CENE 486

Capstone: Trax Team

Wall E. Wallerson & Associates Inc.

Final Report

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1 Project Introduction

The purpose of this project is to create a retaining wall which will allow the land owner, Holiday Inn, to maximize the use of their land. This retaining wall will serve to stabilize the slope that separates the grades of the Trax land and the railroad. The parcel is currently vacant and contains excess soil fill from the 2006 relocation of the railroad tracks.

Photos of the site's current condition were taken during site visit on 9/23. Figure 1-1 displays the south eastern boundary of the Trax land. The proposed retaining wall will roughly parallel the this boundary, and the picture was taken at the approximate location of the beginning of the retaining wall. The apparent path in the picture, void of vegetation, displays what will likely be the approximate location of the FUTS trail.

Figure 1-1: Wall location facing east

Figure 1-2 below, displays the slope which separates the Trax land from the Railroad. This figure is also facing east.

Figure 1-2: Back slope location facing east

Figure 1-3 below, shows the North Fourth Street bridge, which represents the most southwestern boundary of the Trax land.

Figure 1-3: Back slope facing west

1.1 Current Conditions

The current conditions on the site could be generally characterized as undeveloped and includes a steep slope, which can be seen in Figure 1-3 above, on the northeastern property line which separates the Trax property from the railroad. Reports from the client characterize the soil as poor and contains fill material from the construction of the railroad.

1.2 Project Location

The Trax retaining wall project is located on the east side of Flagstaff, at Fourth St. and Route 66. The parcel address is 2251 E. Route 66, Flagstaff AZ 86001. Figure 1-4 below, displays the location of the project in relation to the greater flagstaff area. the approximate project location, the Trax land, is outlined in red. Figure 1-5 shows the property lines of the project site, and the retaining wall will be located along the southeast property line.

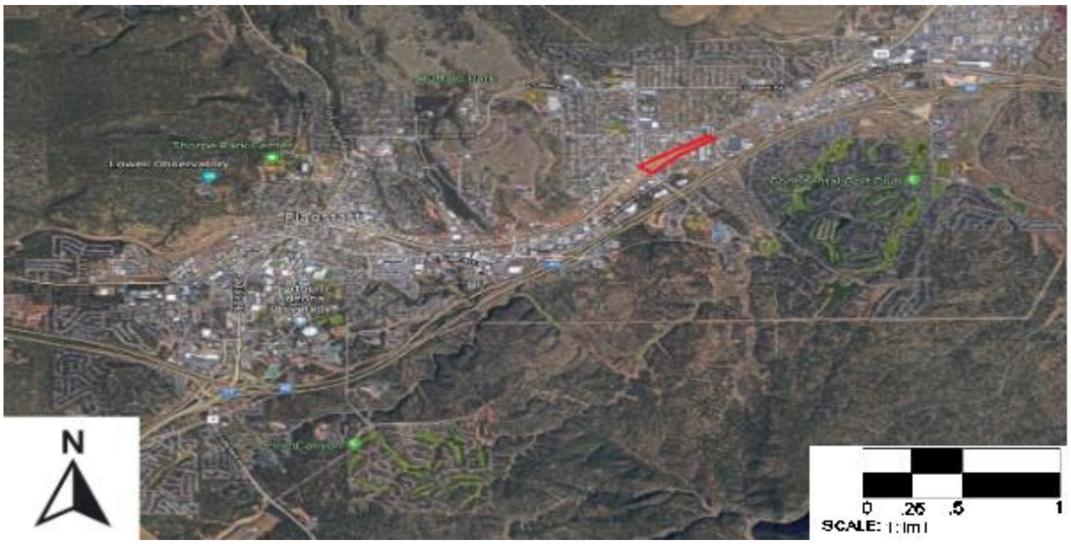

Figure 1-4: Site location relative to Flagstaff

Figure 1-5: Parcel location relative to surrounding locations

1.3 Project Constraints/Limitations

The primary limitation to the project was the lack of proper boring equipment. This limitation was addressed by performing the necessary soil tests on soil samples collected from the soil stockpile on site, as opposed to split spoon samples. Another limitation on the project was the proximity of the proposed wall to the boundary separating the Trax and Railroad properties, which influenced the design of the wall.

2 Field Work

The field work for this project consisted of a site investigation and soil sample collection. A safety and sampling plan was created for the field work and is appended to this report as Appendix A. Soil sampling was conducted from 10am to noon on September 23rd, 2019. The weather was slightly rainy and the team worked quickly to avoid being rained on. Soil conditions were dry on top, but more moist at approximately a foot deep into the stockpile. Ambient temperature was approximately 55 degrees Fahrenheit. Each sample was taken from a different location along the pile. For each sample, four holes were dug into the side of the pile, two on the north side and two in the same location but on the south side of the pile. The sample holes were approximately one to two feet deep towards the center of the pile to avoid external weathering. Each of the six buckets were filled from four different holes at different locations along the soil pile. Figure 2-1 below, shows the west side of the soil sample pile where approximately half of the soil samples were taken from.

Figure 2-1: West side of sampled soil pile

Figure 2-2 located below shows the east side of the soil sample pile where half of the soil samples were taken from.

Figure 2-2: East side of sampled soil pile

Figure 2-3 located below shows the six 5-gallon buckets of soil after the soil collection process.

Figure 2-3: Collected soil samples

3 Testing and Analysis

Testing performed included:

1) Particle-Size Distribution
2) Hydrometer
3) Atterberg Limits
4) Sand Cone (Replaced with Modified Proctor Compaction)
5) Tri-axial
6) Consolidation
7) Direct Shear

The full results of these tests can be seen in Appendix C: Geotechnical Report. Results are summarized in the sections below.

3.1 Particle-Size Distribution

The particle-size distribution test was completed in accordance with ASTM D6913. According to this standard test procedure, the soil was sieved from the bulk composite samples through numbers 10, 20, 40, 60, 100, 140, and 200 sieves. Each of the six samples were sieved, and a percent finer graph was produced (Figure 3-1). Averaging the results of the six tests, it was determined that the soil is comprised of 28.53% gravel, 65.11% sand, 5.8% silt, and 0.56% clay. These data, along with the results of the hydrometer and Atterberg limits tests, were used to determine the soil classification.

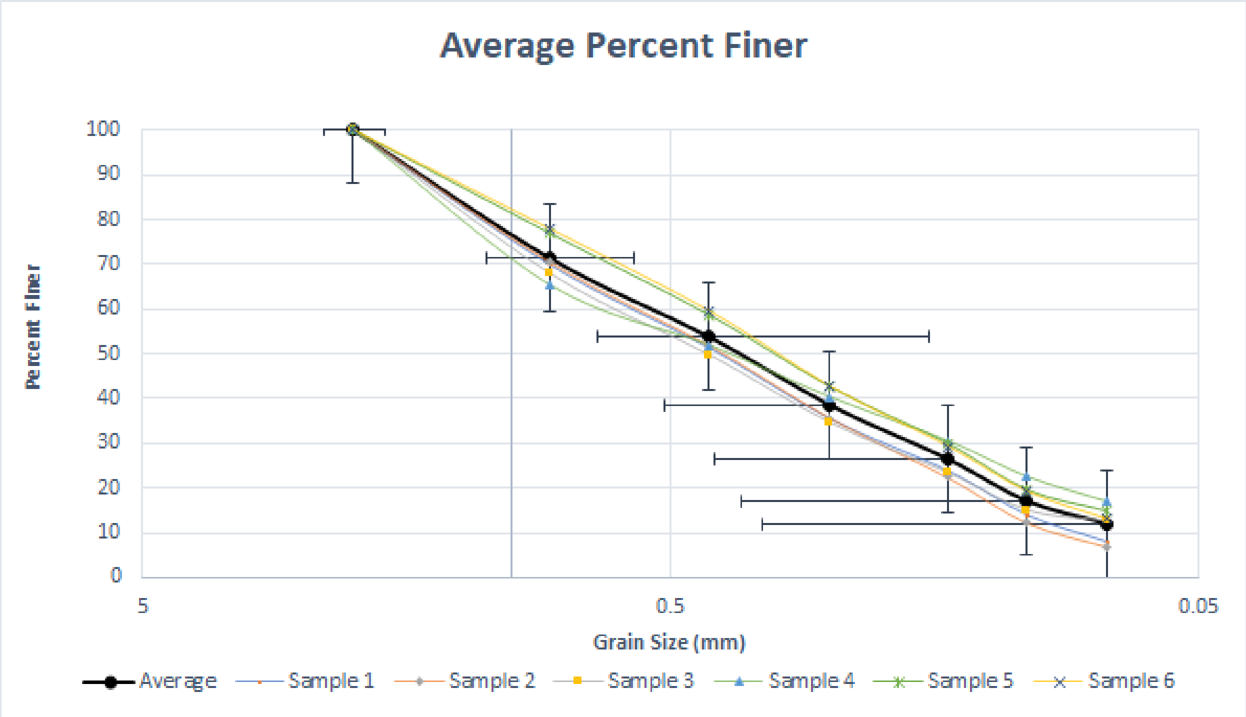

Figure 3-1: Average percent finer graph

3.2 Hydrometer

The hydrometer test followed ASTM 7928-17 where only the soil that passed the number 200 sieve was tested. In order to determine the silt and clay percentages, approximately 50 grams of each soil sample finer that the 200 sieve was placed in a 1000 milliliter graduated cylinder with 125 milliliters of sodium hexametaphosphate and 875 milliliters of water. A hydrometer was placed in each cylinder and measurements were taken at time intervals up to 48 hours. These measurements record how fast the soil particles settle to the bottom of the cylinder and these data were used to determine the fine soil particle size distribution. Figure 3-2 shows each fine soil particle size distribution as well as the average. The results show that the soil contains 65.11% sand, 5.8% silt, and 0.56% clay. This data was used to classify the soil.

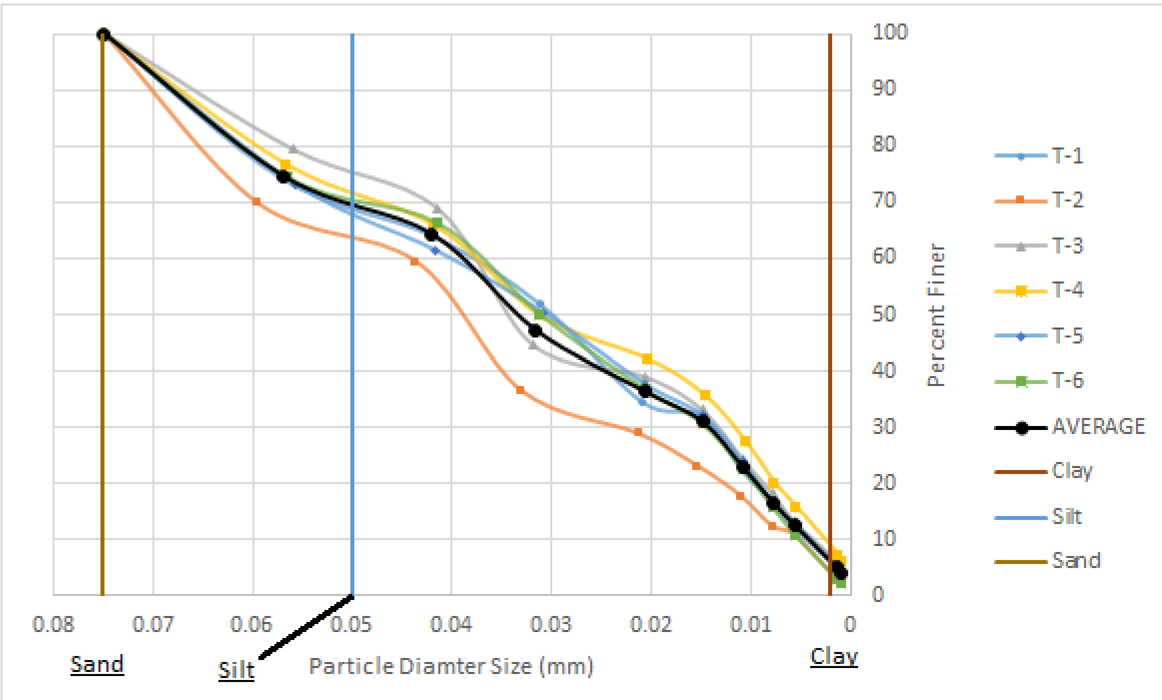

Figure 3-2: Fine soil particle size distribution graph

3.3 Atterberg Limits

The Atterberg Limits tests followed ASTM-D4318-17 which determined the plastic and liquid limits. The plastic limit occurs when the moisture content of the soil reaches a level that the soil begins to act as a plastic and the liquid limit occurs when the moisture content of the soil reaches a level that the soil begins to act as a liquid. The plastic limit was determined by adding water to each soil sample finer than the Number 40 sieve, and then roll it on a glass plate until the rolled soil cracks at a diameter of 1/8th of an inch. Then the soil is dried and the moisture content and plastic limits were determined. Table 3-1 below shows the moisture content of each sample when the soil begins to act as a plastic, as well as the average plastic limit with the standard deviation.

Table 3-1: Plastic limit data results

Plastic Limit %						
Moisture Can ID	T-1	T-2	T-3	T-4	T-5	T-6
Mc (g)	19.5	13.3	19.8	13.6	13.2	13.3
Mm (g)	31.6	25.2	27.5	24.2	31	22.4
Md (g)	29.2	23.2	26	22.2	27.7	20.7
w (%)	24.74	20.20	24.19	23.26	22.76	22.97
PL (%)	24.74	20.20	24.19	23.26	22.76	22.97
AVG PL (%)	23.02	±1.58				

The liquid limit also used the soil finer than the Number 40 sieve and water was added to the soil. The soil was then placed in a Casagrande cup and a cut was made down the middle exposing a two millimeter gap between the soil. The Casagrande cup was then raised 10 millimeters and dropped until the soil closed the gap. This was done four times for each soil sample with different moisture contents to create a liquid limit graph. Figure 3-3 below shows the data collected for each sample and the trend lines for each sample. Samples 4 and 5 have dotted trend lines because the trend line slopes are positive which is incorrect so they were excluded from the liquid limit average. The equation from the average trend line was then used to determine the optimal moisture content at 25 drops, and that was used to determine the liquid limit which was 24.55 percent. These limits were used to classify the soil.

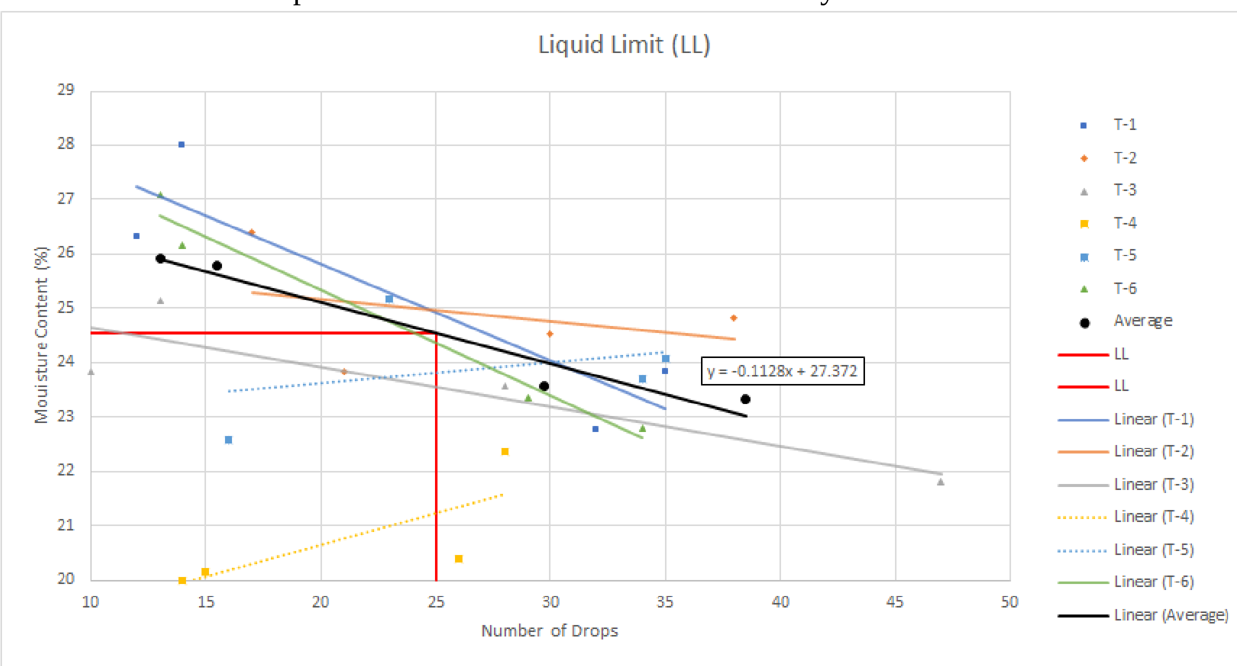

Figure 3-3: Average liquid limit graph

3.4 Modified Proctor Compaction

The modified proctor compaction test followed ASTM-1557-12e1 to determine the dry and moist unit weight of the soil. Soil passing the Number 4 sieve was collected from each sample and water was added to create 4% moisture content. The soil sample was placed in the compaction mold and the proctor hammer was dropped 25 times to compact the soil. A second layer of soil was then placed in the mold and compacted with another 25 hammer drops. A third layer was added and compacted, and the weight of the compacted soil was collected and then the sample was placed in the oven to determine the moisture content. The soil had another 4% moisture content added and the compaction process was repeated. This process of adding 4% moisture content and then compaction was repeated until the weight of the compacted soil began to decrease. Figure 3-4 shows the results from this test. All of the data was averaged except for sample 4 because it does not represent the soil well. The optimal dry unit weight is 1752 (kg/m^3).

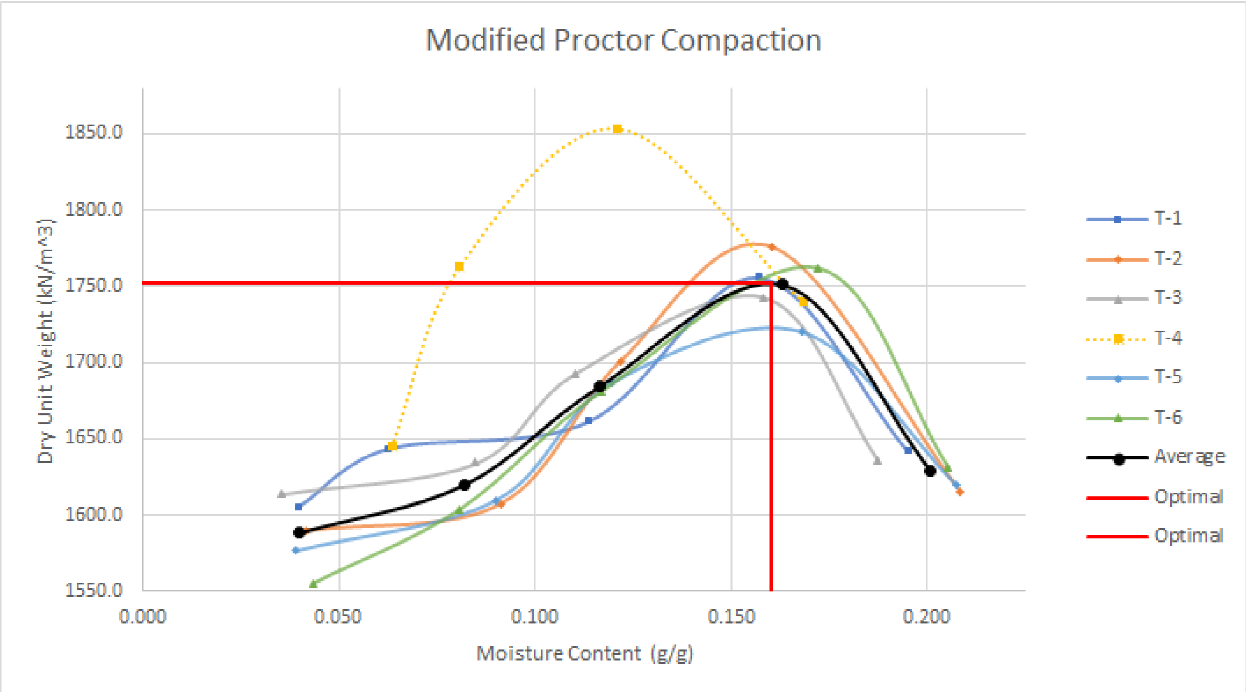

Figure 3-4: Modified proctor compaction graph

3.5 Triaxial Test

The triaxial test results are shown in Figure 3-5 below, and resulted in ambiguous data. The specific triaxial test used was an Unconfined-Unconsolidated test, which is meant for cohesive soils. The tested soil was composed of a large percentage of sand, which is a relatively non-cohesive soil. This created soil specimens that could not bear much stress, and thus failed far earlier than expected. It was determined that a direct shear test would have to be implemented to acquire an adequate shear strength value.

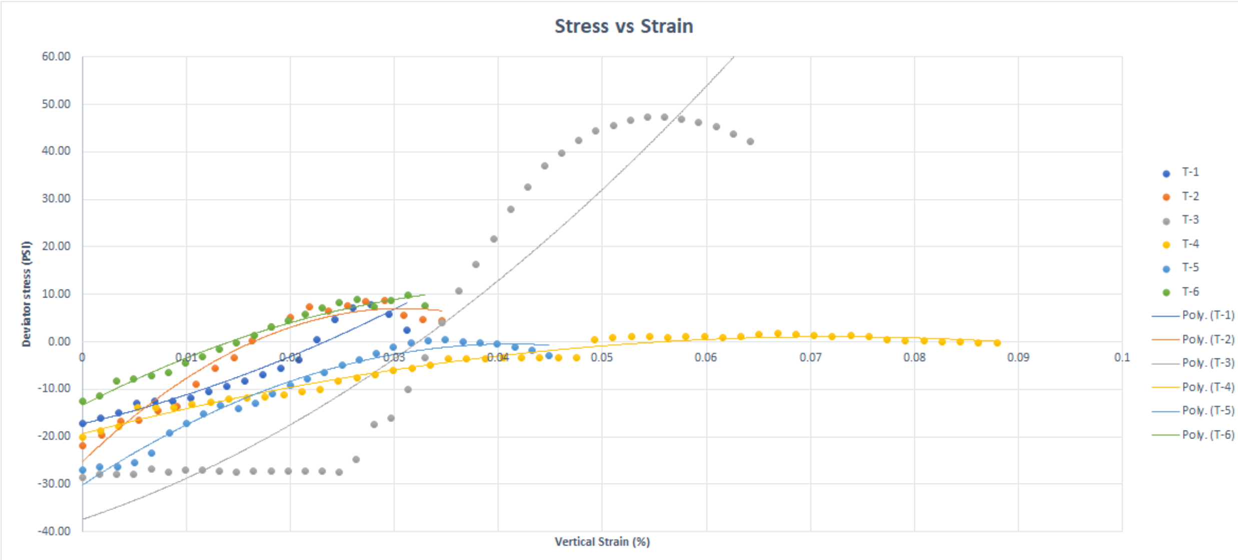

Figure 3-5: Triaxial stress vs strain graph

The expected results from this test was to determine the soil friction angle from various compressive strengths. The results that were obtained did not accurately represent the soil because the majority of the soil is sand and this test is meant to test clay.

3.6 Direct Shear

The direct shear test followed ASTM D3080 to determine the friction angle of the soil. The friction angle was initially determined by piling the dry soil and physically measuring the angle of friction, which was determined to be 35 degrees. This friction angle was a conservative estimate that was going to be changed after the direct shear test. The direct shear test allowed an actual friction angle of 37.9 degrees to be determined. The actual friction angle turned out to be larger than the bulk piled angle (35 degrees), meaning the soil is more cohesive than expected. Figure 3-6 shows the shear stress plotted against the normal stress, which were both recorded through a computer during the testing, and the trend line represents the angle of friction for the soil.

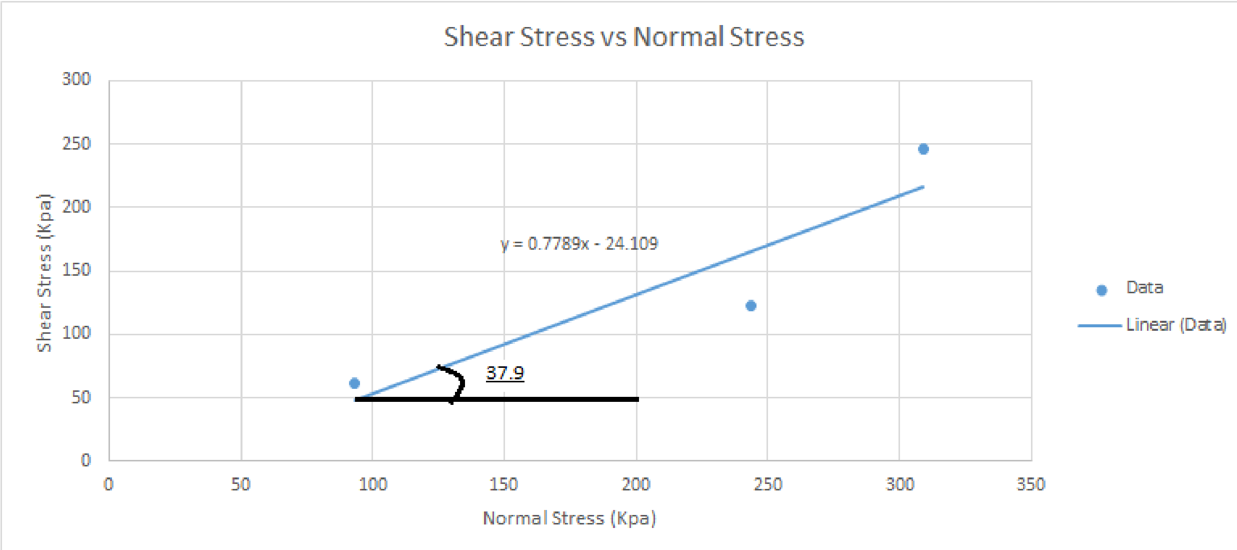

Figure 3-6: Friction angle of soil

3.7 Consolidation Test

In order to analyze soil settlement over time, a consolidation test was run, adhering to ASTM D2435. The objective a consolidation test is to measure settlement over time and attempt to obtain an ultimate settlement value. In order to do this, a vertical strain versus vertical stress curve was developed based upon the test results, and can be seen in Figure 3-7, below. The soil was loaded to a pressure of 2099 psf, which is within the realm of what the actual bearing conditions on sight will be under the load of the proposed retaining wall, FUTS trail, and Holiday Inn. Under this loading, the soil reached a final settlement value of 2.47 mm or approximately .097 inches, as displayed in the raw data of Appendix C. This low level of consolidation is consistent with what would be expected of a soil with low levels of clay, as identified by the soil classification methods. Lastly, Figure 3-8 displays the void ratio of the specimen with logarithmic time. From this Figure, it can be seen that the soil contained approximately 18% voids upon completion of compaction. This is consistent with the final moisture content of the sample which was approximately 18%. This is displayed on the Figure as 0.1845 and since the specimen was fully saturated and under load, it can be safely assumed that the entirety of the voids were due to the presence of water. It must be noted, that due to the time requirements of this test (4 days to reach full loading) that only one specimen was tested.

