toe		
Top reinforcement bars Bottom reinforcement bars	pe_{toe} pi_{toe}	#7 @ 14” O.C. #7 @ 14” O.C.

A stirrup was placed around the reinforcement that extends into the footing to control cracking. A single stirrup was placed as it sufficed for the required spacing of 18”, the max spacing, with the footing depth being 36”.

1.2 CONCLUSION AND RECOMMENDATIONS

The retaining wall design philosophy is safety and simplicity. When a DCR value was too close to 1, the reinforcement bar sizes were either increased or the spacing was decreased. When several bar options were available, the bar sizes were limited to just one or two options to make the reinforcement more constructible.

The retaining wall was designed first by performing wall stability calculations, changing up the retaining wall dimensions until the safety factor requirements were met. Then structural analysis calculations were performed to determine the flexural and shear demands at each component of the wall: stem, heel, and toe. As expected, the stem experienced the largest moment demand, while the slab (toe and heel) experienced the greatest shear demand. The reinforcement bars were designed accordingly until the final retaining wall sufficiently met the load demands, stability checks, and all other reinforcement requirements. The comprehensive design process demonstrates that simply designing for the loads is not enough. The best retaining wall design came about through a combination of changing retaining wall dimensions, adjusting the reinforcement bar size and spacing, and accounting for other relevant reinforcement checks. One surprising check that governed is the minimum reinforcement check for the slab flexural design. Given the large slab thickness of 2 ft, it makes sense that this check would govern, even when the flexural demands were relatively low. Thus, even though it seemed strange for the slab to have a larger moment capacity than the stem—which had the highest moment demand—it makes perfect sense given the minimum reinforcement requirement.

Although the flexural and shear design was relatively straightforward, things became more complicated once our group began to work three-dimensionally. Viewing the wall one section at a time was not sufficient because the spacing didn’t work out when the bars overlapped. In the future, there will be a lot more coordination and thought behind designing reinforcement with a 3D space in mind, rather than viewing things in just a 2D space. That way, the design can get done a lot more quickly and accurately. Another improvement that can be made is actually using the design checklist that was given to the class. Sometimes it was easy to lose track of what else had to be completed for the design. A final improvement that can be implemented in the future is working through the AutoCAD drawings in tandem with the reinforcement design, rather than completing the calculations and having to change up the spacing later anyways in AutoCAD. This would have simplified the process immensely. By thinking logistically about the spacing from the beginning, our group could have optimized the retaining wall design even further.

1.3 REFERENCES

ACI Committee 318. (2019). *Building code requirements for structural concrete (ACI 318-19) and commentary (ACI 318R-19)*. American Concrete Institute.

American Society of Civil Engineers. (2022). *Minimum design loads and associated criteria for buildings and other structures* (ASCE/SEI 7-22). https://doi.org/10.1061/9780784415788

2. ANALYSIS AND DESIGN OF COMBINED SPREAD FOOTING

Along with the cantilever retaining wall design, a combined footing needs to be designed to allow the column to transfer their load into. An existing service pipeline passes directly above Column F8, therefore an isolated spread footing at F8 is not possible, hence the combined footing with Column F7. The footing needs to be designed for flexural and shear demands, as described in the following sections.

2.1.1 Descriptions, Materials and Preliminary Dimensions

This section introduces the basic material properties and preliminary dimensions necessary for the design of the combined spread footing. It includes characteristics of reinforced concrete, soil, and surcharge loads, which serve as foundational inputs for structural analysis and design. The last digit of the compressive strength of the concrete and weight per unit volume of the backfill were assigned to the last digit of a group member's PID, which came out to be ***9***. The Young's modulus was calculated using the compressive strength. The allowable soil stress was given in the project statement.

Table 2.1 lists the physical and mechanical properties of materials used in the design. It includes the compressive strength, modulus of elasticity, and unit weight of reinforced concrete, as well as the surcharge load, unit weight of the soil, and the allowable bearing pressure for the footing foundation.

Table 2.1 Material Properties

Description	Variable	Value
Reinforced Concrete		
Compression Resistance	f_c	5.9 [ksi]
Young's Modulus	E_c	4.378 [ksi]
Weight per unit volume	γ_c	0.150 [kip/ft^3]
Backfill and Surcharge Load		
Surcharge over the footing	w_{sc}	0.379 [kip/ft^2]
Weight per unit volume	γ_s	0.129 [kip/ft^3]
Allowable soil stress	q_t	4 [kip/ft^2]

Table 2.2 defines the geometry of the footing and column layout. It includes the width, length, and thickness of the slab, the spacing between columns, and the distance from column centerlines to the footing edge critical for structural layout and loading analysis. The dimensions were preliminarily sized by assumption. The first iteration of the dimensions gave a LxWxH of 36x10x6 ft. After further calculations in the structural analysis section, the width had to be increased to 20 ft. With the initial dimensions, the eccentricity on the footing could be calculated. The loads applied in this section are unfactored to get a more accurate approximation of real-world soil conditions. Following the calculation of the eccentricity, the stresses at the base of the footing were calculated and checked against the allowable soil stress, q_t. Initially, the footing had a width of 10 ft, but this led to a maximum soil pressure greater than the allowable, therefore leading to an increase of the width to 20 ft. The minimum and maximum applied bearing pressures are found in Table 2.5.

Table 2.2 Dimensions of the combined spread footing

Description	Variable	Value

Column dimensions	L_o	30x30 [in]
Slab width	A	20 [ft]
Slab length	W	36 [ft]
Slab depth	F	3 [ft]
Distance from column centerline P_{8F} to footing edge	S_0	1.25 [ft]
Distance between columns P_{8F} and P_{7F}	S_1	27.75 [ft]
Distance from column centerline P_{7F} to footing edge	S_2	7 [ft]

2.1.2 Structural Analysis

Table 2.3 includes the dead load (DL), superimposed dead load (SDL), and live load (LL) applied to each column. These loads form the basis for calculating pressure distribution and internal stresses in the footing.

This section presents the external forces acting on the combined footing, including axial loads from the columns, bending moments, and shear forces. These are used to verify the footing's performance under service conditions. For this section, the loads were factored using the load combination $1.2*P_D + 1.6*P_L$. First, the ultimate eccentricity relative to the center of the footing was calculated. This was used to find the maximum and minimum gross soil pressure, which was used to calculate the maximum and minimum soil pressure. Using these net values, a net line load equation was applied to the footing. To find the shear equation, the integral of the net line load was taken with respect to *x* (position along the longitudinal axis of the footing). To find the moment equation, the integral of the shear equation was taken with respect to *x*. Each section of the footing had its own equations: the footing section left of Column F8, the footing section in between F8 and F7, and the footing section right of F7. From there, the critical sections for design were found, and the maximum ultimate positive and negative moments acting on the footing were found, as seen in Table 2.4, which outlines the maximum positive and negative bending moments and the shear forces along the footing's longitudinal axis. It also includes punching shear forces around each column, which are used to assess critical shear failure mechanisms.

To find the critical shear demand in one-direction acting on each column, the centerline of each column was first calculated. Then, the left face of each column was found using this centerline value. Next, the critical section was calculated at some distance away from the left face of each column. Using this critical section distance, it was plugged back into the shear force equation depending on what section it was in to find the shear demand.

To find the demand for two-way (punching) shear, the critical perimeter needed to be determined first. The process for calculating the critical sections comes from ACI 318-19 Section 22.6.4. Column F7 is considered an interior column whereas Column F8 is considered an edge column. For Column F7, its critical width and length were taken to be the column dimensions plus the effective depth of the footing, and are equal. For Column F8, the critical width was the same as F7, but its critical length was reduced by half of the effective depth. From here, the critical perimeters and areas were calculated using the critical dimensions. A net average pressure under both columns were estimated from $q_{net,max}$ and $q_{net,min}$ from earlier in the calculations. This is only an approximation. Finally, the punching shear was calculated to be the ultimate axial force on a column subtracted by the quantity of the net average pressure multiplied by the critical area.

Table 2.3 Axial Loads of columns P_{8F} and P_{7F}

Description	Variable	Value
Column P_{8F}		
Dead Load (DL) + Superimposed Dead Load (SDL)	P_{8FD}	385 [kip]
Live Load (LL)	P_{8FL}	96 [kip]
Column P_{7F}		
Dead Load (DL) + Superimposed Dead Load (SDL)	P_{7FD}	800 [kip]
Live Load (LL)	P_{7FL}	234 [kip]

Table 2.4 External Loads on the spread footing

Description	Variable	Value
Positive Moment		
Bending Moment (maximum)	M_{neg}	1195 [kip.ft]
Negative Moment		
Bending Moment (minimum)	M_{pos}	4017 [kip.ft]
Shear Force (perpendicular plane to longitudinal axis)		
Shear force evaluated at a distance F from the face of column P_{8F}	V_{8F}	518 [kip]
Shear force evaluated at a distance F from the face of column P_{7F}	V_{7F}	584 [kip]
Shear Force (Punching shear)		
Shear force evaluated at a distance F around column P_{8F}	V_{P8F}	579 [kip]
Shear force evaluated at a distance F around column P_{7F}	V_{P7F}	1237 [kip]

Table 2.5 Stresses on the Bearing Soil

Description	Variable	Valor
Maximum applied stress ($\sigma_{max} < q_t$)	σ_{max}	3.719 [kip/ft^2]
Minimum applied stress ($\sigma_{min} < q_t$)	σ_{min}	1.697[kip/ft^2]

2.1.3 Footing Flexure Design Calculations

This section focuses on flexural design by comparing the applied bending moments to the footing's moment capacity. Adequate longitudinal reinforcement is determined based on flexural demands. To find the required reinforcing bars needed to ensure the nominal moment was satisfactory, an iterative process was used to find the required area of steel. The negative moment was designed for first since the negative moment demands are higher than that of the positive. An initial A_s value was calculated, assuming an initial moment arm of 0.95 times the depth of the bars to estimate the steel area. From there, a value for the equivalent depth of the stress block was calculated, which was used to find the actual required A_s value. It was then compared with the minimum A_s value, which it was greater than. (40) #8 longitudinal bars were used, for the top reinforcement. Minimum steel area, spacing, and clear spacing checks were then performed to ensure the rebar would fit within the dimensions. For the bottom reinforcement, since the positive moment demand is less than the negative moment demand, the structural requirement was far lower than the minimum steel limit by inspection. Therefore, it was simply assumed that the reinforcement for the negative moment will be adequate for the positive moment, thereby also providing (40) #8 bars for the bottom reinforcement. Again, minimum steel area, spacing, and clear spacing checks were then performed to ensure the rebar would fit within the dimensions. Table 2.6 shows the footing's capacity to resist both positive and negative bending moments. These values are compared against the applied moments to ensure flexural adequacy.

A similar method to calculating the flexural demands and reinforcement was used to determine the bars needed and nominal moment for the transverse reinforcement. This time, an effective band width needed to be calculated. Using this band width, an upward pressure reacting to the factored column load distributed over the effective band area was calculated. This upward pressure was then converted to an upward line load, which was used to find the ultimate transverse mending moment at the face of each column. The method to find the required steel area was the same as the flexural reinforcement, as well as all the spacing and area limit checks. The transverse reinforcement ended up being the same for the area under both columns.

Table 2.6 Resistant Moment Capacity for the elements

Description	Variable	Value
Positive Moment		
Bending Moment Capacity	ϕMr_{pos}	4592 [kip.ft]
Negative Moment		
Bending Moment Capacity	ϕMr_{neg}	4592 [kip.ft]

The longitudinal reinforcement along the footing is summarized in Table 2.7. The bar size and spacing is uniform along the length of the whole footing and the top and bottom reinforcements are the same. The reinforcement on the corner close to P_{8F}, the middle of the footing, and the corner close to P_{7F} are all identical. But the spacing throughout the cross section is not uniform. The bars in the middle of the footing when viewing it from the transverse section are spaced at 6" c.c. while the bars on either end are spaced at 5.5" c.c. When spaced evenly, 40 bars do not fit inside the footing with a nice number of inches so this was designed so that the spacing of the bars would be easy to measure at the site. This arrangement of bars is the same for the top and bottom

Table 2.7 Longitudinal Reinforcement Steel Bar Spacing Through the Footing

Description	Variable	Value
Middle of the footing		
Top reinforcement bars	St_{md}	#8 @ 6" c.c.
Bottom reinforcement bars	Sb_{md}	#8 @ 6" c.c.
Right end of the footing		
Top reinforcement bars	St_{8F}	#8 @ 5.5" c.c.
Bottom reinforcement bars	Sb_{8F}	#8 @ 5.5" c.c.
Left end of the footing		
Top reinforcement bars	St_{7F}	#8 @ 5.5" c.c.
Bottom reinforcement bars	Sb_{7F}	#8 @ 5.5" c.c.

The bars were sliced to the left of column F7 when viewing from the Longitudinal section with the overlap of reinforcements occurring to the left of column F7. The lap slice was placed there for both the top and bottom reinforcements for ease of construction. The overlap is 4' 3" long to satisfy the required development length needed.

The ends of the reinforcement that are at either end of the footing have 12” hooks with a bend radius of 6” that follow the guidelines of Table 25.3.1 in ACI 18-19.

2.1.4 Footing Shear Design Calculations

This section addresses shear design, including one-way shear along the footing and two-way (punching) shear around the columns. It ensures that shear forces do not exceed concrete capacity or require excessive reinforcement.

To find one-way shear capacity, the steel ratio was first calculated, using the provided steel area from the longitudinal reinforcement. The size effect factor λ_s was permissible to be taken as 1.0 according to ACI 318-19 Section 13.2.6.2. The shear capacity was calculated using Equation 22.5.5.1 in ACI 318-19, and was checked against the maximum limit found in ACI 318-19 Section 22.5.5.1.1. From there, a strength reduction factor of 0.75 was applied to the shear capacity, and checked against the ultimate shear demand. The DCR for both columns was less than 1.0, satisfying the shear capacity.

To find two-way shear capacity, the first step was to find the α_s factor for each column, which is a constant used to calculate shear capacity according to ACI 318-19 Section 22.6.5.3. For an interior column (F7), that value was determined to be 40, and for an edge column (F8), that value was determined to be 30. Next, the aspect ratio of the column was to be calculated, coming out to 1.0 as the column dimensions are equal. According to ACI 318-19 Section 22.6.5.2, the two-way shear capacity is the least of three equations: the general formula, the aspect ratio formula, and the perimeter ratio formula. After performing all the calculations, it was found that the general formula provided the governing shear capacity. The capacity was then reduced by 0.75 and checked against the two-way shear demand. The DCR for both columns was less than 1.0, satisfying the shear capacity.

Table 2.9 presents the concrete’s resistance to shear forces at critical locations along and around the footing. It verifies the design against both one-way and punching shear failures.

Table 2.8 Shear Resistant for the elements

Description	Variable	Value
Shear Force (perpendicular plane to longitudinal axis)		
Shear force evaluated at a distance F from the face of column P_{8F}	ϕVr_{8F}	1325 [kip]
Shear force evaluated at a distance F from the face of column P_{7F}	ϕVr_{7F}	1325 [kip]
Shear Force (Punching shear)		
Shear resistance evaluated at a distance F around column P_{8F}	ϕVr_{P8F}	1186 [kip]
Shear resistance evaluated at a distance F around column P_{7F}	ϕVr_{P7F}	1916 [kip]

The number or bars and spacing of the shear reinforcement stirrups are detailed in Table 2.10. This is the spacing seen from the longitudinal section view. The bars were spaced at the maximum reinforcement spacing, 18”, throughout the entire length of the footing.

