Bulk from Test Borings 0-4'	39	40/21	11.8	-	-	-	-	-

*NOTES:

1. We determine the swell pressure as measured in our laboratory using the graphically estimated load-back swell pressure method.
2. Negative Swell-Consolidation Potential indicates compression under conditions of loading and wetting.
3. * = Swell-Consolidation test performed on remolded sample due to rock content. Test results should be considered an estimate only of the swell or consolidation potential at the density and moisture content indicated.
4. ** = High Moisture Content and Low Dry Density due to the High Organic Content Soils/Peat. Total consolidation of the sample in 50+% range.

Direct Shear Strength Tests (Residual Strength Tests): We performed two residual strength direct shear strength tests on minus #10 sieve screen size particles obtained from borings TB-4 at 5-9' and TB-8 at 14-19'. We obtained a range of angle of internal friction (phi) value of 30 degrees and a cohesion of about 85 to 100 pounds per square foot.

4.0 FOUNDATION RECOMMENDATIONS

There are two general types of foundation system concepts, "deep" and "shallow", with the designation being based on the depth of support of the system. We have provided a discussion of viable foundation system concepts for this project below. The choice of the appropriate foundation system for the project is best made by the project structural engineer or project architect. We should be contacted once the design choice has been made to provide consultation regarding implementation of our design parameters.

Base on the subsurface soil conditions encountered, we feel a shallow foundation system will be a viable option for the proposed townhome units located along the western side of the project site in the areas of TB-1 through TB-9 and in the northeastern portion of the site near TB-14 through TB-16 and possibly near TB-13. Due to the high organic content in the soils, high consolidation potential, and shallow ground water near TB-10 through TB-12, the soils in this area are not suitable for shallow foundation systems. We do not recommend structures be located in this area if possible due to the large amount of ariel settlement that will tend to occur under any additional loading from either structures or man placed fill. If structures will be located in this area, the structures will need to be completely supported, including floors, by a deep foundation system.

Preloading of the ground surface and a settlement monitoring program may be necessary prior to construction to limit the amount of post construction ariel settlement. Conceptually, the preloading program would likely consist of placement of a series of steel plates at the base of a controlled fill. The plates would have steel rods that extend to the ground surface as survey monuments. Settlement of the fill mass could then be monitored by a survey program to determine amount of settlement and when settlement ceases.

4.1 Shallow Foundation System Concepts

Subsurface data indicate that clayey gravel with sand and cobbles will likely be the predominant soil type encountered beneath shallow foundations. With the exception of the areas around TB-10 though TB-12, the anticipated soils at the foundation level are considered suitable for shallow foundation support. Deep foundation system design concepts which include isolation of shallow components including floor systems from shallow soils are less likely to experience post-construction movement due to volume changes in the site soil.

There are numerous types of shallow foundation systems and variants of each type. Shallow foundation system concepts discussed below include:

- Spread Footings (continuous) and stem walls

The integrity and long-term performance of each type of system is influenced by the quality of workmanship which is implemented during construction. It is imperative that all excavation and fill placement operations be conducted by qualified personnel using appropriate equipment and techniques to provide suitable support conditions for the foundation system.

4.1.1 Spread Footings

A spread footing foundation system consists of a footing which dissipates, or spreads, the loads imposed from the stem wall (or beam) from the structure above. The soil samples tested from the anticipated support elevations in our test borings had a measured swell pressure of about 0 to 5,000 pounds per square foot and a swell potential magnitude of about -0.8 to 7.6 percent under a 100 or 500 pound per square foot surcharge load. A majority of the samples had to be remolded with only material passing the #10 screen due to the rock content of the site soil; therefore, the overall swell potential of the will likely be lower than the measures swell potential on the remolded samples. The owner must understand that regardless of the expansive soil mitigation design concepts presented below, if the swell pressure generated by the expansive soil on this site exceeds the minimum dead load which is imposed by the spread footing or other structural components, and the expansive site soils become wetted, uplift of the foundation system and other structural components is highly likely. Drilled piers, or other deep foundation system design will provide the least likelihood of post construction movement associated with soil volume changes.

The actual magnitude of the potential uplift of the foundation system depends on the volume (or depth) of the support soils which become moistened after construction. It is difficult to predict the amount of soil which will become moistened after construction, some theories suggest that with time the entire soil mantle may become moistened. Based on our experience in the area we feel that it is possible for at least 4 to 5 feet of soil below the footings to be influenced by subsurface moisture. Based on the assumed depth of moistened soil, laboratory test data, and the soil characteristics we estimate that the magnitude of the potential uplift associated with swelling of the expansive support soil materials may be in the range of about 1 to 1½ inches. If the entire soil mantle becomes moistened the total potential uplift may be considerably higher. The project structural engineer or architect should determine if the potential uplift is tolerable for the proposed structure on this project site.

Uplift associated with swelling soils occurs only where the foundation support soils have been exposed to water; therefore, the uplift may impose shear stresses in the foundation system. The magnitude of the imposed shear stress is related to the swell pressure of the support soil, but is difficult to estimate. Properly designed and constructed continuous spread footings with stem walls (or beams) have the ability to distribute the forces associated with swelling of the support soil. The rigidity of the system helps reduce differential movement and associated damage to the overlying structure. Swelling of the soil supporting isolated pad footings will result in direct uplift of the columns and structural components supported by the columns. Damage to the structure due to this type of movement can be severe. We recommend that isolated pad footings be avoided and that the foundation system be designed as rigid as is reasonably possible.

High foundation dead load, careful preparation of the support soils, placement of granular compacted structural fill, careful placement and compaction of stem wall backfill and positive surface drainage adjacent to the foundation system all help reduce the influence of swelling soils on the performance of the spread footing foundation system.

We recommend that the footings be designed with a high dead load and supported by a layer of moisture conditioned and compacted natural soil which is overlain by a layer of compacted structural fill material. This concept is outlined below:

- The foundation excavation should be excavated to 18 inches below the proposed footing support elevation.
- The natural soils exposed in the bottom of the excavation should be scarified to a depth of about 6 to 8 inches
- The scarified soil should be thoroughly moisture conditioned to about 2 percent above the laboratory determined optimum moisture content and then compacted.
- After completion of the compaction of the moisture conditioned natural soil an 18-inch-thick layer of granular aggregate base course structural fill material should be placed, moisture conditioned and compacted.
- The moisture conditioned natural soil material, and the granular soils should be compacted as discussed under the Compaction Recommendations portion of this report below.
- In the absence of structural engineering design and for general geotechnical engineering purposes, we recommend the stem walls be designed to act as beams and reinforced with continuous steel reinforcement, 4 reinforcement bars, 2 top and 2 bottom. Taller walls may require additional reinforcement bar.
- The structural engineer should be contacted to provide the appropriate reinforcement bar diameter and locations.

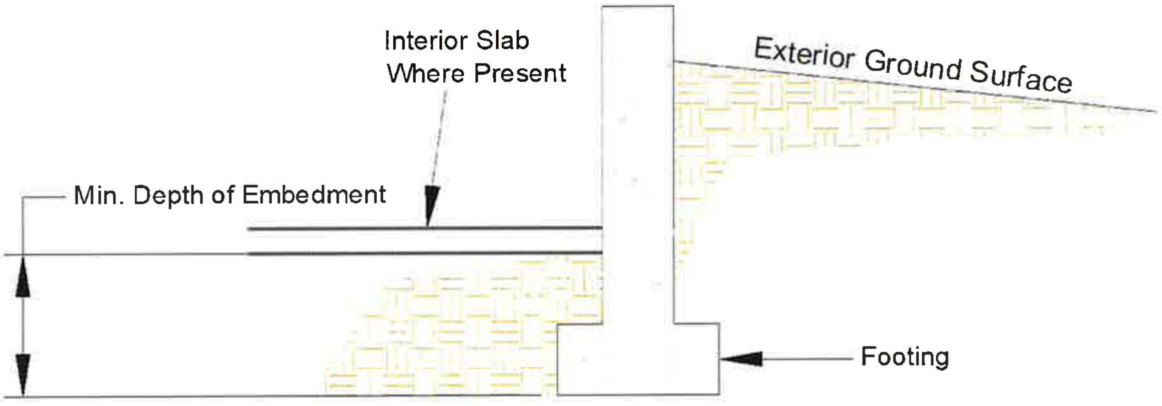
We recommend that particular attention and detail be given to the following aspects of the project construction for this lot;

- A subsurface drain system should be installed adjacent to the residential structure foundation system. Concepts for a subsurface drain system are presented in Section 6.0 of this report.
- The landscaping drainage concept provided in Section 8.5 below is imperative for this site to limit the moisture available to the foundation bearing soils.
- The exterior foundation backfill must be well compacted and moisture conditioned to above optimum moisture content. Recommendations for exterior foundation backfill are provided later in this report.

We recommend below-grade construction, such as retaining walls, crawlspace and basement areas, be protected from wetting and hydrostatic pressure buildup by an underdrain and wall drain system. Topographic conditions on the site may influence the ability to install a subsurface drain system which promotes water flow away from the foundation system. The subsurface drain system concept is discussed under the Subsurface Drain System section of this report below.

The footing embedment is a relatively critical, yet often overlooked, aspect of foundation construction. The embedment helps develop the soil bearing capacity, increases resistance of the footing to lateral movement and decreases the potential for rapid moisture changes in the footing support soils, particularly in crawl space areas. Interior footing embedment reduces the exposure of the crawl space support soils to dry crawl space air. Reduction in drying of the support soil helps reduce downward movement of interior footings due to soil shrinkage.

All footings should have a minimum depth of embedment of at least one 1 foot. The embedment concept is shown below.



Footing Embedment Concept

Not to Scale

Spread footings located away from sloped areas may be designed using the allowable gross bearing capacity information tabulated below.

Minimum Depth of Embedment (Feet)	Continuous Footing Design Capacity (psf)	Isolated Footing Design Capacity (psf)
1	1,500	Not Recommended
2	1,700	
3	1,900	

The bearing capacity values tabulated above may be increased by 20 percent for transient conditions associated with wind and seismic loads. Snow loads are not transient loads.

The bearing capacity values above were based on footing placed directly on the natural soils and on a continuous spread footing width of 1.5 feet. Larger footings and/or footings placed on a blanket of compacted structural fill will have a higher design soil bearing capacity. Development of the final footing design width is usually an iterative process based on evaluation of design pressures, footing widths and the thickness of compacted structural fill beneath the footings. We should be contacted as the design process continues to re-evaluate the design capacities above based on the actual proposed footing geometry.

Footings located on, or near slopes may need to have an additional embedment to establish a suitable footing/slope stability condition for the system. We should be contacted to provide additional information for footings located on, or near, sloped areas.

Due to the relatively high measured swell pressure of the soils tested we recommend isolated footings for support of interior column loads be avoided. A more rigid structure consisting of interior continuous footings and grade beams will help reduce the potential for damage due to swelling soils.

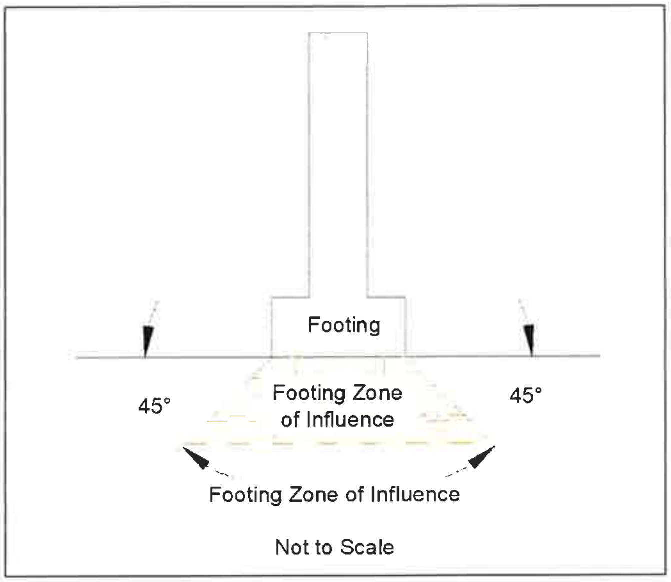
The settlement of the spread footing foundation system will be influenced by the footing size and the imposed loads. We estimated the total post construction settlement of the footings based on our laboratory consolidation data, the type and size of the footing. Our analysis below assumed that the highest bearing capacity value tabulated above was used in the design of the footings. The amount of post construction settlement may be reduced by placing the footings on a blanket of compacted structural fill material.

The estimated settlement for continuous footing with a nominal width of about 1½ to 2½ feet are tabulated below.

Thickness of Compacted Structural Fill (feet)	Estimated Settlement (inches)
0	½ - ¾
B/2	¼ - ½
B	About ¼

B is the footing width

The compacted structural fill should be placed and compacted as discussed in the Construction Considerations, “Fill Placement Recommendations” section of this report, below. The zone of influence of the footing (at elevations close to the bottom of the footing) is often approximated as being between two lines subtended at 45 degree angles from each bottom corner of the footing. The compacted structural fill should extend beyond the zone of influence of the footing as shown in the sketch below.



A general and simple rule to apply to the geometry of the compacted structural fill blanket is that it should extend beyond each edge of the footing a distance which is equal to the fill thickness.

We estimate that the footings designed and constructed above will have a total post construction settlement of about 1 inch or less.

All footings should be support at an elevation deeper than the maximum depth of frost penetration for the area. This recommendation includes exterior isolated footings and column supports. Please contact the local building department for specific frost depth requirements.

The post construction differential settlement may be reduced by designing footings that will apply relatively uniform loads on the support soils. Concentrated loads should be supported by footings that have been designed to impose similar loads as those imposed by adjacent footings.

Under no circumstances should any footing be supported by more than 3 feet of compacted structural fill material unless we are contacted to review the specific conditions supporting these footing locations.

The design concepts and parameters presented above are based on the soil conditions encountered in our test borings. We should be contacted during the initial phases of the foundation excavation at the site to assess the soil support conditions and to verify our recommendations

4.1.2 General Shallow Foundation Considerations

Some movement and settlement of any shallow foundation system will occur after construction. Movement associated with swelling soils also occurs occasionally. Utility line connections through and foundation or structural component should be appropriately sleeved to reduce the potential for damage to the utility line. Flexible utility line connections will further reduce the potential for damage associated with movement of the structure.

4.2 Deep Foundation System Concepts

Deep foundation system design concepts will provide the least likelihood of post-construction movement associated with volume changes within the soil. Due to the high consolidation potential, we recommend a deep foundation system for the structures located near TB-10 through TB-12. Deep Foundation System Concepts Discussed below include:

- Driven Piles

Cased micropiles or helical piers may also be alternatives for deep foundation support; however, due to the subsurface conditions additional field testing should be completed to determine if these options are feasible. This would likely include installation of a series of test piles/piers. We are available to discuss these options in further detail and aid in coordinating additional field testing.

Regardless of the type of deep foundation system concept utilized, the system design must include provisions to isolate and structurally support and building components, including flatwork, that may be influenced by volume changes within the site soil. Grade beams are utilized with most deep foundation system design concepts to facilitate isolation and structural support of various building elements. Grade beams, and any other horizontal component of a deep foundation system must be isolated from the support soil with void forms, or similar concept.