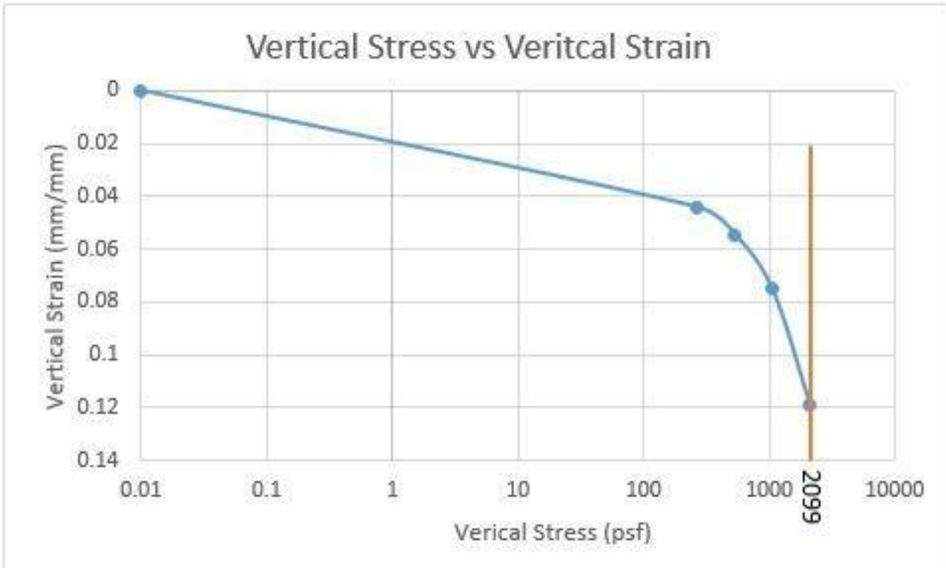

Figure 3-7: Vertical stress vs strain curve

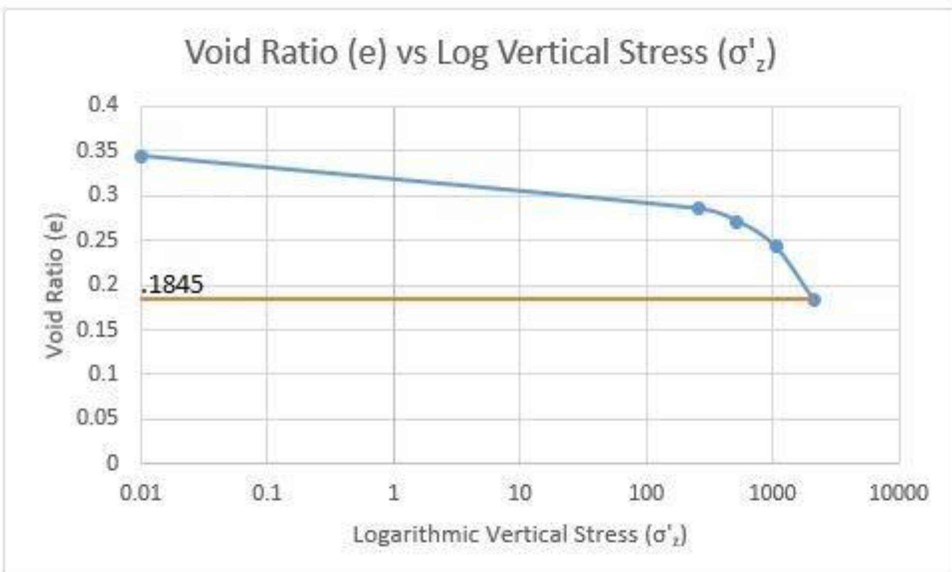

Figure 3-8: Void ratio vs log vertical stress curve

3.8 Heavy Metals Tests

Table 3-2 below displays the results of the heavy metal contaminants testing. The test was performed using a Thermofisher Niton XL3T that uses X ray Fluorescence to detect concentrations of heavy metals identified by the Arizona Soil Remediation Standard for Residential Limits [6]. Twenty four test specimens (4 per each of the 6 samples) were placed in the ring and cap plastic containers with a thin translucent film over the top of them. Then, environmental consultant and NAU graduate student, Wyatt LaFave tested the samples using a lead encased portable test stand.

Table 3-2 below, shows that the soil only slightly exceeds Arsenic and Vanadium levels. Heavy metal contamination is not considered a concern.

Table 3-2: XRF Chemicals of Interest

Contaminant	Detected Average (ppm)	Error (ppm)	**Threshold (ppm)
Strontium (Sr)	432.7	6.3	47000
*Molybdenum (Mo)	4.7	3.8	390
*Cadmium (Cd)	11.6	9.3	39
*Tin (Sn)	11.0	5.5	47,000
*Antimony (Sb)	23.5	8.5	31
*Mercury (Hg)	8.9	7.9	23
*Uranium (U)	6.8	6.3	16
Lead (Pb)	30.3	4.8	400
*Arsenic (As)	9.5	4.0	10
Titanium (Ti)	6108.0	110.7	310,000
Vanadium (V)	117.1	26.5	78
Cromium (Cr) III	38.0	9.3	120,000
Manganese (Mn)	876.2	62.4	3300
*Cobalt (Co)	165.1	144.6	900
Nickel (Ni)	62.6	16.3	1600
Copper (Cu)	45.8	12.3	3100
Zinc (Zn)	101.1	9.2	23,000
*	These elements yielded results which did not meet the minimum levels of detection (LODs) in some or all of the samples, and were thus not accounted for in the average.		
**	AZ Residential Soil Remediation Standards Threshold for Remediation		

3.9 Soil Classification

Through the use of the AASHTO soil classification system, seen in Figure 3-9 below, the soil has been classified as A-1-b: stone fragments, sand, and gravel. If the gravel contents were to be ignored, it would change the percentage of soil passing the #40 sieve (step 5) and would then be classified as A-3: fine sand.

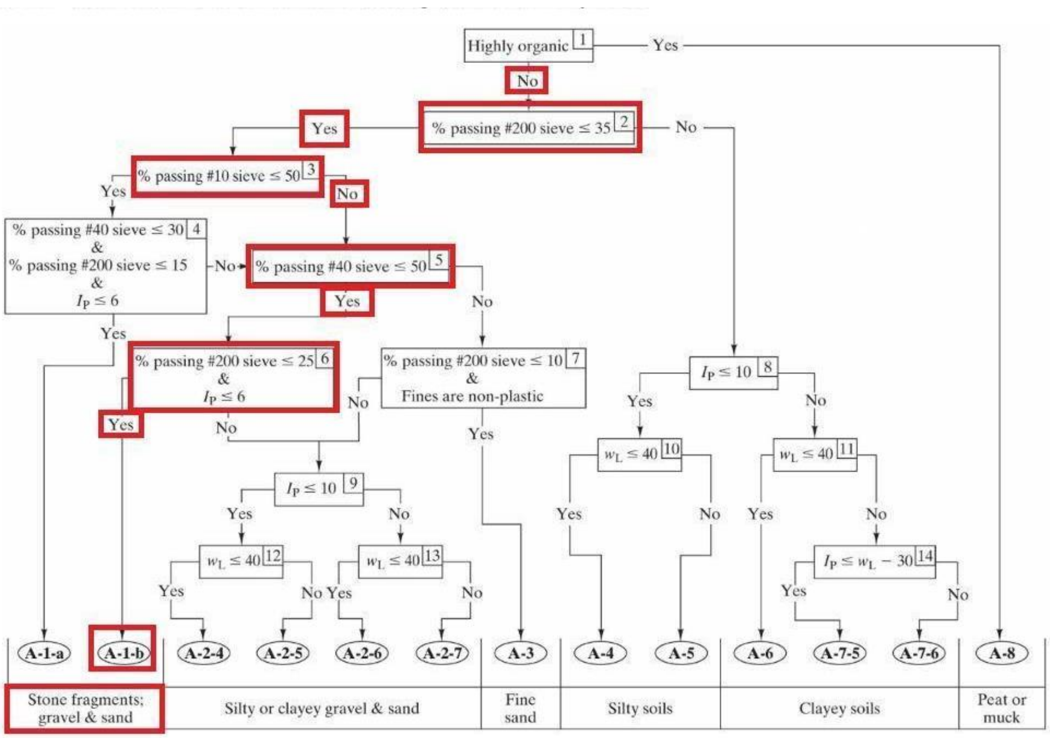

Figure 3-9: AASHTO soil classification

4 Hydrology

The determination of the amount of precipitation on the parcel was completed using National Oceanic and Atmospheric Administration, NOAA. The determination of how the water moves to the parcel is shown as a major basin in figure 4-1. The intensity of rainfall that will be present on site when an average storm event occurs is 0.690 inches in 10-minutes, the average amount of precipitation for a storm in Flagstaff. All intensities that are located on the parcel are shown in Appendix D, showing the determination of storm intensities. Using the area of the parcel, 8.7 acres, and the amount of intensity that NOAA provides, the amount of water that is present on the parcel during a storm is 43.13 cubic ft per second on the entire parcel using the 10 year storm data. This shows that an average 10 year storm has minimal effect on the parcel. And precipitation that is directly on the parcel can be neglected.

The determination of the flow of water to the parcel uses stream stats to calculate the path of flow to the parcel and the total amount of water making it to the parcel. The determination of the amount of total water behind the wall will determine the wall restriction in design.

Figure 4-1: Streamstats defined basin

The movement of water on the site has been determined using the Streamstats[7] program to delineate the major watershed that leads to the site. Streamstats determined that the amount of flow to the parcel for the 100-year storm is 507cfs. The flow from the basin is south and floods Fourth Street during heavy rains. Arizona Department of Transportation, ADOT, uses a series of catch basins to move the water to an underground storm sewer. On the northern side of the parcel, the storm drains can be seen as part of the curb and gutter on Route 66. These are identified to be 25 feet apart and run along the full length of the parcel. This is in place due to the flooding that happens on Fourth Street and runs into Route 66. The parcel was raised from due to the fill during the railroad relocation in 2006, putting it slightly higher than the floodplain. The current infrastructure that is in place will not allow flooding from the basin to reach the parcel.

Figure 4-2: Catch basins along route 66

Shephard Wesnitzer Inc., SWI, has provided the drainage plans for the current site, with flow directions and flow mitigation. The plans show that the increase in impervious surfaces (116,100 square feet because of the Holiday Inn.) The impervious surfaces will reduce the amount of infiltration into the soil, which will allow the water on the parcel to be negligible. The drainage plan shows that the water will be diverted from impervious surfaces to the storm sewer management that will run underneath the FUTS trail. In conclusion, the wall will have some form of drainage to release any excess water from behind the wall, however, this will follow a predetermined detail. The predetermined detail will show a weep hole that will be used in the wall.

5 Hydraulics

City of Flagstaff and Coconino County do not provide standard details for retaining wall drainage. Maricopa County design standards, also known as M.A.G.[9], were used for the drainage of the wall. Weep holes will be spaced 20 ft apart, evenly along the base of the wall. The holes will be made of 4" PVC pipe and cut to fit the length of the wall with a ½" slope per foot. Maricopa County uses a filter material that is either gravel or coarse sand directly behind the wall and filtering to the weep hole. The fill will be 18" tall and 18" wide and will run along the base of the wall. The filter material will be determined by the contractor, and will need to be placed between the wall and the existing soil. The final design will be using a weep holes as the cost and will fit with the elevation change for the wall. Weep holes that will be used for the design are shown in the design detail below.

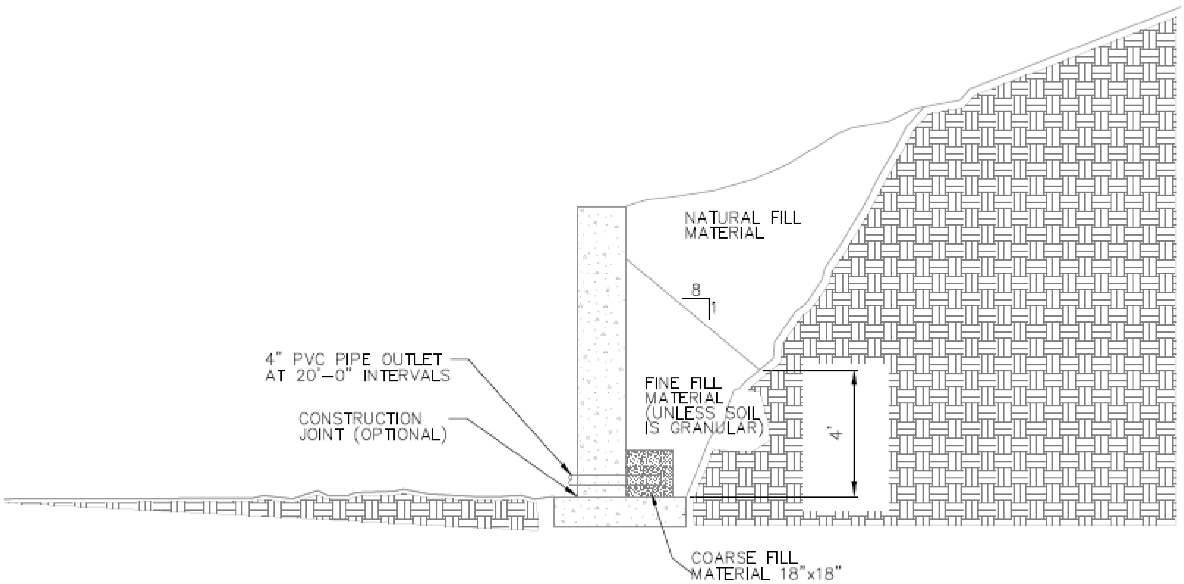

Figure 5-1: Weep hole Detail [9]

6 Wall Design Alternatives

Design alternatives were determined according to the decision matrix shown in Table 6-1 below. In the initial decision matrix, 7 possible alternatives were analyzed, using a positive, neutral, or negative weight for the prescribed categories. Rough sketches of each of these seven wall types can be seen in Figure 6-1, below. The seven wall alternatives included: a concrete gravity wall, concrete cantilever wall, reinforced concrete cantilever wall, anchored retaining wall, mechanically stabilized earth wall, concrete masonry unit wall, and a geotextile wall. Each category was equally weighted, and the three alternatives with the highest total points were chosen as design alternatives to be further evaluated. These more detailed designs are discussed in Section 7.0 Final Wall Design Recommendation.

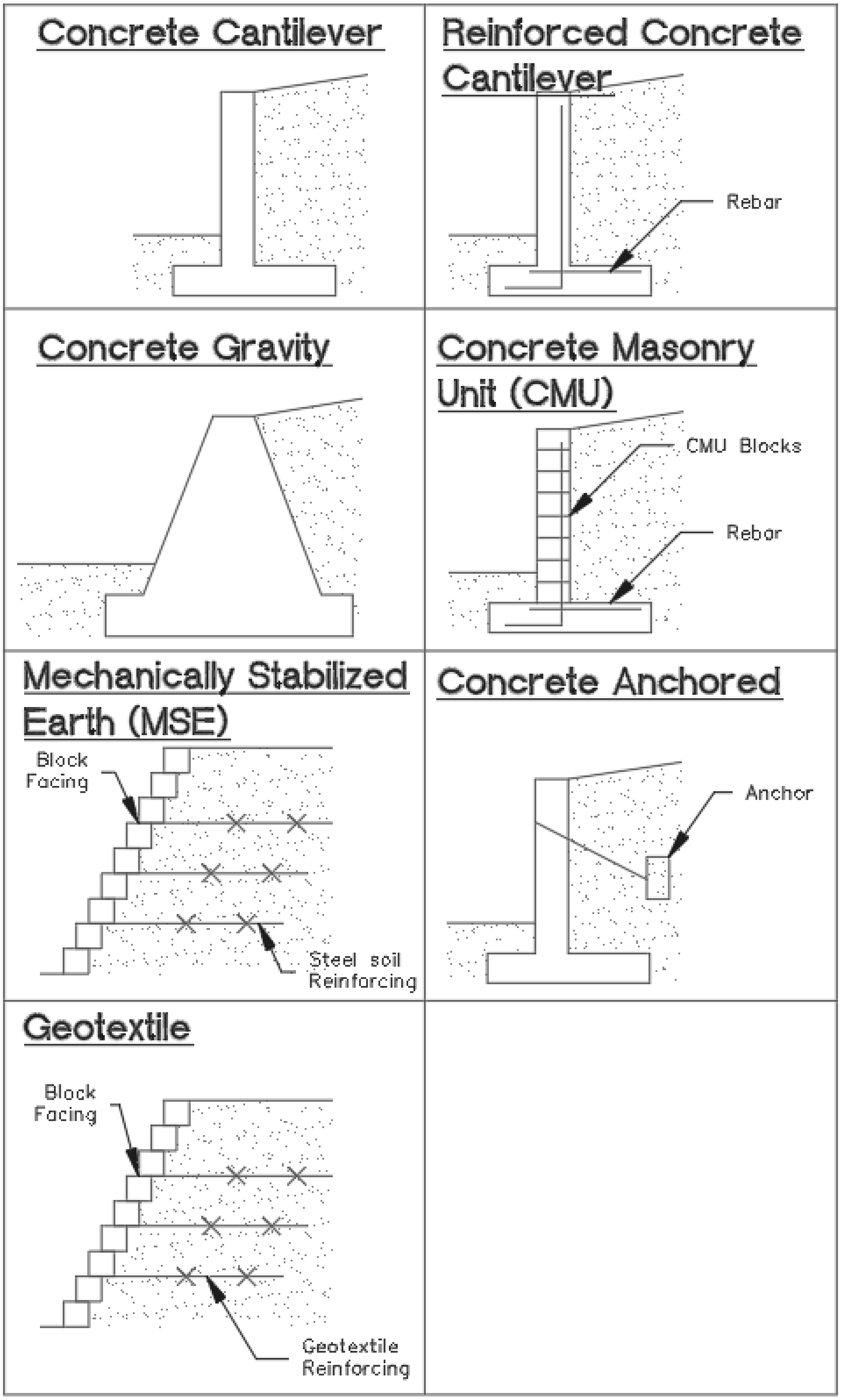

Figure 6-1: Wall alternative sketches

Figure 6-1 above, displays the 7 preliminary designs that were considered for further design. The concrete cantilever is a conventional design option which tends to use smaller footings than its reinforced counterpart, but trades off for depth of excavation required. Similar to the concrete cantilever wall, a concrete gravity wall does not require reinforcement due to its sheer size and volume, but because it tends to be a larger wall, it may not be a suitable option for this design. An anchored retaining wall can utilize a variety of designs, but the idea is that the anchor is attached or buried to something outside of the failure envelope of the wall. This alternative may not be viable due to the proposed storm drain. A Mechanically Stabilized Earth retaining wall, uses a combination of compaction and layered reinforcements to stabilize the slope. This option may not be viable due to the proposed storm drain. A retaining wall made of Concrete Masonry Units essentially acts as a cantilevered wall, but differs due to the lighter unit weight of the concrete masonry units, and the thinner dimensions of the wall. Lastly, a geotextile wall utilizes a synthetic plastic in lifts to stabilize the slope. It may also conflict with the proposed storm drain.

Table 6-1: Wall alternative decision matrix

Decision Matrix Criteria	Concrete Gravity Wall	Concrete Cantilever Wall	Reinforced Concrete Canteliever Wall	Anchored Retaining Wall	Mechanically Stabilized Earth	Concrete Masonry Unit	Geotextile Wall
Foundation Size (6 inch restriction)	-1	0	0	1	1	-1	1
Required Rienforcement (Amount needed)	1	1	-1	-1	1	0	0
Wall Asthetics (Doesn't stand out)	-1	1	0	-1	1	1	1
Estimated Construction Time	1	1	0	-1	-1	0	-1
Sum	0	3	-1	-2	2	0	1

Table 6-2: Decision matrix scaling key

Decision Matrix Key	
Point Value	Description
-1	The wall does not meet the teams requirements and is not practical for wall size or construction.
0	The wall does not have a negative or positive impact on the surroundings. The wall will meet rquirements, but is not the best option.
1	The wall exceeds expectations and is practical for design in this category.
	Selected walls for design.

In the preliminary decision of which walls the team would further evaluate, the concrete gravity wall was not considered because of its large footing and thick base, which would likely require more land use than the project allows for. The reinforced concrete cantilever wall was not considered because after examining the unreinforced concrete cantilever wall, it was determined that no reinforcement was needed. The anchored wall was not considered because of the anchor reinforcement required will add cost and time on the design as well as the drainage may be affected by the anchor. The geotextile wall was not considered because of the complexity of the design and the large estimated cost of the wall. The conventional cantilevered wall was chosen because of its simplicity, the low estimated cost, and the fast estimated construction time. The MSE wall was chosen because of its alternative material type and a more contemporary design could be evaluated. The CMU wall was selected because of the existing CMU retaining wall that was used on the south west side of the 4th Street bridge; this would provide a better look for the area.

These alternatives were further evaluated through the use of a decision matrix to provide a final design recommendation.

6.1 Concrete Cantilever Retaining Wall

The first wall design option is a conventional concrete cantilever retaining wall as shown in Figure 6-2. The dimensioning for this wall were determined from the calculations that are shown later in the report.

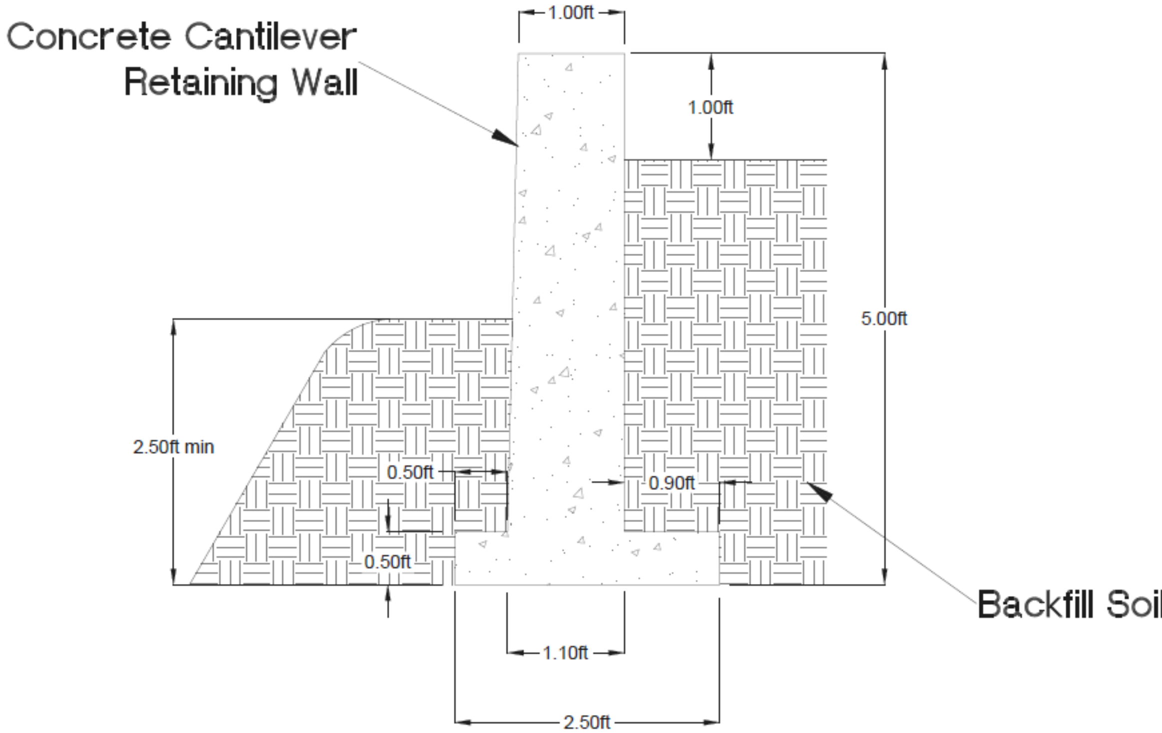

Figure 6-2: Cross-section of concrete cantilever retaining wall

As shown above, the designed wall is five feet high with a minimum buried depth of 2.5 feet. The footing at the bottom of the retaining wall is 2.5 feet wide and these dimensions can be shown in the cross-section of the wall (Figure 6-2). The values used for design, both those determined from testing and those calculated, are located in Table 6-3. These values were used to ensure that the concrete cantilever wall meets the minimum required factors of safety for overturning, sliding, and bearing. These design checks are located in tables 6-4 through 6-6.