Table 2.9 Shear Reinforcement Steel Bars (perpendicular plane to longitudinal axis)

Description	Variable	Value
Middle of the footing (vertical reinforcement)		

Description	Variable	Value
Number of bars (Shear area) Rebar size schedule	N_{md} R_{md}	9 rows of 3 #4 bars #4 @ 18” c.c.
Corner side, close to P_{8F} (vertical reinforcement)		
Number of bars (Shear area) Rebar size schedule	N_{8F} R_{8F}	8 rows of 3 #4 bars #4 @ 18” c.c.
Corner side, close to P_{7F} (vertical reinforcement)		
Number of bars (Shear area) Rebar size schedule	N_{7F} R_{7F}	5 rows of 3 #4 bars #4 @ 18” c.c.

The rebar length of 40’ is not enough to wrap around the whole footing circumference of 46’ so the stirrups were split into 3 smaller stirrups that overlap to provide shear reinforcement and also internal reinforcement that prevents cracking. One stirrup wraps around the right edge, another wraps around the middle, and the another wraps around the left. The directions given in Table 2.10 are from the perspective of the transverse section view.

Table 2.10 Stirrup Location

Description	Variable	Value
Center of the footing		
Number of transverse bars wrapped along both the top and bottom	N_{Center}	17 bars @ 6” c.c
Right edge		
Number of transverse bars wrapped along both the top and bottom	N_{Right}	8 bars @ 5.5” c.c. & 9 bars @ 6” c.c
Left edge		
Number of transverse bars wrapped along the top and bottom	N_{Left}	8 bars @ 5.5” c.c. & 9 bars @ 6” c.c

2.1.5 Other Miscellaneous Calculations

This section includes serviceability checks such as bar spacing and development length to ensure proper steel anchorage and compliance with design codes. Table 2.11 ensures that the spacing of the longitudinal bars are within the allowable range of max and minimum spacing.

Table 2.11 Serviceability Criteria - Longitudinal Reinforcement Steel Bars

Description	Variable	Value
Middle of the footing (horizontal spacing)		
User-defined spacing Maximum Spacing Minimum Spacing	S_{md} Smx_{md} Smn_{md}	5.5 - 6.0 [in] 18.0 [in] 1.0 [in]
Corner side, close to P_{8F} (horizontal spacing)		
User-defined spacing Maximum Spacing	S_{8F} Smx_{8F}	5.5 - 6.0 [in] 18.0 [in]

Description	Variable	Value
Minimum Spacing	Smn_{8F}	1.0 [in]
Development Length	L_{p8F}	50.8 [in]
Corner side, close to P_{7F} (horizontal spacing)		
User-defined spacing	S_{7F}	5.5 - 6.0 [in]
Maximum Spacing	Smx_{7F}	18.0 [in]
Minimum Spacing	Smn_{7F}	1.0 [in]
Development Length	L_{p7F}	50.8 [in]

Skin reinforcement and internal reinforcement bars are detailed in Table 2.12 and were placed to control cracking and temperature effects and ensure structural integrity as the footing is very wide at 20'. The internal reinforcement supports the stirrups as well. Reinforcements are uniform across the length of the footing. There are four rows of internal rebar and they are symmetric across the y axis. The directions given in Table 2.12 are from the perspective of the transverse section view.

Table 2.12 Reinforcement steel bars placed perpendicular to the direction of the applied moment

Description	Variable	Value
Edge of the footing		
Number of bars & rebar size schedule on left edge	R_{left}	2 #4 bars @ 10" c.c.
Number of bars & rebar size schedule on right edge	R_{right}	2 #4 bars @ 10" c.c.
Internal rebar first row from the right and left		
Number of bars & rebar size schedule	R_1	2 #4 @ 10" c.c.
Distance from edge rebars	d_1	5' 8.5"
Internal rebar second row from the right and left		
Number of bars & rebar size schedule	R_2	2 #4 @ 10" c.c.
Distance from edge rebars	d_2	7' 8.5"

2.2 CONCLUSION AND RECOMMENDATIONS

Provides final thoughts based on the analysis and offers suggestions for design or future improvements.

The design of footings is essentially an amalgamation of all that we have learned over the past few years. It incorporates aspects of design from the beginning of college, such as statics and structural analysis, to concepts learned near the end of college, such as concrete and steel design. This also made the calculations quite lengthy, with many variables and design checks to consider and manage. The process was procedural, following step-by-step methods. There were some unexpected challenges that I faced while performing the calculations. The first was that the preliminary width was not sufficient, and caused the maximum soil bearing pressure to be greater than the allowable soil bearing pressure. However, it was a simple fix, and by increasing the width by a factor of 2, I was able to get the DCR below 1.0. Another challenge was getting accurate and reasonable values for the maximum positive moment acting on the bottom steel. By contacting the TAs for help, I was able to resolve this issue. As verified by the example design, the negative moment governed for the longitudinal design, having a much higher moment demand than the positive moment. After finishing the calculations, our DCRs were quite low for some of the design checks. In the real world, this would be considered very conservative and safe, but for the sake of design, we wanted to not be as conservative. By decreasing the thickness of the slab, the DCR became less conservative. This also changed the loads acting on the footing. Initially, the design stated that the top of the footing would be flush with the soil grade, leaving no soil weight acting on top of the footing. By reducing the thickness of the footing, this caused soil to suddenly act on the footing. This change attributed around 300 kips of extra force acting on the footing, which, surprisingly, did not change much in our design.

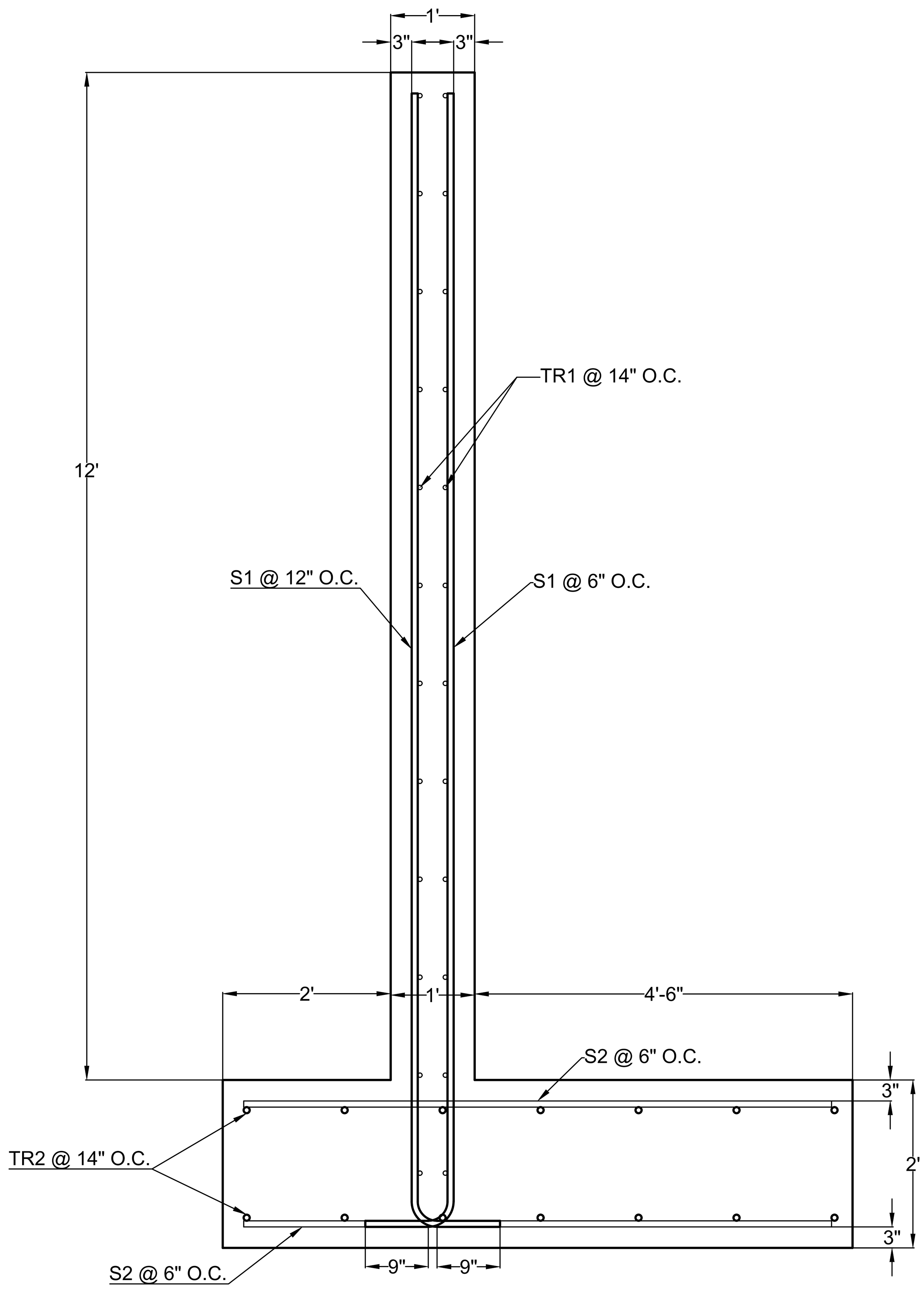
One of the more complicated aspects of the design was detailing the reinforcement. There were longitudinal reinforcements for the moment, stirrups to control cracking caused by shear, and skin and internal reinforcement in the footing. Designing all the reinforcements in the footing was complicated as there were small details to consider for all the types of reinforcement and it was difficult to know what checks and details had to be considered. The best approach to solving this issue was to envision how the reinforcements would be constructed while keeping in mind the material constraints like the limit to the rebar length being 40' and the cracking that would occur with the footing being so wide. In the future, further consideration as to how the footing would perform in terms of cracking due to the depth of the footing being only 3' would be helpful.

2.3 REFERENCES

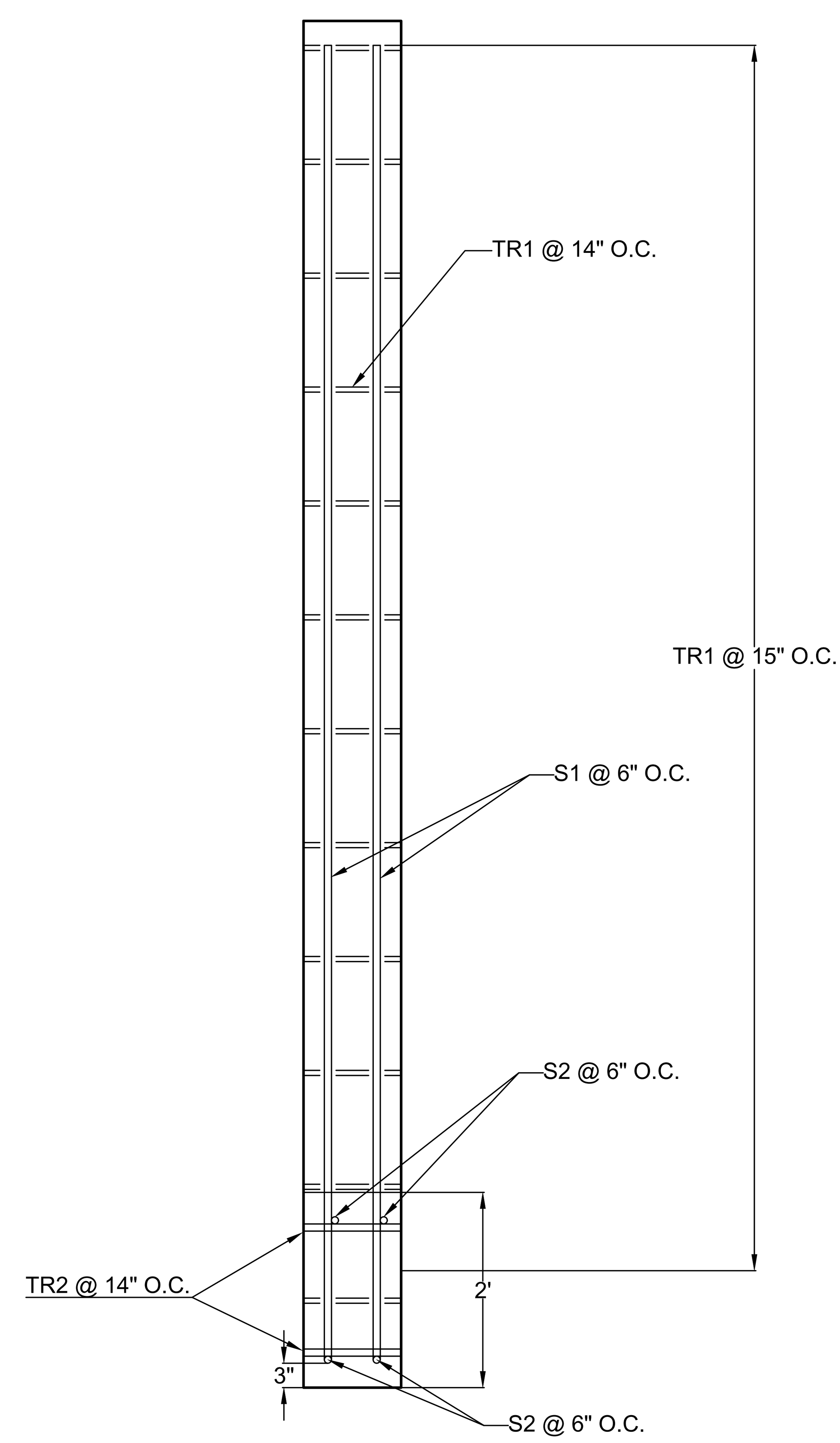
ACI Committee 318. (2019). *Building code requirements for structural concrete (ACI 318-19) and commentary (ACI 318R-19)*. American Concrete Institute.