The elevation of the existing ground surface at our test boring locations at the time the borings were advanced should be established as part of the design process for deep foundation systems for this project. It is critical that the depths to various strata delineated in our test borings logs can be correlated to final project elevations.

4.2.1 Driven Piles

We encountered formational shale, sandstone or limestone at depths that ranged from 6.5 to 32.5 feet in our test borings. We encountered auger refusal approximately two to three feet into the formational prior to auger refusal or termination.

Driven piles that are end/tip bearing in the competent formational materials that underlie the project site may be used to support the proposed bridge abutments and potential associated wingwall structures. Based on the subsurface conditions encountered in our test borings, obtaining a tip bearing condition on the hard formational material should be readily obtained for H-section piles. We anticipate that about 3 to 5 feet of penetration into the formational shale materials may be obtained for H-section piles.

There are numerous methods used to calculate the bearing capacity of driven piles. We typically prefer to establish the bearing capacity of the driven piles based on dynamic formulae which incorporates the rated energy of the installation hammer and the size, weight, depth of the driven pile, and the soil characteristics. We have provided depth and general pile load carrying capacity estimates below, but the actual load capacity of the driven piles must be determined once the pile type (and depth) and energy of the hammer to be used for installation have been determined.

H-piles typically can be driven on sites with difficult installation conditions which may be caused by the presence of large cobbles and boulders. We recommend that H-piles be fitted with reinforcement driving tips to reduce the potential for damage to the pile tip during installation.

We encountered formational material in our test borings at a depth of about 33 feet below the ground surface. We recommend that the H-Piles be driven to an end-bearing support condition. For budgeting and planning purposes we suggest that you consider HP10x or HP12 x H-piles driven to a depth of about 20 to 35 feet below the ground surface. An allowable design capacity of 25 kips may be used if a pile hammer with a minimum rated energy of 20,000 foot pounds per stroke is used for pile installation. The actual depth of penetration of the H-piles into the formational material to establish the desired set criteria and associated bearing capacity will need to be determined during the initial phase of the installation operation.

Any tendency for pile deviation due to obstructions should be corrected immediately during the pile installations process. Piles that are installed out of plumb will have a lower support potential than the estimates provided above. Companion piles may need to be installed adjacent to piles which were installed out-of-plumb. If pile groups are planned, the minimum center to center spacing between the individual piles should be 30 inches or 2.5 times the pile diameter, whichever is greater.

We are available to provide a driving record for the installed piles and to provide geotechnical engineering consultation during the pile driving operations.

We anticipate that refusal will occur within 3 to 5 feet once the tip of the pile encounters the formational materials. We anticipate that damage to the pile could easily and rapidly occur if the potential energy of the hammer is greater than the yield stress of the selected pile section. The piles should be driven with high strength tip protection.

We recommend that the piles be driven with an appropriately sized hammer and/or adjustable stroke/energy hammer to avoid damage to the pile. When the tip elevation seats against the formational shale materials, then a set-criteria of 5 blows per 1/2 inch of pile penetration may be used to verify the set of the pile. Again, the energy output of the pile driving equipment must not exceed the structural capacity of the selected pile. We recommend that at least one pile per bridge abutment be monitored with signal matching pile driving analyzer (PDA) equipment, to verify that the needed capacity of the pile is obtained, and that the pile is not damaged at the set criteria discussed above (based on an allowable hammer energy for the selected pile).

We anticipate that penetration of the piles into the formational materials may be necessary to resolve lateral forces that act on the piles. Battered piles may be utilized to resolve lateral forces for the project. As discussed above, we anticipate that embedment of the piles into the formational materials will be relatively limited, and the penetration that does occur may cause fracturing/disturbance to the formational materials surrounding the pile. Achieving embedment of the piles into the formational materials may require predrilling the formational materials to the desired depth of pile embedment.

4.2.2 Grade Beams

Grade beams are utilized in a pier and grade beam foundation system to distribute the structure loads to each of the piers. The grade beam reinforcement and associated span distance is developed by the project structural engineer. The structural considerations of the grade beam in association with an assessment of the structure being supported by them will, in part determine the spacing between each of the deep foundation components, such as drilled piers (or drilled shafts), helical piers, micropiles and driven piles.

5.0 RETAINING STRUCTURES

We understand that laterally loaded walls will be constructed as part of this site development. Lateral loads will be imposed on the retaining structures by the adjacent soils and, in some cases, additional surcharge loads will be imposed on the retained soils from vehicles or adjacent structures. The loads imposed by the soil are commonly referred to as lateral earth pressures. The magnitude of the lateral earth pressure forces is partially dependent on the soil strength characteristics, the geometry of the ground surface adjacent to the retaining structure, the subsurface water conditions and on surcharge loads.

Due to the expansive nature of the site soils, we do not recommend that the natural soils be used for retaining wall backfill. The retaining walls may be designed using the equivalent fluid pressure values for imported granular soil that are tabulated below.

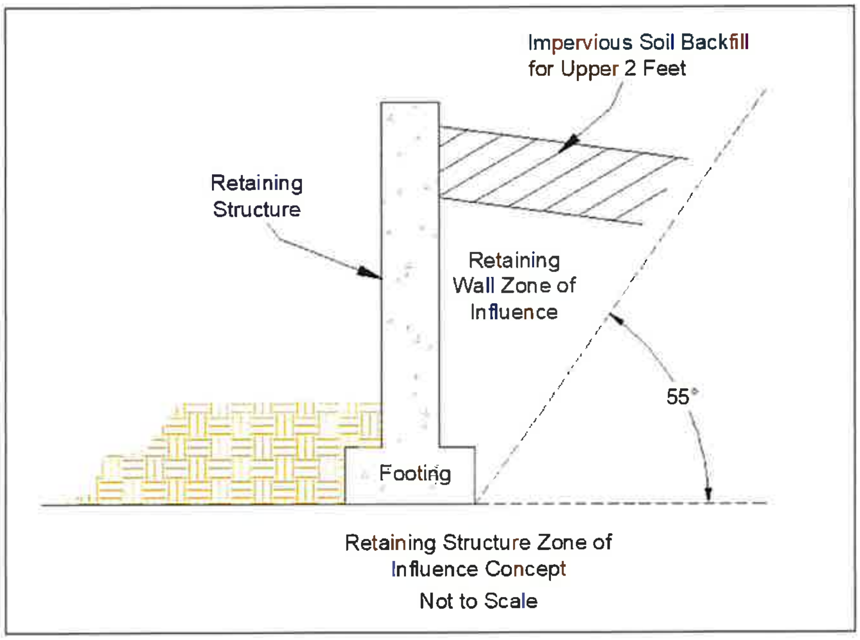
Type of Lateral Earth Pressure	Level Imported Granular Soil Backfill (pounds per cubic foot/foot)
Active	35
At-rest	55
Passive	460
Allowable Coefficient of Friction	0.45

Unit Weight on Imported Gravel = 135.0 pcf ; Angle of Internal Friction = 35 degrees

The granular soil that is used for the retaining wall backfill may be permeable and may allow water migration to the foundation support soils. There are several options available to help reduce water migration to the foundation soils, two of which are discussed here. An impervious geotextile layer and shallow drain system may be incorporated into the backfill, as discussed in Section 9.5, Landscaping Considerations, below. A second option is to place a geotextile filter material on top of the granular soils and above that place about 1½ to 2 feet of moisture conditioned and compacted site clay soils. It should be noted that if the site clay soils are used volume changes may occur which will influence the performance of overlying concrete flatwork or structural components.

The values tabulated above are for well drained backfill soils. The values provided above do not include any forces due to adjacent surcharge loads or sloped soils. If the backfill soils become saturated the imposed lateral earth pressures will be significantly higher than those tabulated above.

The granular imported soil backfill values tabulated above are appropriate for material with an angle of internal friction of 35 degrees, or greater. The granular backfill must be placed within the retaining structure zone of influence as shown below in order for the lateral earth pressure values tabulated above for the granular material to be appropriate.



If an open graded, permeable, granular backfill is chosen it should not extend to the ground surface. Some granular soils allow ready water migration which may result in increased water access to the foundation soils. The upper few feet of the backfill should be constructed using an impervious soil such as silty-clay and clay soils from the project site, if these soils are available. The 55 degree angle shown in the figure above is approximately correct for most clay soils. The angle is defined by 45 + (φ/2) where "φ" if the angle of internal friction of the soil.

Backfill should not be placed and compacted behind the retaining structure unless approved by the project structural engineer. Backfill placed prior to construction of all appropriate structural members such as floors, or prior to appropriate curing of the retaining wall concrete, may result in severe damage and/or failure of the retaining structure.

6.0 LIMITED SLOPE STABILITY ANALYSIS

This section of the report provides limited, conceptual stability modeling based on our understanding of the proposed excavation cuts that will be required for construction. We performed a limited slope stability analysis of the slope geometry cross section. We obtained measurements of the existing slopes during our field study and utilized cross sections produced by CHC Engineers LLC. The specific design of slope stabilization and shoring structures for the project is beyond our scope of services. The following analyses and concepts presented below are limited in nature and are intended to provide general, conceptual stabilization techniques that are applicable for the subject project. The specific design of the retaining and excavation shorting structures should be performed by a retaining/shoring system specialist. There are firms local to the area that specialize in the design and construction of these systems. We are available to assist you in selecting competent design professionals for the project.

Due to auger refusal on the formational material and/or boulders, we do not know the competency or characteristics of the formational material. Based on and as shown in our analysis below, the upper soil mantel will need to be stabilized, while the lower sandstone and shale layers may only need to incorporate face netting with shallow rock anchors to allow for a safe excavation and to prevent loose rock from scaling away from the rock face during construction. Due to the variability of the subsurface soil, water, and formational material conditions, we recommend a site-specific geotechnical engineering slope stability study be conducted for the structures planned in this portion of proposed development area.

The retaining wall excavations will likely need to be constructed in a top-down excavation strategy utilizing placement of soil nail anchors with steel reinforced shotcrete facing due to the steep nature and extent of the slope surfaces above the proposed rear structure retaining wall, and the potential for rock fall hazard from the excavation itself. It may be possible to utilize a heavy gauge mesh material such as Tecco Mesh for the north and south sides of the excavation that are oriented parallel with the slope fall line as these excavations are less critical with regards to slope stability.

We anticipate that seasonal subsurface water may be present within the slope mass during periods of snow melt or periods of heavy precipitation and included a water table in our analysis. Adequate surface drainage must be constructed in conjunction with the cut/fills to prevent the accumulation of water and hydrostatic pressures.

Our study included a parametric study to assess the sensitivity of the results of the analysis to the changes in the various parameters that were used in our analysis. Our study included observations of the topography and geomorphology of the project site and adjacent areas.

The geometry of the slope cross section that we analyzed is based on site measurements obtained during our field study and provided by CHC Engineers LLC.

There are numerous methods and techniques available for slope stability analysis. Most methods include an evaluation of;

- the strength of the soil materials within the slope,
- anisotropies within the slope materials, such as formational material bedding planes, and anomalous soil contacts,
- the subsurface water and soil moisture conditions, and,
- the pre-construction and post-construction geometry of the slope areas where development and construction are proposed.

The data developed during the analysis is condensed and used to estimate the forces within a soil mass that tend to drive movement and the forces that tend to resist movement. The ratio of resisting forces to driving forces is often referred to as the "theoretical slope factor of safety" (FOS) which is a somewhat misleading term to describe this ratio. The ratio is not a true factor of safety, but is a useful mathematical characterization of the forces within a soil mass and the associated stability condition of the slope being analyzed.

A ratio of less than 1.0 indicates that the driving forces within a soil mass are greater than the resisting forces, therefore movement of the slope is occurring. A ratio of 1.0 indicates that the driving forces are equal to the resisting forces, which indicates that movement within the soil can be triggered by only slight increases in the driving forces or slight reductions in the resisting forces. A ratio of greater than 1.0 is an indication that the driving forces are less than the resisting forces and the slope is not moving. Since there are numerous variables and incongruities within most soil masses, a slope is generally not considered as stable unless the ratio is about 1.5 or greater. Generally, slopes or slope/structure combinations with a theoretical factor of safety that is greater than 1.5 are considered appropriate for sites where structures are planned. A factor of safety greater than about 1.3 is often considered as being stable for roadways and other inhabitable structures. A ratio of 1.2 is often considered suitable for temporary excavation stability.

We used Slide® slope stability software to evaluate the stability of computer modeled slope cross sections of select portions of this site. We primarily used the Modified Bishop's Method of slices to analyze the computer modeled slopes. The Modified Bishop's Method of Slices evaluates the resisting and driving forces within slices of the sloped soil mass along a theoretical semi-circular failure plane. The semicircular failure plane with the lowest theoretical factor of safety is labeled the critical circle.

We have utilized two basic soil/rock horizon in our analyses below. The green-colored region represents the formational material. We estimated an angle of internal friction (phi) of 35 degrees, drained cohesion of 500 pounds per square foot (psf), and a density of 140 pounds per cubic foot for the formational material. The yellow-colored region represents the soil material. We estimated an angle of internal friction (phi) of 30 degrees, drained cohesion of 100 pounds per square foot (psf), and a density of 130 pounds per cubic foot for the formational material.

We analyzed profile cross sections 4, 5, and 6, as provided by CHC Engineers, LLC and shown below on Figure 2 and Figure 3.

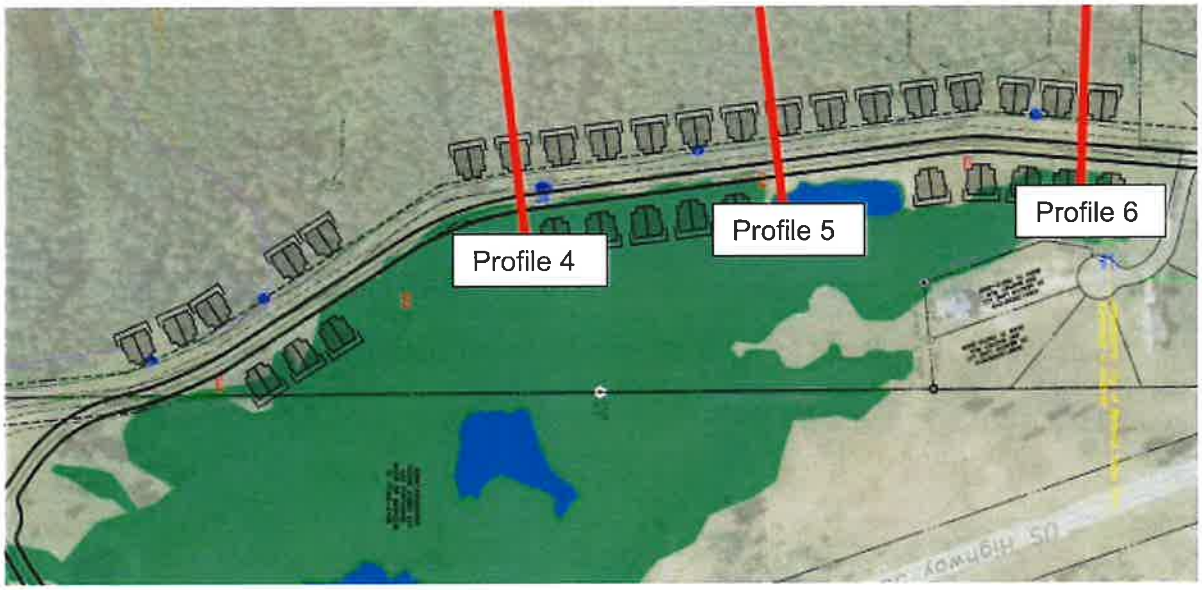


Figure 3. Plan View Locations Profiles 4, 5, and 6.