Table 6-3: Concrete cantilever retaining wall givens/assumptions

Givens		
Heel Space	Setback (ft)	1
Unit Weight	γ concrete (psf)	150
Cohesion	C (lb/ft^2)	0
Friction Angle	Φ (degrees)	37.9
Unit Weight	γ soil (psf)	109.37
Bearing Capacity Factor	Nc	46.12
Bearing Capacity Factor	Nq	33.3
Bearing Capacity Factor	Nγ	48.03
Footing Depth	Df (ft)	2.5
Height	H (ft)	5
Footing Width	B (ft)	2.5
Active Earth Pressure Coefficient	Ka	0.2715
Passive Earth Pressure Coefficient	Kp	4.228
Weighted Footing Width	B' (ft)	1.957
Length	L (ft)	1500
Alpha	α	0
Top of Wall Width	Top B (ft)	1

Table 6-4 shows the calculations that were completed to determine the factor of safety check for overturning. Overturning failure occurs when the active moment force acting on the wall is significantly larger than the resisting moment force causing the wall to overturn. The results of checking this design, as displayed in the green highlighted cells of Table 6-4, displays the overturning factor of safety is 3.09, which passes the minimum of 3.0.

Table 6-4: Overturning factor of safety check calculations

Overturning			
Major Principle Stress	σ0 (lb/ft^2)	546.85	$\sigma 0 = \gamma soil * H$
Minor Principle Stress	σa (lb/ft^2)	148.46978	$\sigma a = (\sigma 0 * Ka) - \left(2 * C * \sqrt{Ka}\right)$
Total Active Force	Pa (lb/ft)	560561.66	$Pa = \sigma a * H * 0.5 * L$
Vertical Active Force	Pv (lb/ft)	0	$Pv = Pa * \sin \alpha$
Horizontal Active Force	Ph (lb/ft)	560561.66	$Ph = Pa * \cos \alpha$
Section 1 Weight	W1 (lb/ft)	1012500	$W1 = (H - (0.1 * H))$*Top B*γconc*L
Section 2 Weight	W2 (lb/ft)	101250	W2=(1.2*TopB)*0.5*(H-(0.1*H))*γc*L
Section 3 Weight	W3 (lb/ft)	281250	$W3$ =B*(0.1*H)*γc*L
Section 4 Weight	W4 (lb/ft)	590598	$W4 = B * \left((0.1 * H) + 1.2\right) * \left(H - (0.1 * H)\right) * \gamma s * L$
Section 5 Weight	W5 (lb/ft)	0	$W5 = 0$
Section 1 Moment Distance	M1 (ft)	1.2	Distance to bottom heel corner
Section 2 Moment Distance	M2 (ft)	0.6333333	Distance to bottom heel corner
Section 3 Moment Distance	M3 (ft)	1.25	Distance to bottom heel corner
Section 4 Moment Distance	M4 (ft)	2.1	Distance to bottom heel corner
Section 5 Moment Distance	M5 (ft)	2.236	Distance to bottom heel corner
Driving Moment	Md (ft-lb/ft)	934269.43	$Md = Ph * (1/3) * H$
Resisting Moment	Mr (ft-lb/ft)	2870943.3	$Mr = (W1 * M1) + (W2 * M2) + (W3 * M3) + (W4 * M4) + (W5 * M5) + (Pv * M4)$
Overturing Factor of Safety	F.S.	3.0729287	$3 < Md/Mr$
Check	GOOD	3.09>3	

Table 6-5 shows the calculations that were completed to determine the factor of safety check for sliding. Sliding failure occurs when the active horizontal force acting on the wall is significantly larger than the resisting horizontal force causing the wall to slide. The result of checking this design for sliding can be seen in the green highlighted cells of Table 6-5 and displays that the sliding factor of safety is 1.64, which exceeds the minimum required value of 1.5.

Table 6-5: Sliding factor of safety calculations

Sliding			
Sum of Vertical Force	V (lb/ft)	1985598	$V = sum(W)$
Resisting Friction Force	fr (lb/ft)	937175	$fr = V * \tan((2/3) * \Phi)$
Cohesion Force	fc (lb/ft)	0	$fc = 0$
Passive Stress	σp' (0) (lb/ft)	0	$\sigma p(0) = 2 * C * \sqrt{Kp}$
Passive Stress	σp' (H) (lb/ft)	1156.063	$\sigma p(H)$ =γs*Df*Kp*σp(0)
Passive Force	Pp1 (lb/ft)	0	$Pp1 = \sigma p(0) * Df * L$
Passive Force	Pp2 (lb/ft)	2167619	$Pp2 = \sigma p(H) * Df * 0.5 * L$
Total Passive Force	Pp (lb/ft)	2167619	$Pp = Pp1 + Pp2$
Total Driving Force	Fr (lb/ft)	3104794	$Fr = fr + fc + Pp$
Total Resisting Force	Fd (lb/ft)	560561.7	$Fd = Ph$
Sliding Factor of Safety	FS(sliding)	5.53872	$1.5 < Fr/Fd$
Check	GOOD	1.64>1.5	

Table 6-6 shows the calculations that were completed to determine the factor of safety check for bearing. Bearing capacity failure occurs when the vertical bearing pressure acting on the wall is significantly larger than the resisting pressure force pushing up on the footing causing the wall to sink. The results of checking this design for bearing capacity can be seen in the green highlighted cells of Table 6-6, and displays that the factor of safety for the bearing capacity was calculated as 13.08, which well exceeds the minimum required factor of safety of 3.0.

Table 6-6: Bearing capacity factor of safety calculations

Bearing Capacity			
Pressure	q (lb/ft)	410137.5	$q = (V/B) * (1 \pm (6e/B))$
Shape Factor	Fqs	1.001012	From Table
Shape Factor	Fγs	0.99948	From Table
Depth Factor	Fqd	1.231638	From Table
Depth Factor	Fγd	1	From Table
Momnent Difference	Mn (ft-lb/ft)	1936674	$Mn = Mr - Md$
Eccentricity	e (ft)	0.274639	$e = (B/2) - (Mn/V)$
Maximum Pressure	q_max (lb/ft^2)	1317750	$qmax = (V/B) * (1 + (6e/B))$
Minimum Pressure	q_min (lb/ft^2)	270728.5	$qmin = (V/B) * (1 - (6e/B))$
Beta	β (degrees)	0	$\beta = angle\ of\ vertical\ pressure$
Inclination Factor	Fqi	1	From Table
Inclination Factor	Fγi	1	From Table
Bearing Capacity	qu (lb/ft^2)	16843340	$qu = (q * Nq * Fqd * Fqi) + (0.5 * \gamma s * B * N\gamma * F\gamma d * F\gamma i)$
Bearing Factor of Safety	F.S.	12.78189	$3 < qu/qmax$
Check	GOOD	13.08>3	

This wall design meets all of the retaining wall checks and works with the proximity constraints as well as the grade elevations. Since the grade along the wall varies, the wall needed to include steps to keep the minimum depth at 2.5 feet and to keep the top of wall one-foot minimum above grade. Table 6-7 includes the grade elevations along the wall as well as the elevations of the top and bottom of the wall. It also includes the step locations and the above and below grade values.

Table 6-7 shows the stationing of the wall starting from the west side, and provides the grade elevation, top and bottom of wall elevations, stepping locations, step sizes, and the above and below grade lengths of the wall.

Table 6-7: Retaining wall elevations and stepping

			Elevations (ft)					
Station	Grade	Top of Wall	Bottom of Wall	Steps	Step Size	Above Grade	Below Grade	Wall Height
0+00	6851	6853.5	6848.5			2.5	2.5	5
0+50	6852	6854	6849	STEP UP	0.5	2	3	5
1+00	6853.7	6856	6851	STEP UP	2	2.3	2.7	5
1+50	6855	6857	6852	STEP UP	1	2	3	5
2+00	6856.2	6857	6852			0.8	4.2	5
2+50	6856.6	6859	6854	STEP UP	2	2.4	2.6	5
3+00	6857	6859	6854			2	3	5
3+50	6857.3	6859	6854			1.7	3.3	5
4+00	6857.6	6859	6854			1.4	3.6	5
4+50	6858	6859	6854			1	4	5
5+00	6858.4	6860.5	6855.5	STEP UP	1.5	2.1	2.9	5
5+50	6859.6	6860.5	6855.5			0.9	4.1	5
6+00	6860	6860.5	6855.5			0.5	4.5	5
6+50	6859.8	6860.5	6855.5			0.7	4.3	5
7+00	6860.5	6862.5	6857.5	STEP UP	2	2	3	5
7+50	6861.3	6862.5	6857.5			1.2	3.8	5
8+00	6862.2	6862.5	6857.5			0.3	4.7	5
8+50	6861.6	6862.5	6857.5			0.9	4.1	5
9+00	6860.9	6862.5	6857.5			1.6	3.4	5
9+50	6860.2	6862	6857	STEP DOWN	0.5	1.8	3.2	5
10+00	6860.5	6862	6857			1.5	3.5	5
10+50	6861.2	6863	6858	STEP UP	1	1.8	3.2	5
11+00	6861.9	6863	6858			1.1	3.9	5
11+50	6862.1	6863	6858			0.9	4.1	5
12+00	6861.7	6863	6858			1.3	3.7	5
12+50	6861	6861.5	6856.5	STEP DOWN		0.5	4.5	5
13+00	6860.4	6861.5	6856.5			1.1	3.9	5
13+50	6859.7	6860	6855	STEP DOWN	1.5	0.3	4.7	5
14+00	6859.1	6860	6855			0.9	4.1	5
14+50	6858.4	6860	6855			1.6	3.4	5
15+00	6857.7	6860	6855			2.3	2.7	5

Figure 6-3 below, shows a plan view of the concrete cantilever design discussed above. This alignment utilized very few steps, since the wall remains the same height throughout the alignment.

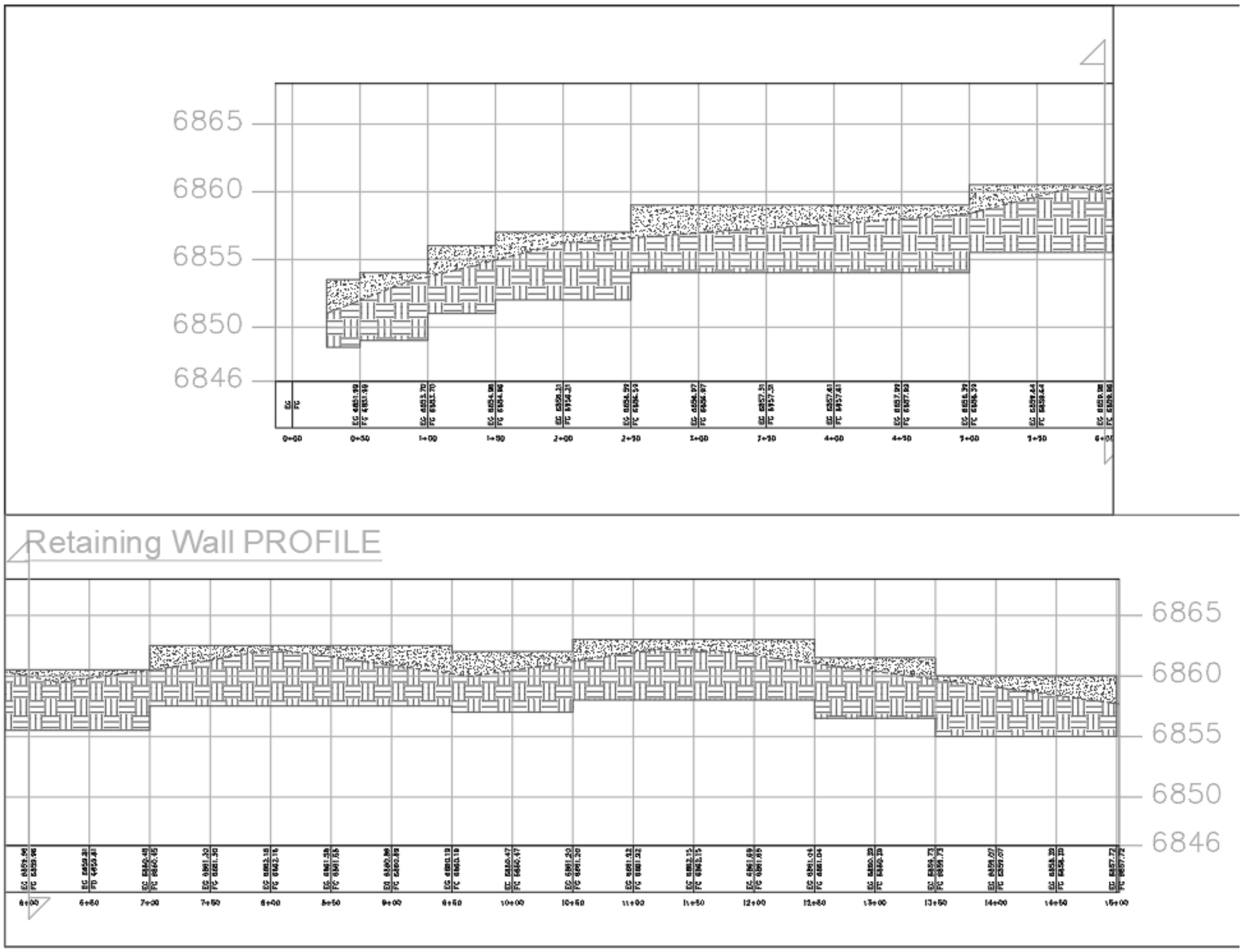

Figure 6-3: Concrete cantilever retaining wall profile

6.2 Mechanically Stabilized Earth Retaining Wall (MSE)

The second design alternative is a Mechanically Stabilized Earth retaining wall shown in Figure 6-4. The dimensioning for this design was determined from the calculations that are shown later in the report.

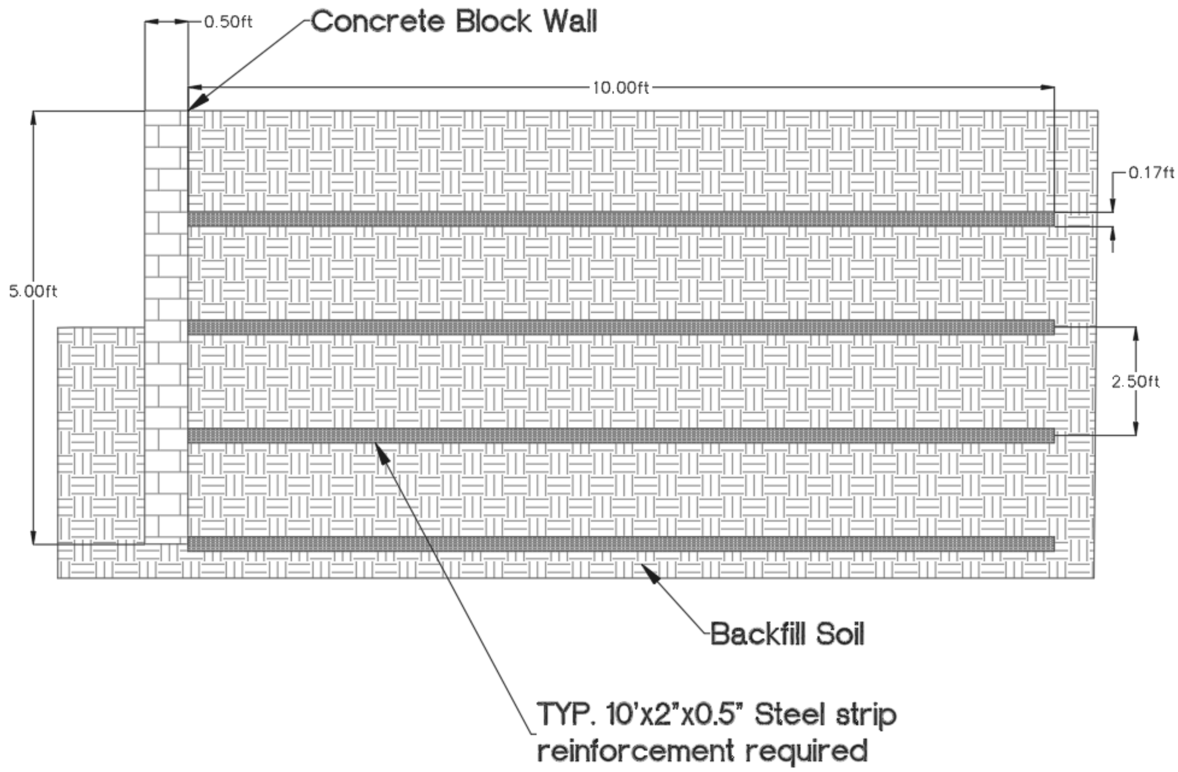

Figure 6-4: MSE retaining wall cross section

Table 6-8 shows the given values from the soil testing as well as the assumed height of the wall. The assumed wall height was determined by inputting various heights into the calculations until the factor of safety checks met the requirements.

Table 6-8: MSE retaining wall givens/assumptions

Givens			
Friction Angle	Φ	37.9	Given
Soil Unit Weight	γ (psf)	109.37	Given
Cohesion	C	0	Given
Height	H (ft)	5	Assume

Table 6-9 shows the assumed steel strip dimensions and calculations used to determine the length of the steel straps. The purpose of this table was to determine the required length of the steel straps. The dimensions and spacing of the straps were assumed through a trial and error process to determine the required length of the steel straps, which is 10 feet.

Table 6-9: MSE steel reinforcement assumptions/calculations

Steel Reinforcement			
Width of Tie	w (in)	2.00	Assume
Vertical Tie Spacing	S_V (ft)	1.25	Assume
Horizontal Tie Spacing	S_H (ft)	4.00	Assume
Yield Strenght of Tie	f_y (lbs/ft^2)	5012504.23	Assume
Soil-Tie Friction Angle	Φ_u	20	Assume
Tie Thickness	t (in)	0.057594397	$t = (\sigma a * Sv * SH * FS(B))/(w * fy)$
Corrosion Tie Thickness	t_c (in)	0.12	$tc = t + (corrosion * lifespan)$
Breaking Factor of Safety	FS_B	3	Assume
Pulling Factor of Saftey	FS_P	3	Assume
Lateral Pressure	σ_a (lbs/ft)	267.31	$\sigma a = \gamma * H * Ka$
Active Pressure Coefficient	K_a	0.489	$Ka = \tan(45 + \Phi/2)$
Steel Strap Length	L	10	

Table 6-10 shows the calculations used to determine the overturning factor of safety. The result from this table is highlighted in green which shows that the overturning factor of safety is greater than the required value of 3.

Table 6-10: MSE overturning factor of safety check

Overturing Check			
Soil Weight	W	5468.5	$W = \gamma * H * L$
Distance to Soil Load	x	5	$x = L/2$
Lateral Soil Force	Pa	668.3	$Pa = 0.5 * \gamma * Ka * H^2$
Depth of Lateral Force	z	1.67	$z = H/3$
Overturining Factor of Safety	FS	24.5	$3 < (W * x)/(Pa * z)$

Table 6-11 shows the calculation used to determine the sliding factor of safety. The result from this table is highlighted in green which shows that the sliding factor of safety is greater than the required value of 3.

Table 6-11: MSE sliding factor of safety check

Sliding Check			
Sliding Factor of Safety	FS	3.86	$3 < (W * \tan((2/3) * \Phi))/Pa$

Table 6-12 shows the calculations used to determine the bearing capacity factor of safety. The result from this table is highlighted in green which shows that the bearing capacity factor of safety is greater than the required value of 3.

Table 6-12: MSE bearing capacity factor of safety check

Bearing Check			
Ultimate Bearing Pressure	q_{ult}	26265.206	$qult = 0.5 * \gamma * L * N\gamma$
Vertical Stress	σ_o (lbs/ft)	546.85	$\sigma 0 = \gamma * H$
Bearing Factor of Safety	FS	48.0	$5 < qult/\sigma 0$

Figure 6-5 shows a preliminary profile view of the MSE retaining wall. There are steps shown along the profile, and these steps occur at the same elevations and locations as the concrete cantilever retaining wall. Table 6-7 shown earlier in the report shows the stepping locations and elevation changes along the profile.

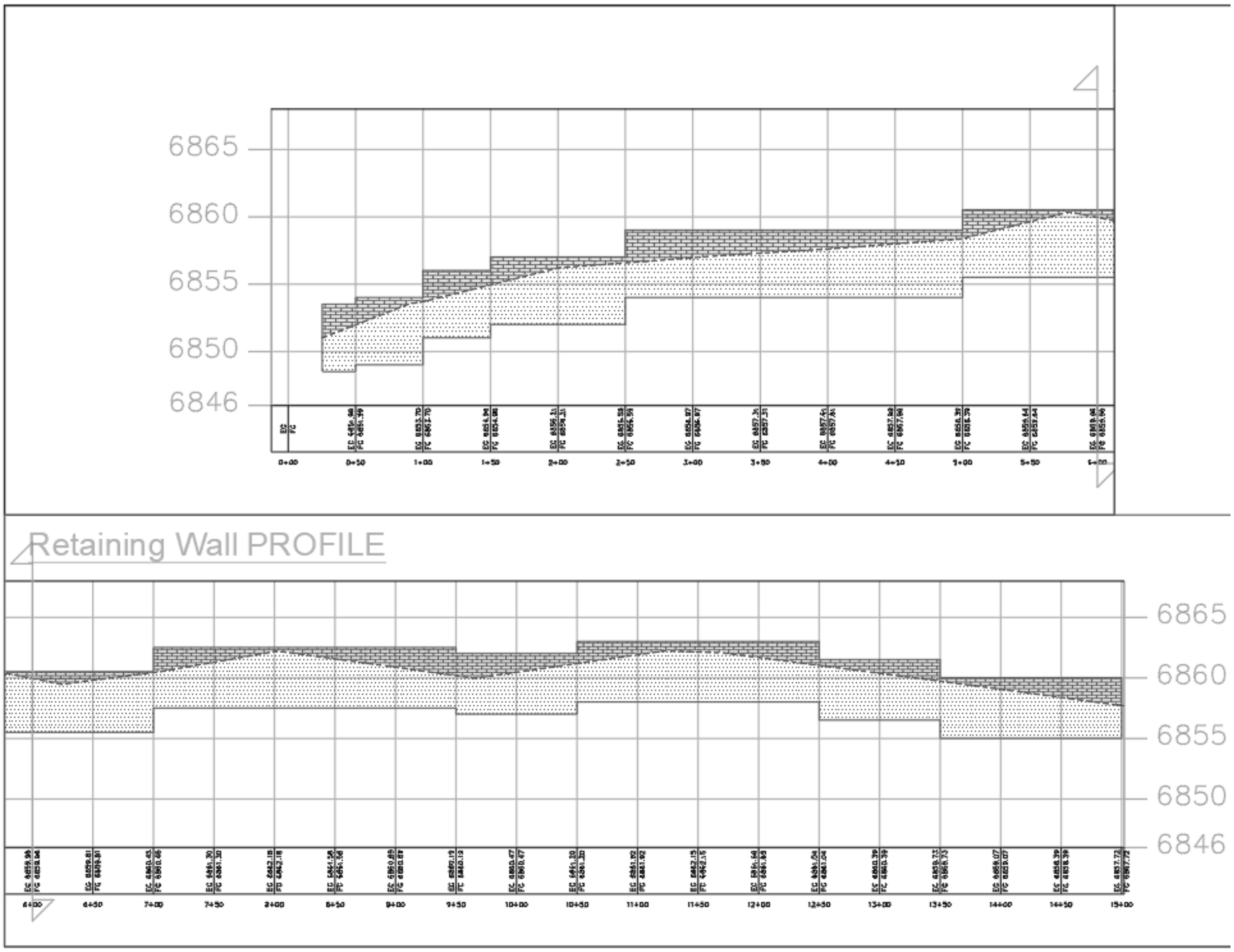

Figure 6-5: MSE retaining wall profile

6.3 Concrete Masonry Unit Retaining Wall

The third design alternative will be a Concrete Masonry Unit retaining wall which has a cross-section shown in Figure 6-6. Figure 6-6 includes the dimensions of the wall as well as the varying wall heights/base widths that occur along throughout the length of the wall. The wall heights vary because the depth of footing was maintained at 5 feet while the top of wall varied as the elevation of the finished grade changed along the alignment. The table in the upper right corner of Figure 6-6 shows the different heights used along the wall with the footing sizes for that wall height. The rebar required in the footing and stem was designed from the ACI Reinforced Concrete Code [11].

BLOCKS	H (FT)	B (FT)
9	5.72	9.0
10	6.35	9.0
11	6.99	10.0
12	7.63	10.0

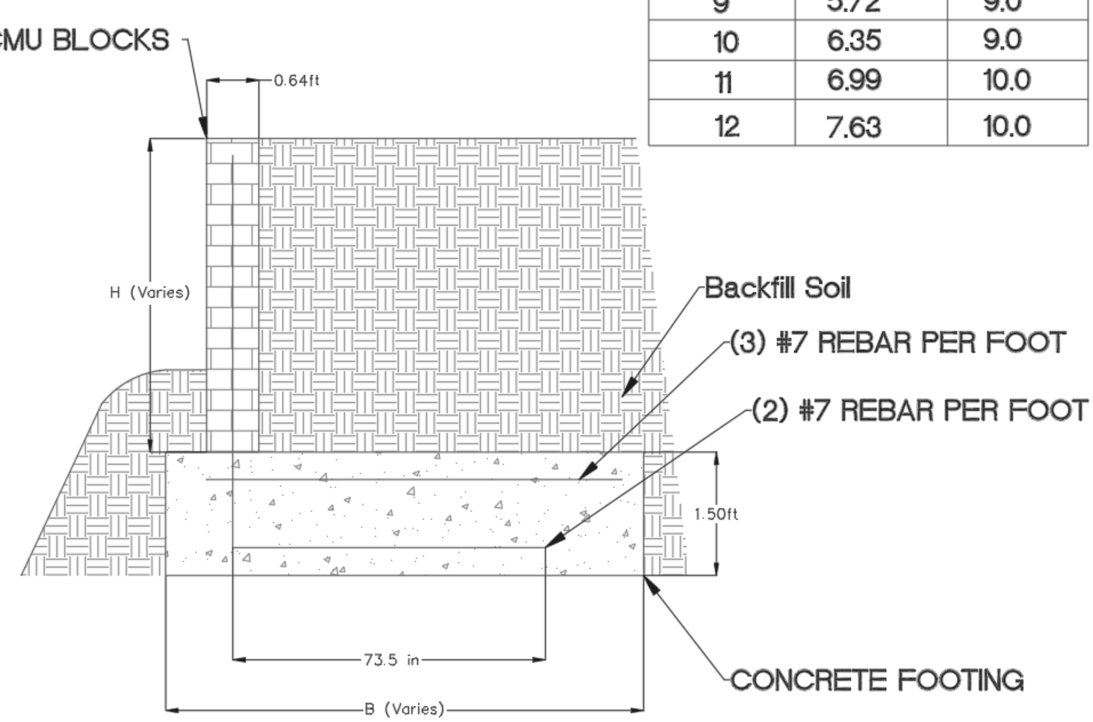

Figure 6-6: CMU retaining wall cross section

Figure 6-7 includes the profile of the wall, which shows that the wall includes steps at various locations along the top and bottom of the wall. The footing along the profile never exceeds the 30 inch frost depth and the top of the wall steps half a foot or less whenever the finish grade elevation is equal to the top of the wall elevation.