American Society of Civil Engineers. (2022). *Minimum design loads and associated criteria for buildings and other structures* (ASCE/SEI 7-22). https://doi.org/10.1061/9780784415788

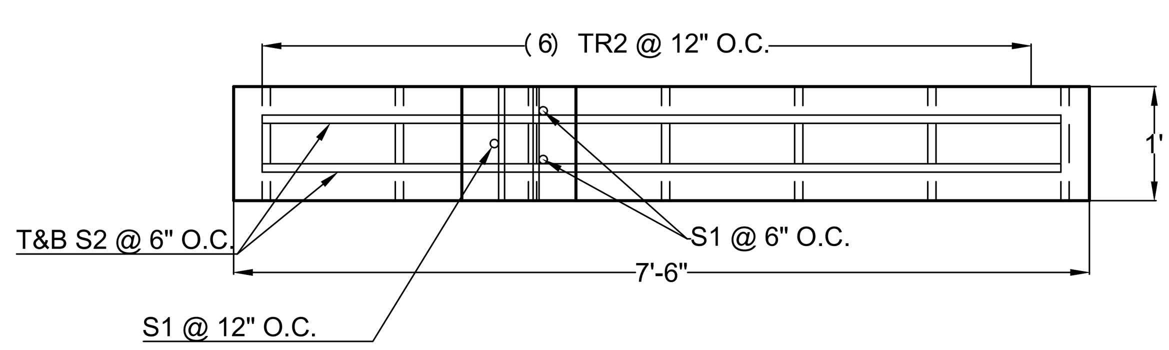
APPENDIX A – Technical drawings (A1 – retaining wall; A2 - footing)



RETAINING WALL LONGITUDINAL SECTION



RETAINING WALL TRANSVERSE SECTION



RETAINING WALL PLAN VIEW

BAR SCHEDULE

ID	BAR #	LENGTH	SPECIFICATIONS
S1	#7	14'-4"	13'-2.375" straight section, followed by 2.75" radius, 90° hook with 9" tail
S2	#7	7'-0"	
TR1	#5	12'-0"	
TR2	#7	12'-0"	

General Notes

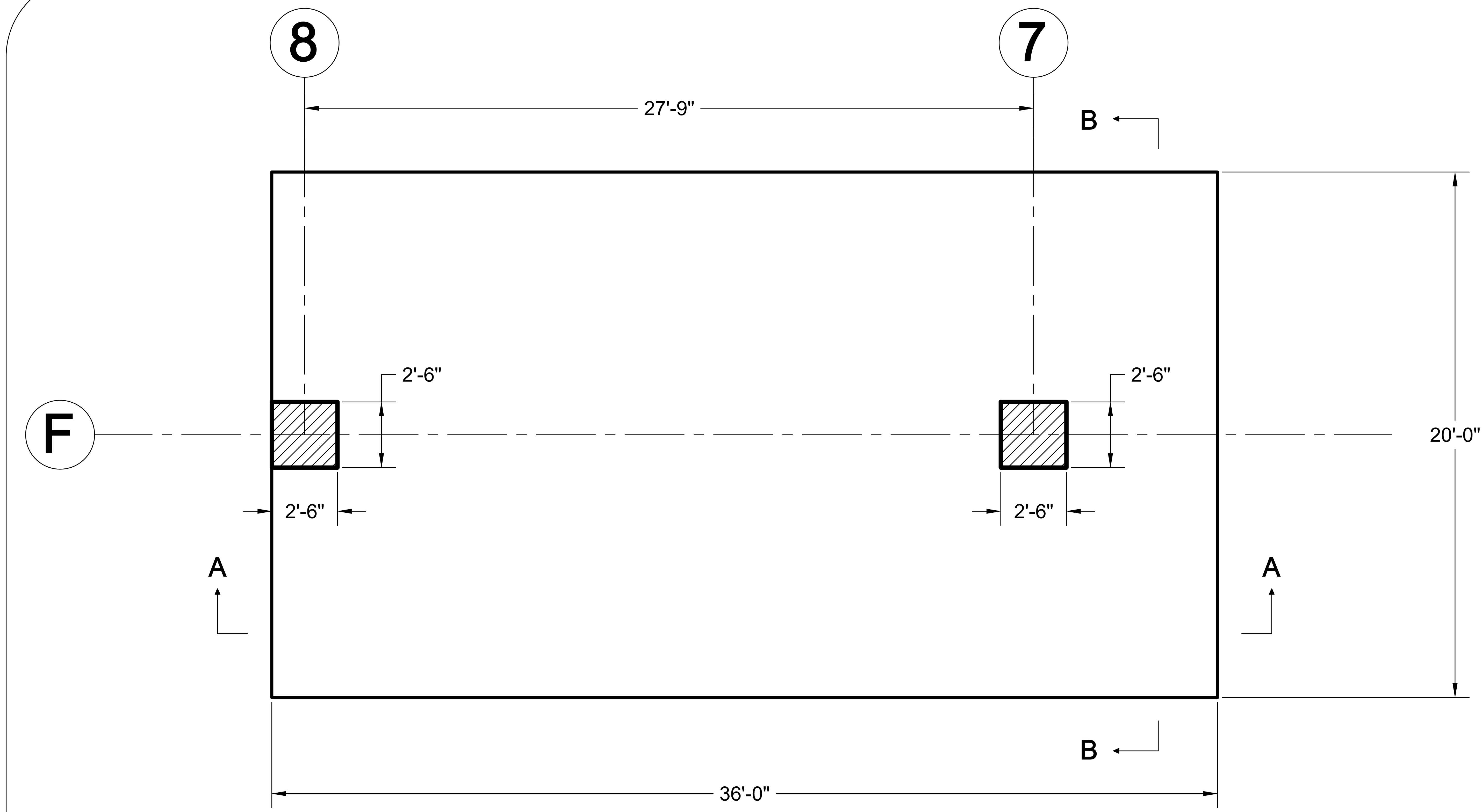
RETAINING WALL DESIGN

No.	Revision/Issue	Date

Firm Name and Address

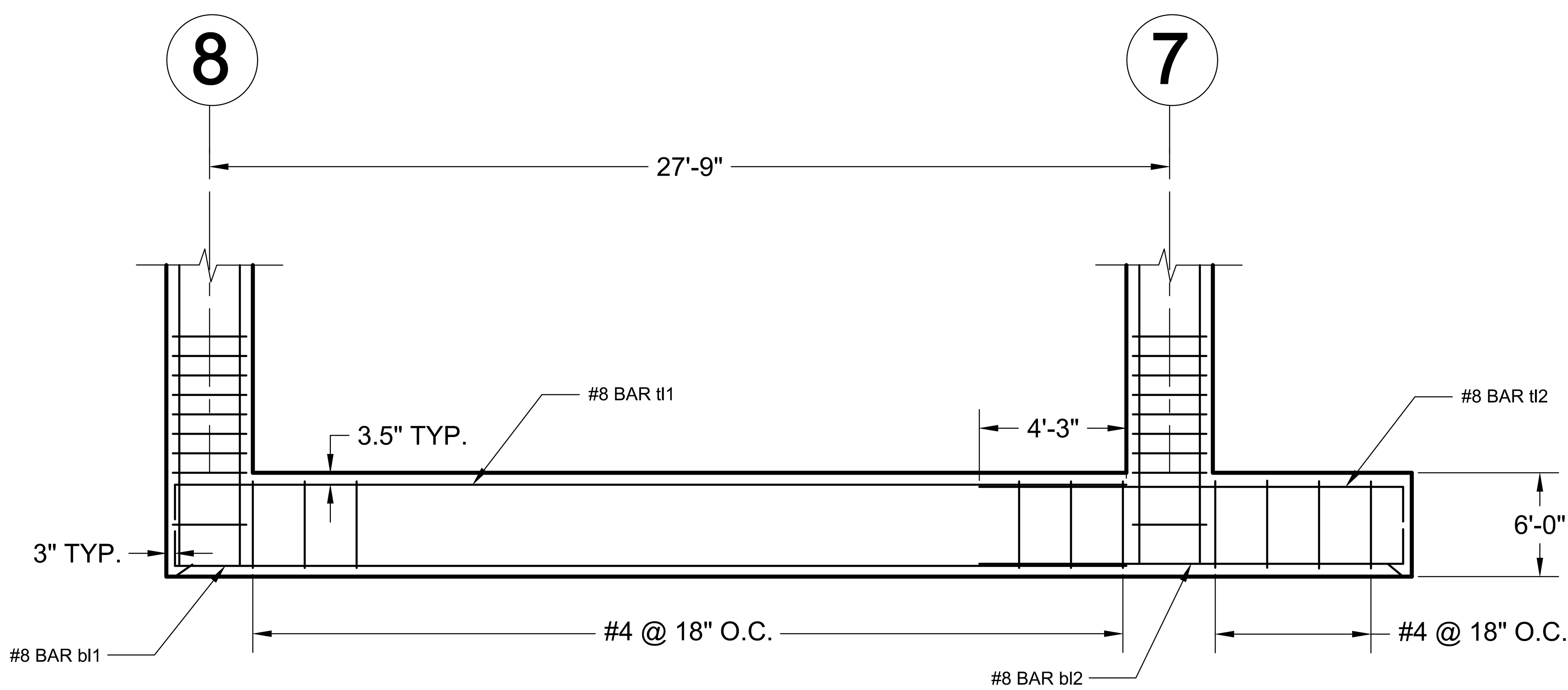
Project Name and Address

Project: SE 140B Project 2
Date: 5/19/2026
Scale: 3/4" = 1'-0"
Sheet: S-01



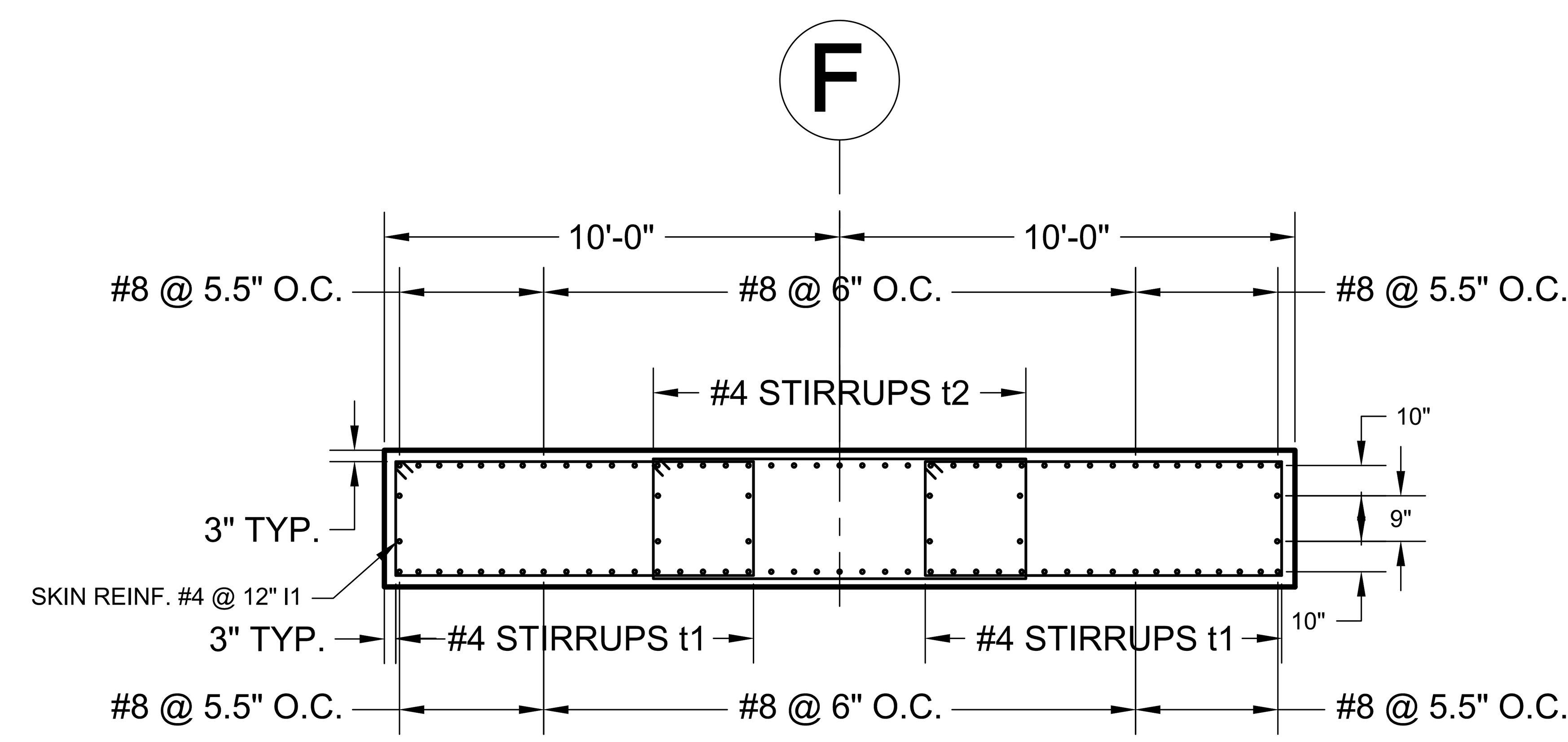
1 FOOTING PLAN VIEW
S-02 1/3" = 1'-0"

BAR SCHEDULE					
LONGITUDINAL BARS					
ID	BAR #	NO. BARS	LENGTH	NOTES	SCHEDULE
bl1	#8	82	27'-6" (12" HOOK)	90° HOOK	
bl2	#8	82	13'-5" (12" HOOK)	90° HOOK	
tl1	#8	82	27'-6" (12" HOOK)	90° HOOK	
tl2	#8	82	13'-5" (12" HOOK)	90° HOOK	
l1	#8	12	35'-6" (24" HOOK)	90° HOOK	
TRANSVERSE BARS					
ID	BAR #	NO. BARS	LENGTH	NOTES	SCHEDULE
t1	#4	42	21'-10"		
t2	#4	21	22'-5"		



NOTE: STIRRUPS, TRANSVERSE BARS, AND SKIN REINFORCEMENT BARS WERE NOT FULLY DRAWN IN 2/S-02 FOR THE SAKE OF A SIMPLER AND CLEANER DRAWING ACCORDING TO PROFESSOR PALERMO'S RECOMMENDATIONS.

2 LONGITUDINAL SECTION A-A
S-02 1/3" = 1'-0"



3 TRANSVERSE SECTION B-B
S-02 1/3" = 1'-0"

General Notes

COMBINED FOOTING F7-8

No.	Revision/Issue	Date

Firm Name and Address

Project Name and Address

Project SE 140B Project 2
Date 5/11/2026
Scale As Noted
Sheet S-02

APPENDIX B – Step-by-Step calculations (B1 – retaining wall; B2 -footing)

1. (B1) RETAINING WALL CALCULATIONS: EXCEL

[SE 140B] Project 2				
Codes: ACI 318-19, ASCE 7-22				
Problem: Retaining Wall Design				
1. Material Properties				
f'c	Specified compressive strength of concrete	5.9	*ksi*	
β₁	Stress block factor: = 0.85 - 0.05 * (f'c - 4000) / (1000 psi)	0.755	*—*	
fy	Reinforcement yield strength (ASTM Grade 60)	60	*ksi*	
fr	Rupture modulus = 7.5 * sqrt(f'c)	0.576	*ksi*	
λ	Concrete density factor (normal-weight = 1.0)	1	*—*	
E_c	Young's Modulus of Concrete	4378.25	*ksi*	
E_s	Young's Modulus of Steel	29000	*ksi*	Concrete
γ_c	Unit Weight of Concrete	0.15	*kcf*	
γ_s	Unit Weight of Soil	0.129	*kcf*	
q_all	Allowable Soil Stress	4	*ksf*	
Φ_f	Internal Friction Angle	32	*degrees*	Soil Backfill
2. Geometry				
i	Hannah PID "i" value	6		
j	Hannah PID "j" value	2		
k	Hannah PID "k" value	9		
w_trib	12 in	12	*in*	
H	12 ft	12	*ft*	
G	1 ft	1	*ft*	
B	Initial B = 2 ft (Iterate as Needed)	2	*ft*	
C	Initial C = 0.5 * H = 6 ft (Iterate as Needed)	4.5	*ft*	
F	Initial F = 3 ft (Iterate as Needed)	2	*ft*	
D	2 ft	2	*ft*	
c-c	3 in	3	*in*	
3. Gravity Loads				
3.1. Self-Weight of Retaining Wall				
W_stem	w_trib * H * g * yc	1.8	*kips*	
W_slab	F*(B+G+C)	2.25	*kips*	
W_wall	W_wall = W_stem + W_slab	4.05	*kips*	
l_stem	COGx of stem = B + G/2	2.5	*ft*	
h_stem	COGy of stem = F + H/2	8	*ft*	
l_slab	COGx of slab = 0.5 * (B+G+C)	3.75	*ft*	
h_slab	COGy of slab = F/2	1	*ft*	
l_wall	(W_stem*l_stem + W_slab * l_slab) / (W_stem + W_slab)	3.19	*ft*	
h_wall	(W_stem*h_stem + W_slab * h_slab) / (W_stem + W_slab)	4.11	*ft*	
3.2. Weight of Backfill				
W_backfill	C*w_trib * H * ys	6.966	*kips*	
l_backfill	B+G+ C/2	5.25	*ft*	
h_backfill	H/2 + F	8	*ft*	
3.3. Weight of Surcharge				
sigma_sc	300 psf + i * 10psf	360	*psf*	
W_sc	C * w_trib * sigma_sc	1.62	*kips*	
l_sc	B + G + 0.5 * C	5.25	*ft*	
3.4. Weight of Soil at Toe				
W_toe	B * w_trib * D * ys	0.516	*kips*	
l_toe	B/2	1	*ft*	
h_toe	D/2 + F	3	*ft*	
4. Earth Pressures over Retaining Wall				
4.1. Active Pressure				
Ka	(1 - sin(theta)) / (1 + sin(theta))	0.307		
h_ta	H + F	14	*ft*	
Ha	0.5 * Ka * ys * h_ta^2 * w_trib	3.88	*kips*	
h_a	h_ta / 3	4.67	*ft*	
H_sc	Ka * sigma_sc * h_ta * w_trib	1.55	*kips*	
h_sc	h_ta / 2	7.00	*ft*	
4.2. Passive Pressure				
Kp	(1 + sin(theta)) / (1 - sin(theta))	3.25		
h_tp	D + F	4	*ft*	
Hp	- 0 .5 * Kp * ys * h_tp ^2 * w_trib	-3.36	*kips*	
h_p	h_tp / 3	1.33	*ft*	