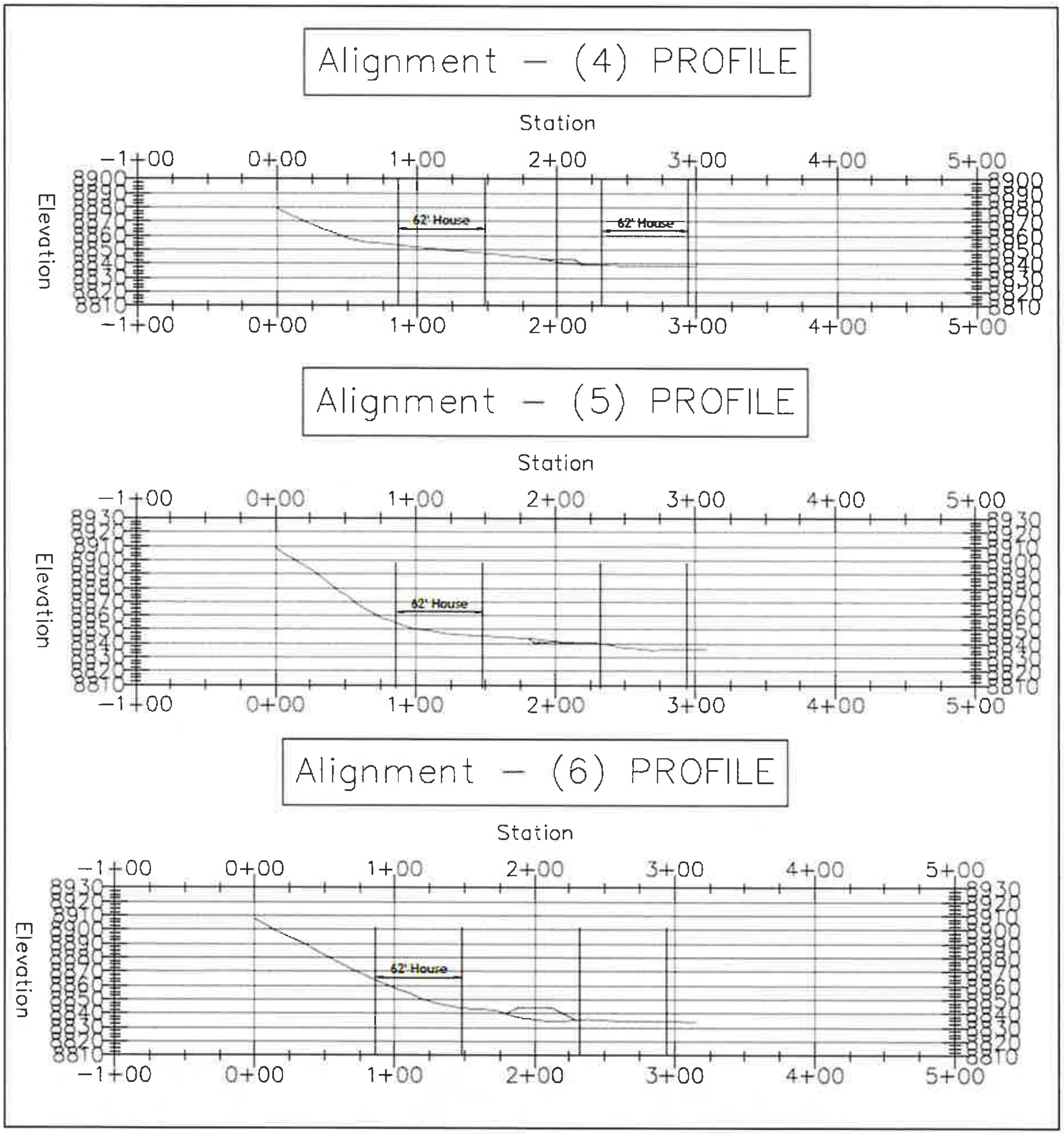


Figure 4. Profile View of Profiles 4, 5, and 6 as provided by CHC Engineers, LLC.

We modeled the existing slope along Profile 4 (not shown) and the resultant estimated factor of safety for the existing slope profile along Profile 4 is 2.125, which should be considered stable given the site soil and water conditions.

The slope profile and stability analysis for an estimated unrestrained 6-foot excavation cut along Profile 4 is shown below on Figure 5.

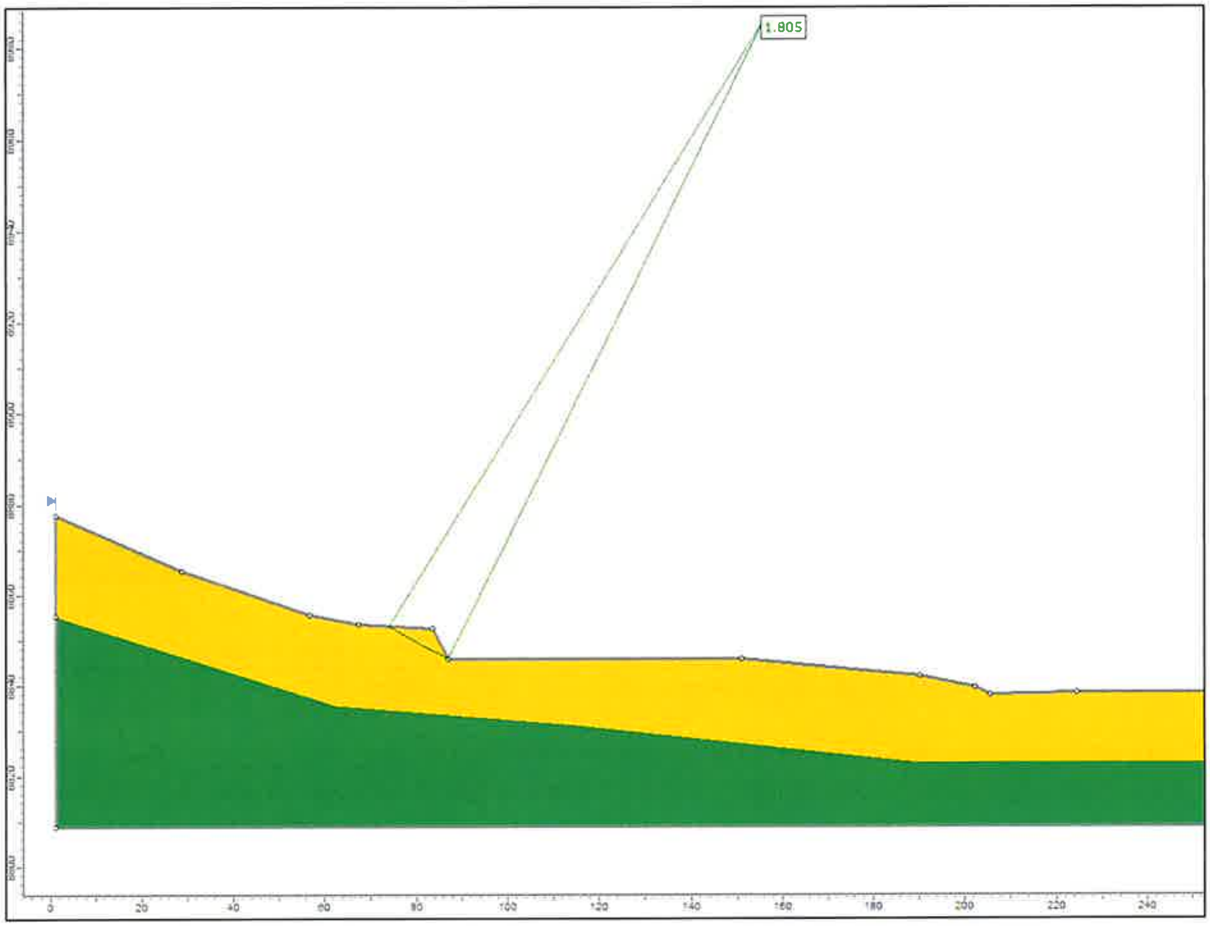


Figure 5: Theoretical F.O.S. for the estimated cut excavation slope conditions (Profile 4), FOS=1.805

The analysis above indicates the estimated factor of safety for the proposed unrestrained excavation cuts for profile 4 is 1.805, which should be considered stable given the site soil and water conditions. The estimated cut height is approximately 6 feet in the above model. If taller excavation cuts are required in this area, we should be contacted to perform an additional analysis.

The existing slope profile and stability analysis along Profile 5 is shown below on Figure 6.

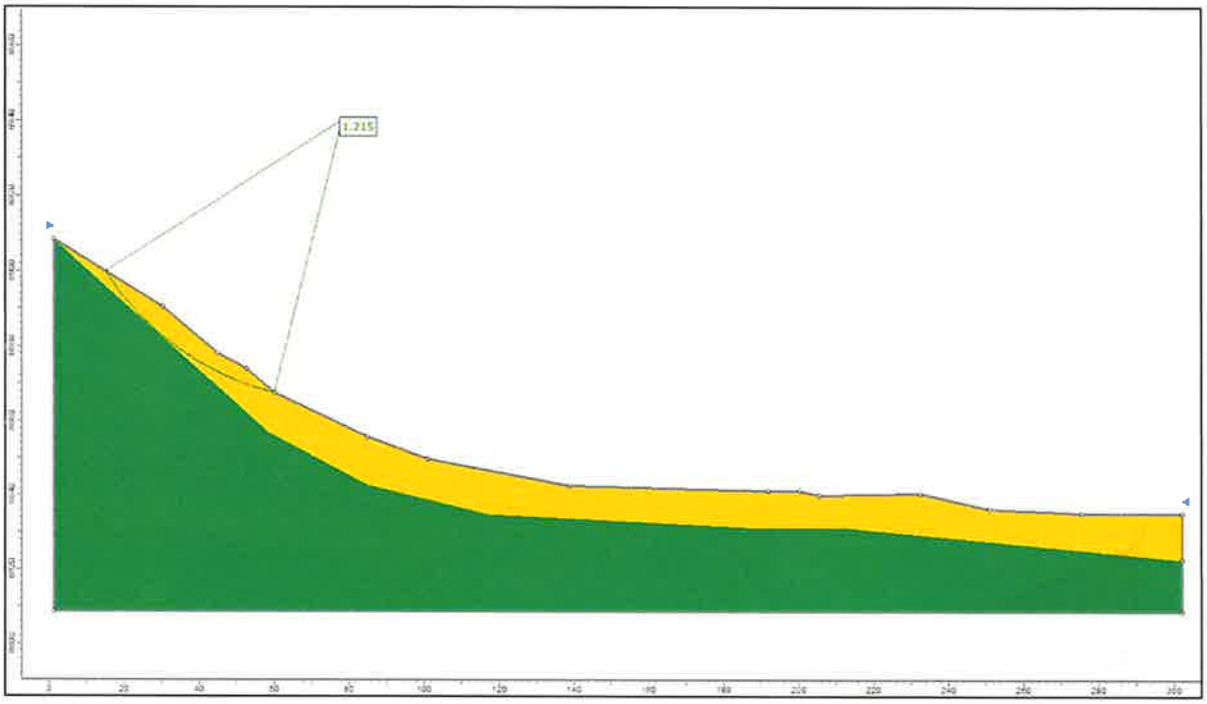


Figure 6: Theoretical F.O.S. for the existing slope conditions (Profile 5), FOS=1.215

The analysis above indicates the estimated factor of safety for the existing slope along Profile 5 is 1.215, which should be considered marginally stable given the site soil and water conditions.

The slope profile for an unrestrained estimated 12-foot excavation cut along Profile 5 is shown below on Figure 7.

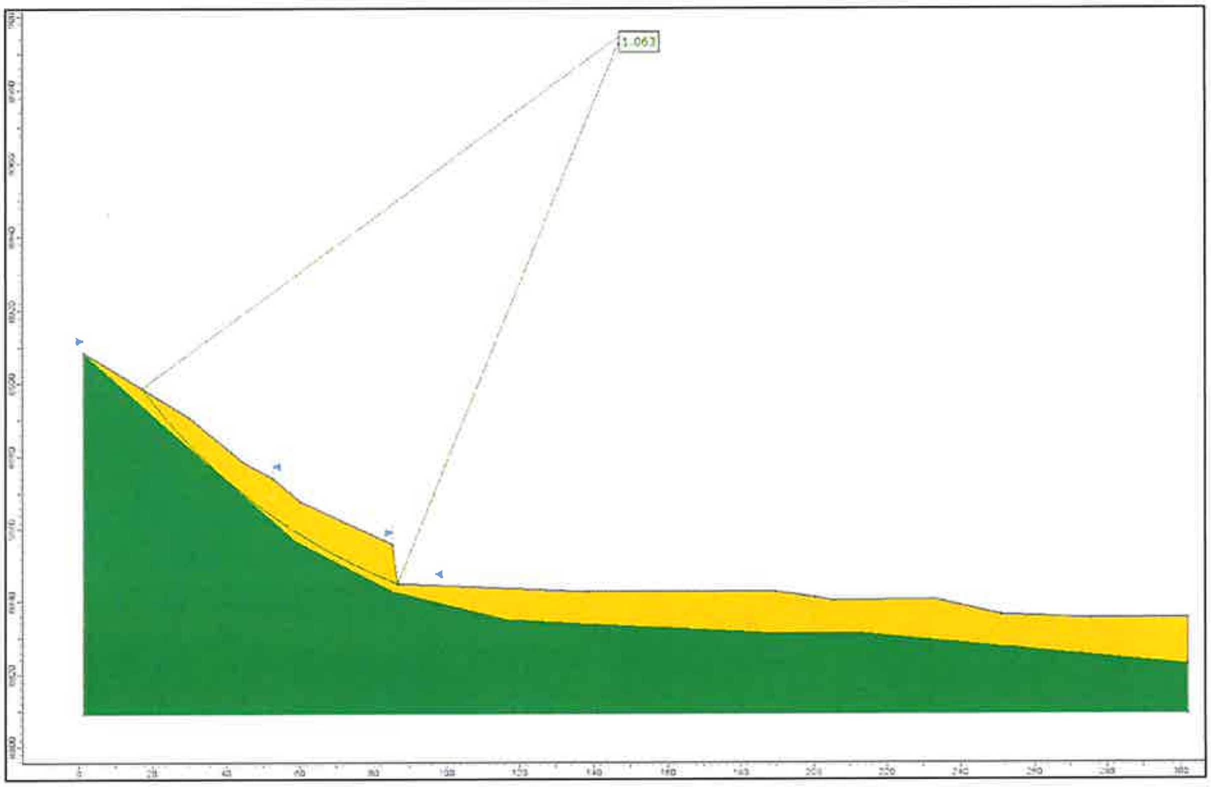


Figure 7: Unrestrained Estimated excavation cut slope conditions along Profile 5.