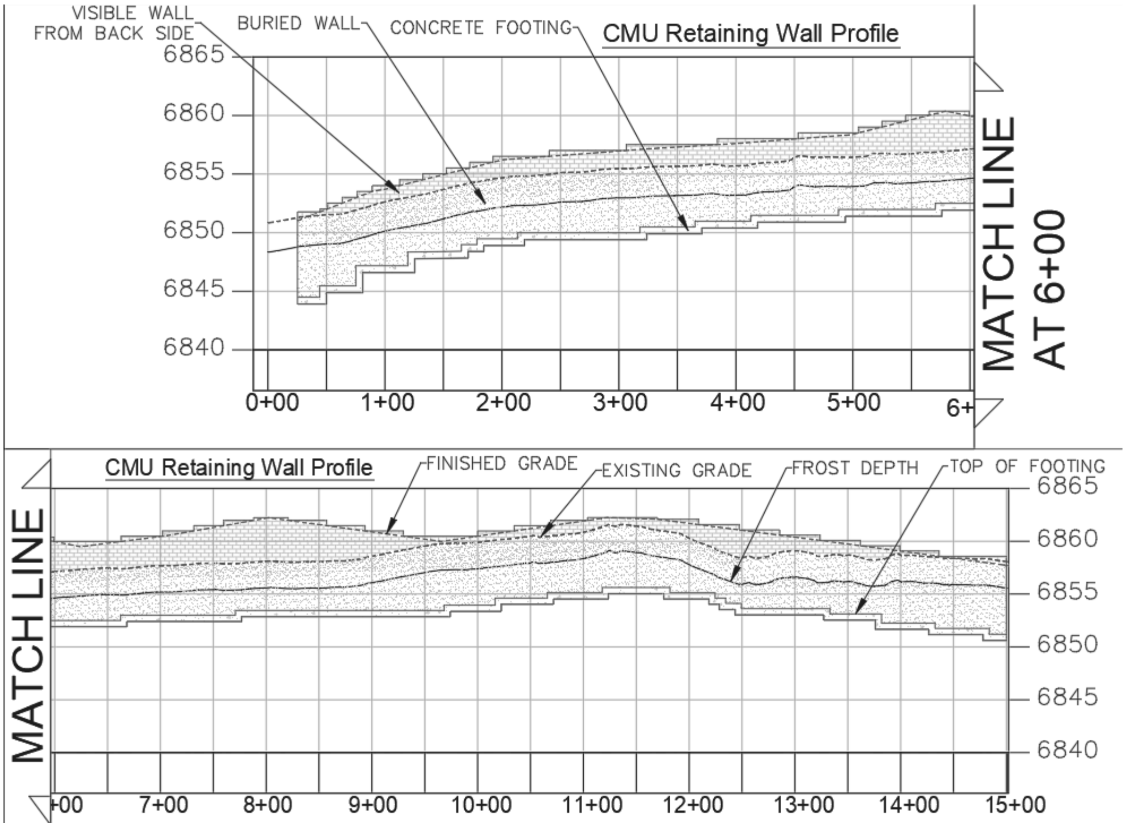

Figure 6-7: CMU retaining wall profile

Table 6-13 below, displays the complete calculations and variables used for checking that the CMU retaining wall meets the minimum factors of safety for bearing, sliding, and overturning.

Table 6-13: CMU retaining wall equations used

	Formulas			Notes
1	Rankine Coeffiecient of Active Pressure	$k_a =$	$\tan^2(45-\phi'/2)$	
2	Active Stress	$\sigma'_a =$	$\gamma*H*ka$	C=0
3	Resultant Active Pressure	$Pa=$	$\sigma'_a*H*.5+Pq$	
4	Applied Vertical Pressure of Soil	$Pv=$	$Pa*\sin(\alpha)$	
5	Applied Horizontal Soil Pressure	$P_H =$	$Pa*\cos(\alpha)$	
6	Factor of Safety for Overturning	$FS_{overturn}=$	$M_r/M_d \geq 2$	
7	Sum of Resistive Forces	$M_r=$	$\Sigma V*(Marm)+Pv*(Marm)$	
8	Driving Moment	$M_d=$	$P_H*(H/3)$	
9	Net Moment	$M_N=$	M_r-M_d	
10	Factor of Safety for Sliding	$FS_{Sliding}=$	$F_r/F_d \geq 1.5$	
11	Resisting Force	$F_r=$	$fr+fc+P_P$	fc=0
12	Driving Force	$F_d=$	P_H	
13	Force of Friction	$fr=$	$(Pv+\Sigma V)*\tan\delta$	
14	Soil-Pile Friction Angle	$\delta=$	$2/3*\phi'$	
15	Coefficient of Friction	Coefficient= $\tan(\delta)$		
16	Resultant Passive Pressure	$P_P=$	$\sigma'_P/2*D_f$	
17	Passive stress	$\sigma'_P =$	$k_P*\gamma*D_f$	C=0
18	Rankine Coefficient of Passive Pressure	$k_P=$	$\tan^2(45+\phi'/2)$	
19	Factor of Safety for Bearing	$FS_{Bearing}=$	$q_u/q_{max} \geq 3$	
20	Bearing Pressure on Toe	$q_{max} =$	$\Sigma V/B*(1+6e/B)$	
21	Eccentrictiy of Load	$e=$	$B/2-M_N/\Sigma V$	
22	Bearing Pressure on Heel	$q_{min}=$	$\Sigma V/B*(1-6e/B)$	
23	Unconfined Compressive Strength	$q_u=$	$c'*N_c*F_{cd}*F_{ci}+q*N_q*F_{qd}*F_{qi}+0.5*\gamma*B'*N_\gamma*F_{\gamma d}*F_{\gamma i}$	See Table 6.3 for factors
24	Bearing Pressure	$q=$	$\gamma*D$	
25	Effective Base Dimension	$B'=$	$B-2*e$	
26	Cohesion	$c'=$	0	
27	Bearing Capacity Factor	$N_c=$	60.78	For ϕ' = 37.9 degrees (values interpolated)
28	Bearing Capacity Factor	$N_q=$	48.33	
29	Bearing Capacity Factor	$N_\gamma=$	76.85	
30	Depth Factor	$F_{cd}=$	$F_{qd}-[(1-F_{qd})/(N_c\tan(\phi'))]$	For $D_f/B \leq 1$ and $\phi' > 0$
31	Depth Factor	$F_{\gamma d}=$	1	
32	Depth Factor	$F_{qd}=$	$1+2\tan\phi'(1-\sin\phi')^2 D_f/B$	
33	Angle of Resultant of ΣV and P_H	$\beta=$	$\arctan(P_H/\Sigma V)$	
34	Inclination Factor	$F_{ci}=F_{qi}=$	$(1-\beta/90)^2$	For β=30.33 degrees
35	Inclination Factor	$F_{\gamma i}=$	$(1-\beta/\phi')^2$	
36	Weight of Area 1	V1=	A1*γ (concrete)	
37	Weight of Area 2	V2=	A2*γ (concrete)	
38	Weight of Area 3	V3=	A3*γ (concrete)	
39	Weight of Area 3	V4=	A4*γ (soil)	
40	Weight of Area 4	V5=	A5*γ (soil)	
41	Weight of Area 5	ΣV=	V1+V2+V3+V4+V5	
42	Allowable Soil Bearing Pressure	qall=	qu/FS	2408.71459 psf

Table 6-14 below, displays some of the more important values used in the design checks.

Table 6-14: CMU retaining wall givens/assumptions

Determined Variable Values:			
Friction Angle	ϕ'	37.900	degrees
	ϕ'	0.661	radians
Unit weight	γ (soil)	109.370	pcf
Unit weight	γ (concrete)	150.000	pcf
Unit weight	γ (normal weight CMU)	125.000	pcf
Total Wall Height	H	7.219	feet
Footing Depth	D_f	2.500	feet
Active Coefficient	k_a	0.239	
Angle of Soil at Top of Wall	α	0.000	degrees
	α	0.000	radians
Active Stress	σ'_a	188.645	psf
Active Pressure	Pa	4480.892	lbs/ft
Surcharge (not from soil)	Pq	3800.000	lbs/ft
Vertical Pressure	Pv	0.000	lbs/ft
Horizontal Pressure	P_H	4480.892	lbs/ft

Table 6-15 below, displays the design check to ensure that factor of safety for overturning for the tallest section of the CMU retaining wall meets the required minimum. From the cells highlighted in green, it can be seen that the design meets the minimum required factor of safety of 3.0.

Table 6-15: CMU overturning factor of safety check

Overturning Check			
Area 1	A1	5.719 ft^2	
Area 3	A3	13.500 ft^2	
Area 4	A4	44.975 ft^2	
			Moment arm
Weight 1	V1	714.844 lbs/ft	0.817708333 ft
Weight 3	V3	2025.000 lbs/ft	4.5 ft
Weight 4	V4	4918.928 lbs/ft	5.067708333 ft
Toal Weight	ΣV	7658.771 lbs/ft	
Resisting Moment	M_r	34624.725 lb-ft/ft	
Driving moment	M_d	10782.146 lb-ft/ft	
Net Moment	M_N	23842.578 lb-ft/ft	
Factor of Safety	$FS_{overturn}$	3.211 ≥3	

Table 6-16 below, displays the design check to ensure that factor of safety for sliding for the tallest part of the CMU retaining wall meets the required minimum. From the cells highlighted in green, it can be seen that the design meets the required minimum factor of safety for sliding of 1.5.

Table 6-16: CMU sliding factor of safety check

Sliding Check		
Force of Friction	fr	3614.835
Force of Cohesion	fc	0.000
Passive Pressure	P_P	4130.149 lbs/ft
Resiting Force	F_r	7744.984
Driving Froce	F_d	4480.892 lbs/ft
Factor of Safety	$FS_{sliding}$	1.728 ≥1.5

Table 6-17 below, displays the design check to ensure that the tallest section of the CMU retaining wall meets the minimum required factor of safety for bearing capacity. From the cells highlighted in green, it can be seen that the design meets the minimum required factor of safety of 3.0.

Table 6-17: CMU bearing capacity factor of safety check

Bearing Capacity Check		
Soil-Pile Friction Angle	δ	25.267 degrees
	δ	0.441 radians
Passive Stress	σ'_p	3304.119 psf
Passive Coefficient	k_p	4.185
Width of Footing	B	9.000 feet
Eccentricity of Load	e	1.387 feet
Effective Width of Footing	B'	6.226 feet
Angle of Resultant of ΣV and P_H	β	30.3305 degrees
	β	0.52937 radians
Bearing Cpacity of Wall	qu	7226.144 psf
Bearing Pressure at Foundation Toe	qmax	1637.782 psf
Bearing Pressure at Heel	qmin	64.168 psf
Soil overburden	q	273.425 psf
Depth Factor	F_{cd}	1.066
Depth Factor	$F_{\gamma d}$	1.000
Depth Factor	F_{qd}	1.064
Inclination Factor	$F_{ci}=F_{qi}$	0.440
Inclination Factor	$F_{\gamma i}$	0.040
Factor of Safety	$FS_{bearing}$	4.412 ≥3

Table 6-13 through Table 6-17 displays the complete calculations for the tallest section of the CMU wall. All similar calculations for the various wall heights can be seen in Appendix F. This reinforced concrete masonry wall varies in height and depth, as seen from the associated dimensions schedule of Figure 6-6. The design uses a global footing thickness of 1.5 feet, but the wall slab, composed of the CMU blocks, varies from 9 to 12 stacked CMU blocks. The CMU block used for design is a split face, normal weight masonry block, of nominal dimensions 8" thick x 8" wide x 16" long. The specified compressive strength of the specified CMU block (f'm) is 1.5 ksi. The wall features a commonly used and easily constructed single wythe design, and is to be grouted at 16" spacing on center. No. 5 rebar is to be used in conjunction with this grouting, and lateral reinforcing wire to be placed as needed. The wall was designed utilizing a conservative approach, such that it should not fail under max loading condition, and is expected to be able to withstand not only the induced load of the FUTS trail, but also a maintenance vehicle driving on the FUTS trail being as large as an F-350, weighing 7762 lbs.

7 Final Design Recommendation

In order to determine the final wall design recommendation, a second decision matrix was performed, which comparatively evaluated the three alternatives that were chosen to be further developed after the preliminary decision matrix. This decision matrix featured a more in depth explanation of the 6 grading criteria and can be seen below in Figure 7-1 below. These grading criteria were composed of 6 major criteria to determine the most feasible option for the client. These criteria were:

1) The ability to easily implement drainage, such as weep holes, into the design.
2) The size of foundation as the railroad restricts the size of the foundation at the toe to 6 inches.
3) The amount of reinforcement required, evaluated from both economic and construction standpoints.
4) The aesthetics of the wall, based upon its cohesive appearance with the surrounding infrastructure.
5) Cost of construction and implementation.
6) The estimated time of construction.

Table 7-1: Final decision matrix

Decision Matrix Criteria	Concrete Cantilever Wall	Mechanically Stabilized Earth	Concrete Masonry Unit
Drainage Natrual and with the addition of Weep holes. Determination of the ability to add weep holes	All walls are able to add weep holes during or after the process of building. Drainage of all walls are similar and are determined to drain efficiently.		
	1	1	1
Foundation Size Size of foundation as the wall is restricted by the railroad and the FUTS trail for proposed Holiday Inn	This issue with the wall is the toe is too small to not design with an extra anchor.	No foundation is used on an MSE wall as reinforcement is built into the back of the wall, using gravity to anchor it.	This issue with the wall is the toe is too small to not design with an extra anchor.
	0	1	0
Required Reinforcement How much rienforcement is required to build the wall based on cost and the ability for contractor to impliment	Rebar will be needed for the reinforcement of the concrete, however, minimal rebar will be needed.	Reinforcement is used through out the wall and is needed to hold the stablized soil together.	Reinforcements are minimum as the foundation is concrete and will have #5 rebar running throughout.
	1	0	0
Wall Asthetics How the wall blends with natural surroundings and infrastructure	Wall is bleak and does not fit in with the landscape surrounding it. This can be painted or dye the concrete, however, is not practice in this application.	The wall will blend into the environment, however, wont match exisiting infrastructure as the wall that is located on the other trax pracel is a CMU wall.	The wall will match exisiting walls and is common in Flagstaff.
	-1	0	1
Estimated Material Cost The overall cost of materials for the contractor to build the 1500 ft wall	$88,200 (Doesn't include cost of transporting or rebar cost)	$115,920	$106,162.5 (CMU block can be made locally)
	1	-1	0
Estimated Construction Time The time it takes to construct the wall and the man hours that are required to impliment the wall	The estimated time of construction increased with the amount of concrete that is needed to construct the whole wall.	The wall reinforcements are assumed to be assembled easily, this is more time consuming as the reinforcements will take time to impliment.	This is a common wall in Flagstaff and is easily assemble using concrete blocks.
	-1	0	1
Sum	1	1	3

As can be seen in Table 7-1, the final wall recommendation is the CMU wall design. This design features a normal weight CMU split face brick with nominal dimensions 8" x 8" x 16" with a compressive masonry strength (f'm) equal to 1.5 ksi. The smallest section of the wall utilizes 9 blocks stacked, and stepping occurs by 1 block up to a maximum of 12 blocks. Type M mortar is to be used along the lateral joints of the CMU blocks, and every other block column is to be grouted. The grouted cells are to have one #7 rebar placed in the center of the cell. In the footing, three #7s per foot are to be placed four inches below the top of the footing. The wall was designed using a conservative approach, meaning that the wall should not fail under maximum loading conditions, which includes the surcharge of a maintenance vehicle weighing as much as 7762 lbs.

The client asked for a railing that would match the standards for the City of Flagstaff, which is shown in Figure 7-1. Using the engineering detail 14-01-010 from the city, the contractor will attach the railing to the top of the CMU retaining wall [10]. It will be up to the contractor how the railing will be mounted to the wall, however, the contractor is required to follow the details in the construction plan set.

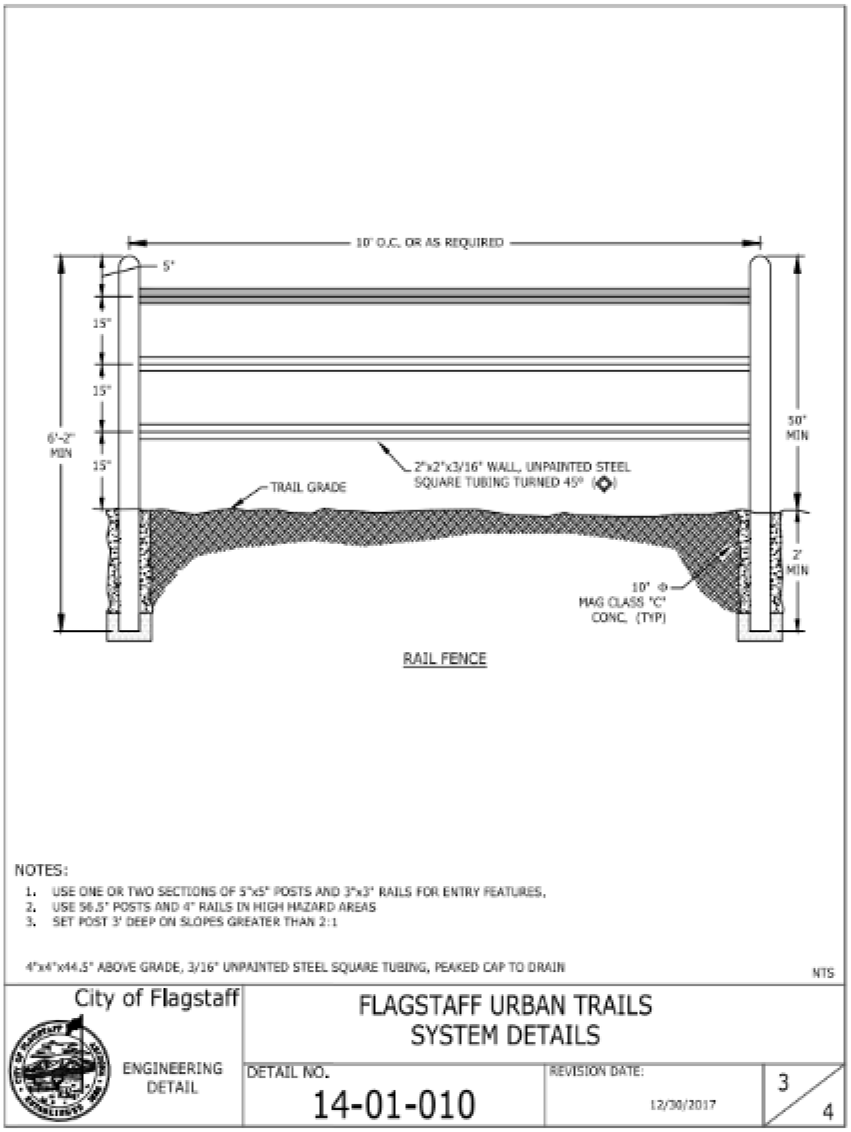

Figure 7-1: Flagstaff Urban Trail handrail detail

8 Impacts the Design

8.1 Environmental Impact

The environmental impact this project may have would be the large amount of concrete that is required for the footing. The footing for the wall requires concrete that will most likely have to be transported from Phoenix, Arizona. The transportation of the concrete and the pouring will produce CO2 that pollutes the air. The construction of the wall will cause noise pollution for nearby businesses.

8.2 Social Impact

The social impact this project would have is the extension of the FUTS path with the retaining wall being located next to the path. The retaining wall with the handrail will support the path extension and provide better access through the area for pedestrians and bike. The handrail will also help prevent people from walking or falling into the railroad. This retaining wall will also have the same look as the existing retaining walls on the west side of the 4th Street bridge so the proposed wall will continue the aesthetic look of the surrounding location.

8.3 Economic Impact

The primary economic impact of this project is to support local businesses, from buying construction materials from local manufacturers in Flagstaff. The CMU blocks that are proposed in the retaining wall design can be manufactured and purchased in Flagstaff. Masonry contractor are also common in Flagstaff so this project would also support their business. Also, the handrail used in the proposed design is a Flagstaff standard handrail with is used all around the city so the manufacturing and installation of that handrail will also be done locally.

9 Cost of Implementing Design

The total costs of implementation for the CMU design alternative is displayed below in Table 9-1. These costs were developed using the 2005 version of the RS Means Cost of Construction book [8]. The costs estimates form RS Means Cost of Construction include the labor and material costs. Maintenance may also be required which could include spraying the wall with salt to reduce the freeze thaw process that occurs in Flagstaff as well as cleaning weep holes and checking for cracks. The majority of the maintenance that will be required for this project will be on the FUTS trail because the trail will have users. The maintenance for this project will be conducted by the City of Flagstaff.

Table 9-1: Engineering opinion of proposed construction cost estimate

EOPC- Engineering Opinion of Proposed Construction					
Item Number	Quantity	Units	Description of Item	Unit Cost	Cost
Dirt Excavation and Demolition					
1	$2,778	CY	Dirt Excavation and Removal	$25	$69,444
				Total	$69,444
Retaining Wall Proposed Cost and Items					
2	$833	CY	Concrete for Foundation	$93	$77,053
3	$38,063	LF	#7 Rebar	$0.75	$28,547
4	$10,500	SF	Unit Masonry Assemblies (Split Face 8" Thick)	$9.5	$99,750
5	$1,500	LF	Cost of FUTS Handrail	$95	$136,500
6	$75	LF	PVC Pipe for Weep holes (4")	$2	$150
7	$3,375	CY	Granular Coarse Fill (18'X18") along wall	$25	$84,375
				Total	$426,374
				Total Cost:	$495,819

10 Summary of Engineering Work

Summaries of the proposed engineering design hours and the actual engineering design hours completed are shown in Figures 10-1. Comparing the two tables, one can see a number of discrepancies between what was proposed and what actually occurred. First it was originally proposed that the field work would take approximately 30 hours, but as discussed in the Field Work Plan, the soil sample acquisition methods incurred some unexpected difficulties that ultimately simplified sample collection greatly, so that the actual sample collection took only 5.5 hours total. Second, the scope of the soil testing expanded beyond that which was originally proposed, but took only 2 hours longer than what was originally expected. Third, as discussed in Section 11.0, the existing surface water runoff conveyance of the area surrounding the site and the proposed storm drain on site greatly simplified the work on hydrology and hydraulics, cutting 84 hours of proposed work to 18 hours of actual work for those tasks. Last, the inherently ambiguous nature of project management led to a total discrepancy of approximately 70 hours (about 25%), compared to the proposed, across the sum of all the subtasks associated with that major task.

Table 10-1: Comparison of Proposed versus Actual hours

Task	Hours Per Staff Member						Total Proposed Hours	Total Actual Hours
	Proposed			Actual				
	Sr. ENG	Assoc. ENG	EIT	Sr. ENG	Assoc. ENG	EIT		
1.0 Site Investigation	1	1	1	3	3	3	9	3
2.0 Field Sampling								
2.1 Field Work Plan	2	1	9	1	1	7	9	12
2.2 Field Work	0	0	5.5	1	9	20	30	5.5
3.0 Geotechnical Analysis								
3.1 Sieve Analysis	0	2	8	1	2	15	18	10
3.2 Hydrometer	3	1	10.5	1	2	15	18	14.5
3.3 Atterberg Limits	0	2	9	1	2	15	18	11
3.4 Sand-Cone Test	4	2	8	1	2	15	18	14
3.5 Tri-axial	3	9	13	1	2	15	18	25
3.6 Consolidation	3	5	14.5	1	2	15	18	22.5
3.7 XRF Contaminants Test	0	0	6					6
3.8 Direct Shear	0	0	7	1	2	15	18	7
4.0 Hydrology								
4.1 Watershed Delineation	0	0	9	1	3	8	12	9
4.2 Time of Concentration	0	2	0	2	6	16	24	2
4.3 Storm Event Runoff	0	0	1	1	3	8	12	1
5.0 Hydraulics								0
5.1 LID Development	0	0	1	1	3	8	12	1
5.2 Pre/Post Floodplain Map	0	0	1	1	3	8	12	1
5.3 Proposed Water Disbursement	0	0	4	1	3	8	12	4
30% Milestone								
6.0 Wall Design Process								
6.1 Wall Designs	6	15.5	24	4	48	38	90	45.5
6.2 Plan and Profiles	0	5	15	1	1	7	9	20
6.3 Final Wall Design Selection	0	0	4	2	6	1	9	4
60% Milestone								
7.0 Impacts	3	3	3	3	3	3	9	9
8.0 Project Management								
8.1 Meetings	0	2	3					5
8.1.1 Team Meetings	12	12	12	10	10	10	30	36
8.1.2 Grading Instructor Meetings	6	6	11	15	15	15	45	23
8.1.3 Technical Advisor Meetings	0	3	6	8	8	8	24	9
8.1.4 Client Meetings	2.5	4.5	6.5	2	2	2	6	13.5
8.2 Schedule and Resource Management	2	3	7	16	3	1	20	12
8.3 Deliverables								0
8.3.1 30% Submittal and Revisions	0	10	7	1	6	17	24	17
8.3.2 60% Submittal and Revisions	4	11	17.5	1	6	17	24	32.5
8.3.3 90% Submittal and Revisions	2	0	11	6	12	30	48	13
8.3.4 100% Submittal	9	12	24	1	6	17	24	45
8.3.5 Website	2	4	6	4	10	14	28	12
PROJECT TOTALS	61.5	113	261.5	92	182	356	630	436

The proposed schedule for this project started August 25th with a site investigation and ended December 10th with a final presentation, report, and website. The proposed schedule is located on page 49 and shows the original schedule. The actual schedule has the same start and finish dates as the proposed schedule, but the project progression is much different as shown on page 50 The proposed schedule projected that the field sampling would be completed by September 12th and the geotechnical testing and analysis would be completed by October 8th, but the actual schedule shows that the field sampling was not completed until September 23rd and the geotechnical testing and analysis was not completed until October 14th. The field sampling took much longer than expected because it took about two weeks to get the field sampling and lab access approved by the lab coordinator. This was unexpected and caused the project to have a delay. The proposed schedule projected they hydrology and hydraulics portion of the project to start on September 13th and end on October 2nd, while the actual schedule shows that the hydrology and hydraulics did not start until October 15th and ended on October 28th. These tasks were expected to be completed during the geotechnical analysis but the soil testing and analysis was much more work than expected because the team was only able to complete 1-2 tests per week. This delay caused the hydrology and hydraulics not to be started until after the geotechnical analysis was completed. The proposed schedule also shows that the wall design process was to start on October 9th and end on November 8th, followed by a week to work on project impacts. The actual schedule shows that started on October 15th and ended on November 15th, followed by 3 days for impacts. The wall design process began during the hydraulics and hydrology section of the project to try to make up some time. The wall design options were expected to be completed at the same time but they were actually completed one after the other, with some overlap between the three designs. These wall designs took longer than expected but the impacts were shortened to make sure the project was completed on time.