5. All Participating Moments			
M_stem	W_stem * l_stem	4.5	k-ft
M_slab	W_slab * l_slab	8.4375	k-ft
M_wall	M_stem + M_slab	12.9375	k-ft
M_backfill	W_backfill * l_backfill	36.5715	k-ft
M_toe	W_toe * l_toe	0.52	k-ft
M_sc_vertical	W_sc * l_sc	8.5	k-ft
M_a	Ha * h_a	18.1	k-ft
M_p	Hp * h_p	-4.5	k-ft
M_sc_lateral	H_sc * h_sc	10.840	k-ft
6. Stability Forces			
u	Sliding Coefficient = tan(theta)	0.625	
F_s	Stabilizing Sliding Force = (W_wall + W_backfill + W_sc + W_toe)*u + abs(H_p)	11.577	kips
F_us	M_stem + M_slab	5.43	kips
M_stabilizing	W_backfill * l_backfill	63.01	k-ft
M_overturning	W_toe * l_toe	28.97	k-ft
7. Stability Control			
FS_sliding	Fs / Fus	**2.131**	
FS_overturning	M_stabilizing / M_overturning	**2.175**	
1. Sliding Check	**DCR: FS_sliding > 1.5**	**Y**	
2. Overturning Check	**DCR: FS_overturning > 2.0**	**Y**	
8. Demand on Allowable Soil Stresses			
8.1 Resultant Moment of Retaining Wall w.r.t Point "c" (COGx of Retaining Wall)			
l'_stem	0.5 * (B + G+ C) - l_stem	1.25	ft
M'_stem	W_stem * l'_stem	2.25	k-ft
l'_slab	0.5 * (B + G + C) - l_slab	0	ft
M'_slab	W_slab * l'_slab	0.00	k-ft
M'_wall	M'_stem + M'_slab	2.25	k-ft
8.2. Backfill Moment Demand ab COG			
l'_backfill	0.5 * (B + G + C) - l_backfill	-1.500	kips
M'_backfill	W_backfill * l'_backfill	-10.449	k-ft
8.3. Soil at Toe Moment Demand ab COG			
l'_toe	0.5 * (B + G + C) - l_toe	3	ft
M'_toe	W_toe * l'_toe	1.42	k-ft
8.4. Moment of Surcharge ab COG			
l'_sc	0.5 * (B + G + C) - l_sc	-1.500	ft
M'_sc	W_sc * l'_sc	-2.430	k-ft
8.5 Earth Passive Pressure			
M'p	M'p = Mp	-4.478	k-ft
8.6 Bearing Stress Calcs			
M'_rs	M'_wall + M'_backfill + M'_toe + M'_sc + M'_p	-13.69	k-ft
M'_urs	M'_urs = M_overturning	28.97	k-ft
M_c	M'_rs + M'_urs	15.28	k-ft
R_v	W_wall + W_backfill + W_sc	12.64	kips
A	(B + G + C)*w_trib	7.50	ft^2
I_footing	(w_trib) * (B + G + C)^3 /12	35.16	ft^4
q_max	R_v / A + Mc * (l_slab) / I_footing	3.31	ksf
q_min	R_Vv / A - Mc * (l_slab) / I_footing	0.055	ksf
Bearing Stress Check: q_max	**DCR: q_max / q_all < 1.0**	**0.83**	
Bearing Stress Check: q_min	**DCR: q_min / q_all < 1.0**	**0.01**	
9. Structural Analysis			
9.1. Stem			
P_a	Ka * ys * H * w_trib	0.476	klf
P_sc	Ka * sigma_sc * w_trib	0.111	klf
Mu_stem	1.6 * (Pa/2 * H * H/3) + 1.6 * (P_sc * H * H/2)	31.01	k-ft
Vu_stem	1.6 * (Pa * H/2) + 1.6 * (P_sc * H)	6.690	kips
9.2. Toe			
w_toe	ys * D * w_trib	0.258	klf
q_toe	(q_max - q_min) * B / (B + G + C)	0.87	ksf
w_q_toe	q_toe * w_trib	0.869	klf
Mu_toe	0.9 * (w_toe * B) * (B / 2) - 1.6 * (q_toe * w_trib * B) * (B / 2) - 1.6 * (q_max - q_toe) * w_trib * (B / 2) * (B / 3)	-4.93	k-ft
Vu_toe	0.9 * (w_toe * B) - 1.6 * (q_toe * B * w_trib) - 1.6 * (q_max - q_toe) * w_trib * (B / 2)	-6.230	kips
9.3. Heel			
w_s	yc * F * w_trib	0.300	klf
w_b	ys * H * w_trib	1.548	klf
w_sc	sigma_sc * w_trib	0.360	klf
q_heel	(q_max - q_min) * (G + B) / (G + B + C)	1.304	ksf
w_q_heel	q_heel * w_trib	1.304	klf
Mu_heel	1.2 * (w_s + w_b) * C * (C / 2) - 0.5 * q_min * w_trib * C * (C / 2) - 0.5 * (q_heel - q_min) * w_trib * (C / 2) * (2 * C / 3)	17.960	k-ft
Vu_heel	1.2 * (w_s + w_b) * C - q_min * C * w_trib - 0.5 * w_trib * (q_heel - q_min) * C	8.450	kips

10. Flexural Design

10.1 Moment Design

		Stem	Slab: Toe and Heel	
d_stem	G - 3 - 0.5 (conservative for smaller bars)	8.5		*in*
d_slab	F - 3 - 0.5 (conservative for smaller bars)		20.5	*in*
Bar Size		#7	#7	
d_b	Bar Diameter	0.875	0.875	*in*
s_bar	Bar Spacing	6	6	*in*
a_it	Iterative value of compression block "a"	0.851	0.195	*in*
As_it	Iterative value of Steel Area = Mu / (0.9 * fy * (d - a_it /2))	0.853	0.196	*in^2*
a	True Depth of Whitney's Compression Block = As_it * fy / (0.85 * f'c * b)	0.851	0.195	*in*
Equilibrium	Equilibrium: a_it - a = 0; Iterate until this is true.	0.00	0.00	
Bar Selection	**#7 BARS @ 6" O.C. for Tension, #7 BARS @ 12" O.C. for Compression (min. requirements)**	#7	#7	
As	Steel area per tributary width (12"): (2) #7 Bars @ 6"	1.2	1.2	*in^2*
p_s	Reinforcement Ratio = As / (d*b)	0.0118	0.0049	
ϕMn	**Nominal Moment Capacity = 0.9 * As*fy*(d-a/2)**	43.60	110.17	*k-ft*

10.2 Moment Design Checks

		Stem	Slab: Toe and Heel	
1. Flexural Check	**DCR: Mu / ϕMn < 1?**	**0.71**	**0.16**	
Mu	Mu_stem = Mu_stem; Mu_slab = max(Mu_toe, Mu_heel)	31.01	17.96	*k-ft*
ϕMn	Nominal Moment Capacity	43.60	110.17	*k-ft*
2. Minimum Area Check	**DCR: As_min / As < 1?**	**0.29**	**0.69**	
p_min	Minimum Reinforcement Ratio = max(0.35 * fr / fy, 0.0018)	0.0034	0.0034	
As_min	Minimum Steel Area = p_min * b * d	0.343	0.827	*in^2*
As_tension	Design Area of Tension Steel	1.20	1.20	*in^2*
As_compression	Design Area of Compression Steel (minimum used):	0.60	1.20	*in^2*
3. Max Area Check	**DCR: As / As_max < 1?**	**0.31**	**0.13**	
p_max	Max Reinforcement Ratio = 0.85 * B1 * f'c / fy * 0.003 / (0.005)	0.038	0.038	
As_max	Maximum Steel Area = p_max * b * d	3.862	9.314	*in^2*
As	Area of Steel	1.2	1.2	*in^2*
4. Min Bar Spacing Check	**DCR: s_min / s < 1**	**0.17**	**0.17**	
s_bar	Set Spacing of Bars	6.0	6.0	*in*
s_min	max(1 in, d_b, 4/3*ϕag)	1	1	*in*
5. Max Bar Spacing Check	**DCR: s_min / s < 1**	**0.33**	**0.33**	
s_bar	Set Spacing of Bars	6.0	6.0	*in*
s_max	s_max_stem = min(2*F, 18 in); s_max_slab = min(2*G, 18 in)	18	18	*in*

10.3 Development Length Calculations for Stem

		Stem: Tension	Stem: Compression	
d_b_stem		0.875	0.875	*in*
ψ_t	Location Factor = 1.0	1.0	1.0	
ψ_e	Uncoated Reinforcement Factor = 1.0	1.0	1.0	
ψ_s	No.7 and Larger Bars = 1.0	1.0	1.0	
lambda	NWC = 1.0	1.0	1.0	
ψ_r	No.11 and Smaller Bars Factor = 1.0	1.0	1.0	
ψ_o	Side Cover Normal to Plane of Hook >= 2.5 in Factor = 1.0	1.0	1.0	
ψ_c	Concrete Strength Factor = f'c (psi) / 15000 + 0.6	0.993	0.993	
l_development	roundup((3 * f_y * ψ_t * ψ_e * ψ_s * d_b_stem) / (50 * lambda * sqrt(f_c)))	42.0	42.0	*in*
l_extension	MAX(d_stem, d_b_stem)	9.0	9.0	*in*
l_dh	MAX((f_y * ψ_e * ψ_r * ψ_o * ψ_c * d_b_stem^1.5) / (55 * lambda * SQRT(f_c)), 8 * d_b_stem, 6)	12.0	12.0	*in*
diam	Inner bar turn diameter = 6*d_b	5.3	5.3	*in*
design diam	Round up to reasonable value	5.5	5.5	*in*

11. Shear Design

11.1 Shear Design

		Stem	Slab	
Vu	Vu_stem = Vu_stem; Vu_slab = max(Vu_toe, Vu_heel)	6.69	8.45	*kips*
λs	Size Effect Modification Factor = min(sqrt(2 / (1 +d/10)), 1)	1	0.8098	
f_lms	Limited Material Strength = min(sqrt(f'c) , 100 psi)	76.81	76.81	*psi*
ϕ_s	Shear Strength Reduction Factor	0.75	0.75	
ϕV_nc	Nominal Concrete Strength = min (ϕs * (2*λs*b*d*f_lms), ϕs * (8*λs*ps^1/3 *b*d*f_lms))	10.691	15.570	*kips*

11.2 Shear Design Check

1. Shear Capacity Check	**DCR: Vu / ϕVn < 1?**	**0.63**	**0.54**	
Vu	Maximum Shear Demand	6.69	8.45	*kips*
ϕVn	Nominal Shear Capacity	10.69	15.57	*kips*

12. Shrinkage/Temperature Steel Design

		Stem	Slab	
p_min	max(0.35*fr/fy, 0.0018)	0.0034	0.0034	*kips*
T	Thickness = (Stem: G), (Slab: F)	12.0	24.0	*in*
s_max_shrinktemp	min(5*Thickness, 18 in)	18.0	18.0	*in*
s_shrink_temp	Design Spacing Value	14.0	14.0	*in*
As_min_shrinktemp	Required Shrinkage/Temperature Area = p_min * s_shrink_temp * T	0.565	1.129	*in^2*
Bar Size	(2) BARS per given spacing	#5	#7	
As_shrinktemp		0.62	1.2	*in^2*
Minimum Area Check	**DCR: As_min / As < 1?**	**0.91**	**0.94**	

2. (B1) RETAINING WALL CALCULATIONS: STEP-BY-STEP EXAMPLES

[SE 140B] Project 2

PID Value: 629
i = 6
j = 2
k = 9

1. Material Properties
Concrete:

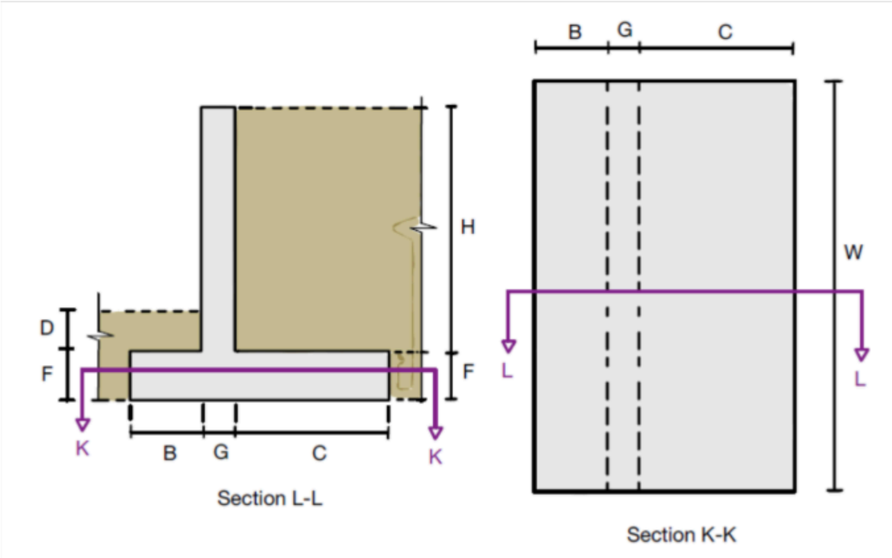
$f'_c = 5000 \text{ psi} + k * 100 \text{ psi} = 5900 \text{ psi}$	(Concrete compressive strength)
$\beta = 0.85 - \dfrac{0.05 * \left(f'_c - 4000 \text{ psi}\right)}{1000 \text{ psi}} = 0.755$	(Beta value for Concrete)
$f_y = 60000 \text{ psi}$	(Steel Yield Strength)
$f_r = 7.5 * \sqrt{f'_c} * 1 \text{ psi}^{0.5} = 576.086 \text{ psi}$	(Rupture Strength)
$E_c = 57000 * \sqrt{f'_c} * 1 \text{ psi}^{0.5} = 4378253.076 \text{ psi}$	(Young's Modulus of Concrete)
$E_s = 29000 \text{ ksi}$	(Young's Modulus of Steel)

Backfill:

$\gamma_c = 150 \text{ pcf}$	(Unit Weight of Concrete)
$\gamma_s = 120 \text{ pcf} + k * 1 \text{ pcf} = 129 \text{ pcf}$	(Unit Weight of Soil)
$q_{all} = 4000 \text{ psf}$	(Allowable Soil Stress)
$\phi = 30 \text{ deg} + \text{Min}(i, j, k) * 1 \text{ deg} = 32 \text{ deg}$	(Internal Friction Angle)

2. Geometry of the Structure

$w_{trib} = 1 \text{ ft}$		(Tributary Width)
H = 12 ft		(Height of Stem)
G = 12 in		(Thickness of Stem)
B = 2 ft	[initial size = 0.5 * H = 6 ft]	(Toe Length)
C = 4.5 ft	[initial size = 0.5 * H = 6 ft]	(Heel Length)
F = 2 ft	[initial size = 3 ft]	(Slab Depth)
D = 2 ft		(Interior Slab Elevation)



3. Gravity Loads

Self-Weight of Retaining Wall

$W_{stem} = w_{trib} * H * G * \gamma_c = 1.8 \text{ kip}$ (Self-Weight of Stem)

$W_{slab} = F * (B + G + C) * w_{trib} * \gamma_c = 2.25 \text{ kip}$ (Self-Weight of Slab: Toe + Heel)