The analysis above indicates the estimated factor of safety for an unrestrained estimated 12 foot excavation cut for Profile 5 is 1.063, which should be considered unstable to marginally stable given the site soil and water conditions.

The slope profile and analysis for the estimated existing slopes along Profile 6 is shown below on Figure 8.

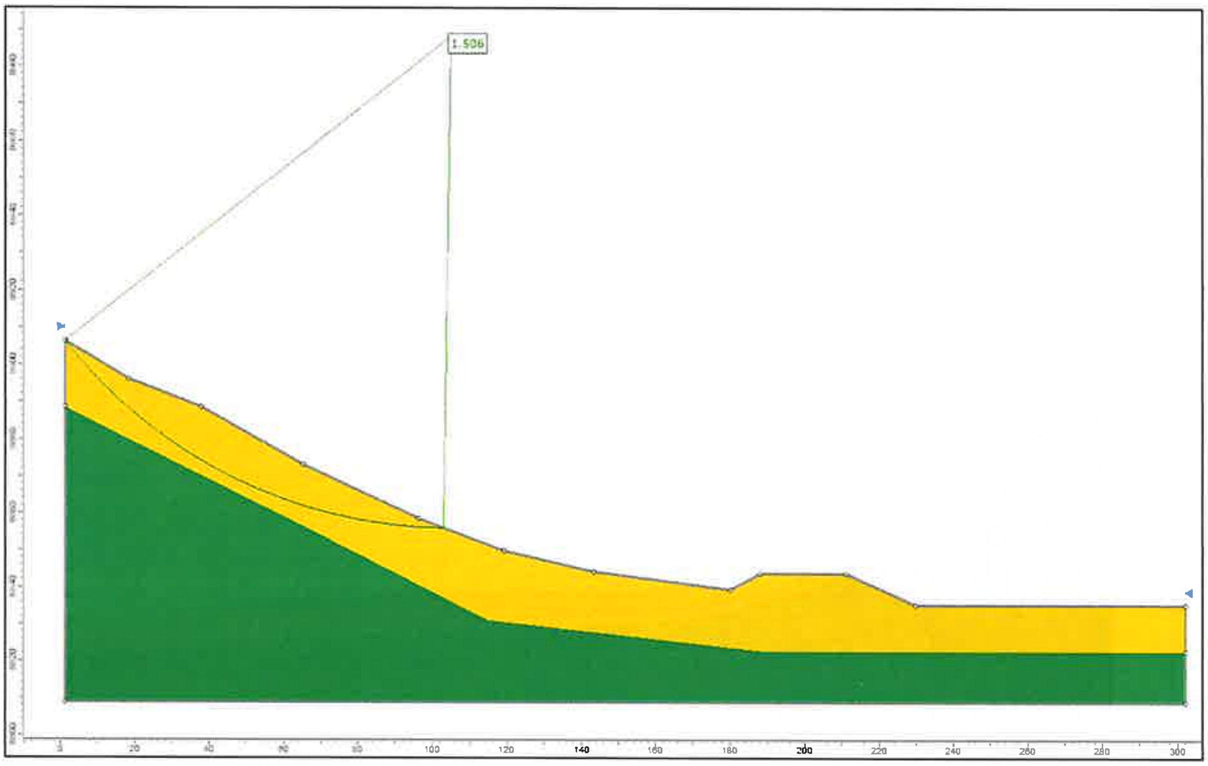


Figure 8: Theoretical F.O.S. for the estimated existing slope conditions along Profile 6, FOS=1.506

The analysis above indicates the estimated factor of safety for the estimated existing slope conditions along Profile 6 is 1.506, which should be considered stable given the site soil and water conditions.

The slope profile for an unrestrained estimated 14-foot excavation cut along Profile 6 is shown below on Figure 9.

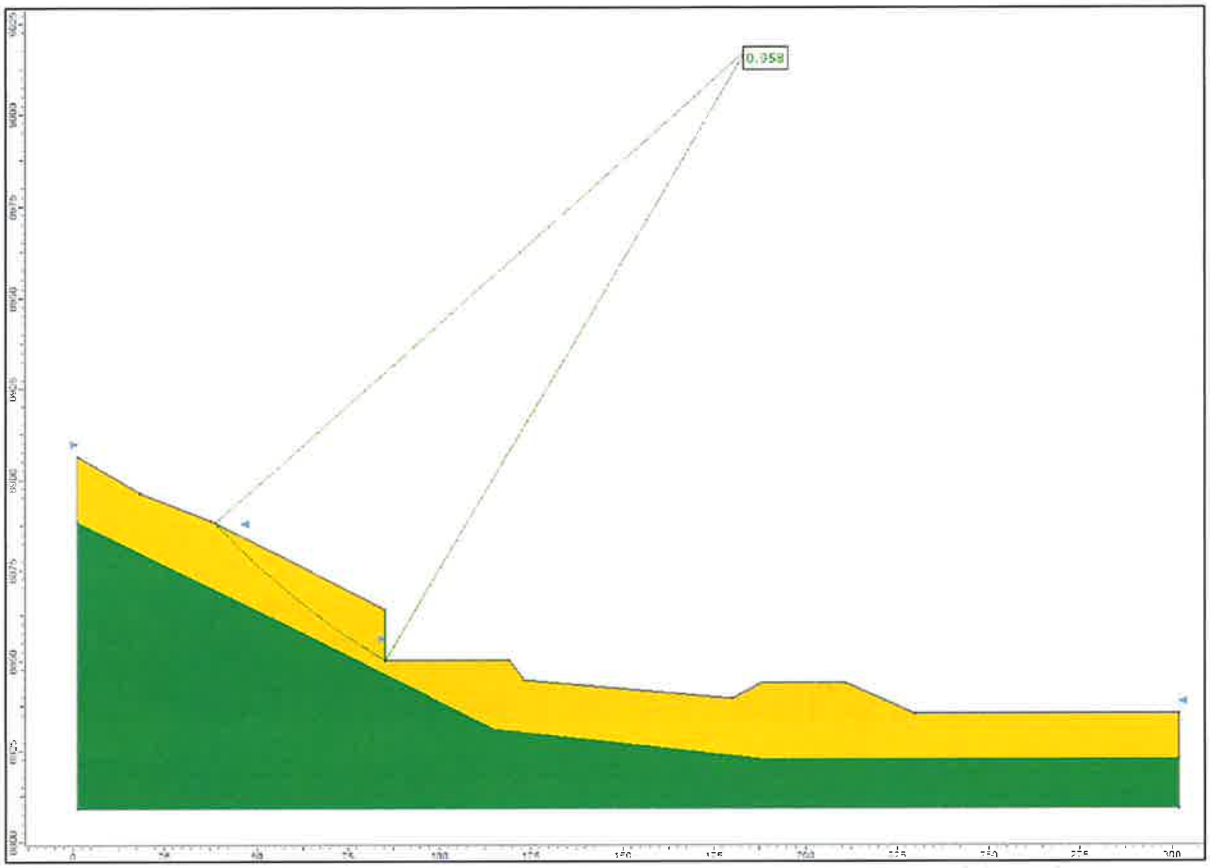


Figure 9: Unrestrained Estimated 14 foot excavation cut slope conditions along Profile 6, F.O.S. 0.958.

The analysis above indicates the estimated factor of safety for an unrestrained estimated 14 foot excavation cut for Profile 6 is 0.958, which should be considered unstable given the site soil and water conditions.

Due to the unstable to marginally unstable cut slope conditions along Profile 5 and Profile 6, we do not recommend additional excavation into the existing cut slope without temporary and/or permanent shoring. We have provided conceptual modeling for soil nail slope revetment for permanent shoring in Figures 10 and 11 below.

We anticipate that soil nails will need to be utilized to stabilize the upper project excavations in the soil mantel and into the site formational materials. The soil nails shown in the analysis below are modeled at 4 feet on center horizontally and vertically with a total embedment depth of 25 feet. The soil nails were modeled with a plunge inclination of about 15 degrees down from the horizontal.

Based on our limited field data to date, we have estimated an allowable soil to grout bond capacity of 1,500 pounds per square foot of nail embedment was used in our analysis and may be used in the design of temporary and/or permanent shoring system(s).

The grout should have a minimum 28 day compressive strength of at least 4,000 pounds per square inch. The amount of grout used to grout each soil nail anchor should be closely monitored in order to insure that the entire volume of the soil nail anchor boring is adequately filled.

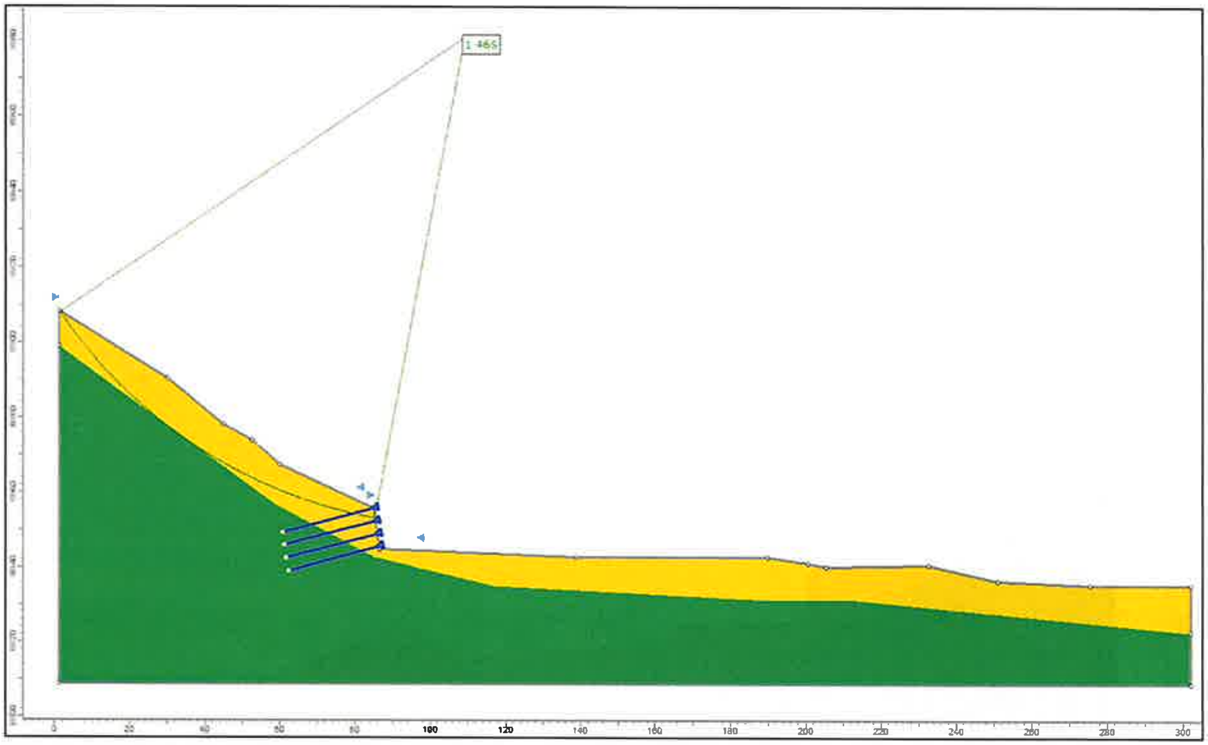


Figure 10: Theoretical F.O.S. for the conceptual cut excavation slope revetment conditions (Section F), FOS=1.465

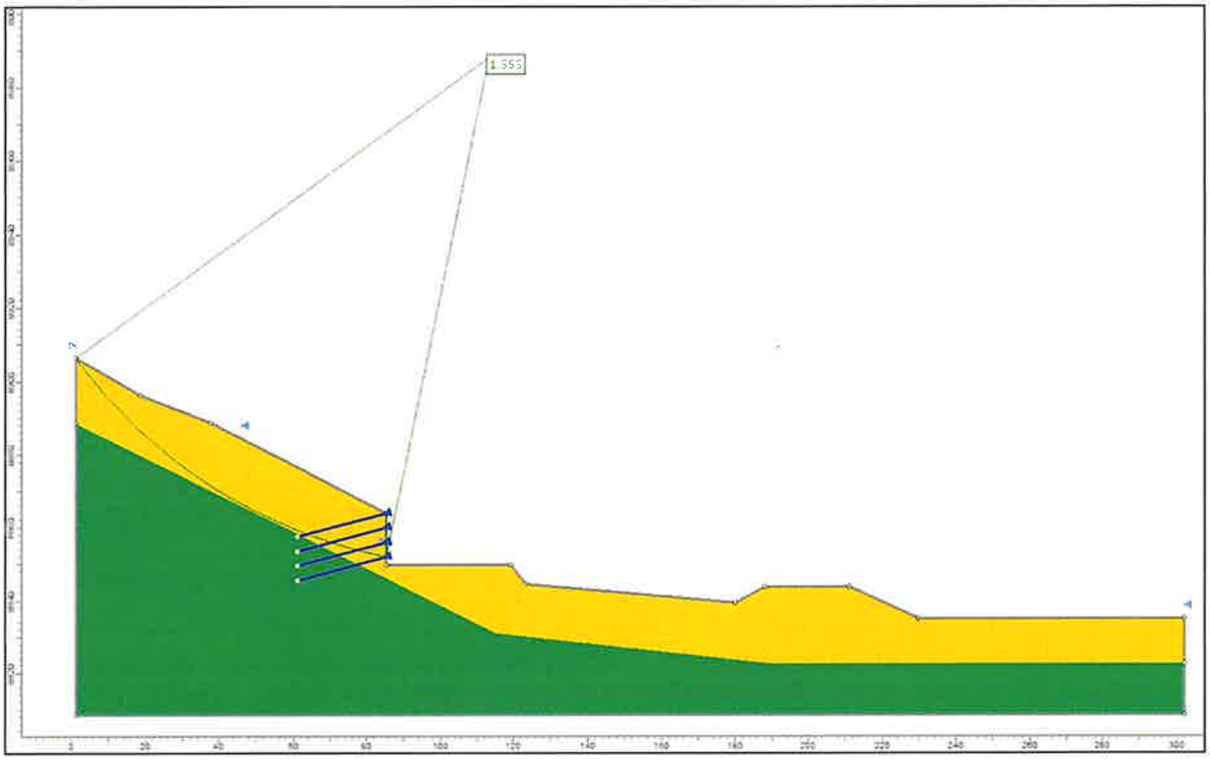


Figure 11: Theoretical F.O.S. for the conceptual cut excavation slope revetment conditions (Profile 6), FOS=1.555

As shown in the analyses presented above, a theoretical factor of safety of 1.465 to 1.555 was achieved in our analysis based on our approximation of the potential excavation cut slopes in these areas of the project. The formational material (green shaded area) will likely require some form of face netting coupled with some shallow nail lengths for where the formational material is encountered to reduce the potential for rocks generated by raveling of these faces from impacting and injuring workers below. We should be contacted to observe the formational material as it is being blasted/excavated to provide additional recommendations.