11 Summary of Engineering Costs

The proposed summary of engineering costs, can be seen in Figure 11-1 below. The summary of the actual costs can be seen in Figure 11-2, below Figure 11-1. Comparing the two, it can be seen that the design fee was approximately ⅔ of the proposed design fee. The greatest contributors to this discrepancy were the discrepancies between proposed and actual hours on the project due to soil testing and the simplified hydrological analysis. It can be seen in Figure 11-1 that the actual cost of engineering services incurred by the client was $60,815.

Table 11-1: Proposed cost of engineering service

Item	Description	Cost per Unit	Number of Units	Units	Cost
1.0 Personnel:	Sr. Eng.	$200.00	92	Hours	$18,400.00
	Assoc. Eng.	$140.00	182	Hours	$25,480.00
	EIT	$90.00	356	Hours	$32,040.00
	Total Personnel:				$75,920.00
2.0 Supplies:	Lab Rental	$100.00	108	Hours	$10,800.00
3.0 Total					$86,720.00

Table 11-2: Actual cost of engineering service

Item	Description	Cost per Unit	Number of Units	Units	Cost
1.0 Personnel:	Sr. Eng.	$200.00	67.5	Hours	$13,500.00
	Assoc. Eng.	$140.00	126	Hours	$17,640.00
	EIT	$90.00	282.5	Hours	$25,425.00
	Total Personnel:				$56,565.00
2.0 Supplies:	Lab Rental	$100.00	42.5	Hours	$4,250.00
3.0 Total					$60,815.00

12 Conclusion

The objective of this project was to produce three possible retaining wall design alternatives which would adequately support the proposed FUTS trail and Holiday Inn. Prior to designing the alternatives, soil analysis was needed to determine the type of soil that was retained. The soil testing along with the determination of other soil property factors, were used to evaluate the wall designs. The three alternatives were determined based on decision matrices to narrow the best option. This final recommendation was determined to be a CMU wall that was adjusted based on the existing grade and the client's proposed grade. The project was completed on time.

13 References

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[10]Flagstaff.az.gov. (2019). *Engineering Design Standards | City of Flagstaff Official Website*. [online] Available at: https://www.flagstaff.az.gov/482/Engineering-Design-Standards [Dec. 2019].
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Appendices

Appendix A- Field Safety and Sampling Plan

<u>Trax Retaining Wall Team Field Work Plan</u>

Wall E. Wallerson Inc. and Associates

Josh Endersby
Hunter Scnoebelen
Chris Cook

9/18/2019

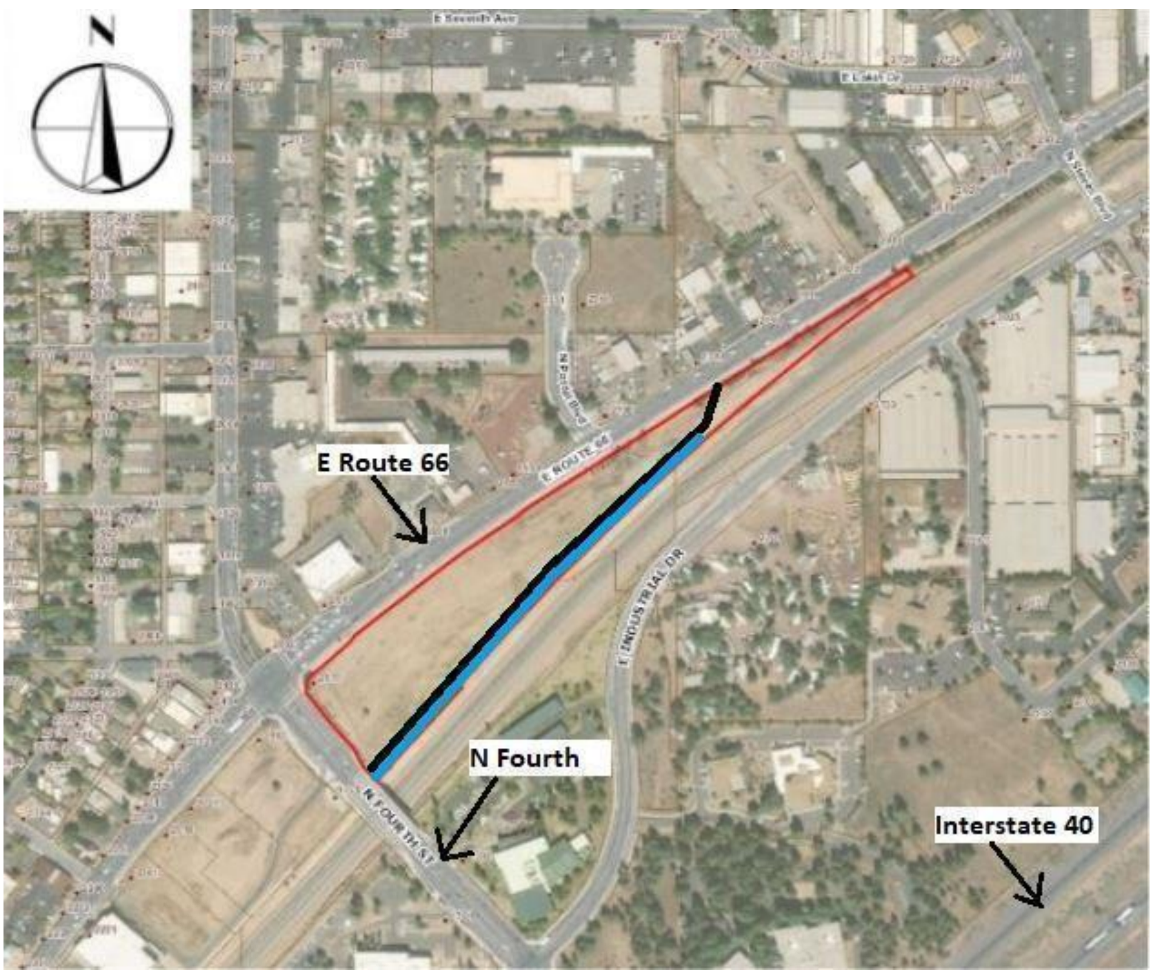


Figure 1: Project Location in Flagstaff, Arizona

1.0 Sampling Location:

Figure 2.0: Sampling Location

2.0 Sampling

The Trax Team plans to acquire a total of 4-8 samples from 4 soil piles. The soil piles, shown in Figure 2.0: Sampling Locations, show that the sampling will start from the undeveloped lot West of Fourth Street while the rest of the sampling will occur on the East side of the site. Samples will be taken at multiple locations around the soil piles and placed in a five-gallon bucket. Each pile samples will be placed in different buckets and labeled accordingly.

2.1 Equip

The equipment that will be used for sample collection will include:

- 3 shovels
- 3 pairs of working gloves
- maximum of 8 five-gallon buckets
- Hand auger (if necessary)
- tape and sharpie for labeling samples.

2.2 Sample Protocol

Per each soil pile, the soil on the outside will be scraped off of the pile to eliminate weathered soil from the sample. The samples will then be in areas around the pile, as shown in Figure 3.0. The samples will be taken as close to the center of the pile as possible to provide an accurate representation of the entire pile. The soil sampled from each location around the pile will be placed in a bucket and label to prevent confusion. The labels will include a "T" for Trax, and a number representing which pile the soil was taken from. An example would be "T-1" for pile 1, if multiple sample buckets are used on the same pile the labeling would be "T-1.1" and "T-1.2".

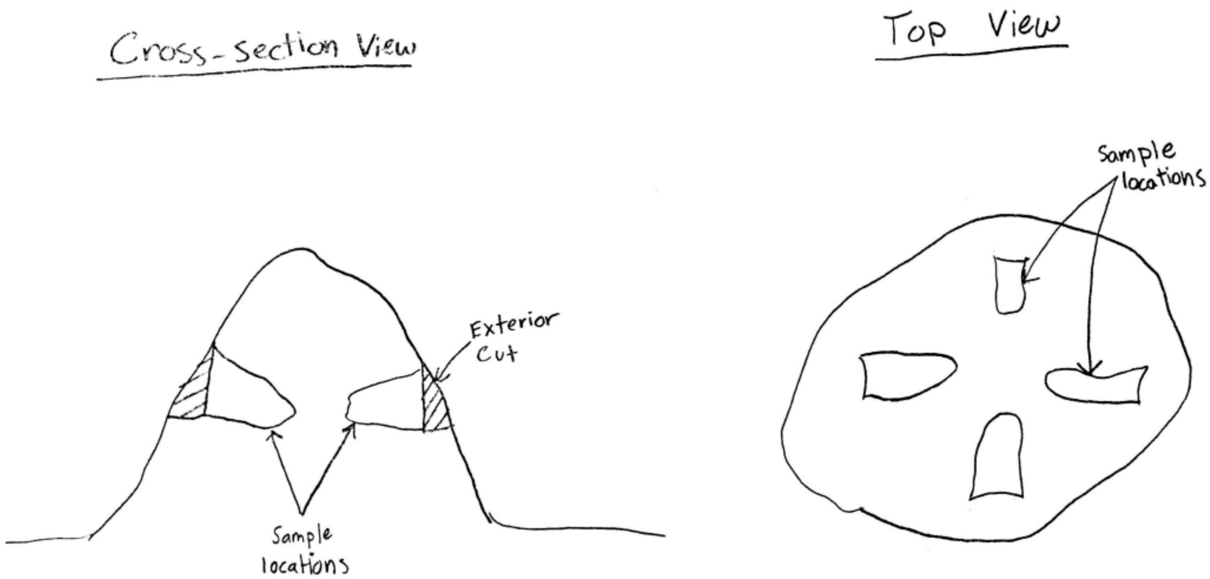


Figure 3.0: Sapling Method

2.3 Deviations from Plan

In the event that the team is unable to acquire the samples in the manner(s) described above, the following deviations will be executed as necessary.

1. If dense, large rock is uncovered, preventing the acquisition of a sample, then smaller samples will be taken at any possible location around the pile.
2. If an acceptable sample cannot be obtained for lab testing, the pocket penetrometer and Torvane tests will be used in place of the triaxial test.
3. After speaking to the client, the team is aware that there is a large amount of fill and "bad soil" on the site. If the tests or samples indicate something other than these results, the client will provide the team with the geotechnical report to use for design
4. In the very worst-case event, that no good sample can be acquired, the team will discuss with the Technical Advisor and Client the option to entirely replace the geotechnical testing with a site survey.

3.0 Safety

In the event of an emergency, the nearest hospital is the Flagstaff Medical Center, located at 1200 N Beaver St, Flagstaff, AZ, 86001. This location is approximately 3 miles or 8 minutes away from the site. The map below shows the approximate times and distances of alternate routes to the hospital.

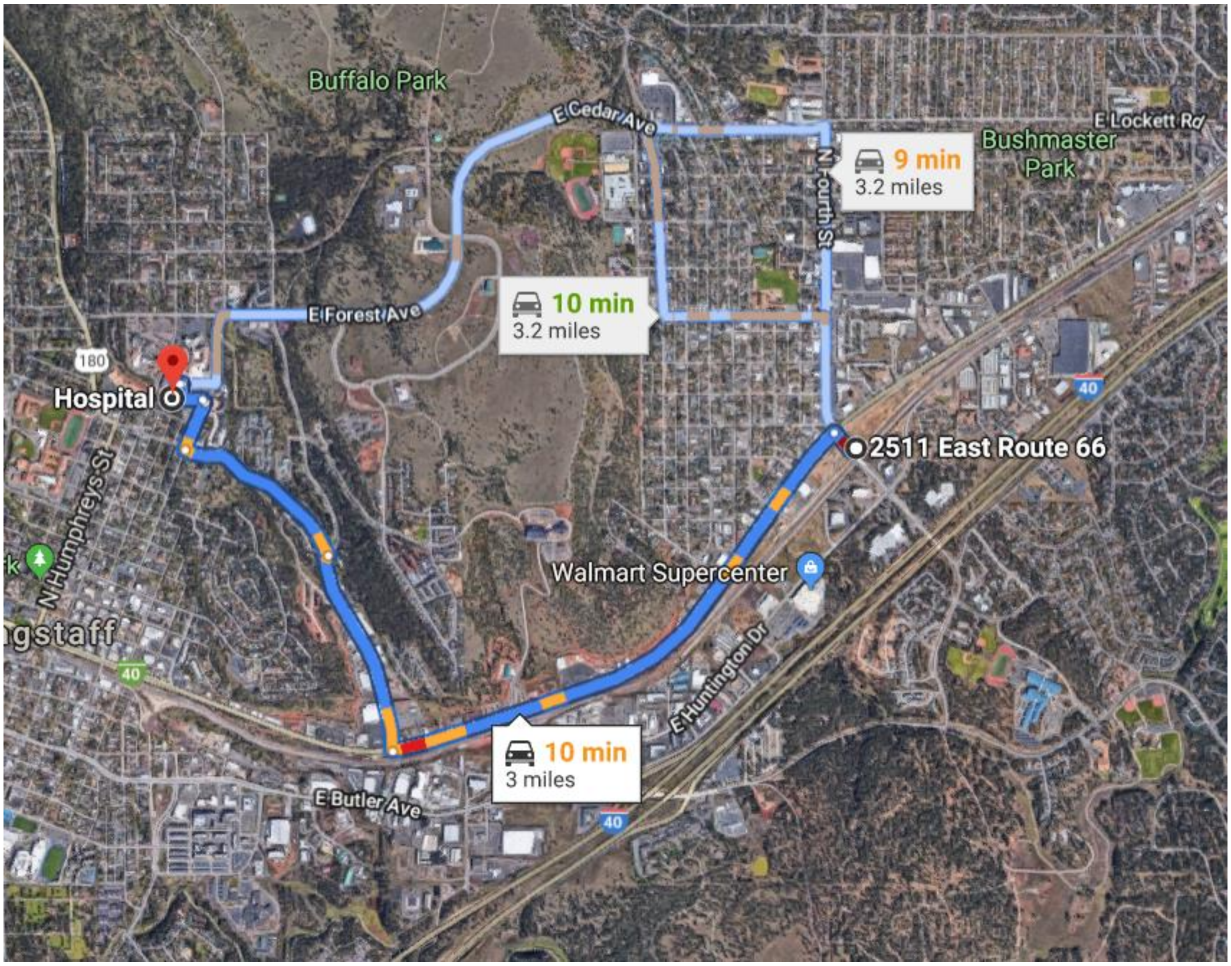


Figure 3.0: Emergency Route Map

The map shows that the two main routes are either:

West along Route 66 to Switzer Canyon Drive,
North to San Francisco Street,
Hospital on left

Or

Go North on Fourth Street,
(Sub-option: Take E 6th Ave to N West St)
To West on E Cedar Ave
E Cedar turns into E Forest Ave
Follow E Forest Ave West to San Francisco Street
South on San Francisco St,
Hospital on Right.

NAU Field Safety Checklist

This form is designed to assist the Principal Investigator (PI), or Supervisor with assessing potential hazards of fieldwork. **The completed checklist must be shared with all the members of the field team and a copy must be kept on file on campus.** Multiple trips to the same location can be covered by a single checklist, as long as any changes in hazards and/or participants are documented. NAU's Regulatory Compliance groups are available to review these plans, and will conduct periodic reviews of departmental checklists.

Before you go:

¨This checklist must be completed, with a copy maintained on campus, prior to departure for any fieldwork.
¨Prepare first aid kit and any documentation needed (SOPs, Chemical Safety Data Sheets, etc)
¨Assemble and check safety provisions
¨Check to assure all required immunizations are current for all team members
¨Check to assure all emergency health care and insurance requirements have been met.

Principal Investigator/Supervisor: Bridget Bero

Type of Field Work: Academic Field Trip Field Research Observation Other

Dates of Travel:
September 23, 2019

Mode of Transportation:
Personal Vehicle

Location of Field Work:
Country: USA Geographical Site: 4th Street/Route 66

Nearest City: Flagstaff, AZ Distance from

Site: 0 Miles Nearest Hospital/Distance (Attach map when applicable):

See attached

Field Work:
The team plans to sample soil from existing piles with shovels and buckets. The samples will then be transported back to the soils lab in the engineering building.

No-Go Criteria:
High winds, thunderstorms (lightning within 6 miles or 30 seconds for thunder to sound)

Emergency Procedures: See attached
University Contact (Name/ Phone): Adam Bringhurst/435-668-6799
Local Field Contact (Name/ Phone): Stephen Irwin/928-242-5641
Special Medical Requirements: None
First Aid Training: None
Physical Demands: Carrying and lifting 5-gallon buckets filled with soil. Shoveling the soil from test piles.

Risk Assessment: Please list identified risks associated with the activity or the physical environment and the appropriate safety measures to be taken to reduce the risks (personal protective equipment, training, SOPs, etc); Include a separate sheet if necessary. Attach Safety Data Sheets (SDSs) and training documentation for any chemicals that will be used.

Identified Risk	**Safety Measures**
Temperature Extremes	Team will not go out in the field if temperatures are below 30 or above 90 degrees.
Cuts From Vegetation	Wear closed toe shoes and pants
Plants/Insect Allergies	Route to hospital planned, and first aid kit in nearby car
Tripping	Work in the daytime and stay back from the road property
Blisters	Wear work gloves

Animal Studies: A field study is defined as any study conducted on free-living wild animals that does not involve an invasive procedure or materially alter the behavior of the animal under study. In order to help you determine if your study fits this criteria, please answer the following questions.

1. **Does your study greatly disturb the animals under study?** YesNo
   (ex. testing predator vocalization, supplemental feeding, nest manipulation)
2. **Does your study involve an invasive procedure?** YesNo
   (ex. blood sampling, tagging)
3. **Does your study cause potential harm/injury to the animal?** YesNo
   (ex. net and trap capture, bagging)

If you answered **YES to any** of these questions, your study involves invasive procedures or materially alters the behavior of the animal under study. Please fill out the full IACUC protocol application form. http://www.research.nau.edu/compliance/iacuc/

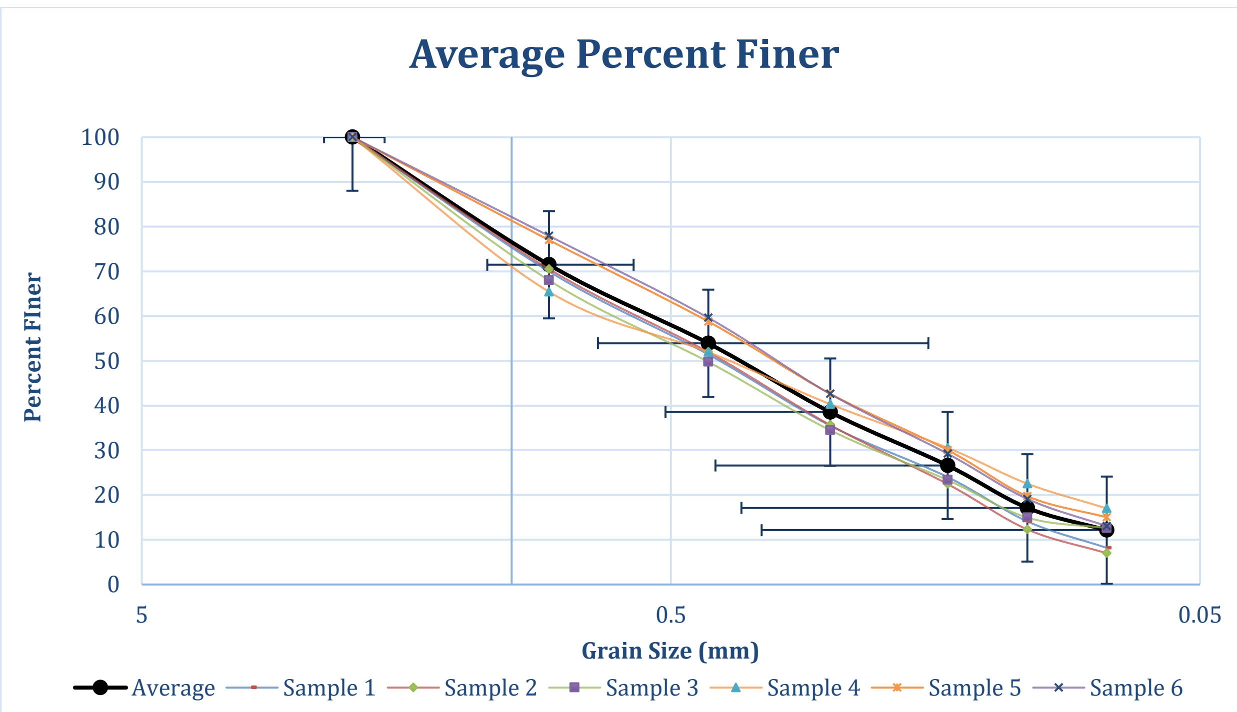
If you answered **NO to all three** of these questions and your study will only involve observation of free ranging animals, then an IACUC protocol is not required.