$W_{wall} = W_{stem} + W_{slab} = 4.05 \text{ kip}$ (Self-Weight of Retaining Wall)

$l_{stem} = \left(B + \dfrac{G}{2}\right) = 2\text{ft } 6\text{in}$ (COGx of Stem)

$h_{stem} = F + \dfrac{H}{2} = 8 \text{ ft}$ (COGy of Stem)

$l_{slab} = 0.5 * (B + G + C) = 3\text{ft } 9\text{in}$ (COGx of Slab)

$h_{slab} = \dfrac{F}{2} = 1 \text{ ft}$ (COGy of Slab)

$l_{wall} = \dfrac{W_{stem} * l_{stem} + W_{slab} * l_{slab}}{W_{stem} + W_{slab}} = 3.194 \text{ ft}$ (Total COGx of Retaining Wall)

$h_{wall} = \dfrac{W_{stem} * h_{stem} + W_{slab} * h_{slab}}{W_{stem} + W_{slab}} = 4.111 \text{ ft}$ (Total COGy of Retaining Wall)

Weight of Backfill

$H = 12 \text{ ft}$ (Height of Backfill)

$W_{backfill} = C * w_{trib} * H * \gamma_s = 6.966 \text{ kip}$ (Weight of Backfill)

$l_{backfill} = B + G + \dfrac{C}{2} = 5\text{ft } 3\text{in}$ (COGx of Backfill)

$h_{backfill} = \dfrac{H}{2} + F = 8 \text{ ft}$ (COGy of Backfill)

Weight of Surcharge:

$\sigma_{sc} = 300\ psf + i * 10\ psf = 360\ psf$ (Surcharge Pressure)

$W_{sc} = C * w_{trib} * \sigma_{sc} = 1.62\ kip$ (Surcharge Weight on Heel)

$l_{sc} = B + G + 0.5 * C = 5ft\ 3in$ (COGx of Surcharge)

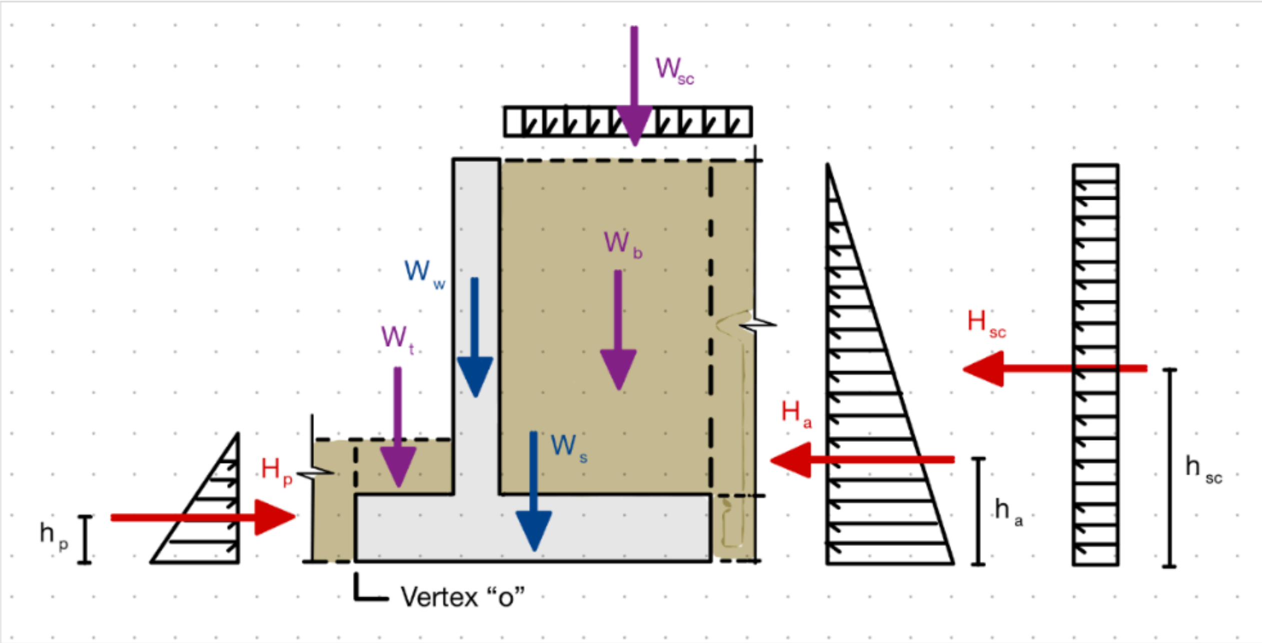
Weight of Soil at Toe:

$D = 2\ ft$ (Height of Soil at Toe)

$W_{toe} = B * w_{trib} * D * \gamma_s = 0.516\ kip$ (Weight of Soil at Toe)

$l_{toe} = \frac{B}{2} = 1\ ft$ (COGx of Soil at Toe)

$h_{toe} = \frac{D}{2} + F = 3\ ft$ (COGy of Soil at Toe)



4. Earth Pressures over Retaining Wall

Active Pressure:

$K_a = \frac{1 - \sin(\phi)}{1 + \sin(\phi)} = 0.307$ (Coulomb Active Pressure Coefficient)

$h_{ta} = H + F = 14\ ft$ (Foundation Depth of Active Soil)

$H_a = 0.5 * K_a * \gamma_s * h_{ta}^2 * w_{trib} = 3.884\ kip$ (Earth Force due to Active Backfill)

$h_a = \frac{h_{ta}}{3} = 4.667\ ft$ (Vertical Distance to w.r.t Point O)

$H_{sc} = K_a * \sigma_{sc} * h_{ta} * w_{trib} = 1.549\ kip$ (Earth Force due to Surcharge)

$$h_{sc} = \frac{h_{ta}}{2} = 7\ ft$$ (Vertical Distance w.r.t Point O)

Passive Pressure:

$$K_p = \frac{1 + \sin(\phi)}{1 - \sin(\phi)} = 3.255$$ (Coulomb Active Pressure Coefficient)

$h_{tp} = D + F = 4\ ft$ (Foundation Depth of Passive Soil)

$H_p = -0.5 * K_p * \gamma_s * h_{tp}^2 * w_{trib} = -3.359\ kip$ (Earth Force due to Passive Backfill)

$$h_p = \frac{h_{tp}}{3} = 1.333\ ft$$ (Vertical Distance w.r.t Point O)

5. ALL Participating Moments

$M_{stem} = W_{stem} * l_{stem} = 4.5\ kip * ft$ (Moment of Stem)

$M_{slab} = W_{slab} * l_{slab} = 8.438\ kip * ft$ (Moment of Slab: Toe + Heel)

$M_{wall} = M_{stem} + M_{slab} = 12.938\ kip * ft$ (Total Moment of Retaining Wall)

$M_{backfill} = W_{backfill} * l_{backfill} = 36.572\ kip * ft$ (Moment of Backfill)

$M_{toe} = W_{toe} * l_{toe} = 0.516\ kip * ft$ (Moment of Toe)

$M_{sc,vertical} = W_{sc} * l_{sc} = 8.505\ kip * ft$ (Moment of Vertical Surcharge)

$M_a = H_a * h_a = 18.127\ kip * ft$ (Moment of Earth Active Pressure)

$M_p = H_p * h_p = -4.478\ kip * ft$ (Moment of Earth Passive Pressure)

$M_{sc,lateral} = H_{sc} * h_{sc} = 10.840\ kip * ft$ (Moment of Lateral Surcharge)

External Force	Stabilizing Moment	Overturning Moment
Retaining Wall	12.94 kip*ft	
Backfill	36.57 kip*ft	
Soil at Toe	0.52 kip*ft	
Vertical Surcharge	8.51 kip*ft	
Earth Passive Pressure	18.13 kip*ft	
Earth Active Pressure		-4.48 kip*ft
Earth Pressure Surcharge		10.84 kip*ft

6. Stability Forces

$\mu = \tan(\phi) = 0.625$ (Sliding Coefficient btwn Concrete and Soil)

$F_s = \left(W_{wall} + W_{backfill} + W_{sc} + W_{toe}\right) * \mu + \left|H_p\right| = 11.577\ kip$ (Stabilizing Sliding Force)

$F_{us} = H_a + H_{sc} = 5.433\ kip$ (Unstabilizing Sliding Force)

$M_{stabilizing} = M_{wall} + M_{backfill} + M_{toe} + M_{sc,vertical} + \left|M_p\right| = 63.008\ kip * ft$ (Stabilizing Moment)

$M_{overturning} = M_a + M_{sc,lateral} = 28.967\ kip * ft$ (Overturning Moment)

7. Stability Control

$FS_{sliding} = \dfrac{F_s}{F_{us}} = 2.13$ ≥ 1.5: G (Sliding Factor of Safety)

$FS_{overturning} = \dfrac{M_{stabilizing}}{M_{overturning}} = 2.18$ ≥ 2.0: G (Moment Factor of Safety)

8. Demand on Allowable Soil Stresses:

Resultant Moment of Retaining Wall w.r.t Point "c" (COGx of retaining wall)

$l'_{stem} = 0.5 * (B + G + C) - l_{stem} = 1ft\ 3in$ (Lever Arm of Stem from COG)

$M'_{stem} = W_{stem} * l'_{stem} = 2.25\ kip * ft$ (Moment of Stem ab COG)

$l'_{slab} = 0.5 * (B + G + C) - l_{slab} = 0ft\ 0in$ (Lever Arm of Slab from COG)

$M'_{slab} = W_{slab} * l'_{slab} = 0\ kip * ft$ (Moment of Slab ab COG)

$M'_{wall} = M'_{stem} + M'_{slab} = 2.25\ kip * ft$ (Resultant Moment of Retaining Wall ab COG)

Backfill:

$l'_{backfill} = 0.5 * (B + G + C) - l_{backfill} = -1ft\ 6in$ (Level Arm of Backfill ab COG)

$M'_{backfill} = W_{backfill} * l'_{backfill} = -10.449\ kip * ft$ (Moment of Backfill ab COG)

Soil at Toe:

$l'_{toe} = 0.5 * (B + G + C) - l_{toe} = 2ft\ 9in$ (Level Arm of Soil at Toe ab COG)

$M'_{toe} = W_{toe} * l'_{toe} = 1.419\ kip * ft$ (Moment of Soil at Toe ab COG)

Surcharge:

$l'_{sc} = 0.5 * (B + G + C) - l_{sc} = -1ft\ 6in$ (Level Arm of Surcharge ab COG)

$M'_{sc} = W_{sc} * l'_{sc} = -2.43\ kip * ft$ (Moment of Surcharge ab COG

Earth Passive Pressure:

$M'_p = M_p = -4.478\ kip * ft$ (Moment of Earth Passive Pressure ab COG)

$M'_{rs} = M'_{wall} + M'_{backfill} + M'_{toe} + M'_{sc} + M'_p = -13.688\ kip * ft$ (Stabilizing Moment)

$M'_{urs} = M_{overturning} = 28.967\ kip * ft$ (Unstabilizing Moment)

$M_c = M'_{rs} + M'_{urs} = 15.279\ kip * ft$ (Resultant Moment ab COG)

$R_v = W_{wall} + W_{backfill} + W_{sc} = 12.636\ kip$ (Vertical Resultant Force)

$A = (B + G + C) * w_{trib} = 7.5\ ft^2$ (Area of Footing)

$$I_{footing} = \frac{\left(w_{trib}\right) * (B + G + C)^3}{12} = 35.156\ ft^4$$ (Inertia of Footing)

DOUBLE CHECK THE BEARING STRESS VALUES

$$q_{max} = \frac{R_v}{A} + \frac{M_c * \left(l_{slab}\right)}{I_{footing}} = 3.315\ \frac{kip}{ft^2}$$ (Bearing Stress on the left side of footing; Min?)

$$q_{min} = \frac{R_v}{A} - \frac{M_c * \left(l_{slab}\right)}{I_{footing}} = 0.0551\ \frac{kip}{ft^2}$$ (Bearing Stress on the right side of footing; Max?)

$q_{max} < q_{all}$ = True **G**

$q_{min} < q_{all}$ = True **G**

9. Structural Analysis

Stem: Load Combo

$P_a = K_a * \gamma_s * H * w_{trib} = 475.636\ plf$ (Lateral Load due to Active Pressure)

$P_{sc} = K_a * \sigma_{sc} * w_{trib} = 110.613\ plf$ (Lateral Load due to Surcharge)

$$M_{u,stem} = 1.6 * \left(\left(\frac{P_a}{2}\right) * (H) * \left(\frac{H}{3}\right)\right) + 1.6 * \left(P_{sc} * H * \left(\frac{H}{2}\right)\right) = 31.007\ kip * ft$$ (Stem Moment)

$$V_{u,stem} = 1.6 * \left(\frac{P_a * H}{2}\right) + 1.6 * \left(P_{sc} * H\right) = 6.690\ kip$$ (Stem Shear)

Toe: Load Combo

$w_{toe} = \gamma_s * D * w_{trib} = 258\ plf$ (Vertical Load due to Weight at Toe)

$$q_{toe} = \frac{\left(q_{max} - q_{min}\right) * B}{B + G + C} = 869.193\ psf$$ (Pressure at Toe near Fixed End)

$w_{q,toe} = q_{toe} * w_{trib} = 869.193\ plf$ (Vertical Load due to Soil Pressure)

$$M_{u,toe} = 0.9 * \left(w_{toe} * B\right) * \left(\frac{B}{2}\right) - 1.6 * \left(q_{toe} * w_{trib} * B\right) * \left(\frac{B}{2}\right) - 1.6 * \left(q_{max} - q_{toe}\right) * w_{trib} * \left(\frac{B}{2}\right) * \left(\frac{B}{3}\right)$$

$= -4.925\ kip * ft$ (Moment Demand at Toe)

$$V_{u,toe} = 0.9 * \left(w_{toe} * B\right) - 1.6 * \left(q_{toe} * B * w_{trib}\right) - 1.6 * \left(q_{max} - q_{toe}\right) * w_{trib} * \left(\frac{B}{2}\right) = -6.230\ kip$$

(Shear Demand at Toe)

Heel: Load Combo

$w_s = \gamma_c * F * w_{trib} = 300$ plf (Vertical Load due to Self-Weight)

$w_b = \gamma_s * H * w_{trib} = 1548$ plf (Vertical Load due to Backfill)

$w_{sc} = \sigma_{sc} * w_{trib} = 360$ plf (Vertical Load due to Surcharge)

$$q_{heel} = \frac{\left(q_{max} - q_{min}\right) * (G + B)}{G + B + C} = 1303.790 \text{ psf}$$ (Pressure at Heel near Fixed End)

$w_{q,heel} = q_{heel} * w_{trib} = 1303.790$ plf (Vertical Load due to Soil Pressure)

$$M_{u,heel} = 1.2 * \left(w_s + w_b\right) * (C) * \left(\frac{C}{2}\right) - 0.5 * \left(q_{min} * w_{trib} * C * \left(\frac{C}{2}\right)\right) - 0.5 * \left(q_{heel} - q_{min}\right) * w_{trib} * \left(\frac{C}{2}\right) * \left(\frac{2 * C}{3}\right)$$

$= 17.960$ kip * ft (Moment Demand at Heel)

$$V_{u,heel} = 1.2 * \left(w_s + w_b\right) * (C) - \frac{q_{min} * C * w_{trib}}{2} - \frac{0.5 * w_{trib} * \left(q_{heel} - q_{min}\right) * C}{2} = 8.450 \text{ kip}$$

(Shear Demand at Heel)

10. Flexural Design

Stem Design

$d_{stem} = G - 3$ in $- 0.5$ in $= 8.5$ in (Depth of Stem Reinforcement)