Saturation of the soil materials retained by the wall system will greatly reduce the stability of the wall system. Surface and subsurface drain systems must be constructed above and/or adjacent to the soil nail retaining wall, and any other retaining walls associated with the structure to help relieve buildup of hydrostatic pressures exerted on the wall systems. A drain blanket such as a Mira Drain product may be installed behind shotcrete structures. Surface water must not be allowed to pond in areas above the retaining wall structure and other unreinforced excavation cut slopes associated with the project.

The specific design of slope stabilization and shoring structures for the project is beyond our scope of services. The specific design of any retaining and excavation shorting structures should be performed by a retaining/shoring system specialist/engineer. There are firms local to the area that specialize in the design and construction of these systems. We are available to assist you in selecting competent design professionals for the project.

This section of our report provides geotechnical engineering design parameters but does not provide a shoring design. The project designer must be contacted to provide a design based on the information presented in this report.

We are available to review and tailor our recommendations as the project progresses and additional information which may influence our recommendations becomes available.

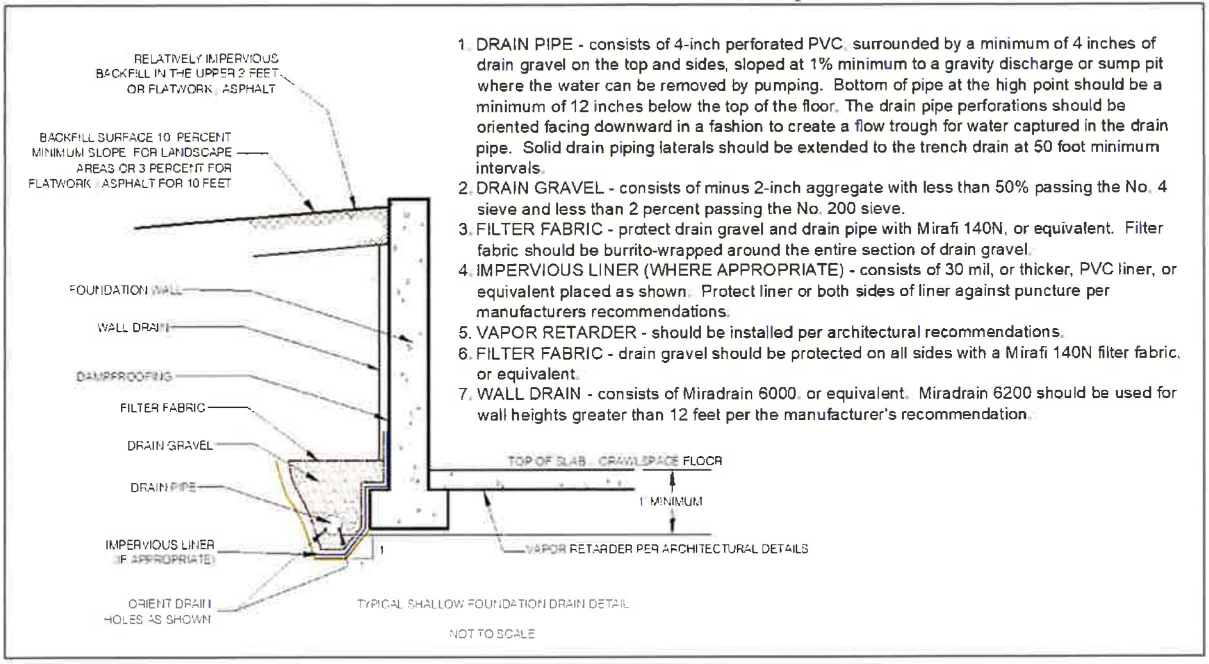
7.0 SUBSURFACE DRAIN SYSTEM

We recommend below-grade construction, such as retaining walls, crawlspace and basement areas, be protected from wetting and hydrostatic pressure buildup by an underdrain and wall drain system. Exterior retaining structures may be constructed with weep holes to allow subsurface water migration through the retaining structures. Topographic conditions on the site may influence the ability to install a subsurface drain system which promotes water flow away from the foundation system. The subsurface drain system concept is discussed under the Subsurface Drain System section of this report below.

A drain system constructed with a free draining aggregate material and a 4 inch minimum diameter perforated drain pipe should be constructed adjacent to retaining structures and/or adjacent to foundation walls. The drain pipe perforations should be oriented facing downward. The system should be protected from fine soil migration by a fabric-wrapped aggregate which surrounds a rigid perforated pipe. We do not recommend use of flexible corrugated perforated pipe since it is not possible to establish a uniform gradient of the flexible pipe throughout the drain system alignment. Corrugated drain tile is perforated throughout the entire circumference of the pipe and therefore water can escape from the perforations at undesirable locations after being collected. The nature of the perforations of the corrugated material further decreases its effectiveness as a subsurface drain conduit.

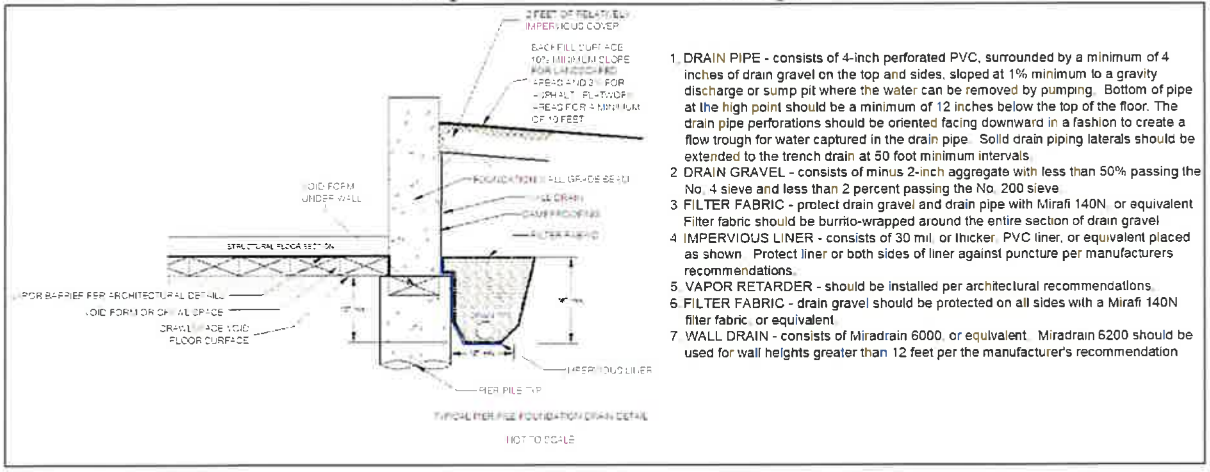
The drain should be placed at each level of excavation and at least 12 inches below lowest adjacent finish floor or crawlspace grade. The drain system pipe should be graded to surface outlets or a sump vault. The drain system should be sloped at a minimum gradient of about 2 percent, but site geometry and topography may influence the actual installed pipe gradient. Water must not be allowed to pool along any portion of the subsurface drain system. An improperly constructed subsurface drain system may promote water infiltration to undesirable locations. The drain system pipe should be surrounded by about 2 to 4 cubic feet per lineal foot of free draining aggregate. If a sump vault and pump are incorporated into the subsurface drain system, care should be taken so that the water pumped from the vault does not recirculate through pervious soils and obtain access to the basement or crawl space areas. An impervious membrane should be included in the drain construction for grade beam and pier systems or other foundation systems such as interrupted footings where a free pathway for water beneath the structure exists. Generalized subsurface drain system concepts are shown below.

Shallow Foundation Drain Concept



1. DRAIN PIPE - consists of 4-inch perforated PVC, surrounded by a minimum of 4 inches of drain gravel on the top and sides, sloped at 1% minimum to a gravity discharge or sump pit where the water can be removed by pumping. Bottom of pipe at the high point should be a minimum of 12 inches below the top of the floor. The drain pipe perforations should be oriented facing downward in a fashion to create a flow trough for water captured in the drain pipe. Solid drain piping laterals should be extended to the trench drain at 50 foot minimum intervals.
2. DRAIN GRAVEL - consists of minus 2-inch aggregate with less than 50% passing the No. 4 sieve and less than 2 percent passing the No. 200 sieve.
3. FILTER FABRIC - protect drain gravel and drain pipe with Mirafi 140N, or equivalent. Filter fabric should be burrito-wrapped around the entire section of drain gravel.
4. IMPERVIOUS LINER (WHERE APPROPRIATE) - consists of 30 mil, or thicker, PVC liner, or equivalent placed as shown. Protect liner or both sides of liner against puncture per manufacturers recommendations.
5. VAPOR RETARDER - should be installed per architectural recommendations.
6. FILTER FABRIC - drain gravel should be protected on all sides with a Mirafi 140N filter fabric, or equivalent.
7. WALL DRAIN - consists of Miradrain 6000, or equivalent. Miradrain 6200 should be used for wall heights greater than 12 feet per the manufacturer's recommendation.

Deep Foundation Drain Concept



1. DRAIN PIPE - consists of 4-inch perforated PVC, surrounded by a minimum of 4 inches of drain gravel on the top and sides, sloped at 1% minimum to a gravity discharge or sump pit where the water can be removed by pumping. Bottom of pipe at the high point should be a minimum of 12 inches below the top of the floor. The drain pipe perforations should be oriented facing downward in a fashion to create a flow trough for water captured in the drain pipe. Solid drain piping laterals should be extended to the trench drain at 50 foot minimum intervals.
2. DRAIN GRAVEL - consists of minus 2-inch aggregate with less than 50% passing the No. 4 sieve and less than 2 percent passing the No. 200 sieve.
3. FILTER FABRIC - protect drain gravel and drain pipe with Mirafi 140N, or equivalent. Filter fabric should be burrito-wrapped around the entire section of drain gravel.
4. IMPERVIOUS LINER - consists of 30 mil, or thicker, PVC liner, or equivalent placed as shown. Protect liner or both sides of liner against puncture per manufacturers recommendations.
5. VAPOR RETARDER - should be installed per architectural recommendations.
6. FILTER FABRIC - drain gravel should be protected on all sides with a Mirafi 140N filter fabric, or equivalent.
7. WALL DRAIN - consists of Miradrain 6000, or equivalent. Miradrain 6200 should be used for wall heights greater than 12 feet per the manufacturer's recommendation.

There are often aspects of each site and structure which require some tailoring of the subsurface drain system to meet the needs of individual projects. Drain systems that are placed adjacent to void forms must include provisions to protect and support the impervious liner adjacent to the void form. We are available to provide consultation for the subsurface drain system for this project, if desired.

Water often will migrate along utility trench excavations. If the utility trench extends from areas above the site, this trench may be a source for subsurface water within the proposed basement or crawl space. We suggest that the utility trench backfill be thoroughly compacted to help reduce the amount of water migration. The subsurface drain system should be designed to collect subsurface water from the utility trench and direct it to surface discharge points.

8.0 CONCRETE FLATWORK

We anticipate that both interior and exterior concrete flatwork will be considered in the project design. Concrete flatwork is typically lightly loaded and has a limited capability to resist shear forces associated with uplift from swelling soils and/or frost heave. It is prudent for the design and construction of concrete flatwork on this project to be able to accommodate some movement associated with swelling soil conditions.

The soil samples tested have a measured swell pressure up to about 5,000 pounds per square foot and a magnitude swell potential of about 7.6 percent under a 100 pound per square foot surcharge load. Due to the measured swell potential and swell pressure, interior floors supported over a crawl space are less likely to experience movement than are concrete slabs support on grade. The following recommendations are appropriate for garage floor slabs and for interior floor slabs if the owner is willing to accept the risk of potential movement beyond normal tolerances.

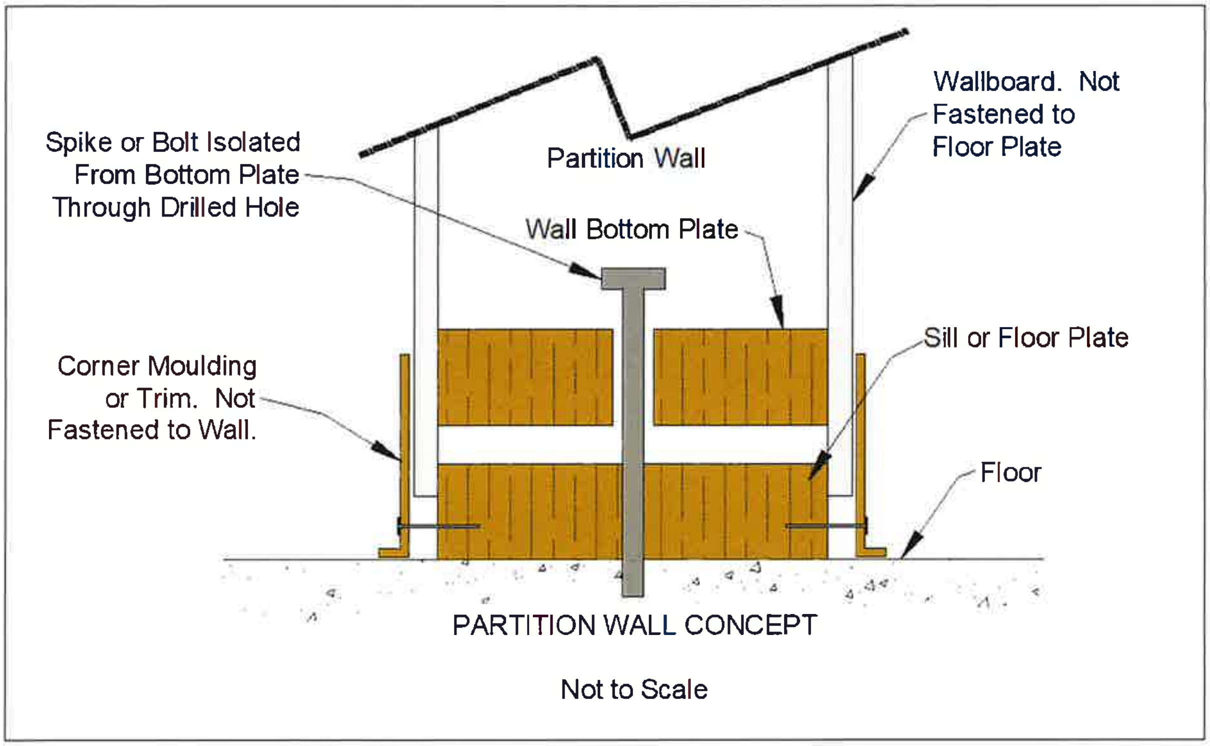
We do not recommend slab-on-grade floor construction in the areas noted to have high organic content soils with a high consolidation potential which are generally the areas between TB-10 and TB-12. If development is planned in these areas, all flooring systems should be structurally supported.

8.1 Interior Concrete Slab-on-Grade Floors

A primary goal in the design and construction of concrete slab-on-grade floors is to reduce the amount of post construction uplift associated with swelling soils, or downward movement due to consolidation of soft soils. A parallel goal is to reduce the potential for damage to the structure associated with any movement of the slab-on-grade which may occur. There are limited options available to help mitigate the influence of volume changes in the support soil for concrete slab-on-grade floors, these include:

- Preconstruction scarification, moisture conditioning and re-compaction of the natural soils in areas proposed for support of concrete flatwork, and/or,
- Placement and compaction of granular compacted structural fill material

Damage associated with movement of interior concrete slab-on-grade floor can be reduced by designing the floors as “floating” slabs. The concrete slabs should not be structurally tied to the foundations or the overlying structure. Interior walls or columns should not be supported on the interior floor slabs. Movement of interior walls or columns due to uplift of the floor slab can cause severe damage throughout the structure. Interior walls may be structurally supported from framing above the floor, or interior walls and support columns may be supported on interior portions of the foundation system. Partition walls should be designed and constructed with voids above, and/or below, to allow independent movement of the floor slab. This concept is shown below.