Field Team Membership (Please list the names of all members of the field team, and the Field Team Leader.) *Include a separate sheet if necessary.*
Name/Cell Phone Number (if applicable on site)
1. Chris Cook, 951-970-0947
2. Josh Endersby, 760-468-9711
3. Hunter Schnoebelen, 623-680-8462

Appendix B-1: Soil Test Results: Particle Size Distribution

Average Results

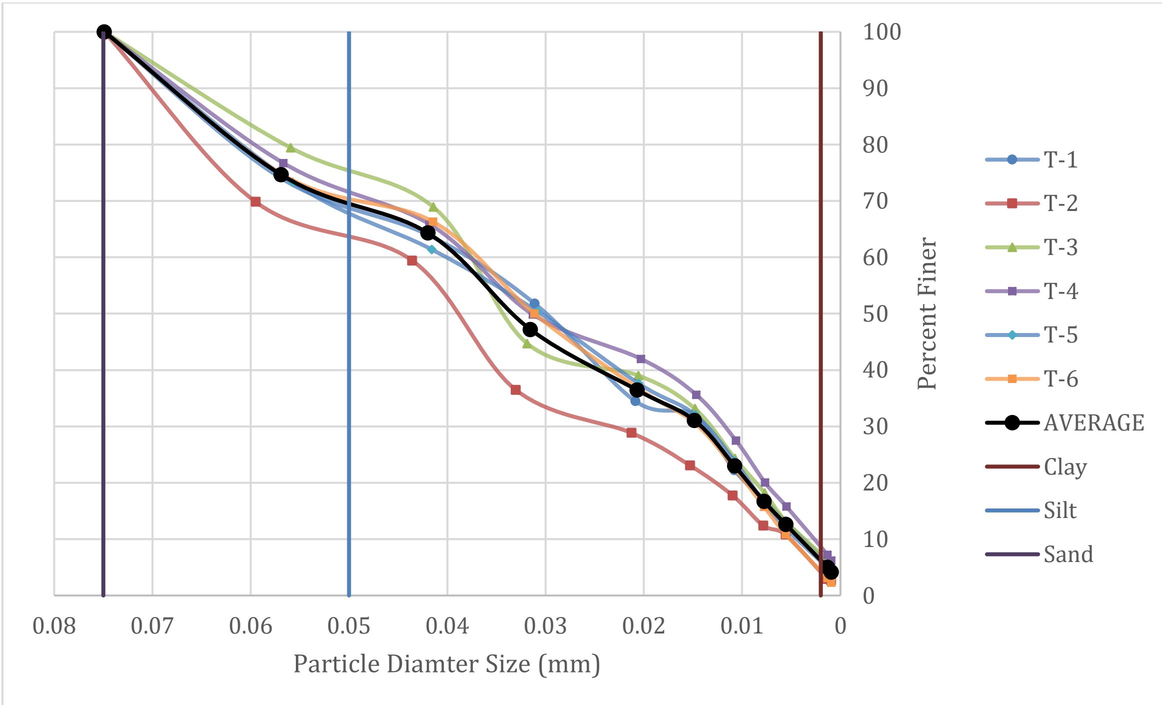
Percent Finer									
Sieve NO.	**Size (mm)**	**Sample #1**	**Sample #2**	**Sample #3**	**Sample #4**	**Sample #5**	**Sample #6**	**Average**	**STD DEV**
10	2	100.00	100.00	100.00	100.00	100.00	100.00	100.00	0.00
20	0.85	69.98	70.51	67.99	65.43	76.98	77.92	71.47	4.97
40	0.425	51.43	51.94	49.76	51.96	58.78	59.61	53.91	4.18
60	0.25	35.51	35.64	34.48	40.30	42.64	42.59	38.53	3.75
100	0.15	24.01	22.45	23.35	30.55	30.05	29.23	26.61	3.71
140	0.106	14.09	12.30	14.95	22.53	19.72	19.12	17.12	3.93
200	0.075	8.18	6.99	12.61	17.00	15.00	13.03	12.13	3.87
Pan	N/A	3.05	2.45	6.40	11.28	8.16	6.82	6.36	3.29



Appendix B-2: Soil Test Results: Hydrometer

Average Results

Sample	Time (min)	% Finer	RcL	L (cm)	A	D (mm)	T ©	
AVG	0	100				0.0749		STD DEV
	0.5	74.66129	1	16.562	0.013048	0.056922		3.24
	1	64.31705	1	17.22709	0.013048	0.041976	21.83	3.47
	2	47.21005	1	17.89218	0.013048	0.031555		5.79
	5	36.44125	1	18.55727	0.013048	0.02068		4.49
	10	31.08584	1	19.22236	0.013048	0.014872		4.27
	20	23.01147	1	19.88745	0.013048	0.010776		3.21
	40	16.67164	1	20.55255	0.013048	0.007758		2.57
	80	12.59113	1	21.21764	0.013048	0.005549		1.86
	1440	5.026157	1	21.88273	0.013048	0.001335		1.73
	2880	4.130919	1	22.54782	0.013048	0.000947		1.45

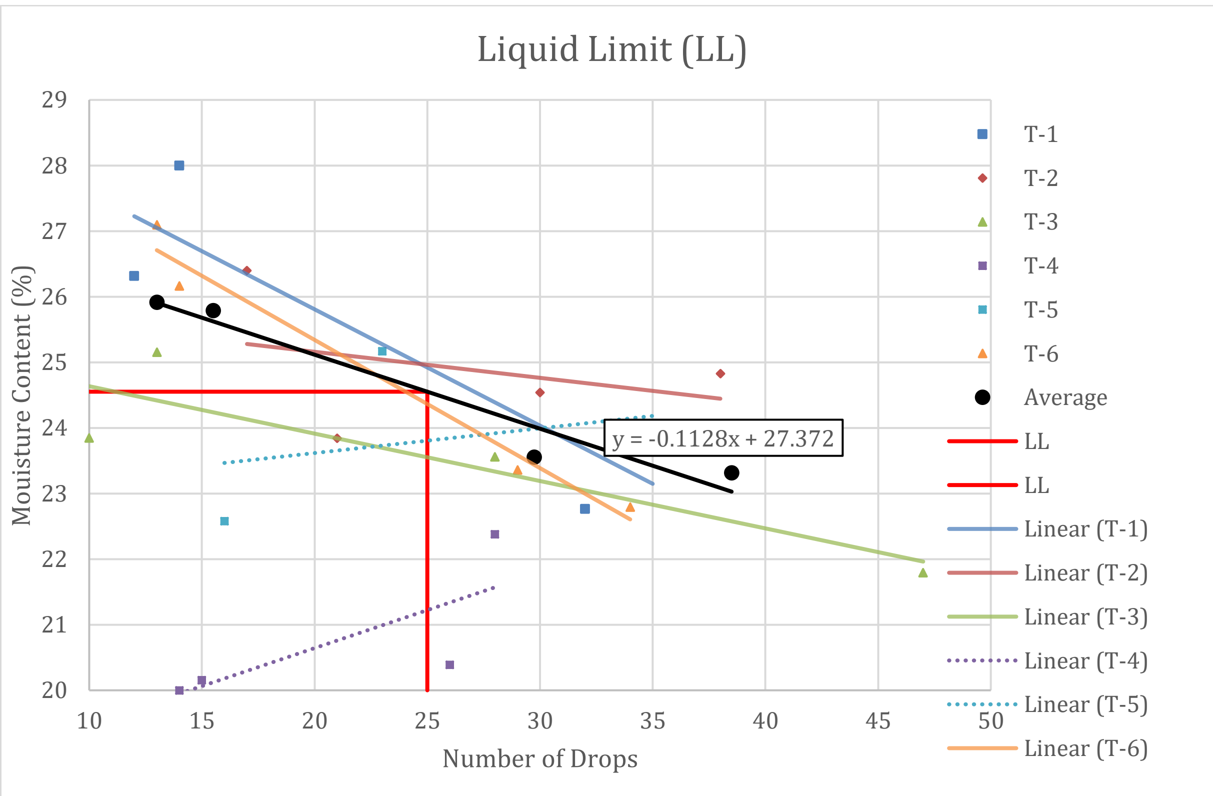


100
90
80
70
60
50
40
30
20
10
0
0.08
0.07
0.06
0.05
0.04
0.03
0.02
0.01
0
Particle Diamter Size (mm)
Percent Finer
T-1
T-2
T-3
T-4
T-5
T-6
AVERAGE
Clay
Silt
Sand

Appendix B-3: Soil Test Results: Atterberg Limits

Average Results

Plastic Limit						
Moisture Can ID	T-1	T-2	T-3	T-4	T-5	T-6
Mc (g)	19.5	13.3	19.8	13.6	13.2	13.3
Mm (g)	31.6	25.2	27.5	24.2	31	22.4
Md (g)	29.2	23.2	26	22.2	27.7	20.7
w (%)	24.74	20.20	24.19	23.26	22.76	22.97
PL (%)	24.74	20.20	24.19	23.26	22.76	22.97
AVG PL (%)	23.02			± 1.58		

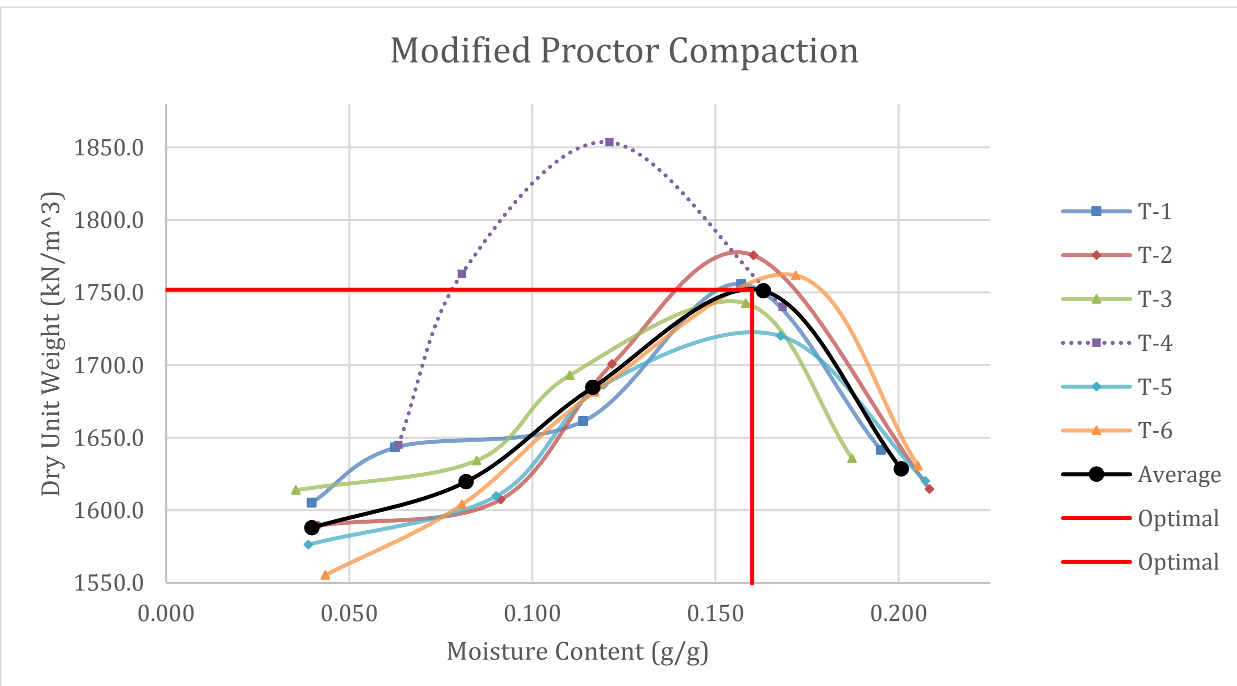


LL	24.552

Appendix B-4: Soil Test Results: Modified Proctor Compaction

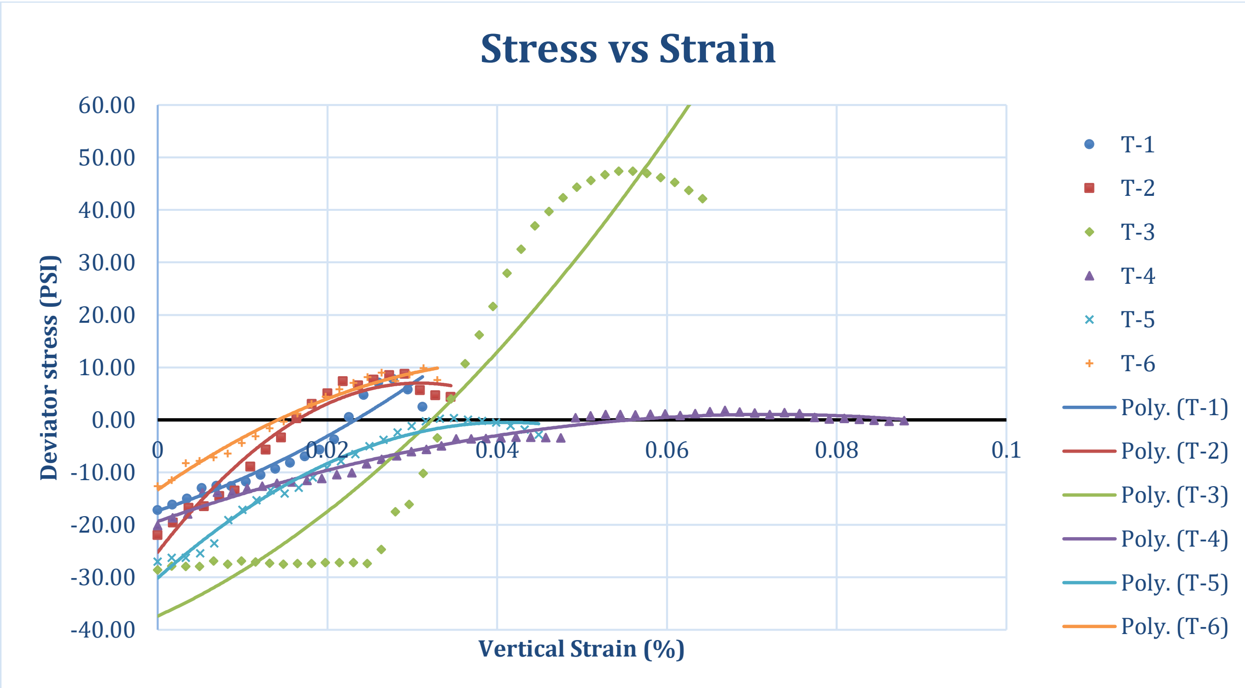
Average Results

Modified Proctor Compaction- Average					
Trial	1	2	3	4	5
moisture content	0.040	0.082	0.116	0.163	0.201
Std Dev Moisture Content	0.003	0.012	0.005	0.006	0.009
weight of compacted soil	1571.9	1675.7	1803.6	1919.3	1845.2
moist unit weight	1667.6	1777.8	1913.6	2036.2	1955.3
dry unit weight	1588.1	1619.7	1684.6	1751.2	1628.6
Std Dev Dry Unit Weight	23.3	17.8	14.8	21.1	11.0
Optimal dry unit weight	1752				
Optimal dry unit weight (lb/ft^3)	109.37				



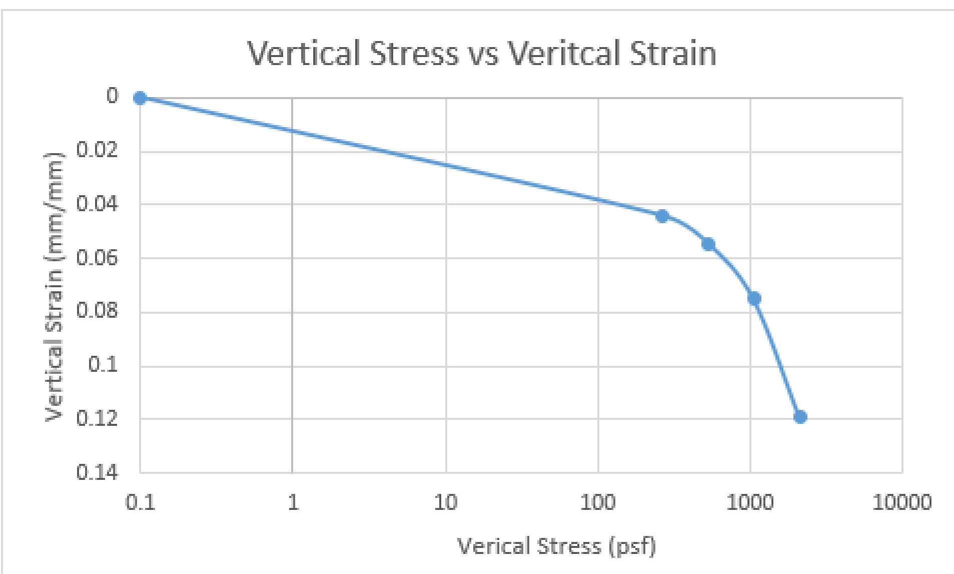
Appendix B-5: Soil Test Results: Unconfined, Unconsolidated Triaxial Compressive Test

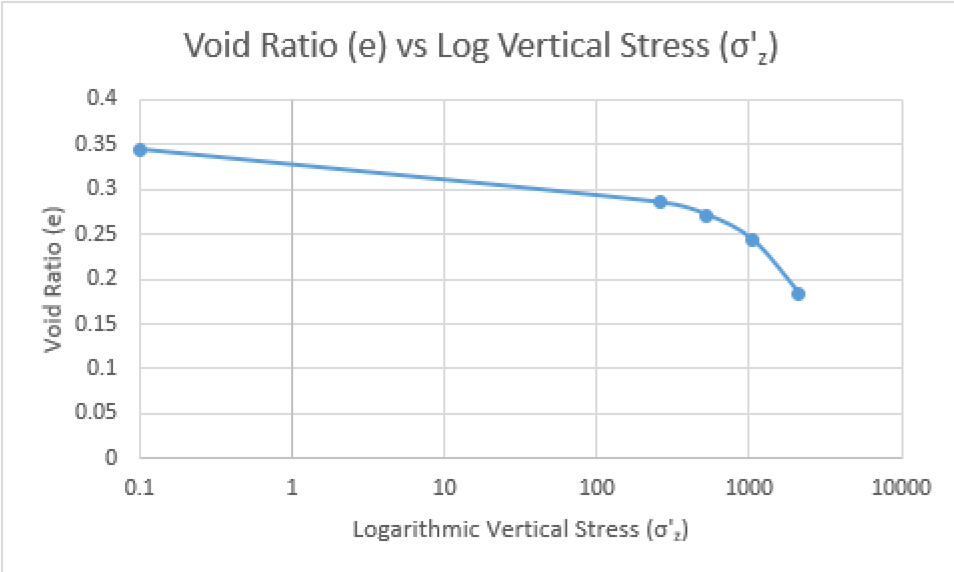
Results



Appendix B-6: Soil Test Results: Consolidation

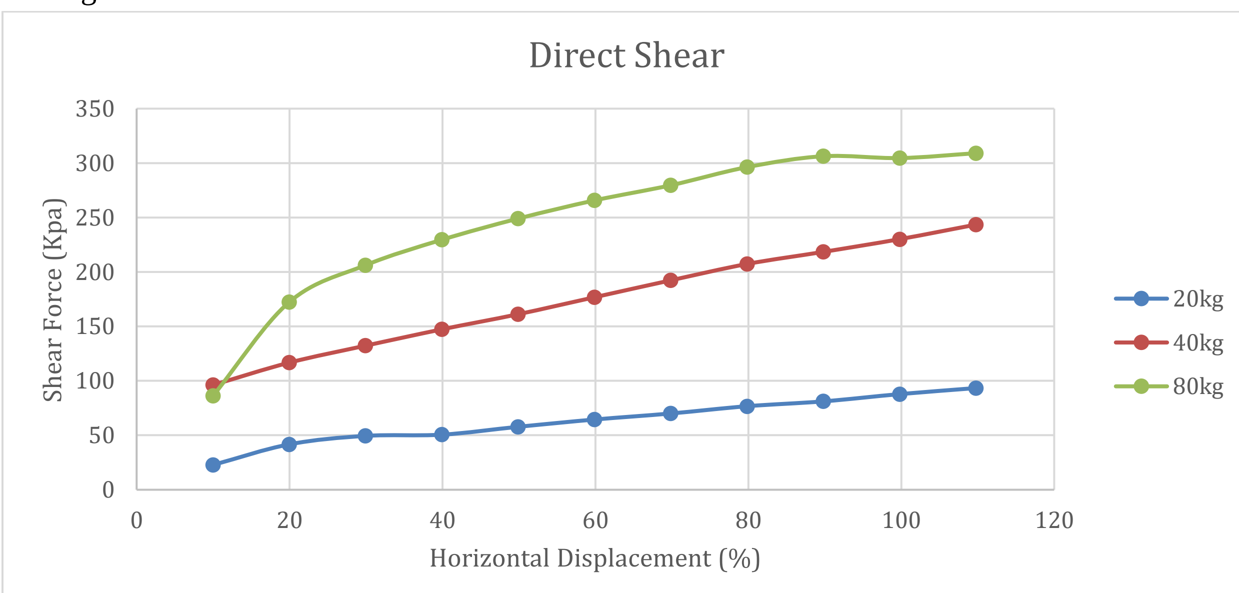
Average Results

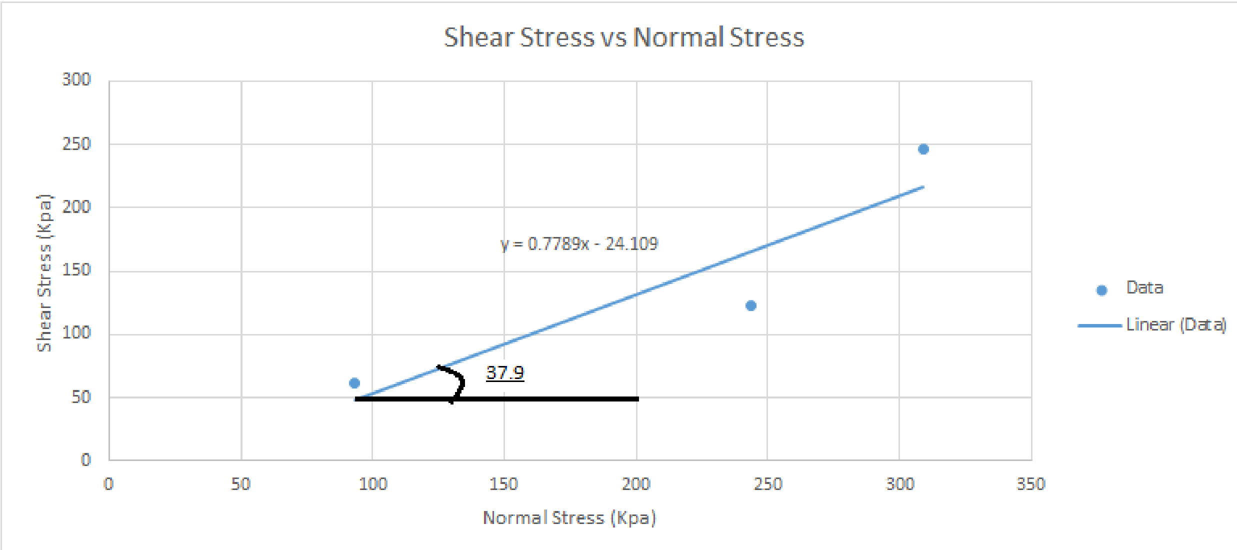




Appendix B-7: Soil Test Results: Direct Shear

Average Results





Shear Stress vs Normal Stress
300
250
200
150
100
50
0
Shear Stress (Kpa)
0
50
100
150
200
250
300
350
Normal Stress (Kpa)
y = 0.7789x - 24.109
37.9
Data
Linear (Data)

Appendix C: Geotechnical Report

Geotechnical Report

Trax Team: Retaining Wall for Proposed Holiday Inn

Site Address:
2511 E Route 66
Flagstaff, AZ 86001

Prepared For:
Stephen Irwin
Shephard Wesnitzer INC.
110 W. Dale Ave
Flagstaff, AZ 86001

Prepared By:
Wall E. Wallerson & Associates Inc.
Flagstaff, AZ 86001

1.0 Introduction:

1.1 Project Information

The following report is the result of the soil testing to design the retaining wall for the proposed Holiday Inn. Geotechnical work was performed on the parcel, 107-13-009, of the proposed Holiday in to be located at 4th Street and Route 66 in Flagstaff, Arizona. Scope of geotechnical work for the retaining wall consisted of collecting 6 samples on the proposed fill of the site. Soil collection locations and soil collection plan is located in *Appendix A* of the report.

The purpose of this report is to determine the following for the retaining wall on site:

- Soil type
- Soil Attributes
  - Unit Weight of Soil
  - Bearing Capacity
  - Settlement
  - Liquid and Plastic Limits
  - Percentage of Clay

1.2 Project Location

The project is outlined in the image below in red. The project location is confined by Route 66 and Fourth Street, as well as the railroad located to the South of the parcel. APN for the parcel is 107-13-009 in Coconino County.

Figure 1.0: Project parcel outlined in red.

1.3 Field Work

Sampling location was the fill pile that is located on the north eastern part of the parcel as shown in Figure 2.0 above. Sampling was located in this position due to the determination that the parcel was mostly fill and the safety restrictions of sampling. Sampling was done following the field work plan and safety plan shown in *Appendix A*.

Figure 2.0: Soil collection location (outlined in black) on the project parcel (outlined in red).

1.4 Current Site Condition

Site condition has changed historically due to the movement of the railroad that used to run through the parcell. The movement and excavation of the railroad leaves 3 feet of disturbed soil that has not been properly compacted. Minimal structures exist on the property and natural vegetation is abundant.

Table 1.0: Site condition description

Items	Description
Existing Structures on Site	Fence located on the southeastern part of the parcel. Pathway that connects to the existing FUTS trail from the sidewalk on Route 66 and 4th Street.
Visual Soil Condition	Ground contains soil or vegetation, mostly weeds and dead vegetation
Existing Grading	Contains soil fill from existing railroad project movement.

2.0 Results

2.1 Testing and Procedures

The results of geotechnical analysis were completed using the following tests and procedures.

Table 2.0: Table of test performed using ASTM standards.

Testing for Analysis Performed	Outcome for Each Test
Sieve Analysis (ASTM D6913M-17)	Particle size distribution curve to determine percentage of gravel, sand and clay/silt.
Hydrometer Analysis (ASTM D7928-17)	Determine the percent of clay in the soil.
Atterberg Limits (ASTM D4318-17)	Determine the liquid and plastic limit of the soil as well as the soil type.
Tri-axial (ASTM D4767-11)	Determine the bearing capacity of the soil.
Consolidation (ASTM D2435M-11)	Determine the settlement of the soil.
Direct Shear Analysis (ASTM)	Determine the bearing capacity of the soil. (substitute for triaxial test)
Proctor Compaction (ASTM)	Determine the unit weight of the soil

2.2 Results of Testing

The results of testing is based on the average of the samples to provide a basis of information to the client. The soil is also a mixed sample as it was collected in buckets from a fill pile. Note that this is used in place of the boring holes that were proposed.

2.2.1 Sieve Analysis

Determined the following percentages all graphs and tables in *Appendix B.*

Table 3.0: Soil particle distribution for the soil collected.

Type of Soil	Sieve Number (NO.)	Particle Size (mm)	Percentage (%)
Gravel	10 > X	2 > X	28.53
Sand	10 > X > 200	2 > X > .05	67.07
Silt/Clay	X > 200	.05 > X	4.4

2.2.2 Hydrometer

Provided the percentage of clay from silt and sand that passed the 200 sieve. The following table shows the percentages.

Table 4.0:Particle distribution for soil that passed the 200 sieve.

Type of Soil	Sieve Number (NO.)	Particle Size (mm)	Percentage (%)
Sand	X > 200	X > 0.05	30.13
Silt	X > 200	.05 > X > 0.002	63.5
Clay	X > 200	0.002 > X	6.37

2.2.3 Atterberg Limits

The testing results are as follows for the determination of the plastic and liquid limits of the soil. Note that this is an average of all of the testing as 6 samples were tested.