$a_{it,stem} = 0.851$ in (Iterative Compression Block Depth)

$$A_{s,it,stem} = \frac{M_{u,stem}}{0.9 * f_y * \left(d_{stem} - \frac{a_{it,stem}}{2}\right)} = 0.853 \text{ in}^2$$ (Iterative Steel Area)

$$a_{stem} = \frac{A_{s,it,stem} * f_y}{0.85 * f'_c * w_{trib}} = 0.851 \text{ in}$$ (Compression Block Depth)

$A_{s,stem,tension} = 2 * 0.6 \text{ in}^2 = 1.2 \text{ in}^2$ **(#7 Tension Bars @ 6" O.C.)**

$A_{s,stem,compression} = 0.6 \text{ in}^2$ **(Stem Area: #7 Rebar @ 12" O.C.)**

$$\phi M_{n,stem} = 0.9 * A_{s,stem,tension} * f_y * \left(d_{stem} - \frac{a_{stem}}{2}\right) = 43.603 \text{ kip} * \text{ft}$$ (Nominal Moment)

$$\rho_{s,stem} = \frac{A_{s,stem,tension}}{w_{trib} * d_{stem}} = 0.0118$$ (Stem Reinforcement Ratio)

1. Flexural DCR:

$M_{u,stem} = 31.007$ kip * ft (Demand Moment)

$\phi M_{n,stem} = 43.603$ kip * ft (Nominal Moment)

$$DCR_{stem,flexure} = \frac{M_{u,stem}}{\phi M_{n,stem}} = 0.711 \; \text{G}$$ (Flexural DCR)

2. Min Area Check:

$$\rho_{min,stem} = Max\left(\frac{0.35 * f_r}{f_y}, 0.0018\right) = 0.00336$$ (Min Reinforcement Ratio)

$$A_{s,min,stem} = \rho_{min,stem} * w_{trib} * d_{stem} = 0.343 \text{ in}^2$$ (Min Reinforcement Area)

$$A_{s,stem,compression} = 0.6 \text{ in}^2$$ **(Stem Area: #7 Rebar @ 12" O.C.)**

$$A_{s,stem,tension} = 1.2 \text{ in}^2$$ **(Stem Area: #7 Rebar @ 6" O.C.)**

$$DCR_{stem,min,area} = \frac{A_{s,min,stem}}{A_{s,stem,tension}} = 0.286 \; \text{G}$$ (Min Area DCR)

$$DCR_{stem,min,area,compression} = \frac{A_{s,min,stem}}{A_{s,stem,compression}} = 0.571 \; \text{G}$$ (Min Area DCR for Comp. Bars)

3. Max Area Check

$$\rho_{max,stem} = \frac{0.85 * \left(\beta * f'_c\right) * (0.003)}{f_y * \left(0.003 + \frac{f_y}{E_s}\right)} = 0.0373$$ (Max Reinforcement Ratio)

$$A_{s,max,stem} = \rho_{max,stem} * G * d_{stem} = 3.810 \text{ in}^2$$ (Max Reinforcement Area)

$$DCR_{stem,max,area} = \frac{A_{s,stem,tension}}{A_{s,max,stem}} = 0.315 \; \text{G}$$ (Max Area DCR)

4. Min Bar Spacing

$$s_{b,min,stem} = Max(1 \text{ in}, 1 \text{ in}, 0.5 \text{ in}) = 1 \text{ in}$$ (Min Bar Spacing)

$$s_{stem,tension} = 6 \text{ in}$$ (Tension Bar Spacing)

$$s_{stem,compression} = 12 \text{ in}$$ (Compression Bar Spacing)

$$DCR_{stem,min,spacing,t} = \frac{s_{b,min,stem}}{s_{stem,tension}} = 0.167 \; \text{G}$$ (Min Bar Spacing DCR)

$$DCR_{stem,min,spacing,c} = \frac{s_{b,min,stem}}{s_{stem,compression}} = 0.0833 \; \text{G}$$ (Min Bar Spacing DCR)

5. Max Bar Spacing

$s_{b,max,stem} = \mathrm{Min}(5 * G, 18\ in) = 18\ in$ (Max Bar Spacing)

$s_{stem,tension} = 6\ in$ (Tension Bar Spacing)

$s_{stem,compression} = 12\ in$ (Compression Bar Spacing)

$$DCR_{stem,max,spacing,t} = \frac{s_{stem,tension}}{s_{b,max,stem}} = 0.333\ \text{G}$$ (Max Bar Spacing DCR)

$$DCR_{stem,max,spacing,c} = \frac{s_{stem,compression}}{s_{b,max,stem}} = 0.667\ \text{G}$$ (Max Bar Spacing DCR)

Stem Development Length Calculation Example: Tension Reinforcement

$d_{b,stem,tension} = 7/8\ in$ **(#7 bars @ 6" O.C. used along span)**

$\psi_{t,tension} = 1$ (location factor)

$\psi_{e,tension} = 1$ (uncoated reinforcement)

$\psi_{s,tension} = 1$ (1.0 for No.7 and larger bars)

$lambda_{tension} = 1$ (NWC)

$l_{e,stem,tension} = \mathrm{Max}\left(d_{stem}, d_{b,stem,tension}\right) = 8.5\ in$ **(extension length, measured from M(x) = 0, use 9")**

$$l_{d,stem,tension} = \left(\frac{3 * \left(\frac{f_y}{1\ psi}\right) * \psi_{t,tension} * \psi_{e,tension} * \psi_{s,tension} * d_{b,stem,tension}}{50 * lambda_{tension} * \sqrt{\frac{f'_c}{1\ psi}}}\right) = 41.010\ in$$

(development length: use 42")

$\psi_r = 1$ (No. 11 and smaller bars)

$\psi_o = 1$ (side cover normal to plane of hook ≥ 2.5 in)

$$\psi_c = \frac{f'_c}{15000\ psi} + 0.6 = 0.993$$ (concrete strength)

$$l_{dh,stem,tension} = \mathrm{RoundUp}\left(\mathrm{Max}\left(\frac{f_y * \psi_{e,tension} * \psi_r * \psi_o * \psi_c * d_{b,stem,tension}^{1.5}}{\left(55 * lambda_{tension} * \sqrt{\frac{f'_c}{1\ psi}}\right) * 1\ in^{.5} * 1\ psi}, 8 * d_{b,stem,tension}, 6\ in\right)\right)$$

$= 12\ in$

$diam_{stem,tension} = 6 * d_{b,stem,tension} = 5\ 1/4\ in$ **(Use 5.5 in bend diameter for #7 bars)**

Toe Design

$F = 2\ ft$ (Depth of Slab)

$d_{toe} = F - 3\ in - .5\ in = 20.5\ in$ (Depth of Toe Reinforcement)

$a_{it,toe} = 0.0533\ in$ (Iterative Compression Block Depth)

$$A_{s,it,toe} = \frac{-M_{u,toe}}{0.9 * f_y * \left(d_{toe} - \frac{a_{it,toe}}{2}\right)} = 0.0535\ in^2$$ (Iterative Steel Area)

$$a_{toe} = \frac{A_{s,it,toe} * f_y}{0.85 * f'_c * w_{trib}} = 0.0533\ in$$ (Compression Block Depth)

$A_{s,toe} = 0.6 * 2\ in^2 = 1.2\ in^2$ **(Steel Area: #7 Bars @ 6"O.C.)**

$$\phi M_{n,toe} = 0.9 * A_{s,toe} * f_y * \left(d_{toe} - \frac{a_{toe}}{2}\right) = 110.556\ kip * ft$$ (Nominal Moment)

$$\rho_{s,toe} = \frac{A_{s,toe}}{w_{trib} * d_{toe}} = 0.00488$$ (Toe Steel Reinforcement Area)

1. Flexural DCR:

$M_{u,toe} = -4.925\ kip * ft$ (Demand Moment)

$\phi M_{n,toe} = 110.556\ kip * ft$ (Nominal Moment)

$$DCR_{toe,flexure} = \frac{-M_{u,toe}}{\phi M_{n,toe}} = 0.0446$$ G (Flexural DCR)

2. Min Area Check:

$$\rho_{min,toe} = Max\left(\frac{0.35 * f_r}{f_y}, 0.0018\right) = 0.00336$$ (Min Reinforcement Ratio)

$A_{s,min,toe} = \rho_{min,toe} * w_{trib} * d_{toe} = 0.827\ in^2$ (Min Reinforcement Area)

$A_{s,toe} = 1.2\ in^2$ (Stem Area)

$$DCR_{toe,min,area} = \frac{A_{s,min,toe}}{A_{s,toe}} = 0.689$$ G (Min Area DCR)

3. Max Area Check

$$\rho_{max,toe} = \frac{0.85 * \left(\beta * f'_c\right) * (0.003)}{f_y * \left(0.003 + \frac{f_y}{E_s}\right)} = 0.0373$$ (Max Reinforcement Ratio)

$$A_{s,max,toe} = \rho_{max,toe} * G * d_{toe} = 9.188\ in^2$$ (Max Reinforcement Area)

$$DCR_{stem,max,toe} = \frac{A_{s,toe}}{A_{s,max,toe}} = 0.131\ \text{G}$$ (Max Area DCR)

4. Min Bar Spacing

$s_{b,min,toe} = Max(1\ in, 1\ in, 0.5\ in) = 1\ in$ (Min Bar Spacing)

$s_{toe} = 6\ in$ (True Bar Spacing)

$$DCR_{toe,min,spacing} = \frac{s_{b,min,toe}}{s_{toe}} = 0.167\ \text{G}$$ (Min Bar Spacing DCR)

5. Max Bar Spacing

$s_{b,max,toe} = Min(5 * G, 18\ in) = 18\ in$ (Max Bar Spacing)

$s_{toe} = 6\ in$ (True Bar Spacing)

$$DCR_{toe,max,spacing} = \frac{s_{toe}}{s_{b,max,toe}} = 0.333\ \text{G}$$ (Max Bar Spacing DCR)

Heel Design

$d_{heel} = d_{toe} = 20.5\ in$ (Depth of Heel Reinforcement)

$a_{it,heel} = 0.195\ in$ (Iterative Compression Block Depth)

$$A_{s,it,heel} = \frac{M_{u,heel}}{0.9 * f_y * \left(d_{heel} - \frac{a_{it,heel}}{2}\right)} = 0.196\ in^2$$ (Iterative Steel Area)

$$a_{heel} = \frac{A_{s,it,heel} * f_y}{0.85 * f'_c * w_{trib}} = 0.195\ in$$ (Compression Block Depth)

$A_{s,heel} = A_{s,toe} = 1.2\ in^2$ **(Steel Area: #7 Bars @ 6"O.C.)**

$$\phi M_{n,heel} = 0.9 * A_{s,heel} * f_y * \left(d_{heel} - \frac{a_{heel}}{2}\right) = 110.173\ kip * ft$$ (Nominal Moment)

$$\rho_{s,heel} = \frac{A_{s,heel}}{w_{trib} * d_{heel}} = 0.00488$$ (Heel Steel Reinforcement Ratio)

1. Flexural DCR:

$M_{u,heel} = 17.960\ kip * ft$ (Demand Moment)

$\phi M_{n,heel} = 110.173\ kip * ft$ (Nominal Moment)

$$DCR_{heel,flexure} = \frac{M_{u,heel}}{\phi M_{n,heel}} = 0.163 \quad \text{G}$$ (Flexural DCR)

2. Min Area Check:

$$\rho_{min,heel} = Max\left(\frac{0.35 * f_r}{f_y}, 0.0018\right) = 0.00336$$ (Min Reinforcement Ratio)

$A_{s,min,heel} = \rho_{min,heel} * w_{trib} * d_{heel} = 0.827\ in^2$ (Min Reinforcement Area)

$A_{s,heel} = 1.2\ in^2$ (Stem Area)

$$DCR_{heel,min,area} = \frac{A_{s,min,heel}}{A_{s,heel}} = 0.689 \quad \text{G}$$ (Min Area DCR)

3. Max Area Check

$$\rho_{max,heel} = \frac{0.85 * \left(\beta * f'_c\right) * (0.003)}{f_y * \left(0.003 + \frac{f_y}{E_s}\right)} = 0.0373$$ (Max Reinforcement Ratio)

$A_{s,max,heel} = \rho_{max,heel} * G * d_{heel} = 9.188\ in^2$ (Max Reinforcement Area)

$$DCR_{stem,max,heel} = \frac{A_{s,heel}}{A_{s,max,heel}} = 0.131 \quad \text{G}$$ (Max Area DCR)

4. Min Bar Spacing

$s_{b,min,heel} = Max(1\ in, 1\ in, 0.5\ in) = 1\ in$ (Min Bar Spacing)

$s_{heel} = s_{toe} = 6\ in$ (True Bar Spacing)

$$DCR_{heel,min,spacing} = \frac{s_{b,min,heel}}{s_{heel}} = 0.167 \quad \text{G}$$ (Min Bar Spacing DCR)

5. Max Bar Spacing

$s_{b,max,heel} = Min(5 * F, 18\ in) = 18\ in$ (Max Bar Spacing)

$s_{heel} = 6\ in$ (True Bar Spacing)

$$DCR_{heel,max,spacing} = \frac{s_{heel}}{s_{b,max,heel}} = 0.333$$ G (Max Bar Spacing DCR)

11. Shear Design

Stem

$V_{u,stem} = 6.690\ kip$ (Stem Ultimate Shear)

$$\lambda_{s,stem} = Min\left(\sqrt{\frac{2\ in}{1\ in + \frac{d_{stem}}{10}}}, 1\right) = 1$$ (Size Effect Modification Factor)

$$f_{lms} = Min\left(\sqrt{f'_c} * 1\ psi^{0.5}, 100\ psi\right) = 76.811\ psi$$ (Limit material strength for shear resistance)

$\phi_s = 0.75$ (shear factor)

$$\phi V_{nc,stem} = Min\left(\phi_s * \left(2 * \lambda_{s,stem} * w_{trib} * d_{stem} * f_{lms}\right), \phi_s * \left(8 * \lambda_{s,stem} * \rho_{s,stem}^{\left(\frac{1}{3}\right)} * w_{trib} * d_{stem} * f_{lms}\right)\right)$$

$= 10.691\ kip$ (Nominal Concrete Shear Capacity)

1. Shear DCR

$$DCR_{stem,shear} = \frac{V_{u,stem}}{\phi V_{nc,stem}} = 0.626$$ G (Max Bar Spacing DCR)

Toe

$V_{u,toe} = -6.230\ kip$ (Toe Ultimate Shear)

$$\lambda_{s,toe} = Min\left(\sqrt{\frac{2\ in}{1\ in + \frac{d_{toe}}{10}}}, 1\right) = 0.810$$ (Size Effect Modification Factor)

$f_{lms} = 76.811\ psi$ (Limit material strength for shear resistance)

$$\phi V_{nc,toe} = Min\left(\phi_s * \left(2 * \lambda_{s,toe} * w_{trib} * d_{toe} * f_{lms}\right), \phi_s * \left(8 * \lambda_{s,toe} * \rho_{s,toe}^{\left(\frac{1}{3}\right)} * w_{trib} * d_{toe} * f_{lms}\right)\right)$$

$= 15.570\ kip$ (Nominal Concrete Shear Capacity)