PARTITION WALL CONCEPT

Not to Scale

The sketch above provides a concept. If the plans include isolation of the partition walls from the floor slab, the project architect or structural engineer should be contacted to provide specific details and design of the desired system.

If the owner chooses to construct concrete slab-on-grade floors, the floors should be supported by a layer of granular structural fill overlying the processed natural soils. Interior concrete flatwork, or concrete slab-on-grade floors, should be underlain by scarification, moisture conditioning and compaction of about 6 inches of the natural soils followed by placement of at least 18 inches of compacted granular structural fill material that is placed and compacted as discussed in the Construction Considerations, "Fill Placement Recommendations" section of this report, below.

The above recommendations will not prevent slab heave if the expansive soils underlying slabs-on-grade become wet. However, the recommendations will reduce the effects if slab heave occurs. All plumbing lines should be pressure tested before backfilling to help reduce the potential for wetting. The only means to completely mitigate the influence of volume changes on the performance of interior floors is to structurally support the floors over a void space. Floors that are suspended by the foundation system will not be influenced by volume changes in the site soils. The suggestions and recommendations presented in this section are intended to help reduce the influence of swelling soils on the performance of the concrete slab-on-grade floors.

8.1.1 Capillary and Vapor Moisture Rise

Capillary and vapor moisture rise through the slab support soil may provide a source for moisture in the concrete slab-on-grade floor. This moisture may promote development of mold or mildew in poorly ventilated areas and may influence the performance of floor coverings and mastic placed directly on the floor slabs. The type of floor covering, adhesives used, and other considerations that are not related to the geotechnical engineering practice will influence the design. The architect, builder and particularly the floor covering/adhesive manufacturer should be contacted regarding the appropriate level of protection required for their products.

Comments for Reduction of Capillary Rise

One option to reduce the potential for capillary rise through the floor slab is to place a layer of clean aggregate material, such as washed concrete aggregate for the upper 4 to 6 inches of fill material supporting the concrete slabs.

Comments for Reduction of Vapor Rise

To reduce vapor rise through the floor slab, a moisture barrier such as a 6 mil (or thicker) plastic, or similar impervious geotextile material is often be placed below the floor slab. The material used should be protected from punctures that will occur during the construction process.

There are proprietary barriers that are puncture resistant that may not need the underlying layer of protective material. Some of these barriers are robust material that may be placed below the compacted structural fill layer. We do not recommend placement of the concrete directly on a moisture barrier unless the concrete contractor has had previous experience with curing of concrete placed in this manner. As mentioned above, the architect, builder and particularly the floor covering/adhesive manufacturer should be contacted regarding the appropriate level of moisture and vapor protection required for their products.

8.1.2 Slab Reinforcement Considerations

The project structural engineer should be contacted to provide steel reinforcement design considerations for the proposed floor slabs. Any steel reinforcement placed in the slab should be placed at the appropriate elevations to allow for proper interaction of the reinforcement with tensile stresses in the slab. Reinforcement steel that is allowed to cure at the bottom of the slab will not provide adequate reinforcement.

8.2 Exterior Concrete Flatwork Considerations

Exterior concrete flatwork includes concrete driveway slabs, aprons, patios, and walkways. The desired performance of exterior flatwork typically varies depending on the proposed use of the site and each owner's individual expectations. As with interior flatwork, exterior flatwork is particularly prone to movement and potential damage due to movement of the support soils. This movement and associated damage may be reduced by following the recommendations discussed under interior flatwork, above. Unlike interior flatwork, exterior flatwork may be exposed to frost heave, particularly on sites where the bearing soils have a high silt content. It may be prudent to remove silt soils from exterior flatwork support areas where movement of exterior flatwork will adversely affect the project, such as near the interface between the driveway and the interior garage floor slab. If silt soils are encountered, they should be removed to the maximum depth of frost penetration for the area where movement of exterior flatwork is undesirable.

If some movement of exterior flatwork is acceptable, we suggest that the support areas be prepared by scarification, moisture conditioning and re-compaction of about 6 inches of the natural soils followed by placement of at least 12 inches of compacted granular fill material. The scarified material and granular fill materials should be placed as discussed under the Construction Considerations, "Fill Placement Recommendations" section of this report, below.

It is important that exterior flatwork be separated from exterior column supports, masonry veneer, finishes and siding. No support columns, for the structure or exterior decks, should be placed on exterior concrete unless movement of the columns will not adversely affect the supported structural components. Movement of exterior flatwork may cause damage if it is in contact with portions of the structure exterior.

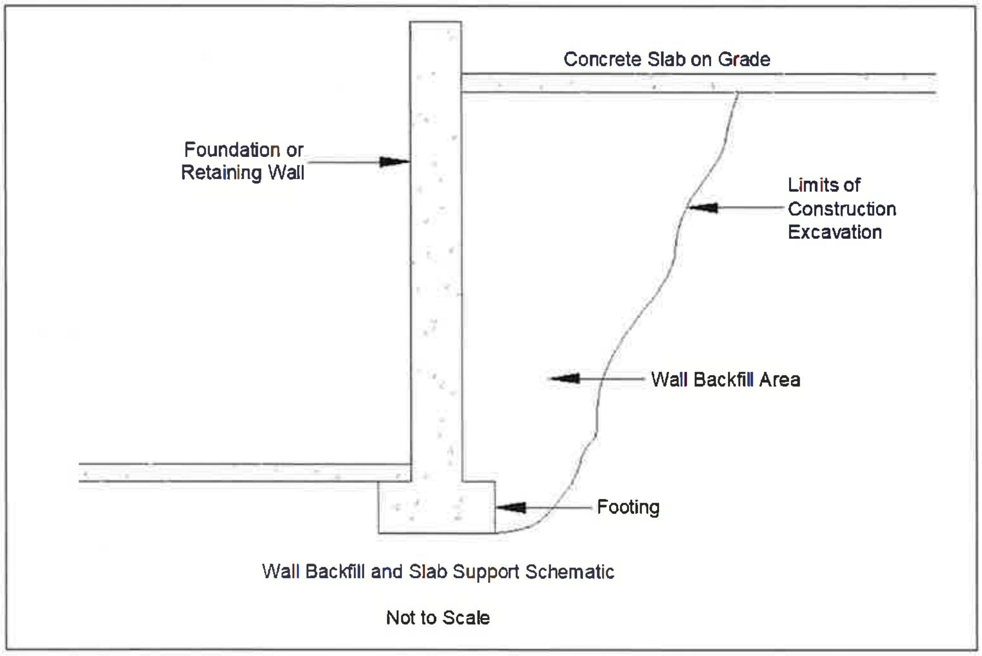
It should be noted that silt and silty sand soils located near the ground surface are particularly prone to frost heave. Soils with high silt content have the ability to retain significant moisture. The ability for the soils to accumulate moisture combined with a relatively shallow source of subsurface water and the fact that the winter temperatures in the area often very cold all contribute to a high potential for frost heave of exterior structural components. We recommend that silty soils be removed from the support areas of exterior components that are sensitive to movement associated with frost heave. These soils should be replaced with a material that is not susceptible to frost heave. Aggregate road base and similar materials retain less water than fine-grained soils and are therefore less prone to frost heave. We are available to discuss this concept with you as the plans progress.

Landscaping and landscaping irrigation often provide additional moisture to the soil supporting exterior flatwork. Excessive moisture will promote heave of the flatwork either due to expansive soil, or due to frost action. If movement of exterior slabs is undesirable, we recommend against placement of landscaping that requires irrigation. The ground surfaces near exterior flatwork must be sloped away from flatwork to reduce surface water migration to the support soil.

Exterior flatwork should not be placed on soils prepared for support of landscaping vegetation. Cultivated soils will not provide suitable support for concrete flatwork.

8.3 General Concrete Flatwork Comments

It is relatively common that both interior and exterior concrete flatwork is supported by areas of fill adjacent to either shallow foundation walls or basement retaining walls. A typical sketch of this condition is shown below.



Settlement of the backfill shown above will create a void and lack of soil support for the portions of the slab over the backfill. Settlement of the fill supporting the concrete flatwork is likely to cause damage to the slab-on-grade. Settlement and associated damage to the concrete flatwork may occur when the backfill is relatively deep, even if the backfill is compacted.

If this condition is likely to exist on this site it may be prudent to design the slab to be structurally supported on the retaining or foundation wall and designed to span to areas away from the backfill area as designed by the project structural engineer. We are available to discuss this with you upon request.

9.0 PAVEMENT SECTION THICKNESS DESIGN RECOMMENDATIONS

We have provided recommendations for a flexible asphalt and rigid Portland concrete pavement sections. We have provided our traffic estimates in Section 9.1 below. Our flexible asphalt pavement section thickness recommendations are provided in Section 9.2 and general asphalt pavement construction recommendations are provided in Section 9.3. Rigid Portland concrete recommendations are provided in Section 9.4.

9.1 Traffic Estimates

Traffic projections and corresponding 18,000 pound (18k) equivalent single axel load (ESAL) factors were not available at the time of this report. We have provided conceptual pavement section thickness recommendations for an assumed 100,000 ESALs. If higher ESAL values are anticipated or if alternative recommendations are required, the pavement sections presented in this report should be re-evaluated.

9.2 Asphalt Pavement Design Recommendations

The aggregate materials used within the pavement section should conform to the requirements outlined in the current Specifications for Road and Bridge Construction, Colorado Department of Transportation (CDOT). The aggregate base material should be a ¾-inch minus material that conforms to the CDOT Class 6 aggregate base course specifications and have an R-value of at least 78. The aggregate sub-base course should conform to the CDOT specifications for Class 2 material and should have a minimum R-value 70. Other material may be suitable for use in the pavement section, but materials different than those listed above should be tested and observed by us prior to inclusion in the project design or construction. Aggregate sub-base and base-course materials should be compacted to at least 95 percent of maximum dry density as defined by the modified Proctor test, ASTM D1557.

We recommend that the asphalt concrete used on this project be mixed in accordance with a design prepared by a licensed professional engineer, or an asphalt concrete specialist. We should be contacted to review the mix design prior to placement at the project site. We recommend that the asphalt concrete be compacted to between 92 and 96 percent of the maximum theoretical density.

We have provided several pavement section design thicknesses for 100,000 estimated ESALs. The project civil engineer, or contractor can evaluate the best combination of materials for economic considerations.

Based on the laboratory analysis of the native soils, we obtained a CBR value of 4.1 and estimated an R-Value of 9 and a resilient modulus of 3,450. Other assumptions made for our analysis are listed below.

- Reliability Factor R(%) = 85%
- Overall Standard Deviation, So = 0.44
- Estimated Total 18K-ESAL value(s) = 100,000
- Effective Roadbed Soils Resilient Modulus, Mr = 3,450
- Change is serviceability index, Delta PSI = 2.5
- Structural Coefficient of Asphalt Pavement = 0.44
- Structural Coefficient of Aggregate Base Course = 0.12
- Structural Coefficient of Aggregate Sub-Base Course = 0.09
- Modifying Structural Layer Coefficients for aggregate base course and aggregate sub-base course layers, mi = 1.0 (fair drainage conditions with 5%-25% saturation frequency)

We have estimated a pavement reliability factor (R) of 85 percent. The Federal Highway Administration defines R as "the probability that a pavement section will perform satisfactorily over the design period. It must account for uncertainties in traffic loading, environmental conditions, and construction materials. The AASHTO design method accounts for these uncertainties by incorporating a reliability level R to provide a factor of safety into the pavement design and thereby increase the probability that the pavement will perform as intended over its design life." A higher R will result in thicker pavement section materials; however, may lead to a greater reliability in the pavement performance. The designer or project civil engineer should evaluate the desired R factor for the intended use. We can provide alternate reliability factors for the proposed pavement section upon request.

Based on the above assumptions and laboratory test data obtained for the native on-site soil materials, we obtained a structural number (SN) equal to 2.91 for an assumed 100,000 18k-ESAL. Our pavement thickness design recommendations are provided below. We have shown alternate pavement sections below that meet the minimum structural numbers.

Pavement Section Design Thickness – 100,000 ESAL (Minimum SN = 2.91)

Pavement Section Component	Alternative Thickness of Each Component (inches)			
Asphalt Concrete	4	4	4.5	5
Class 6 Roadbase	4	10	4	6
Class 2 Sub-Base	8	0	6	0
Structural Number	2.96	2.96	3.00	2.92

We do not recommend use of ¾ inch aggregate base course in layers less than 4 inches or the use of 3-inch minus sub-base in layers less than 6 inches. This may result in total structural numbers that are in excess of the minimum required by the anticipated traffic loading as can be seen in the tables above.

Water intrusion into the pavement section support materials will negatively influence the performance of the parking lot surface. Water from irrigation, water from natural sources that migrates into the soils beneath landscapes surface and water from any source that gains access to the support materials can all decrease the life of the parking lot surface. Care should be taken along curbs and any edge of the parking lot to develop an interface between the material that will reduce subsurface and surface water migration into the support soil and pavement section materials. Landscape islands and other irrigated features often promote water migration since no surface flow from these features typically occurs. The same can occur along perimeter cub areas.

Water will often migrate along the interface of concrete curbs and gutter areas early in the life of any parking area. The tendency for this type of migration often decreases with time but can be reduced by compaction of materials along the outside base of curb areas adjacent to the interface of the concrete curb and the underlying soil prior to placement of landscaping soil above this interface.

9.3 General Asphalt Pavement Recommendations

The asphalt pavement used on this project should be mixed in accordance with a design prepared by a licensed professional engineer, or an asphalt pavement specialist. We should be contacted to review the mix design prior to placement at the project site. We recommend that the asphalt pavement be compacted to between 92 and 96 percent of the maximum theoretical density.

We suspect that the subgrade soils will be well above the optimum moisture content in many areas of the project. We anticipate that conventional scarification and drying of the subgrade soils will be sufficient for most areas of the roadway subgrade provided warm and preferably breezy weather conditions are present during the project construction, and there is adequate time to perform scarification and drying construction procedures. However, it is likely that some areas of the subgrade will require specialty stabilization techniques. We have provided cursory recommendations for stabilization of severely yielding soil materials in Section 5.0 below.

The subgrade soil materials should be scarified to a depth of about 8 inches, moisture conditioned, and compacted to at least 90 percent of the maximum dry density as defined by ASTM D1557 or AASHTO T180 (Modified Proctor). Proof rolling observations should then be performed over the prepared subgrade surface. Any areas of significant yielding should be stabilized as needed prior to placement of the overlying aggregate base course materials. The surface of the subgrade soil should be graded and contoured to be approximately parallel to the finished grade of the asphalt surface.