Table 5.0: Plastic Limit data

Plastic Limit						
Moisture Can ID	T-1	T-2	T-3	T-4	T-5	T-6
Mc (g)	19.5	13.3	19.8	13.6	13.2	13.3
Mm (g)	31.6	25.2	27.5	24.2	31	22.4
Md (g)	29.2	23.2	26	22.2	27.7	20.7
w (%)	24.74	20.20	24.19	23.26	22.76	22.97
PL (%)	24.74	20.20	24.19	23.26	22.76	22.97
AVG PL (%)	23.02			± 1.58		

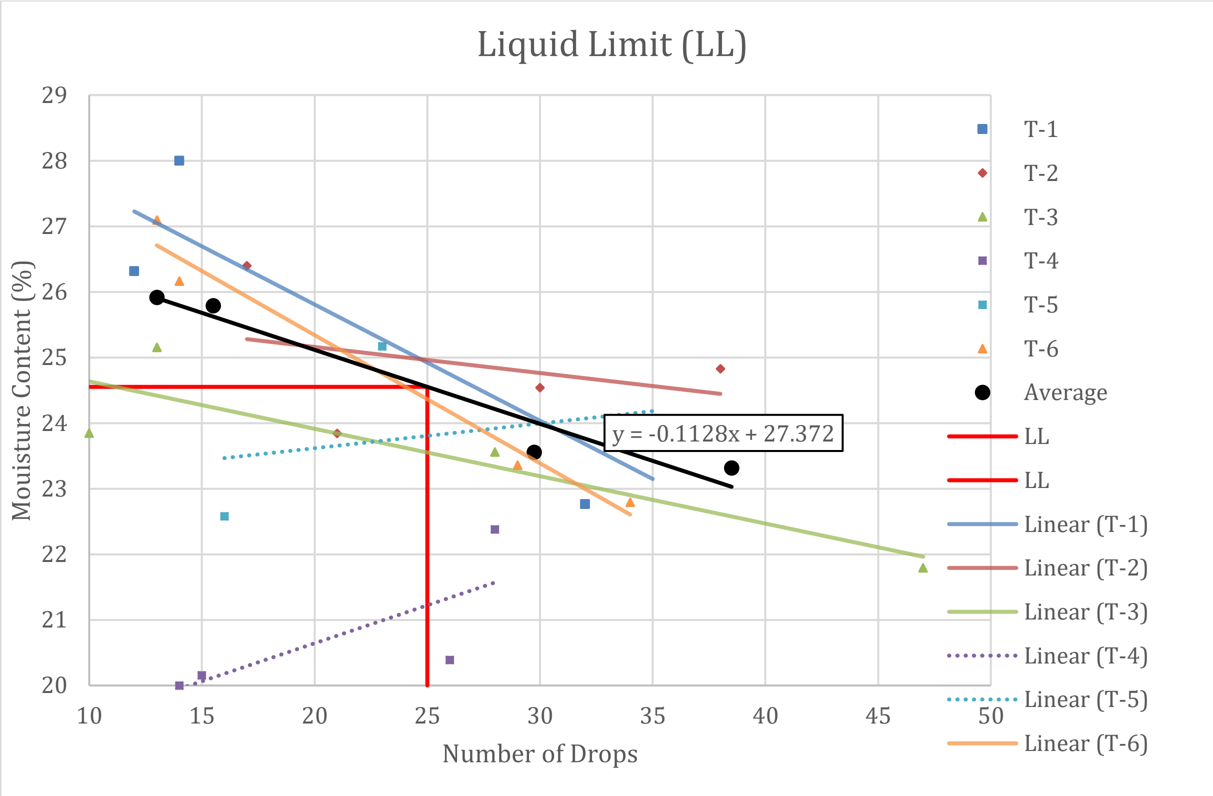


Figure 3.0 Liquid Limit Data
Liquid Limit- 24.92
Plastic Limit- 23.02
Cu- 12.11
Cc- 1.169

To classify the fill soil on the parcel, all methods were used to give a better understanding of the soil. AASHTO, USCS, and USDA are shown with classification and soil descriptions.

Table 6.0: Soil Classification Summary

Classification Method	Classification Description	Soil Description
AASHTO	A-1-b , A-3	Gravel Sand/ Fine Sand
USCS	ML, SW	Welly Graded Sand with Gravel
USDA	N/A	Sand

2.2.4 Triaxial

Testing is for the determination of the bearing capacity of the soil. Note that this is not being used in the determination of the soil or the bearing capacity as testing did not provide sufficient data. The actual The direct shear test will be done instead to determine the bearing capacity of the soil. Results for the triaxial test are shown in *Appendix F*. The

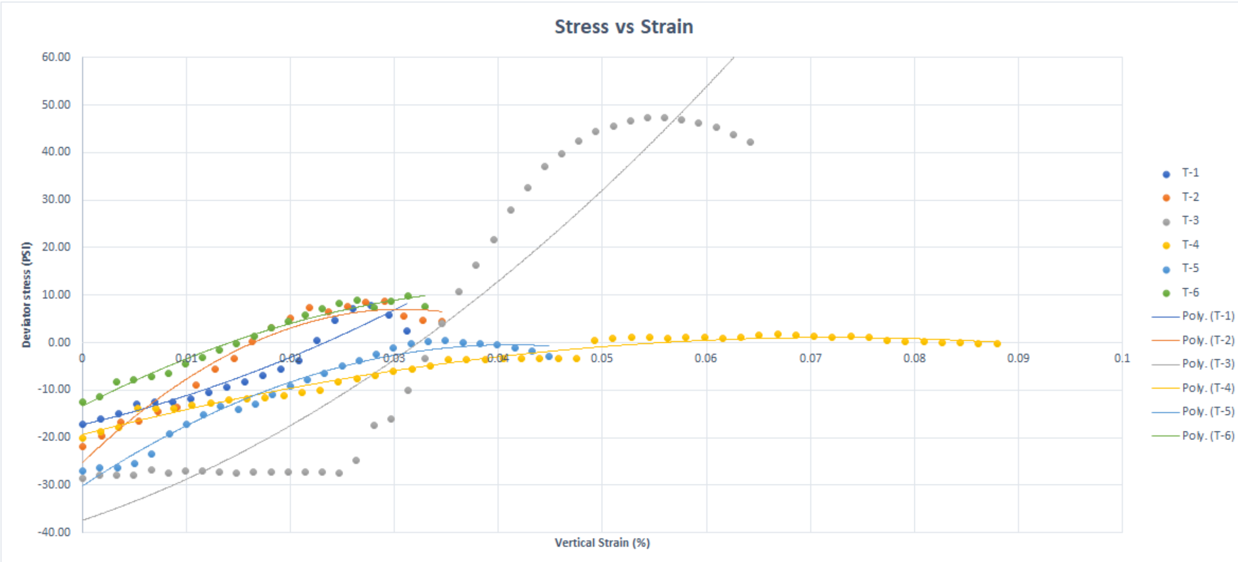


Figure 4.0: Triaxial Test Data

2.2.5 Proctor Compaction

The proctor compaction test provided a unit weight for the samples as shown in the table below. This also was averaged to provide an optimal unit weight of 1752 Kg/m^3 at the peak of the curve provided in *Appendix E*. The following are the results of the testing.

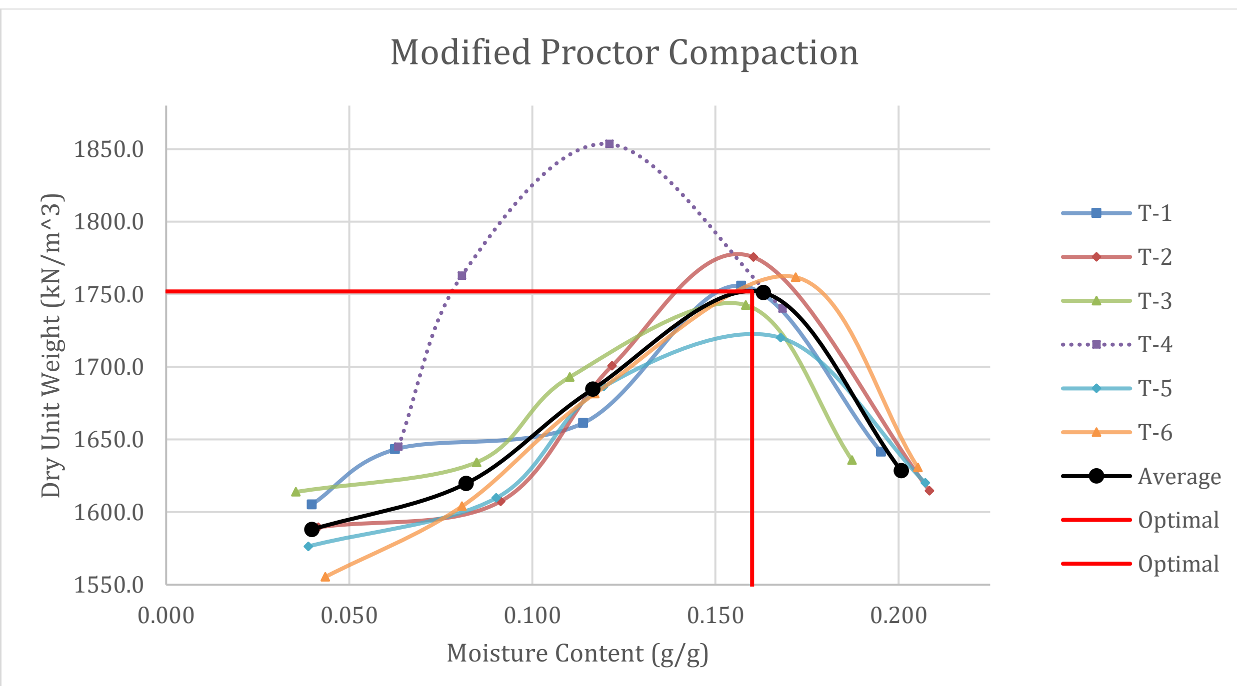


Figure 5.0: Modified Proctor Compaction Data

2.2.6 Consolidation

Consolidation is used to determine the total settlement in the soil when a load is applied. The following is the stress table that was collected from the consolidation testing that was completed. Due to the lack of time and the amount of load on the soil. This was redone to collect accurate results for settling. Unfortunately since the soil sample was disturbed, the results from this test are inconclusive.

Table 7.0: Consolidation Test Data

Vertical Effective Stress (σ'z)=P/A					
P		A		(σ'z)	
4	kg	3231.0806	mm^2	0.00124	kg/mm^2
		0.0032	m^2	1237.97592	kg/m^2
8	kg	3232.0806	mm^2	0.00248	kg/mm^2
		0.0032	m^2	2475.95185	kg/m^2
16	kg	3233.0806	mm^2	0.00495	kg/mm^2
		0.0032	m^2	4951.90370	kg/m^2

2.2.7 Direct Shear

Direct Shear was used to determine the friction angle of the soil. The data collected and plotted in Figure 6.0 is the shear force applied on the soil against the horizontal displacement of the soil. This data was used to create Figure 7.0 and allowed the friction angle to be determined.

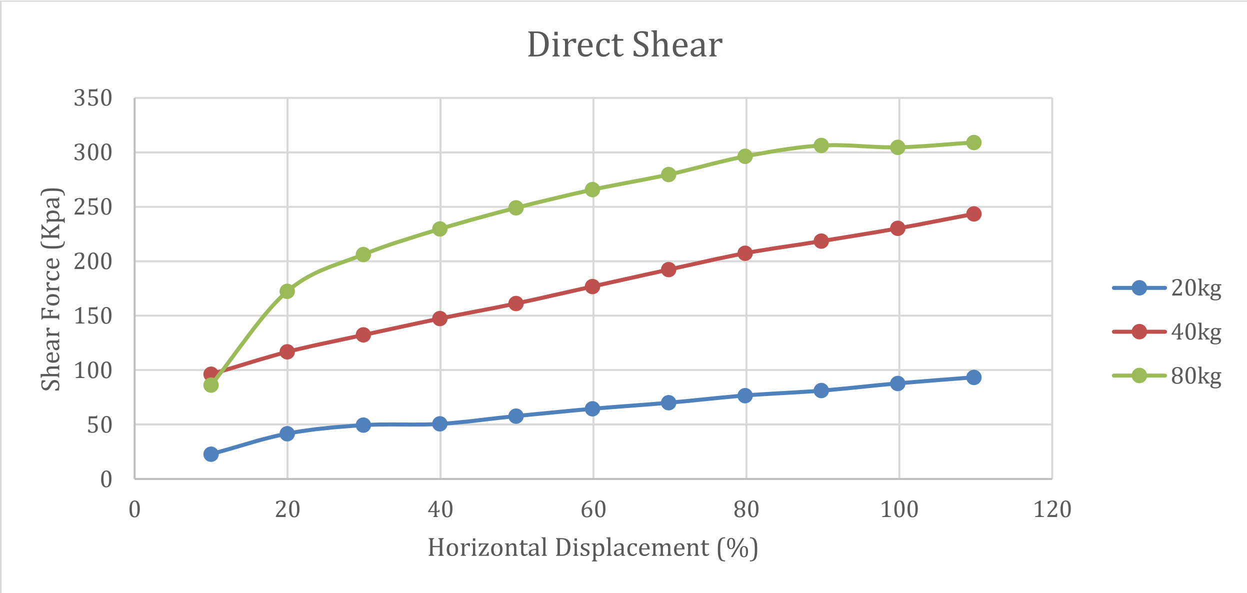


Figure 6.0: Direct Shear Data

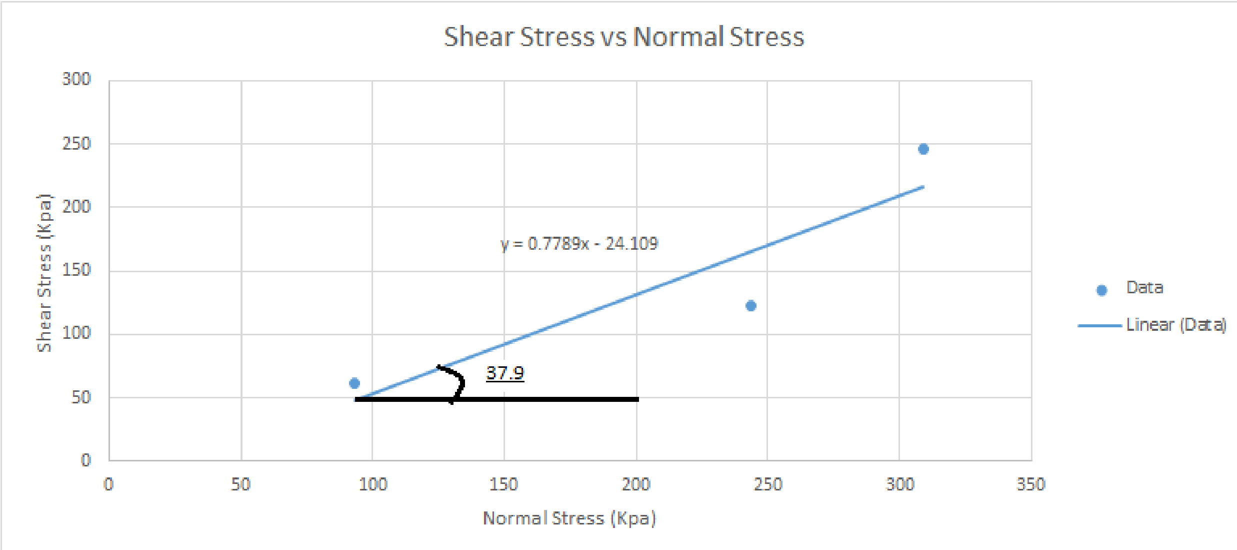


Figure 7.0: Friction Angle of soil

3.0 Summary of Analysis

The soil that has been tested is from a fill that is located on the parcel. This is not a fully accurate report as the soil that is on the site varies near the retaining wall. This will be used to design the wall as parcel is mostly fill that is determined similar to the stockpile that the homogenous samples were taken from. Soil sampling will continue as two more testing sessions are needed prior to wall design.

The type of soil consists of mostly sand and small amounts of gravel and clay. This is ideal for the design as the soil will have little to no settlement after the wall is completed. The soil is defined as fine sand by the AASHTO standards. The amount of clay that the soil is negligible, however, the consolidation test will determine if settlement is an issue due to the clay. The optimal dry unit weight of the soil is 1752 kg/m^3 as the proctor compaction test identified. Triaxial did not provide useable results and a direct shear test will be used in its place once completed for the bearing capacity of the wall.

Overall, the soil will provide sufficient drainage and the soil is sufficient for the wall design as of now. A further and more thorough report will be provided when the testing is finished.

Appendix D: Streamstats

StreamStats Report

Region ID: AZ
Workspace ID: AZ20191017222146767000
Clicked Point (Latitude, Longitude): 35.20720, -111.61004
Time: 2019-10-17 15:22:04 -0700

Basin Characteristics

Parameter Code	Parameter Description	Value	Unit
CONTDA	Area that contributes flow to a point on a stream	6.02	square miles
ELEV	Mean Basin Elevation	7875.722	feet
APRAVTMP	Mean AprilTemperature	40.7	degrees F
AUGAVPRE	Mean August Precipitation	2.6	inches
AUGAVTMP	Mean August Temperature	61.6	degrees F
AZ_HIPERMA	Percent basin surface area containing high permeability aquifer units as defined for Arizona in SIR 2014-5211	97	percent

Appendix D: Streamstats Results

Parameter Code	Parameter Description	Value	Unit
AZ_HIPERMG	Percent basin surface area containing high permeability geologic units as defined for Arizona in SIR 2014-5211	97	percent
BASINPERIM	Perimeter of the drainage basin as defined in SIR 2004-5262	17.24	miles
BSLDEM10M	Mean basin slope computed from 10 m DEM	30	percent
CH92_01DEV	Percent Difference between 1992 and 2001 area covered by developed land using NLCD	3	percent
CH92_01FOR	Percent Difference between 1992 and 2001 area covered by forest using NLCD	2	percent
DECAVPRE	Mean December Precipitation	1.3	inches
DRNAREA	Area that drains to a point on a stream	6.02	square miles
DRNDENSITY	Basin drainage density defined as total stream length divided by drainage area.	2.74	dimensionless
EL5000	Percent of area above 5000 ft	100	percent
EL6000	Percent of area above 6000 ft	100	percent
EL7500	Percent of area above 7500 ft	63	percent
ELEVMAX	Maximum basin elevation		feet
FD_Region	FD_Region	9619	dimensionless
FEBAVPRE	Mean February Precipitation	2.8	inches
IMPNLCD01	Percentage of impervious area determined from NLCD 2001 impervious dataset	4	percent
JANAVPRE	Mean January Precipitation	3.4	inches
JULAVPRE	Mean July Precipitation	2.7	inches
JULYAVTMP	Mean July Temperature	63	degrees F
JUNAVPRE	Mean June Precipitation	0.6	inches
JUNEAVTMP	Mean June Temperature	57.9	degrees F
LAT_CENT	Latitude of Basin Centroid	35.247	decimal degrees
LC01BARE	Percentage of area barren land, NLCD 2001 category 31	0	percent
LC01DEV	Percentage of land-use from NLCD 2001 classes 21-24	8	percent

Parameter Code	Parameter Description	Value	Unit
LC01FOREST	Percentage of forest from NLCD 2001 classes 41-43	90	percent
LC01HERB	Percentage of herbaceous upland from NLCD 2001 class 71	2	percent
LC92FOREST	Percentage of forest from NLCD 1992 classes 41-43	88	percent
LONG_CENT	Longitude Basin Centroid	-111.6228	decimal degrees
LU92DEV	Percent of area covered by all densities of developed land using 1992 NLCD	5	percent
MARAVPRE	Mean March Precipitation	3.2	inches
MARAVTMP	Mean March Temperature	34.9	degrees F
MAYAVPRE	Mean May Precipitation	1	inches
MAYAVTMP	Mean May Temperature	48.5	degrees F
MINBELEV	Minimum basin elevation		feet
NFSL30_10M	Percent area with north-facing slopes greater than 30 percent from 10-meter NED.	9	percent
NOVAVPRE	Mean November Precipitation	1.8	inches
NOVAVTMP	Mean November Temperature	34.7	degrees F
OCTAVPRE	Mean October Precipitation	2.1	inches
OCTAVTMP	Mean October Temperature	44.7	degrees F
OUTLETELEV	Elevation of the stream outlet in thousands of feet above NAVD88.	6852.43	feet
PRECIP	Mean Annual Precipitation	25.4	inches
RELIEF	Maximum - minimum elevation		feet
RELRELF	Basin relief divided by basin perimeter		feet per mi
SEPAVPRE	Mean September Precipitation	2.5	inches
SEPAVTMP	Mean September Temperature	55.1	degrees F
SLOP30_10M	Percent area with slopes greater than 30 percent from 10-meter NED	46	percent
STRMTOT	total length of all mapped streams (1:24,000-scale) in the basin	16.49	miles
TEMP	Mean Annual Temperature	44.01	degrees F

Peak-Flow Statistics Parameters[Peak Region 1 High Elev 2014 5211]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
CONTDA	Contributing Drainage Area	6.02	square miles	1.26	711
ELEV	Mean Basin Elevation	7875.722	feet		

Peak-Flow Statistics Flow Report[Peak Region 1 High Elev 2014 5211]

PII: Prediction Interval-Lower, PIu: Prediction Interval-Upper, SEp: Standard Error of Prediction, SE: Standard Error (other -- see report)

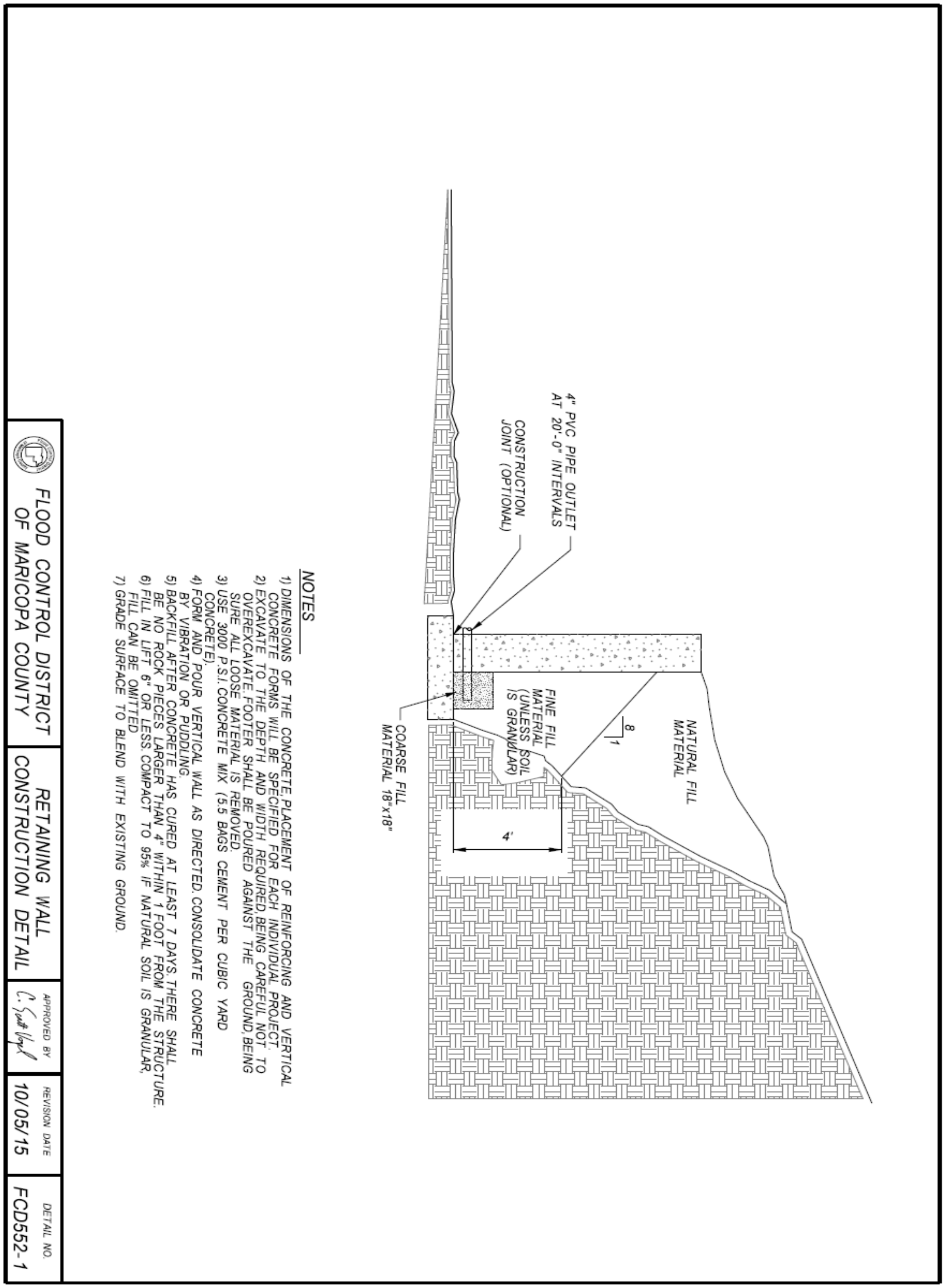
Statistic	Value	Unit	PII	PIu	SEp
2 Year Peak Flood	56.4	ft^3/s	15.9	200	86.1
5 Year Peak Flood	129	ft^3/s	47.5	352	64.4
10 Year Peak Flood	197	ft^3/s	78.4	494	58.2
25 Year Peak Flood	305	ft^3/s	127	732	55
50 Year Peak Flood	400	ft^3/s	166	964	55.1
100 Year Peak Flood	507	ft^3/s	207	1240	56.3
200 Year Peak Flood	630	ft^3/s	249	1590	58.5
500 Year Peak Flood	818	ft^3/s	305	2190	62.7

Peak-Flow Statistics Citations

Paretti, N.V., Kennedy, J.R., Turney, L.A., and Veilleux, A.G.,2014, Methods for estimating magnitude and frequency of floods in Arizona, developed with unregulated and rural peak-flow data through water year 2010: U.S. Geological Survey Scientific Investigations Report 2014-5211, 61 p., http://dx.doi.org/10.3133/sir20145211. (http://pubs.usgs.gov/sir/2014/5211/)

Application Version: 4.3.8

Appendix E: Maricopa Standard Detail



Appendix F: Reinforced CMU Wall

Appendix F contains calculations and design checks for all wall heights smaller than the tallest height design (9 CMU blocks). Figures A through B contain calculations and design checks for a wall of 8 CMU blocks.