Heel

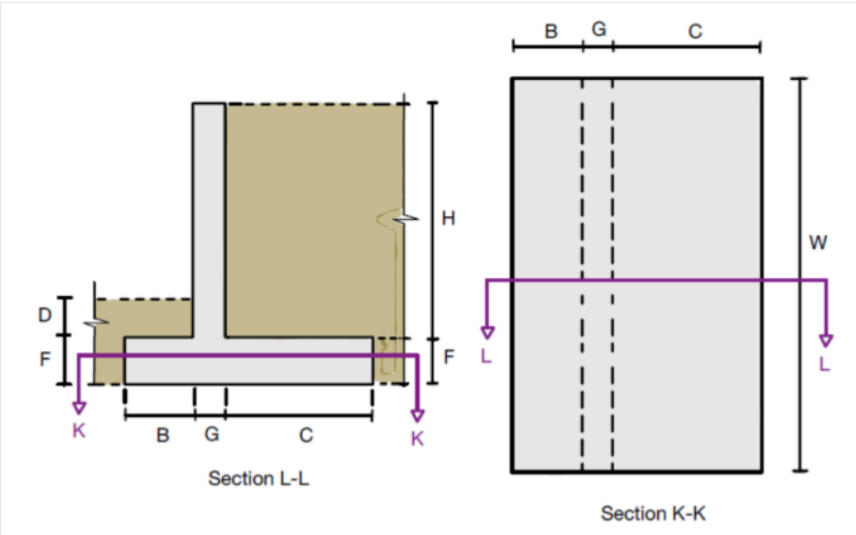
$V_{u,heel} = 8.450\ kip$ (Heel Ultimate Shear)

$$\lambda_{s,heel} = \mathrm{Min}\left(\sqrt{\frac{2\ in}{1\ in + \frac{d_{heel}}{10}}}, 1\right) = 0.810$$ (Size Effect Modification Factor)

$f_{lms} = 76.811\ psi$ (Limit material strength for shear resistance)

$$\phi V_{nc,heel} = \mathrm{Min}\left(\phi_s * \left(2 * \lambda_{s,heel} * w_{trib} * d_{heel} * f_{lms}\right), \phi_s * \left(8 * \lambda_{s,heel} * \rho_{s,heel}^{\left(\frac{1}{3}\right)} * w_{trib} * d_{heel} * f_{lms}\right)\right)$$

$= 15.570\ kip$ (Nominal Concrete Shear Capacity)



12. Temperature and Shrinkage Steel Design

Stem Design

$\rho_{min,stem} = 0.00336$ (Min Reinforcement Ratio)

$G = 12\ in$ (Retaining Wall Thickness)

$s_{max,stem,shrinktemp} = \mathrm{Min}(5 * G, 18\ in) = 18\ in$ (Max Bar Spacing)

$s_{stem,shrinktemp} = 14\ in$ (Chosen Bar Spacing)

$A_{s,min,stem,shrinktemp} = \rho_{min,stem} * G * s_{stem,shrinktemp} = 0.565\ in^2$ (Min Area)

$A_{s,stem,shrinktemp} = 2 * 0.31\ in^2 = 0.62\ in^2$ (Chosen Area)

$$DCR_{stem,shrinktemp} = \frac{A_{s,min,stem,shrinktemp}}{A_{s,stem,shrinktemp}} = 0.911$$ **G** (As_min DCR)

Stem Temp/Shrinkage Steel Design: (2)#5 BARS @ 14" O.C.

Slab Design

$$\rho_{min,slab} = Max\left(\frac{0.35 * f_r}{f_y}, 0.0018\right) = 0.00336$$ (Min Reinforcement Ratio)

F = 2 ft (Slab Thickness)

$$s_{max,slab,shrinktemp} = Min(5 * G, 18\ in) = 18\ in$$ (Max Spacing)

$$s_{slab,shrinktemp} = 14\ in$$ (Chosen Spacing)

$$A_{s,min,slab,shrinktemp} = \rho_{min,slab} * F * s_{slab,shrinktemp} = 1.129\ in^2$$ (Min Area)

$$A_{s,slab,shrinktemp} = 2 * .6\ in^2 = 1.2\ in^2$$ (Chosen Area)

$$DCR_{slab,shrinktemp} = \frac{A_{s,min,slab,shrinktemp}}{A_{s,slab,shrinktemp}} = 0.941\ \text{G}$$ (As_min DCR)

Slab Temp/Shrinkage Steel Design: (2)#7 BARS @ 14" O.C.

3. (B2) FOOTING CALCULATIONS: EXCEL

[SE 140B] Project 2			
Codes: ACI 318-19, ASCE 7-22			
Problem: Spread Footing Design			
1.0 Geometry			
L_o	Side length of the square columns	30	in
S_o	Distance from exterior edge of footing to centerline of column F8	1.25	ft
S_1	Distance between centerlines of columns F8 and F7	27.75	ft
S_2	Distance from center of column F7 to interior edge of footing	7	ft
A	Total assumed length of footing: S_o + S_1 + S_2	36.00	ft
W	Assumed width of footing	20	ft
F_slab	Total thickness of footing	3	ft
d	Effective depth, assuming 3 inches of clear cover to reinforcement centroid: F_slab - 3 in	2.75	ft
D_f	Embedment depth of footing	6	ft
Area_slab	Area of the slab	720	ft^2
Area_columns	Area of the columns	12.5	ft^2
2.0 Material Properties and Design Criteria			
2.1 Concrete Properties			
f'c	Specified compressive strength of concrete	5.9	ksi
γ_c	Unit weight of normal-weight concrete	150	pcf
λ	Concrete density factor (normal-weight = 1.0)	1.0	—
E_c	Modulus of elasticity: 57000*sqrt(fc')	4378.25	ksi
ε	Maximum usable strain	0.003	—
β	Rectangular stress block depth factor	0.755	—
2.2 Steel Properties			
f_y	Strength of longitudinal steel (ASTM A615 Gr 60 deformed bars)	60	ksi
E_s	Modulus of elasticity	29000	ksi
2.3 Soil Properties			
γ_s	Unit weight of backfill soil	129	pcf
q_t	Allowable soil bearing pressure	4000	psf
SF	Factor of safety against failure	2.5	—
q_u	Ultimate soil bearing pressure (Terzaghi)	10000	psf
2.4 Strength Reduction Factors			
ϕ_f	Flexure (assuming tension-controlled section)	0.9	—
ϕ_v	Shear	0.75	—
3.1 Applied Loads (UNFACTORED)			
P_8_F_DL	DL + SDL acting on Column F8	385	kip
P_8_F_LL	LL acting on Column F8	96	kip
P_7_F_DL	DL + SDL acting on Column F7	800	kip
P_7_F_LL	LL acting on Column F7	234	kip
W_slab	Weight of the slab	324	kip
W_soil	Weight of the soil	273.80	kip
W_t	Weight of the soil + slab	597.80	kip
Pt	DL + LL acting on both column	1515	kip
P_tot	Total weight acting from columns and slab	1839	kip
4.1 Calculation of the Eccentricity			
x_F8	Location of F8 about left edge of the footing (x = 0 ft)	1.25	ft
x_F7	Location of F7 about left edge of the footing (x = 0 ft)	29.00	ft
x_footing	Geometric center of the footing	18	ft
X_o	Position of the resultant load	19.80	ft
e_o	Eccentricity relative to the geometric center of the footing	1.80	ft
e_lim	Eccentricity limit (footing remains in compression, no uplift)	6.00	ft
e_o < e_lim?	**DCR CHECK** if eccentricity is within the limit	**0.30**	—
4.2 Stresses at the Base of the Footing (Service Level)			
q_max	Maximum bearing pressure	3322.05	ft
q_min	Minimum bearing pressure	1786.28	ft
q_max < q_t	**DCR CHECK** if the soil bearing capacity is adequate	**0.831**	—
5.1 Factored Loads			
P_uF8	Factored axial load on F8	615.60	kip
P_uF7	Factored axial load on F7	1334.40	kip
P_u	Total factored column load	1950.00	kip
W_u	Factored self weight	388.8	kip
P_u_tot	Total factored vertical load	2338.80	kip
5.2 Ultimate Eccentricity			
X_u	Centroid of the factored loads	19.87	ft
e_u	Ultimate eccentricity relative to the center	1.87	ft
5.3 Gross and Net Ultimate Soil Pressures			
qu_max,gross	Maximum gross ultimate soil pressure	4259.24	psf
qu_min,gross	Minimum gross ultimate soil pressure	2237.43	psf
q_sw	Uniform downward pressure exerted by factored footing self-weight	540.00	psf
qu_net,max	Maximum net ultimate soil pressure	3719.24	psf
qu_net,min	Minimum net ultimate soil pressure	1697.43	psf

6.1 Net Line Load Formulation			
w_max	Maximum distributed line load (at x = 36 ft)	74.38	klf
w_min	Minimum distributed line load (at x = 0 ft)	33.95	klf
w_net(x)	w_min + ((w_max - w_min)/A_slab)*x	33.95 + 1.123x	klf
6.2 Shear Force and Bending Moment Equations			
6.2a Interval 1: Left of F8 (0 < x < 1.25 ft)			
V(x)	Integral of w_net(x)	33.95x + 0.5615x^2	kip
M(x)	Integral of V(x)	16.975x^2 + 0.1872x^3	kip-ft
6.2b Interval 2: Between Columns (1.25 < x < 29 ft)			
V(x)	Shear (33.95x + 0.5615x^2 - P_uF8)	33.95x + 0.5615x^2 - 615.6	kip
M(x)	Moment	16.975x^2 + 0.1872x^3 - 615.6(x - 1.25)	kip-ft
6.2c Interval 3: Right of F7 (29 < x < 36 ft)			
V(x)	Shear (33.95x + 0.5615x^2 - P_uF8 - P_uF7)	33.95x + 0.5615x^2 - 615.6 - 1334.4	kip
M(x)	Moment	16.975x^2 + 0.1872x^3 - 615.6(x - 1.25) - 1334.4(x - 29)	kip-ft
6.4 Critical Sections for Design			
6.4a Maximum Negative Moment (Top Steel)			
x_crit	Occurs where V(x) = 0 **between the columns** (33.95x + 0.5615x^2 - 615.6 = 0)	14.6 ft	kip
M_u,neg	M(x_crit)	-4017.28	kip-ft
6.4b Maximum Positive Moments (Bottom Steel)			
x_crit,F8	Face of F8 (x = 1.25 + 30/(2*12))	2.50	ft
x_crit,F7	Face of F7 (x = 29 - 30/(2*12))	27.75	ft
M_u,pos,F8	Occur at the face of the columns (x = 1.25 ft)	26.89	kip-ft
M_u,pos,F7	Occur at the face of the columns (x = 30.25 ft)	1194.60	kip-ft
7.1 Reinforcement Limits and Spacing Criteria			
A_s,min	Minimum flexural reinforcement needed (0.0018*A_gross)	15.55	in^2
ρ_max	Maximum reinforcement ratio	0.0237	—
A_s,max	Maximum flexural reinforcement needed (ρ_max*b*d)	187.42	in^2
s_clear,min	Assuming maximum aggregrate size of d_agg = 0.75 in and using #8 bars (d_b = 1 in)	1.00	in
s_max	ACI 318-19 Section 8.7.2.2	18.00	in
7.2 Top Reinforcement (Negative Moment)			
M_u,neg	Maximum negative moment	48207.36	kip-in
A_s,init	Assuming an initial moment arm of 0.95d to estimate steel area	28.48	—
a	Depth of the compression stress block	1.42	in
A_s,req	Required steel area given a	27.65	in^2
A_s	Controlling A_s	15.55	in^2
# of bars	# = A_s/A_b of which A_b = 0.79 in^2	40.00	bars
USE (40) # 8 BARS			
A_s,prov	Actual amount of steel area provided	31.60	in^2
A_s,min < A_s,prov < A_s,max	**CHECK** if steel area meets minimum and maximum required	**G**	—
s_cc	Center-to-center spacing (s_cc = (h - 2*(3 in cover) - d_b)/(39 spaces), check spacing	5.83	in
s_cc < s_max	**DCR CHECK** maximum spacing requirements	**0.32**	—
s_clear	s_clear = s_cc - d_b, check spacing	4.83	in
s_clear > s_clear,min	**DCR CHECK** minimum spacing requirements	**0.21**	—
7.3 Bottom Reinforcement (Positive Moment)			
M_u,pos	Maximum positive moment	14335.20	kip-in
M_u,pos << M_u,neg	**DCR CHECK:** By inspection, the structural requirement will be far lower than the minimum steel limit, therefore we can simply assume that the reinforcement for the negative moment will be adequate for the positive moment	**0.30**	—
USE (40) # 8 BARS			
A_s,prov	Provided area of steel	31.60	in^2
A_s,min < A_s,prov < A_s,max	Check if steel area meets minimum and maximum required	**G**	—
s_cc	Center-to-center spacing (s_cc = (h - 2*(3 in cover) - d_b)/(39 spaces)	5.97	in
s_cc < s_max	Check maximum spacing requirements	**0.32**	—
s_clear	s_clear = s_cc - d_b, check spacing	4.97	in
s_clear > s_clear,min	Check minimum spacing requirements	**0.20**	—
7.4 Summary of Longitudinal Reinforcement			
Top Bar Nominal Capacity (ϕM,n_neg)	Required negative moment reinforcement ((40) # 8 bars @ 6" O.C.)	4591.67	kip-ft
DCR Check	Check if capacity is greater than demand (ϕM,n_neg > M_u,neg)	**0.875**	—
Bottom Bar Nominal Capacity (ϕM,n_pos)	Required positive moment reinforcement ((40) # 8 bars @ 6" O.C.)	4591.67	kip-ft
DCR Check	Check if capacity is greater than demand (ϕM,n_pos > M_u,pos)	**0.260**	—

8.1 Effective Band Width and Critical Section

		Column F7	Column F8	
W_eq	Effective band width	66.00	66.00	in
l_c	Cantilever length from column face	8.75	8.75	ft

8.2 Transverse Net Soil Pressure and Moment Demand

		Column F7	Column F8	
qu_net_tr	Upward pressure reacting to the factored column load distributed over the effective band area	12.13	5.60	ksf
w_u,tr	Upward line load acting on the cantilever	66.72	30.78	klf
M_u,tr	Ultimate transverse bending moment at the face of the column	30649.50	14139.56	kip-in

8.3 Reinforcement Limits and Spacing Criteria

		Column F7	Column F8	
d_t	Effective depth for transverse reinforcement (d - 1 in)	32.00	32.00	in
A_s,min	Minimum transverse reinforcement (0.0018*W_eq*F_slab)	4.28	4.28	in^2
A_s,max	Maximum transverse reinforcement (ensuring tension-controlled section, ϕ = 0.9, ρ_max*W_eq*d_t)	49.98	49.98	in^2
s_clear,min	Minimum transverse reinforcement spacing assuming #8 rebar (max(1 in, d_b, 4/3d_agg)	1.00	1.00	in
s_max	Maximum transverse reinforcement spacing	18.00	18.00	in

8.4 Transverse Flexural Reinforcement

		Column F7	Column F8	
A_s,init	Assuming an initial moment arm of 0.95d_t to estimate steel area	18.67	8.61	in^2
a	Depth of the compression stress block	3.38	1.56	in
A_s,req	Required steel area given a	18.73	8.39	in^2
A_s	Controlling A_s	4.28	4.28	in^2

USE (17) #8 DISTRIBUTED EVENLY WITHIN 8.5 FT BAND WIDTH CENTERED UNDER F7 AND F8

		Column F7	Column F8	
A_s,prov	Provided area of steel	13.43	13.43	in^2
A_s,min < A_s,prov < A_s,max	**Check** if steel area meets minimum and maximum required	G	G	—
s_cc	Center-to-center spacing (s_cc = (W_eq - 2*(3 in cover) - d_b)/(16 spaces)	3.69	3.69	in
s_cc < s_max	**DCR Check** maximum spacing requirements	**0.20**	**0.20**	—
s_clear	s_clear = s_cc - d_b, check spacing	2.69	2.69	in
s_clear > s_clear,min	**DCR CHECK** minimum spacing requirements	**0.37**	**0.37**	—

9.1a Critical Shear Demand Evaluated at Column F7

x	Centerline of Column F7 (S_o + S_1)	29.00	ft
x_face	Left face of F7 (x - (L_o/12)/2)	27.75	ft
x_crit	Critical section (x_face - d)	25.00	ft
V_u	Shear equation from Section 6 evaluated at x = 25 ft	584.09	kip