The aggregate materials used within the pavement section should conform to the requirements outlined in the current Specifications for Road and Bridge Construction, Colorado Department of Transportation (CDOT). The aggregate base material should be a ¾ inch minus material that conforms to the CDOT Class 6 aggregate base course specifications and have an R-value of at least 78. The aggregate sub-base course should conform to the CDOT specifications for Class 2 material and should have a minimum R-value 70. Other material may be suitable for use in the pavement section, but materials different than those listed above should be tested and observed by us prior to inclusion in the project design or construction. Aggregate sub-base and base-course materials should be compacted to at least 95 percent of maximum dry density as defined by the modified Proctor test, ASTM D1557.

Thorough proof rolling with a fully loaded tandem axle water truck should be performed across the prepared aggregate surface prior to placement of the asphalt cement. Any areas that are observed to yield should be stabilized as necessary. We should be contacted to observe the proof rolling operations and provide recommendations for stabilization if necessary.

The drainage characteristics of the roadway should be addressed by the project civil engineer. Surface water must not be allowed to pool in areas adjacent to the asphalt pavement roadway.

9.4 Portland Cement Concrete Pavement Recommendations

For concrete pavements (rigid pavements), we recommend a minimum of 5-inches of Portland cement concrete (PCC). Concrete pavement underlain by 12 inches Class 6 aggregate base course is recommended 1) to create a uniform subbase/base, 2) to limit potential of pumping of fines from beneath the pavement, 3) provide a working platform for construction, and 4) to help control frost heave soils.

All concrete should be based on a mix design established by a qualified engineer. A CDOT Class P or D mix would be acceptable. The design mix should consist of aggregate, Portland cement, water, and additives which will meet the requirements contained in this section. The concrete should have a modulus of rupture of third point loading of 650 psi. Normally, concrete with a 28-day compressive strength of 4,200 psi will meet this requirement. Concrete should contain approximately 6 percent entrained air. Maximum allowable slump should not exceed 4 inches.

The concrete should contain joints not greater than 10 feet on centers. Joints should be sawed or formed by pre-molded filler. The joints should be at least 1/3 of the slab thickness. Joints should be reinforced with dowels to provide load transfer between slabs. Concrete pavement joints should meet the requirements of CDOT Standard Plan No. M 412-1 and CDOT Standard Specifications Section 412.13. Expansion joints should be provided at the end of each construction sequence and between the concrete slab and adjacent structures. Expansion joints, where required, should be filled with a ½-inch thick asphalt impregnated fiber. Concrete should be cured by protecting against loss of moisture, rapid temperature changes and mechanical injury for at least three days after placement. After sawing joints, the saw residue shall be removed and the joint sealed.

10.0 CONSTRUCTION CONSIDERATIONS

This section of the report provides comments, considerations and recommendations for aspects of the site construction which may influence, or be influenced by the geotechnical engineering considerations discussed above. The information presented below is not intended to discuss all aspects of the site construction conditions and considerations that may be encountered as the project progresses. If any questions arise as a result of our recommendations presented above, or if unexpected subsurface conditions are encountered during construction we should be contacted immediately.

10.1 Fill Placement Recommendations

There are several references throughout this report regarding both natural soil and compacted structural fill recommendations. The recommendations presented below are appropriate for the fill placement considerations discussed throughout the report above.

All areas to receive fill, structural components, or other site improvements should be properly prepared and grubbed at the initiation of the project construction. The grubbing operations should include scarification and removal of organic material and soil. No fill material or concrete should be placed in areas where existing vegetation or fill material exist.

We observed evidence of previous site use and existing man-placed fill during our field work. We encountered man-placed fill in our test borings. We suspect that man-placed fill and subterranean structures may be encountered as the project construction progresses. All existing fill material should be removed from areas planned for support of structural components. Excavated areas and subterranean voids should be backfilled with properly compacted fill material as discussed below.

Preloading of the ground surface and a settlement monitoring program may be necessary prior to construction to limit the amount of post construction ariel settlement in the areas near TB-11 through TB-13. Conceptually, the preloading program would likely consist of placement of a series of steel plates at the base of a controlled fill. The plates would have steel rods that extend to the ground surface as survey monuments. Settlement of the fill mass could then be monitored by a survey program to determine amount of settlement and when settlement ceases.

10.1.1 Subgrade Soil Stabilization

We suspect that soft, yielding soil conditions may be encountered at various locations on the project site during construction. This material may be challenging to compact in preparation for placement of overlying fill material. We have provided two general categories of concepts to stabilize these soils to provide a suitable substrate for placement and compaction of overlying compacted fill. These include:

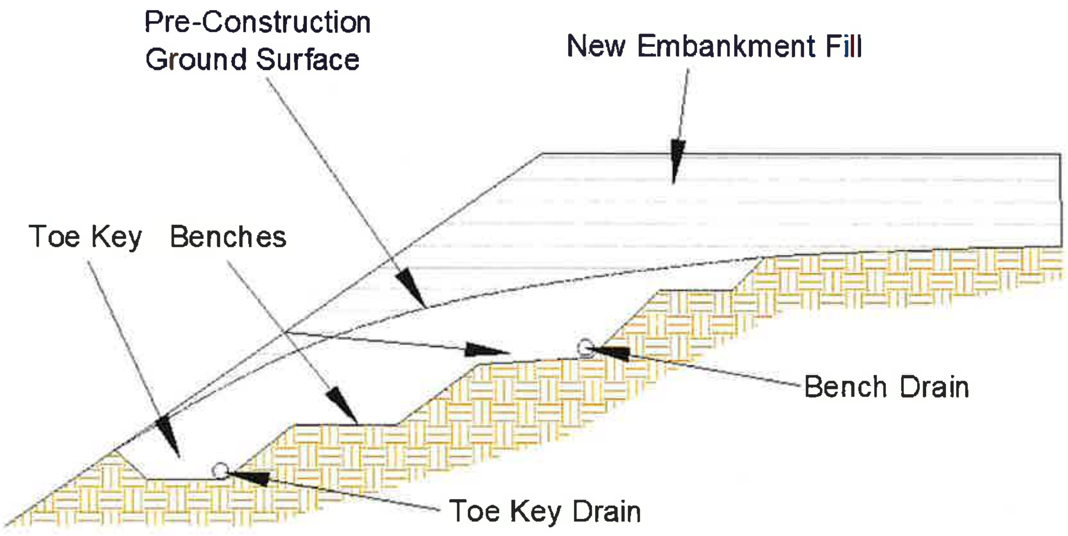
1.) Mechanical Stabilization; using soil and/or geotextile materials, and,
2.) Chemical Stabilization; using dry Portland cement.

Mechanical stabilization of soil often includes placement of aggregate material and/or larger cobbles (3-4 inch size) into an area where the soils are yielding. The most predictable technique is to over-excavate these soft areas by about 8 to 12 inches, (or more, if needed) lightly proof compact the exposed soil, place a layer of woven geosynthetic or geogrid-type material, such as or Mirifi RS 280i or BXG 120 geogrid, followed by placement of a "clean crushed aggregate" material with a nominal maximum size of 3 inches and not more than about 5 percent passing the #4 sieve. This clean crushed aggregate material should then be consolidated with a plate-type compactor. A less robust fabric, such as a non-woven geofabric, (such as Mirifi 140N) is placed on top of this aggregate layer followed by placement and compaction of the overlying fill material. For sites with extremely soft conditions it may be necessary to increase the clean aggregate layer to about 18 inches and place an intermediate layer of geogrid (or fabric) at mid-height of this layer.

Chemical stabilization using Portland cement is effective for most soils. Generally, this technique is more suitable for isolated soft areas. Generally dry Portland cement powder may be placed on the surface of the soft yielding material and subsequently mixed into the soil. The effectiveness of this technique is partially dependent upon the thoroughness of the mixing. If it can be thoroughly mixed the application rate of the Portland cement need not be more than 10 percent, and often an application of 5 to 7 percent will provide a significant decrease in free water and stabilize the material. After mixing, the material should be allowed to "rest" for about two of more hours prior to compaction. The treated material will often yield some during initial compaction, but will generally increase in rigidity as the process of hydration begins takes place. If yielding under compaction is excessive, the material should be allowed "cure" additionally prior to continued compaction effort being applied. Often it takes more time, such as overnight, to allow the cement to fully stabilize the material so this strategy is often implemented in an area at the end of a work day and allowed to cure overnight followed by subsequent fill placement on the following day.

10.1.2 Embankment Fill on Slopes

Embankment fill placed on slopes must be placed in areas that have been properly prepared prior to placement of the fill material. The fill should be placed in a toe key and benches constructed into the slope. The concept is shown below.



Toe Key and Bench Drain
Concept Schematic

Not to Scale

The width of the toe key should be at least one-fourth of the height of the fill. The elevation difference between each bench, width, and geometry of each bench is not critical; however, the elevation difference between each lift should not exceed about 3 to 4 feet. The benches should be of sufficient width to allow for placement of horizontal lifts of fill material; therefore, the size of the compaction equipment used will influence the bench widths.

Embankment fill material thicker than 5 feet should be analyzed on a site-specific basis. The fill mass may impose significant loads on, and influence the stability of the underlying slope. We suggest that no fill slopes steeper than two and one-half to one (2½:1, horizontal to vertical) be constructed unless a slope stability analysis of the site is conducted.

The toe key and bench drains shown above should be placed to reduce the potential for water accumulation in the embankment fill and in the soils adjacent to the embankment fill. The placement of these drains is more critical on larger fill areas, areas where subsurface water exists and in areas where the slopes are marginally stable.

The toe key and bench drains may consist of a perforated pipe which is surrounded by a free draining material which is wrapped by a geotextile filter fabric. The pipe should be surrounded by 4 to 6 cubic feet of free draining material per lineal foot of drain pipe.

10.1.2 Natural Soil Fill

Any natural soil used for any fill purpose should be free of all deleterious material, such as organic material and construction debris. Natural soil fill includes excavated and replaced material or in-place scarified material. Due to the expansive characteristics of the natural soil we do not recommend that it be used as fill material for direct support of structural components. The natural soils may be used to establish general site elevation. Our recommendations for placement of natural soil fill are provided below.

- The natural soils should be moisture conditioned, either by addition of water to dry soils, or by processing to allow drying of wet soils. The proposed fill materials should be moisture conditioned to between about optimum and about 2 percent above optimum soil moisture content. This moisture content can be estimated in the field by squeezing a sample of the soil in the palm of the hand. If the material easily makes a cast of soil which remains in-tact, and a minor amount of surface moisture develops on the cast, the material is close to the desired moisture content. Material testing during construction is the best means to assess the soil moisture content.
- Moisture conditioning of clay or silt soils may require many hours of processing. If possible, water should be added and thoroughly mixed into fine grained soil such as clay or silt the day prior to use of the material. This technique will allow for development of a more uniform moisture content and will allow for better compaction of the moisture conditioned materials.
- The moisture conditioned soil should be placed in lifts that do not exceed the capabilities of the compaction equipment used and compacted to at least 90 percent of maximum dry density as defined by ASTM D1557, modified Proctor test.
- We typically recommend a maximum fill lift thickness of 6 inches for hand operated equipment and 8 to 10 inches for larger equipment.
- Care should be exercised in placement of utility trench backfill so that the compaction operations do not damage underlying utilities.
- The maximum recommended lift thickness is about 6 to 8 inches. The maximum recommended rock size for natural soil fill is about 3 inches. This may require on-site screening or crushing if larger rocks are present. We must be contacted if it is desired to utilize rock greater than 3 inches for fill materials.

10.1.3 Granular Compacted Structural Fill

Granular compacted structural fill is referenced in numerous locations throughout the text of this report. Granular compacted structural fill should be constructed using an imported commercially produced rock product such as aggregate road base. Many products other than road base, such as clean aggregate or select crusher fines may be suitable, depending on the intended use. If a specification is needed by the design professional for development of project specifications, a material conforming to the Colorado Department of Transportation (CDOT) "Class 6" aggregate road base material can be specified. This specification can include an option for testing and approval in the event the contractor's desired material does not conform to the Class 6 aggregate specifications. We have provided the CDOT Specifications for Class 6 material below.

Grading of CDOT Class 6 Aggregate Base-Course Material	
Sieve Size	Percent Passing Each Sieve
1 inch	100
¾ inch	95-100
#4	30-65
#8	25-55
#200	3-12

Liquid Limit less than 30

All compacted structural fill should be moisture conditioned and compacted to at least 90 percent of maximum dry density as defined by ASTM D1557, modified Proctor test. Areas where the structural fill will support traffic loads under concrete slabs or asphalt concrete should be compacted to at least 95 percent of maximum dry density as defined by ASTM D1557, modified Proctor test.

Although clean-screened or washed aggregate may be suitable for use as structural fill on sites with sand or non-expansive silt soils, or on sites where shallow subsurface water is present, clean aggregate materials must not be used on any site where expansive soils exist due to the potential for water to accumulate in the voids of the clean aggregate materials.

Clean aggregate fill, if appropriate for the site soil conditions, must not be placed in lifts exceeding 8 inches and each lift should be thoroughly vibrated, preferably with a plate-type vibratory compactor prior to placing overlying lifts of material or structural components. We should be contacted prior to the use of clean aggregate fill materials to evaluate their suitability for use on this project.

10.1.4 Deep Fill Considerations

Deep fills, in excess of approximately 3 feet, should be avoided where possible. Fill soils will settle over time, even when placed properly per the recommendations contained in this report. Natural soil fill or engineered structural fills placed to our minimum recommended requirements will tend to settle an estimated 1 to 3 percent; therefore, a 3 foot thick fill may settle up to approximately 1 inch over time. A 10 foot thick fill may settle up to approximately 3½ inches even when properly placed. Fill settlement will result in distress and damage to the structures they are intended to support. There are methods to reduce the effects of deep fill settlement such as surcharge loading and surveyed monitoring programs; however, there is a significant time period of monitoring required for this to be successful. A more reliable method is to support structural components with deep foundation systems bearing below the fill envelope. We can provide additional guidance regarding deep fills up on request.

10.2 Excavation Considerations

Unless a specific classification is performed, the site soils should be considered as an Occupational Safety and Health Administration (OSHA) Type C soil and should be sloped and/or benched according to the current OSHA regulations. Excavations should be sloped and benched to prevent wall collapse. Any soil can release suddenly and cave unexpectedly from excavation walls, particularly if the soils is very moist, or if fractures within the soil are present. Daily observations of the excavations should be conducted by OSHA competent site personnel to assess safety considerations.

We did not encounter free subsurface water in our test borings. If water is encountered during construction, it may be necessary to dewater excavations to provide for suitable working conditions.

Scattered boulders were encountered in our test borings and large boulders are known to be present throughout the vicinity. Due to the size of the boulders encountered in the vicinity, if encountered, they may be difficult to remove using conventional excavation techniques and equipment. Removal of large boulders can also create a void of loose soil beneath structural components, which may require additional removal of loose soil and replacement with structural fill. In some instances, it may be preferable to leave boulders in place. Reduction in the thickness of the recommended structural fill beneath footings and slabs may also be prudent to limit disturbance to the bearing soils. If large boulders are encountered in the building footprint, a representative of the geotechnical engineer can provide field observations and provide additional recommendations for subgrade preparation.