Formulas				Notes
1	Rankine Coeffiecient of Active Pressure	k_a =	$\tan^2(45-\phi'/2)$	
2	Active Stress	σ'_a =	γ*H*ka	C=0
3	Resultant Active Pressure	Pa=	σ'_a *H*.5+Pq	
4	Applied Vertical Pressure of Soil	Pv=	Pa*sin(α)	
5	Applied Horizontal Soil Pressure	P_H =	Pa*cos(α)	
6	Factor of Safety for Overturning	$FS_{overturn}$=	$M_r/M_d \geq 2$	
7	Sum of Resistive Forces	M_r=	ΣV*(Marm)+Pv*(Marm)	
8	Driving Moment	M_d=	P_H*(H/3)	
9	Net Moment	M_N=	M_r-M_d	
10	Factor of Safety for Sliding	$FS_{Sliding}$=	$F_r/F_d \geq 1.5$	
11	Resisting Force	F_r=	fr+fc+P_P	fc=0
12	Driving Force	F_d=	P_H	
13	Force of friction	fr=	(Pv+ΣV)*tanδ	
14	Soil-Pile Friction Angle	δ=	2/3*ϕ'	
15	Coefficient of Friction	Coefficient= tan(δ)		
16	Resultant Passive Pressure	P_P=	σ'_P/2*D_f	
17	Passive Stress	σ'_P =	k_P*γ*D_f	C=0
18	Rankine Coefficient of Passive Pressure	k_P=	$\tan^2(45+\phi'/2)$	
19	Factor of Safety for Bearing	$FS_{Bearing}$=	$q_u/q_{max} \geq 3$	
20	Bearing Pressure on Toe	q_{max} =	ΣV/B*(1+6e/B)	
21	Eccentrictiy of Load	e=	B/2-M_N/ΣV	
22	Bearing Pressure on Heel	q_{min}=	ΣV/B*(1-6e/B)	
23	Unconfined Compressive Strength	q_u=	$c'*N_c*F_{cd}*F_{ci}+q*N_q*F_{qd}*F_{qi}+0.5*\gamma*B'*N_\gamma*F_{\gamma d}*F_{\gamma i}$	See Table 6.3 for factors
24	Bearing Pressure	q=	γ*D	
25	Effective Base Dimension	B'=	B-2*e	
26	Cohesion	c'=	0	
27	Bearing Capacity Factor	N_c=	60.78	For ϕ' = 37.9 degrees (values interpolated)
28	Bearing Capacity Factor	N_q=	48.33	
29	Bearing Capacity Factor	N_γ=	76.85	
30	depth Factor	F_{cd}=	F_{qd}-[(1-F_{qd})/($N_c$tan(ϕ'))]	For $D_f/B \leq 1$ and $\phi' > 0$
31	depth Factor	$F_{\gamma d}$=	1	
32	depth Factor	F_{qd}=	$1+2\tan\phi'(1-\sin\phi')^2 D_f/B$	
33	Angle of Resultant of ΣV and P_H	β=	arctan(P_H/ΣV)	
34	Inclination Factor	F_{ci}=F_{qi}=	$(1-\beta/90)^2$	For β=29.93 degrees
35	Inclination Factor	$F_{\gamma i}$=	$(1-\beta/\phi')^2$	
36	Weight of Area 1	V1=	A1*γ (concrete)	
37	Weight of Area 2	V2=	A2*γ (concrete)	
38	Weight of Area 3	V3=	A3*γ (concrete)	
39	Weight of Area 3	V4=	A4*γ (soil)	
40	Weight of Area 4	V5=	A5*γ (soil)	
41	Weight of Area 5	ΣV=	V1+V2+V3+V4+V5	
42	Allowable Soil Bearing Pressure	qall=	qu/FS	2555.766942 psf

Determined Variable Values:		
Friction Angle	ϕ'	37.900 degrees
	ϕ'	0.661 radians
Unit weight	γ (soil)	109.370 pcf
Unit weight	γ (concrete)	150.000 pcf
Unit weight	γ (normal CMU)	125.000 pcf
Total Wall Height	H	6.583 feet
Footing Depth	D_f	2.500 feet
Active Coefficient	k_a	0.239
Angle of Soil at Top of Wall	α	0.000 degrees
	α	0.000 radians
Active Stress	σ'_a	172.040 psf
Active Pressure	Pa	4366.299 lbs/ft
Surcharge (not from soil)	Pq (surcharge)	3800.000 lbs/ft
Vertical Pressure	Pv	0.000 lbs/ft
Horizontal Pressure	P_H	4366.299 lbs/ft

Overturning Check			
Area 1	A1	3.230 ft^2	
Area 3	A3	15.000 ft^2	
Area 4	A4	45.061 ft^2	
			Moment arm
Weight 1	V1	403.754 lbs/ft	0.81771 ft
Weight 3	V3	2250.000 lbs/ft	5 ft
Weight 4	V4	4928.344 lbs/ft	5.56771 ft
Toal Weight	ΣV	7582.099 lbs/ft	
Resisting Moment	M_r	39019.737 lb-ft/ft	
Driving moment	M_d	9581.601 lb-ft/ft	
Net Moment	M_N	29438.136 lb-ft/ft	
Factor of Safety	$FS_{overturn}$	4.072 ≥3	

Sliding Check			
Force of friction	fr	3578.646	
Force of cohesion	fc	0.000	
Passive Pressure	P_p	3766.600	lbs/ft
Resiting Force	F_r	7345.247	
Driving Froce	F_d	4366.299	lbs/ft
Factor of Safety	$FS_{sliding}$	1.682	≥1.5

Bearing Capacity Check			
Soil-Pile Friction Angle	δ	25.267	degrees
	δ	0.441	radians
Passive Stress	σ'_p	3013.280	psf
Passive Coefficient	k_p	4.185	
Width of Footing	B	10.000	feet
Eccentricity of Load	e	1.117	feet
Effective Width of Footing	B'	7.765	feet
Angle of Resultant of ΣV and P_H	β	29.9363	degrees
	β	0.52249	radians
Bearing Cpacity of Wall	qu	7667.301	psf
Bearing Pressure at Foundation Toe	qmax	1266.551	psf
Bearing Pressure at Heel	qmin	249.868	psf
	q	273.425	psf
Depth Factor	F_{cd}	1.059	
Depth Factor	$F_{\gamma d}$	1.000	
Depth Factor	F_{qd}	1.058	
Inclination Factor	$F_{ci}=F_{qi}$	0.445	
Inclination Factor	$F_{\gamma i}$	0.044	
Factor of Safety	$FS_{bearing}$	6.054	≥3

Figures C through D contain calculations and design checks for a wall of 7 CMU blocks.

	Formulas			Notes
1	Rankine Coeffiecient of Active Pressure	k_a =	$\tan^2(45-\phi'/2)$	
2	Active Stress	σ'_a =	γ*H*ka	C=0
3	Resultant Active Pressure	Pa=	σ'_a*H*.5+Pq	
4	Applied Vertical Pressure of Soil	Pv=	Pa*sin(α)	
5	Applied Horizontal Soil Pressure	P_H =	Pa*cos(α)	
6	Factor of Safety for Overturning	$FS_{overturn}$=	$M_r/M_d \geq 2$	
7	Sum of Resistive Forces	M_r=	ΣV*(Marm)+Pv*(Marm)	
8	Driving Moment	M_d=	P_H*(H/3)	
9	Net Moment	M_N=	M_r-M_d	
10	Factor of Safety for Sliding	$FS_{Sliding}$=	$F_r/F_d \geq 1.5$	
11	Resisting Force	F_r=	fr+fc+P_P	fc=0
12	Driving Force	F_d=	P_H	
13	Force of friction	fr=	(Pv+ΣV)*tanδ	
14	Soil-Pile Friction Angle	δ=	2/3*ϕ'	
15	Coefficient of Friction	Coefficient= tan(δ)		
16	Resultant Passive Pressure	P_P=	σ'_P/2*D_f	
17	Passive Stress	σ'_P =	k_P*γ*D_f	C=0
18	Rankine Coefficient of Passive Pressure	k_P=	$\tan^2(45+\phi'/2)$	
19	Factor of Safety for Bearing	$FS_{Bearing}$=	$q_u/q_{max} \geq 3$	
20	Bearing Pressure on Toe	q_{max} =	ΣV/B*(1+6e/B)	
21	Eccentrictiy of Load	e=	B/2-M_N/ΣV	
22	Bearing Pressure on Heel	q_{min}=	ΣV/B*(1-6e/B)	
23	Unconfined Compressive Strength	q_u=	$c'*N_c*F_{cd}*F_{ci}+q*N_q*F_{qd}*F_{qi}+0.5*\gamma*B'*N_\gamma*F_{\gamma d}*F_{\gamma i}$	See Table 6.3 for factors
24	Bearing Pressure	q=	γ*D	
25	Effective Base Dimension	B'=	B-2*e	
26	Cohesion	c'=	0	
27	Bearing Capacity Factor	N_c=	60.78	For ϕ' = 37.9 degrees (values interpolated)
28	Bearing Capacity Factor	N_q=	48.33	
29	Bearing Capacity Factor	N_γ=	76.85	
30	depth Factor	F_{cd}=	F_{qd}-[(1-F_{qd})/($N_c$tan(ϕ'))]	For $D_f/B \leq 1$ and $\phi' > 0$
31	depth Factor	$F_{\gamma d}$=	1	
32	depth Factor	F_{qd}=	$1+2\tan\phi'(1-\sin\phi')^2 D_f/B$	
33	Angle of Resultant of ΣV and P_H	β=	arctan(P_H/ΣV)	
34	Inclination Factor	F_{ci}=F_{qi}=	$(1-\beta/90)^2$	For β=29.19 degrees
35	Inclination Factor	$F_{\gamma i}$=	$(1-\beta/\phi')^2$	
36	Weight of Area 1	V1=	A1*γ (concrete)	
37	Weight of Area 2	V2=	A2*γ (concrete)	
38	Weight of Area 3	V3=	A3*γ (concrete)	
39	Weight of Area 3	V4=	A4*γ (soil)	
40	Weight of Area 4	V5=	A5*γ (soil)	
41	Weight of Area 5	ΣV=	V1+V2+V3+V4+V5	
42	Allowable Soil Bearing Pressure	qall=	qu/FS	2785.686634 psf

Determined Variable Values:		
Friction Angle	ϕ'	37.900 degrees
	ϕ'	0.661 radians
Unit weight	γ (soil)	109.370 pcf
Unit weight	γ (concrete)	150.000 pcf
Unit weight	γ (normal CMU)	125.000 pcf
Total Wall Height	H	5.948 feet
Footing Depth	D_f	2.500 feet
Active Coefficient	k_a	0.239
Angle of Soil at Top of Wall	α	0.000 degrees
	α	0.000 radians
Active Stress	σ'_a	155.435 psf
Active Pressure	Pa	4262.258 lbs/ft
Surcharge (not from soil)	Pq (surcharge)	3800.000 lbs/ft
Vertical Pressure	Pv	0.000 lbs/ft
Horizontal Pressure	P_H	4262.258 lbs/ft

Overturning Check			
Area 1	A1	2.826 ft^2	
Area 3	A3	16.500 ft^2	
Area 4	A4	43.876 ft^2	
			Moment arm
Weight 1	V1	353.285 lbs/ft	0.81771 ft
Weight 3	V3	2475.000 lbs/ft	5.5 ft
Weight 4	V4	4798.770 lbs/ft	6.06771 ft
Toal Weight	ΣV	7627.055 lbs/ft	
Resisting Moment	M_r	43018.921 lb-ft/ft	
Driving moment	M_d	8450.518 lb-ft/ft	
Net Moment	M_N	34568.403 lb-ft/ft	
Factor of Safety	$FS_{overturn}$	5.091 ≥3	

Sliding Check		
Force of friction	fr	3599.865
Force of cohesion	fc	0.000
Passive Pressure	P_P	3403.052 lbs/ft
Resiting Force	F_r	7002.917
Driving Froce	F_d	4262.258 lbs/ft
Factor of Safety	$FS_{sliding}$	1.643 ≥1.5

Bearing Capacity Check		
Soil-Pile Friction Angle	δ	25.267 degrees
	δ	0.441 radians
Passive Stress	σ'_p	2722.441 psf
Passive Coefficient	k_p	4.185
Width of Footing	B	11.000 feet
Eccentricity of Load	e	0.968 feet
Effective Width of Footing	B'	9.065 feet
Angle of Resultant of ΣV and P_H	β	29.1979 degrees
	β	0.5096 radians
Bearing Cpacity of Wall	qu	8357.060 psf
Bearing Pressure at Foundation Toe	qmax	1059.339 psf
Bearing Pressure at Heel	qmin	327.398 psf
Soil overburden	q	273.425 psf
Depth Factor	F_{cd}	1.054
Depth Factor	$F_{\gamma d}$	1.000
Depth Factor	F_{qd}	1.053
Inclination Factor	$F_{ci}=F_{qi}$	0.456
Inclination Factor	$F_{\gamma i}$	0.053
Factor of Safety	$FS_{bearing}$	7.889 ≥3

Figures E through F contain calculations and design checks for a wall of 6 CMU blocks.

		Formulas		Notes
1	Rankine Coeffiecient of Active Pressure	$k_a =$	$\tan^2(45-\phi'/2)$	
2	Active Stress	$\sigma'_a =$	$\gamma*H*ka$	C=0
3	Resultant Active Pressure	Pa=	$\sigma'_a*H*.5+Pq$	
4	Applied Vertical Pressure of Soil	Pv=	$Pa*\sin(\alpha)$	
5	Applied Horizontal Soil Pressure	$P_H =$	$Pa*\cos(\alpha)$	
6	Factor of Safety for Overturning	$FS_{overturn}=$	$M_r/M_d \geq 2$	
7	Sum of Resistive Forces	$M_r=$	$\Sigma V*(Marm)+Pv*(Marm)$	
8	Driving Moment	$M_d=$	$P_H*(H/3)$	
9	Net Moment	$M_N=$	M_r-M_d	
10	Factor of Safety for Sliding	$FS_{Sliding}=$	$F_r/F_d \geq 1.5$	
11	Resisting Force	$F_r=$	$fr+fc+P_P$	fc=0
12	Driving Force	$F_d=$	P_H	
13	Force of friction	fr=	$(Pv+\Sigma V)*\tan\delta$	
14	Soil-Pile Friction Angle	$\delta=$	$2/3*\phi'$	
15	Coefficient of Friction	Coefficient= $\tan(\delta)$		
16	Resultant Passive Pressure	$P_P=$	$\sigma'_P/2*D_f$	
17	Passive Stress	$\sigma'_P =$	$k_P*\gamma*D_f$	C=0
18	Rankine Coefficient of Passive Pressure	$k_P=$	$\tan^2(45+\phi'/2)$	
19	Factor of Safety for Bearing	$FS_{Bearing}=$	$q_u/q_{max} \geq 3$	
20	Bearing Pressure on Toe	$q_{max} =$	$\Sigma V/B*(1+6e/B)$	
21	Eccentrictiy of Load	e=	$B/2-M_N/\Sigma V$	
22	Bearing Pressure on Heel	$q_{min}=$	$\Sigma V/B*(1-6e/B)$	
23	Unconfined Compressive Strength	$q_u=$	$c'*N_c*F_{cd}*F_{ci}+q*N_q*F_{qd}*F_{qi}+0.5*\gamma*B'*N_\gamma*F_{\gamma d}*F_{\gamma i}$	See Table 6.3 for factors
24	Bearing Pressure	q=	$\gamma*D$	
25	Effective Base Dimension	B'=	B-2*e	
26	Cohesion	c'=	0	
27	Bearing Capacity Factor	$N_c=$	60.78	For ϕ' = 37.9 degrees (values interpolated)
28	Bearing Capacity Factor	$N_q=$	48.33	
29	Bearing Capacity Factor	$N_\gamma=$	76.85	
30	depth Factor	$F_{cd}=$	$F_{qd}-[(1-F_{qd})/(N_c\tan(\phi'))]$	For $D_f/B \leq 1$ and $\phi' > 0$
31	depth Factor	$F_{\gamma d}=$	1	
32	depth Factor	$F_{qd}=$	$1+2\tan\phi'(1-\sin\phi')^2 D_f/B$	
33	Angle of Resultant of ΣV and P_H	$\beta=$	$\arctan(P_H/\Sigma V)$	
34	Inclination Factor	$F_{ci}=F_{qi}=$	$(1-\beta/90)^2$	For β=28.96 degrees
35	Inclination Factor	$F_{\gamma i}=$	$(1-\beta/\phi')^2$	
36	Weight of Area 1	V1=	A1*γ (concrete)	
37	Weight of Area 2	V2=	A2*γ (concrete)	
38	Weight of Area 3	V3=	A3*γ (concrete)	
39	Weight of Area 3	V4=	A4*γ (soil)	
40	Weight of Area 4	V5=	A5*γ (soil)	
41	Weight of Area 5	ΣV=	V1+V2+V3+V4+V5	
42	Allowable Soil Bearing Pressure	qall=	qu/FS	2927.248716 psf

Determined Variable Values:		
Friction Angle	ϕ'	37.900 degrees
	ϕ'	0.661 radians
Unit weight	γ (soil)	109.370 pcf
Unit weight	γ (concrete)	150.000 pcf
Unit weight	γ (normal CMU)	125.000 pcf
Total Wall Height	H	5.313 feet
Footing Depth	D_f	2.500 feet
Active Coefficient	k_a	0.239
Angle of Soil at Top of Wall	α	0.000 degrees
	α	0.000 radians
Active Stress	σ'_a	138.830 psf
Active Pressure	Pa	4168.767 lbs/ft
Surcharge (not from soil)	Pq (surcharge)	3800.000 lbs/ft
Vertical Pressure	Pv	0.000 lbs/ft
Horizontal Pressure	P_H	4168.767 lbs/ft

Overturning Check			
Area 1	A1	2.423 ft^2	
Area 3	A3	18.000 ft^2	
Area 4	A4	41.421 ft^2	
			Moment arm
Weight 1	V1	302.816 lbs/ft	0.817708333 ft
Weight 3	V3	2700.000 lbs/ft	6 ft
Weight 4	V4	4530.205 lbs/ft	6.567708333 ft
Toal Weight	ΣV	7533.020 lbs/ft	
Resisting Moment	M_r	46200.677 lb-ft/ft	
Driving moment	M_d	7382.192 lb-ft/ft	
Net Moment	M_N	38818.485 lb-ft/ft	
Factor of Safety	$FS_{overturn}$	6.258 ≥3	

Sliding Check		
Force of friction	fr	3555.482
Force of Cohesion	fc	0.000
Passive Pressure	P_P	3039.503 lbs/ft
Resiting Force	F_r	6594.985
Driving Froce	F_d	4168.767 lbs/ft
Factor of Safety	$FS_{sliding}$	1.582 ≥1.5

Bearing Capacity Check		
Soil-Pile Friction Angle	δ	25.267 degrees
	δ	0.441 radians
Passive Stress	σ'_p	2431.603 psf
Passive Coefficient	k_p	4.185
Width of Footing	B	12.000 feet
Eccentricity of Load	e	0.847 feet
Effective Width of Footing	B'	10.306 feet
Angle of Resultant of ΣV and P_H	β	28.9601 degrees
	β	0.50545 radians
Bearing Cpacity of Wall	qu	8781.746 psf
Bearing Pressure at Foundation Toe	qmax	893.570 psf
Bearing Pressure at Heel	qmin	361.934 psf
Soil Overburden	q	273.425 psf
Depth Factor	F_{cd}	1.049
Depth Factor	$F_{\gamma d}$	1.000
Depth Factor	F_{qd}	1.048
Inclination Factor	$F_{ci}=F_{qi}$	0.460
Inclination Factor	$F_{\gamma i}$	0.056
Factor of Safety	$FS_{bearing}$	9.828 ≥3

Figures G through F contain calculations and design checks for a wall of 5 CMU blocks.

	Formulas			Notes
1	Rankine Coeffiecient of Active Pressure	k_a =	$\tan^2(45-\phi'/2)$	
2	Active Stress	σ'_a =	$\gamma*H*ka$	C=0
3	Resultant Active Pressure	Pa=	$\sigma'_a*H*.5+Pq$	
4	Applied Vertical Pressure of Soil	Pv=	$Pa*\sin(\alpha)$	
5	Applied Horizontal Soil Pressure	P_H =	$Pa*\cos(\alpha)$	
6	Factor of Safety for Overturning	$FS_{overturn}$=	$M_r/M_d \geq 2$	
7	Sum of Resistive Forces	M_r=	$\Sigma V*(Marm)+Pv*(Marm)$	
8	Driving Moment	M_d=	$P_H*(H/3)$	
9	Net Moment	M_N=	M_r-M_d	
10	Factor of Safety for Sliding	$FS_{Sliding}$=	$F_r/F_d \geq 1.5$	
11	Resisting Force	F_r=	$fr+fc+P_P$	fc=0
12	Driving Force	F_d=	P_H	
13	Force of friction	fr=	$(Pv+\Sigma V)*\tan\delta$	
14	Soil-Pile Friction Angle	δ=	$2/3*\phi'$	
15	Coefficient of Friction	Coefficient= $\tan(\delta)$		
16	Resultant Passive Pressure	P_P=	$\sigma'_P/2*D_f$	C=0
17	Passive stress	σ'_P =	$k_P*\gamma*D_f$	
18	Rankine Coefficient of Passive Pressure	k_P=	$\tan^2(45+\phi'/2)$	
19	Factor of Safety for Bearing	$FS_{Bearing}$=	$q_u/q_{max} \geq 3$	
20	Bearing Pressure on Toe	q_{max} =	$\Sigma V/B*(1+6e/B)$	
21	Eccentrictiy of Load	e=	$B/2-M_N/\Sigma V$	
22	Bearing Pressure on Heel	q_{min}=	$\Sigma V/B*(1-6e/B)$	See Table 6.3 for factors
23	Unconfined Compressive Strength	q_u=	$c'*N_c*F_{cd}*F_{ci}+q*N_q*F_{qd}*F_{qi}+0.5*\gamma*B'*N_\gamma*F_{\gamma d}*F_{\gamma i}$	
24	Bearing Pressure	q=	$\gamma*D$	
25	Effective Base Dimension	B'=	B-2*e	
26	Cohesion	c'=	0	For ϕ' = 37.9 degrees (values interpolated)
27	Bearing Capacity Factor	N_c=	60.78	
28	Bearing Capacity Factor	N_q=	48.33	
29	Bearing Capacity Factor	N_γ=	76.85	For $D_f/B \leq 1$ and $\phi' > 0$
30	depth Factor	F_{cd}=	$F_{qd}-[(1-F_{qd})/(N_c\tan(\phi'))]$	
31	depth Factor	$F_{\gamma d}$=	1	
32	depth Factor	F_{qd}=	$1+2\tan\phi'(1-\sin\phi')^2 D_f/B$	
33	Angle of Resultant of ΣV and P_H	β=	$\arctan(P_H/\Sigma V)$	For β=25.82 degrees
34	Inclination Factor	$F_{ci}=F_{qi}$=	$(1-\beta/90)^2$	
35	Inclination Factor	$F_{\gamma i}$=	$(1-\beta/\phi')^2$	
36	Weight of Area 1	V1=	A1*γ (concrete)	
37	Weight of Area 2	V2=	A2*γ (concrete)	
38	Weight of Area 3	V3=	A3*γ (concrete)	
39	Weight of Area 3	V4=	A4*γ (soil)	
40	Weight of Area 4	V5=	A5*γ (soil)	
41	Weight of Area 5	ΣV=	V1+V2+V3+V4+V5	
42	Allowable Soil Bearing Pressure	qall=	qu/FS	4282.399717 psf

Determined Variable Values:		
Friction Angle	ϕ'	37.900 degrees
	ϕ'	0.661 radians
Unit weight	γ (soil)	109.370 pcf
Unit weight	γ (concrete)	150.000 pcf
Unit weight	γ (normal CMU)	125.000 pcf
Total Wall Height	H	4.677 feet
Footing Depth	D_f	2.500 feet
Active Coefficient	k_a	0.239
Angle of Soil at Top of Wall	α	0.000 degrees
	α	0.000 radians
Active Stress	σ'_a	122.225 psf
Active Pressure	Pa	4085.828 lbs/ft
Surcharge (not from soil)	Pq (surcharge)	3800.000 lbs/ft
Vertical Pressure	Pv	0.000 lbs/ft
Horizontal Pressure	P_H	4085.828 lbs/ft

Overturning Check			
Area 1	A1	2.019 ft^2	
Area 3	A3	22.500 ft^2	
Area 4	A4	44.049 ft^2	
			Moment arm
Weight 1	V1	252.346 lbs/ft	0.817708333 ft
Weight 3	V3	3375.000 lbs/ft	7.5 ft
Weight 4	V4	4817.603 lbs/ft	8.067708333 ft
Toal Weight	ΣV	8444.950 lbs/ft	
Resisting Moment	M_r	64385.864 lb-ft/ft	
Driving moment	M_d	6369.919 lb-ft/ft	
Net Moment	M_N	58015.945 lb-ft/ft	
Factor of Safety	$FS_{overturn}$	10.108 ≥3	

Sliding Check		
Force of friction	fr	3985.900
Force of cohesion	fc	0.000
Passive Pressure	P_p	2675.955 lbs/ft
Resiting Force	F_r	6661.855
Driving Froce	F_d	4085.828 lbs/ft
Factor of Safety	$FS_{sliding}$	1.630 ≥1.5

Bearing Capacity Check			
Soil-Pile Friction Angle	δ	25.267	degrees
	δ	0.441	radians
Passive Stress	σ'_p	2140.764	psf
Passive Coefficient	k_p	4.185	
Width of Footing	B	15.000	feet
Eccentricity of Load	e	0.630	feet
Effective Width of Footing	B'	13.740	feet
Angle of Resultant of ΣV and P_H	β	25.81858301	degrees
	β	0.450619282	radians
Bearing Cpacity of Wall	qu	12847.199	psf
Bearing Pressure at Foundation Toe	qmax	704.895	psf
Bearing Pressure at Heel	qmin	421.099	psf
Soil Overburden	q	273.425	psf
Depth Factor	F_{cd}	1.039	
Depth Factor	$F_{\gamma d}$	1.000	
Depth Factor	F_{qd}	1.039	
Inclination Factor	$F_{ci}=F_{qi}$	0.509	
Inclination Factor	$F_{\gamma i}$	0.102	
Factor of Safety	$FS_{bearing}$	18.226	≥3