9.1b Critical Shear Demand Evaluated at Column F8

x	Centerline of Column F8 (S_o + h/2)	1.25	ft
x_face	Left face of F8	0.00	ft
x_crit	Critical section (x_face + d)	2.75	ft
V_u	Shear equation from Section 6 evaluated at x = 2.75 ft	518.00	kip

9.2a One-Way Shear Capacity

		Column F7	Column F8	
ρ_w	Steel ratio (A_s,prov/(b_w*d)	0.0479	0.0479	—
λ_s	Size effect factor (ACI 318-19, Section 13.2.6.2)	1.0	1.0	—
V_c	Shear capacity (8*λ_s*λ*(ρ_w)^(1/3)*sqrt(f_c')*b_w*d, ACI 318-19 Equation 22.5.5.1)	1767.21	1767.21	kip
V_c,max	Maximum limit for V_c (5*λ*sqrt(f_c')*b_w*d, ACI 318-19 Table 22.5.5.1)	3041.73	3041.73	kip
V_c < V_c,max	**DCR Check** if shear capacity is less than maximum allowable capacity	**0.58**	**0.58**	—
ϕV_c	Design capacity using shear reduction (ϕ = 0.75)	**1325.41**	**1325.41**	kip
V_u < ϕV_c	**DCR Check** if ultimate shear is less than nominal capacity	**0.44**	**0.39**	—

10.1a Critical Perimeter and Demand for Two-Way (Punching) Shear Design for Column F7

c_1	Length of column (L_o)	2.50	ft
c_2	Width of column (L_o)	2.50	ft
b_1	Length of punching shear area (c_1 + d)	5.25	ft
b_2	Width of punching shear area (c_2 + d)	5.25	ft
b_o	Critical perimeter (2*(b_1 + b_2))	21.00	ft
A_punch	Area inside perimeter (b_1*b_2)	27.56	ft^2
q_net,avg	Average net pressure under F7, approximate	3.50	ksi
V_u,punch	P_u_F7 - (q_net,avg*A_punch)	1237.93	kip

10.1b Critical Perimeter and Demand for Two-Way (Punching) Shear Design for Column F8

c_1	Length of column (L_o)	2.50	ft
c_2	Width of column (L_o)	2.50	ft
b_1	Length of punching shear area (c_1 + d/2)	3.88	ft
b_2	Width of punching shear area (c_2 + d)	5.25	ft
b_o	Critical perimeter (2*(b_1) + b_2)	13.00	ft
A_punch	Area inside perimeter (b_1*b_2)	20.34	ft^2
q_net,avg	Average net pressure under F8, approximate	1.80	ksi
V_u,punch	P_u_F8 - (q_net,avg*A_punch)	578.98	kip

10.2 Two-Way Shear Capacity

		Column F7	Column F8	
α_s	Interior column (ACI 318-19 Section 22.6.5.3)	40.00	30.00	—
β	Aspect ratio of the column (c_long/c_short)	1.00	1.00	—
V_c1	General formula (4*λ*sqrt(f_c')*b_o*d)	2555.06	1581.70	kip
V_c2	Aspect ratio formula ((2 + 4/β)*λ*sqrt(f_c')*b_o*d)	3832.58	2372.55	kip
V_c3	Perimeter ratio formula ((2 + (α_s*d)/b_o)*λ*sqrt(f_c')*b_o*d)	4623.44	3300.28	kip
Governing V_c	Minimum of V_c1, V_c2, V_c3	2555.06	1581.70	kip
ϕV_c	Design capacity using shear reduction (ϕ = 0.75)	**1916.29**	**1186.28**	kip
V_u < ϕV_c	**DCR Check** if ultimate shear is less than nominal capacity	**0.646**	**0.488**	—

4. (B2) FOOTING REINFORCEMENT LENGTHS:

Longitudinal Reinforcement

#8 Bars $d_b = 1.00"$

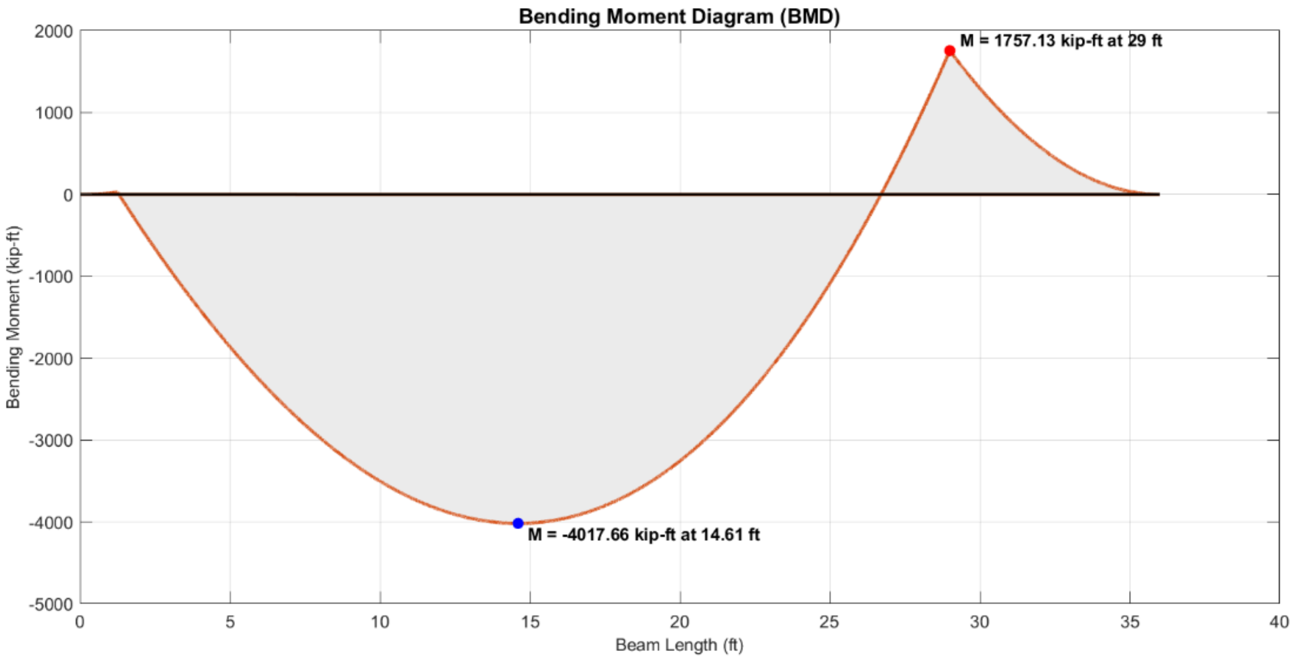
Extension Length: $l_o = 12d_b = 12"$ (ACI 318-19 Table 25.3.1)

- Extension Length is used as hook length at end of reinforcements

Bar Bend Radius: $r = max(6d_b, 3") = 6"$

Development Length: $l_d = 50.77"\ or\ 4.23' \rightarrow 4.25'\ or\ 4'\ 3"$

Lap Splice Length: $l_{st} = 1.3 * l_d = 5.5'\ or\ 5'\ 6"$



Splice was placed to the left of column F7 on both the top and bottom because the moment at that location is low.

Reinforcement between columns:

$$S_o + S_1 - clear\ cover - column\ width/2 = 1.25' + 27.75' - 3" - \frac{30"}{2} = 27'\ 6"$$

Reinforcement under F7 and beyond:

$$S_2 + column\ width/2 + l_d - clear\ cover = 7' + \frac{30"}{2} + 4.25' - 3" = 13'\ 5"$$

Stirrups

Hook Length: $l_h = 6d_b = 6.0"$

Stirrup Lengths: Around longitudinal reinforcements + bar bend + hook length

Ends:

$$2 * (7 * 5.5" + 9 * 6" + 36" - 2 * 3.5") + 2\pi * 1" + 2 * 6" = 21.774'\ or\ 21'\ 10"$$

Middle:

$$2 * (16 * 6" + 36" - 2 * 3.5") + 2\pi * 1" + 2 * 6" = 22.357'\ or\ 22'\ 5"$$

APPENDIX C

Reflections

INDIVIDUAL REFLECTIONS

Chris Liu

- My primary role in the project was to perform the footing design and calculations, as well as part of its drawings and writing up the report for my calculations. In the first week or so, I preliminarily sized the footing, performed structural analysis for positive moment, negative moment, and shear (one-way and punching), picked rebar sizes, and started the drafting. The original plan was that I would also perform the serviceability checks and detailing, including splices and temperature and shrinkage steel. Unfortunately, my week ramped up in business for the second week, including assignments for other classes and a midterm for a graduate class. As a result, I had to ask my other group members for help. This included swapping the serviceability checks and detailing with another group member for their drafting task. I felt like I was the most comfortable with drafting since I had done most of it for the previous capstone class and had been learning Autocad since middle school. Thus, I was still able to help finish part of the project while still being able to take time for my other classes. I also had to ask my other group members to help with writing my part of the report.
- A technical challenge that we faced was discrepancies with the bar detailing and reinforcement for the footing. Some of the numbers and diagrams were confusing, so I had to ask the group member who made them for any clarifications that could assist in my drafting. She responded promptly and thus cleared up any confusions that I had.
- Honestly, there were not any major disagreements for this project. The only thing I can think of that was even close to a disagreement was deciding how to divide the tasks. When we were undergoing this process, one way that helped us finish was to play into each others' strengths, giving tasks to those that have a lot of experience in it previously.
- The most important thing I learned from working on this project as a team is that it is never wrong or bad to ask for help. With my personality, I tend to not ask others for help even if I'm overwhelmed or struggling, in fact, I tend to go out of my way to help others despite that. Maybe it stems from not wanting to burden others or having too much pride in my own work to have someone else work on it. Whatever the reason, it has caused me some amount of pain and struggle in the past, and I think by learning to let that go a bit, it's helping me improve as a team player and on my communication skills as well.

Isaac Kim

- My primary role in the project was to work through all of the retaining wall calculations, including all stability checks, structural analysis, and structural design. Since I worked directly with the retaining wall, I also worked directly on the technical drawings. This is slightly different from the way we typically run things, where the drawings and design are split by team member. However, through the last project, we learned the value of maintaining consistency throughout the whole design process of an element. Thus, the change was welcome.
- One technical challenge our team encountered was finding the optimal design that keeps the DCR below 1.0, while designing for the smallest and minimal number of steel bars. Meeting the DCR was surprisingly simple, but the iterative design process was the most time-consuming since there are so many working combinations of bars. Also since all of the initial calculations were performed in BlockPad and took up nearly 14 pages, it was difficult

to view the holistic process and spot mistakes. To address this, I ended up transferring all of the calculations from BlockPad to Excel. This was a good way to double-check my work and make any adjustments to the design.

- Our team managed communication about design choices and workload by communicating in our iMessage group chat. We didn't have too many disagreements to discuss since we had one person designing the retaining wall all the way through and one person designing the foundation all the way through. The other two people were mainly in charge of the bearing pressure calculations, double-checking the others' calculations, and writing up relevant parts of the report. This unidimensional philosophy worked well in maintaining consistency throughout the design process. However, it was also a lot of work for just two people to handle. Since the design was so stratified, it made communication all the more important to design a cohesive structure. In the future, one thing that we'd do differently is splitting up the work more evenly. which is not my teammates' fault. It was my fault since I volunteered for it.
- The most important thing I learned through this project is the value of teamwork. When this project was first released, I thought it wouldn't take too long to just power through by the work of a few people. However, after a few days, it became quickly overwhelming. I learned that working collaboratively is the best approach to generating quality work even if it takes a little bit more time. Another thing I learned is how it may be valuable to write up the report while working through the calculations. Writing the report several days after doing the calculations makes me more prone to forgetting. This is something that I will do moving forward in Project 3.

Hannah Ahn

My primary role was detailing the reinforcement in the footing. I began with the role of organizing the report, checking calculations, and assisting with the drawings but in the end I took on reinforcement detailing instead helping with drawings. This was to help with other team members in evening out the calculation work and because I had experience doing the reinforcement detailing. When our team initially discussed roles, I was willing to help with figuring out details needed for the drawings so it was not a difficult task. Working on the reinforcement sizing and layout in the previous project also helped as I knew where in ACI 318-19 I had to check for specific sizings.

One technical challenge our team encountered was figuring out the detailing and drawings for the footing and retaining wall because we had to make design choices that were not explicitly told to us. We worked through it by envisioning what would be necessary to actually construct this structural member and details and measurements required would come to mind. Looking at the project as if we were a professional engineer helped to eliminate areas of uncertainty in terms of what our deliverables had to cover. The details considered for the drawings were a lot easier to choose with this mindset.

When my role was adjusted, open communication and flexibility allowed for things to work through smoothly. We began the project with a meeting to discuss who wanted to do what part of the project. This gave all of our team members a good idea of what they had to be in charge of. As time went on though, our roles shifted and we settled into responsibilities that fit us best. It was a good approach to assign all responsibilities in the beginning and shifting things around as we went because that means that no responsibilities fall between the cracks. This strategy works for us but it requires all team members to know the whole scope of the project from the very beginning. But there is room for change throughout the project in case responsibilities were not allotted well in the beginning. All of our team members have different strengths and skillsets they bring to the project so there were not many overlaps over aspects of the project we wanted to be in charge of.

The most important thing I am taking away from the project is the need for open teamwork. Last project, we split up responsibilities in the calculations more and that proved difficult since we relied on one another's calculations to do our own. This was not hard but rather inconvenient. This time, we had people carry through the whole calculation of a part themselves instead of splitting it up. This had its own challenges as the work was overwhelming for certain people and others were not involved with the problem solving at all. Rather than focusing on convenience, next time I need to prioritize the team's experience and choose the more inconvenient route that requires more teamwork.

Brian Tran

- My primary role in the project was to double check the calculations performed by my teammates in addition to the individual reflection. Over the course of the two weeks however, I also helped with the report write up because the calculations ended up taking up more time than we had initially expected.
- One technical challenge that my team encountered was an unexpected drawing issue. For one of the drawings, what the calculator was trying to convey to the drafter was not completely clear. This was quickly resolved by the two members asking each other questions and clearing up any confusion.
- My team managed disagreements about design choices, workload, and approach by communicating with each other whenever problems would come up. We listened to what each other had to say, which helped us quickly resolve any issues that occurred.
- The most important thing I learned from working on this project as a team is that responsibilities can change quickly and I need to be prepared to do what I can to help out my team. The two members working on the calculations had a lot of work to do for another class, so me and another team member helped out where we could with the report. This allowed us to get work done on time.

TEAM SUMMARY

Our team began the project by distributing the various roles and responsibilities needed. Brian was responsible for the loading convention and bearing pressure calculations as well as double checking other team members' calculations. Isaac was responsible for the retaining wall calculations which includes the stability check, structural analysis, flexure and shear design, and servicing with detailing. He was also responsible for the retaining wall drawing. Chris was responsible for the combined spread footing which includes the structural analysis, flexure and shear design, serviceability calculations, and the drawings. Hannah was responsible for the report formatting, checking over the drawings, and footing reinforcement detailing. We each chose what responsibilities we wanted to take on. There were times where we asked other members to take on other responsibilities but it was done through clear and open communication.

Moving forward, we will prioritize distributing the calculations and drawings more evenly so that we are all doing portions of the project instead of leaving it to select members. This will make us naturally peer review one another's work as we do calculations together and will help in preventing errors or wrong judgement. At the beginning of the project, we distributed roles but did not set intermediate deadlines for the various aspects of the report. It would be helpful to set intermediate deadlines in the next project as this will make sure that expectations are clear for each responsibility. This will also help as we are trying to distribute the workload more. Overall, we are continuing to learn what works and does not work for our team. Despite some challenges in the work distribution, it was a good learning experience that we can improve on for the next project.