If possible, excavations should be constructed to allow for water flow from the excavation the event of precipitation during construction. If this is not possible it may be necessary to remove water from snowmelt or precipitation from the foundation excavations to help reduce the influence of this water on the soil support conditions and the site construction characteristics.

10.2.1 Excavation Cut Slopes

We anticipate that some permanent excavation cut slopes may be included in the site development. Temporary cut slopes should not exceed 5 feet in height and should not be steeper than about 1:1 (horizontal to vertical) for most soils. Permanent cut slopes greater than 5 feet or steeper than 2½:1 must be analyzed on a site-specific basis.

Excavation cut slopes must be analyzed on a case/situation specific basis and restrained as necessary. The project shoring design engineer should be contacted for the design of the project shoring needs.

10.3 Utility Considerations

Subsurface utility trenches will be constructed as part of the site development. Utility line backfill often becomes a conduit for post construction water migration. If utility line trenches approach the proposed project site from above, water migrating along the utility line and/or backfill may have direct access to the portions of the proposed structure where the utility line penetrations are made through the foundation system. The foundation soils in the vicinity of the utility line penetration may be influenced by the additional subsurface water. There are a few options to help mitigate water migration along utility line backfill. Backfill bulkheads constructed with high clay content soils and/or placement of subsurface drains to promote utility line water discharge away from the foundation support soil.

Some movement of all structural components is normal and expected. The amount of movement may be greater on sites with problematic soil conditions. Utility line penetrations through any walls or floor slabs should be sleeved so that movement of the walls or slabs does not induce movement or stress in the utility line. Utility connections should be flexible to allow for some movement of the floor slab.

If utility line trenches are excavated using blasting techniques it is relatively common for surface and subsurface water to migrate along the fractures in the rock that may be created by blasting. If this water gains access to a utility line trench that has a gradient down toward the structure the water may gain access to the foundation support materials and/or subsurface portions of the proposed structure. Provisions should be made in the project construction plans to create an impervious barrier to prevent water from migrating into undesirable locations.

10.4 Exterior Grading and Drainage Comments

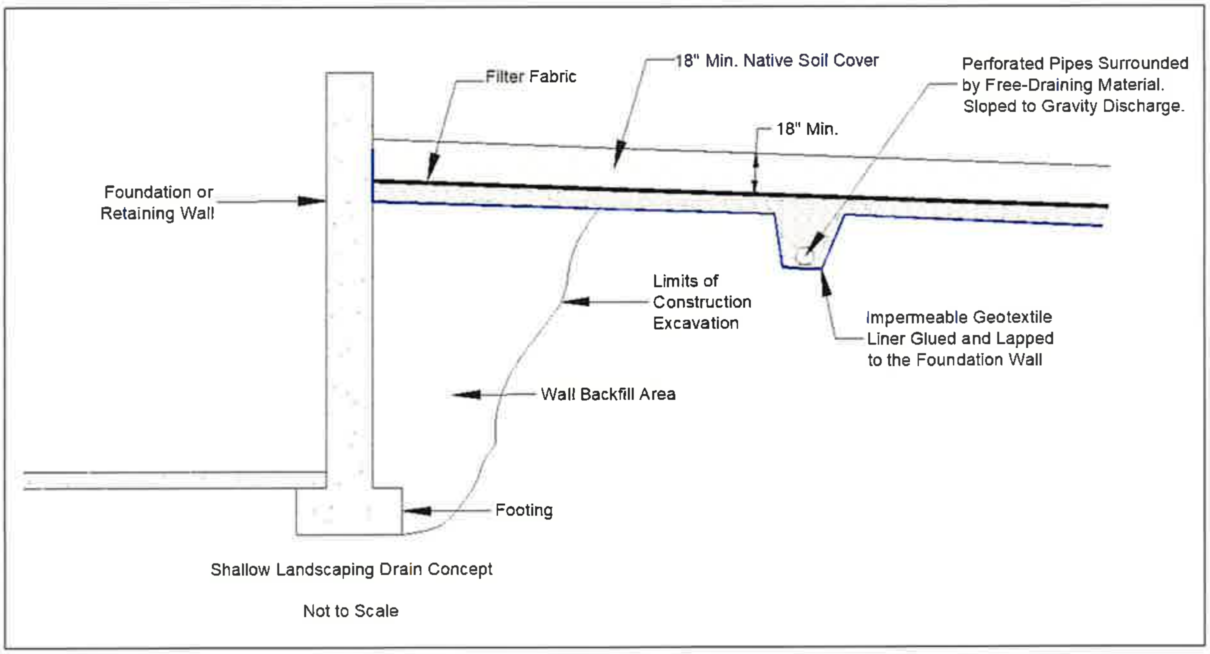
The following recommendations should be following during construction and maintained for the life of the structure with regards to exterior grading and surface drainage.

- The ground surface adjacent to the structure should be sloped to promote water flow away from the foundation system and flatwork.
- Snow storage areas should not be located in areas which will allow for snowmelt water access to support soils for the foundation system or flatwork.
- The project civil engineer, architect or builder should develop a drainage scheme for the site. We typically recommend the ground surface surrounding the exterior of the building be sloped to drain away from the foundation in all directions. We recommend a minimum slope of 12 inches in the first 10 feet in unpaved areas and a minimum slope of 3 inches in the first 10 feet in paved areas.
- Water flow from the roof of the structure should be captured and directed away from the structure. If the roof water is collected in an eave gutter system, or similar, the discharge points of the system must be located away from areas where the water will have access to the foundation backfill or any structure support soils. If downspouts are used, provisions should be made to either collect or direct the water away from the structure.
- Care should be taken to not direct water onto adjacent property or to areas that would negatively influence existing structures or improvements.

10.5 Landscaping Considerations

We recommend against construction of landscaping which requires excessive irrigation. Generally landscaping which uses abundant water requires that the landscaping contractor install topsoil which will retain moisture. The topsoil is often placed in flattened areas near the structure to further trap water and reduce water migration from away from the landscaped areas. Unfortunately, almost all aspects of landscape construction and development of lush vegetation are contrary to the establishment of a relatively dry area adjacent to the foundation walls. Excess water from landscaped areas near the structure can migrate to the foundation system or flatwork support soils, which can result in volume changes in these soils.

A relatively common concept used to collect and subsequently reduce the amount of excess irrigation water is to glue or attach an impermeable geotextile fabric or heavy mill plastic to the foundation wall and extend it below the topsoil which is used to establish the landscape vegetation. A thin layer of sand can be placed on top of the geotextile material to both protect the geotextile from punctures and to serve as a medium to promote water migration to the collection trench and perforated pipe. The landscape architect or contractor should be contacted for additional information regarding specific construction considerations for this concept which is shown in the sketch below.



A free draining aggregate or sand may be placed in the collection trench around the perforated pipe. The perforated pipe should be graded to allow for positive flow of excess irrigation water away from the structure or other area where additional subsurface water is undesired. Preferably the geotextile material should extend at least 10 or more feet from the foundation system.

Care should be taken to not place exterior flatwork such as sidewalks or driveways on soils that have been tilled and prepared for landscaping. Tilled soils will settle which can cause damage to the overlying flatwork. Tilled soils placed on sloped areas often "creep" down-slope. Any structure or structural component placed on this material will move down-slope with the tilled soil and may become damaged.

10.6 Soil Sulfate and Corrosion Issues

The requested scope of our services did not include assessment of the chemical constituents of corrosion potential of the site soils. Most soils in southwest Colorado are not typically corrosive to concrete. There has not been a history of damage to concrete due to sulfate corrosion in the area.

We are available to perform soluble sulfate content tests to assess the corrosion potential of the soils on concrete if desired.

10.7 Radon Issues

The requested scope of service of this report did not include assessment of the site soils for radon production. Many soils and formational materials in western Colorado produce Radon gas. The structure should be appropriately ventilated to reduce the accumulation of Radon gas in the structure. Several Federal Government agencies including the Environmental Protection Agency (EPA) have information and guidelines available for Radon considerations and home construction. If a radon survey of the site soils is desired, please contact us.

10.8 Mold and Other Biological Contaminants

Our services do not include determining the presence, prevention or possibility of mold or other biological contaminants developing in the future. If the client is concerned about mold or other biological contaminants, a professional in this special field of practice should be consulted.

11.0 CONSTRUCTION MONITORING AND TESTING

Engineering observation of subgrade bearing conditions, compaction testing of fill material and testing of foundation concrete are equally important tasks that should be performed by the geotechnical engineering consultant during construction. We should be contacted during the construction phase of the project and/or if any questions or comments arise as a result of the information presented below. It is common for unforeseen, or otherwise variable subsurface soil and water conditions to be encountered during construction. As discussed in our proposal for our services, it is imperative that we be contacted during the foundation excavation stage of the project to verify that the conditions encountered in our field exploration were representative of those encountered during construction. Our general recommendations for construction monitoring and testing are provided below.

- Consultation with design professionals during the design phases: This is important to ensure that the intentions of our recommendations are properly incorporated in the design, and that any changes in the design concept properly consider geotechnical aspects.
- Grading Plan Review: A grading plan was not available for our review at the time of this report. A grading plan with finished floor elevations for the proposed construction should be prepared by a civil engineer licensed in the State of Colorado. Trautner Geotech should be provided with grading plans once they are complete to determine if our recommendations based on the assumed bearing elevations are appropriate.
- Observation and monitoring during construction: A representative of the Geotechnical engineer from our firm should observe the foundation excavation, earthwork, and foundation phases of the work to determine that subsurface conditions are compatible with those used in the analysis and design and our recommendations have been properly implemented. Placement of backfill should be observed and tested to judge whether the proper placement conditions have been achieved. Compaction tests should be performed on each lift of material placed in areas proposed for support of structural components.

- We recommend a representative of the geotechnical engineer observe the drain and dampproofing phases of the work to judge whether our recommendations have been properly implemented.
- If asphaltic concrete is placed for driveways or aprons near the structure we are available to provide testing of these materials during placement.

12.0 CONCLUSIONS

While we feel that it is feasible to develop this site as planned using relatively conventional techniques we feel that it is prudent for us to be part of the continuing design of this project to review and provide consultation in regard to the proposed development scheme as the project progresses to aid in the proper interpretation and implementation of the recommendations presented in this report. This consultation should be incorporated in the project development prior to construction at the site.

13.0 LIMITATIONS

This study has been conducted based on the geotechnical engineering standards of care in this area at the time this report was prepared. We make no warranty as to the recommendations contained in this report, either expressed or implied. The information presented in this report is based on our understanding of the proposed construction that was provided to us and on the data obtained from our field and laboratory studies. Our recommendations are based on limited field and laboratory sampling and testing. Unexpected subsurface conditions encountered during construction may alter our recommendations. We should be contacted during construction to observe the exposed subsurface soil conditions to provide comments and verification of our recommendations.

The recommendations presented above are intended to be used only for this project site and the proposed construction which was provided to us. The recommendations presented above are not suitable for adjacent project sites, or for proposed construction that is different than that outlined for this study.

This report provides geotechnical engineering design parameters, but does not provide foundation design or design of structure components. The project architect, designer or structural engineer must be contacted to provide a design based on the information presented in this report.

This report does not provide an environmental assessment nor does it provide environmental recommendations such as those relating to Radon or mold considerations. If recommendation relative to these or other environmental topics are needed and environmental specialist should be contacted.

The findings of this report are valid as of the present date. However, changes in the conditions of the property can occur with the passage of time. The changes may be due to natural processes or to the works of man, on the project site or adjacent properties. In addition, changes in applicable or appropriate standards can occur, whether they result from legislation or the broadening of knowledge. Therefore, the recommendations presented in this report should not be relied upon after a period of two years from the issue date without our review.

We are available to review and tailor our recommendations as the project progresses and additional information which may influence our recommendations becomes available.

Please contact us if you have any questions, or if we may be of additional service.

Respectfully,
TRAUTNER GEOTECH

THOMAS RICHARD HARRISON
PE0047810
01-27-25

Tom R. Harrison, P.E.
Geotechnical Engineer

APPENDIX A

Field Study Results

TRAUTNER GEOTECH LLC

Field Engineer : Tom Harrison
Hole Diameter : 4" Solid
Drilling Method : Continuous Flight Auger
Sampling Method : Mod. California Sampler
Date Drilled : 12/11/2024
Total Depth (approx.) : 7 feet
Location : See Figure in Report

LOG OF BORING TB-1

Cascade Village Townhomes- South
Durango, Colorado
Lauren Davis, AIA,AICP.
ldavis@ra-ae.com

58656 GE

Sample Type
- ■ Mod. California Sampler
- ▦ Standard Split Spoon
- ▨ Bag Sample

Water Level
- ▼ Water Level During Drilling
- ▽ Water Level After Drilling

Depth in feet	DESCRIPTION	USCS	GRAPHIC	Samples	Blow Count	Water Level	REMARKS
0	SANDY,SILTY LEAN CLAY WITH ORGANICS; soft to medium stiff, moist, dark brown.	CL	▨	▨			
1 2							
3 4 5 6	CLAYEY, SANDY GRAVEL WITH COBBLES; medium dense, moist, brown.	GC	▨	▨ ■ ▨	9/6 13/6		
	SANDSTONE BOULDER OR FORMATION; very hard, dry, brown to gray.	SS	▤	▨			
7	Practical auger drilling refusal on sandstone boulder or formation at 7 feet.						

TRAUTNER GEOTECH LLC

Field Engineer : Tom Harrison
Hole Diameter : 4" Solid
Drilling Method : Continuous Flight Auger
Sampling Method : Mod. California Sampler
Date Drilled : 12/11/2024
Total Depth (approx.) : 13 feet
Location : See Figure in Report

LOG OF BORING TB-2

Cascade Village Townhomes-South
Durango, Colorado
Lauren Davis, AIA,AICP.
ldavis@ra-ae.com

58656 GE

Sample Type
- ■ Mod. California Sampler
- ▦ Standard Split Spoon
- ▨ Bag Sample

Water Level
- ▼ Water Level During Drilling
- ▽ Water Level After Drilling

Depth in feet	DESCRIPTION	USCS	GRAPHIC	Samples	Blow Count	Water Level	REMARKS
0	SANDY LEAN CLAY WITH ORGANICS; medium stiff, moist, dark brown.	CL					
2				■	6/6 10/6		
3	CLAYEY, SANDY GRAVEL WITH COBBLES; medium dense to dense to very dense, slightly moist to moist, brown.	GC					
4				▦	9/6 11/6 12/6		
9				▦	25/6 39/6 40/6		Very dense at 10 feet.
12	FRACTURED SANDSTONE FORMATION; very hard, dry, brown.	SS					
13	Practical auger drilling refusal on sandstone formation at 13 feet.						

TRAUTNER GEOTECH LLC

Field Engineer : Tom Harrison
Hole Diameter : 4" Solid
Drilling Method : Continuous Flight Auger
Sampling Method : Mod. California Sampler
Date Drilled : 12/11/2024
Total Depth (approx.) : 10 feet
Location : See Figure in Report

LOG OF BORING TB-3

Cascade Village Townhomes-South
Durango, Colorado
Lauren Davis, AIA,AICP.
ldavis@ra-ae.com

58656 GE

Sample Type
■ Mod. California Sampler
▦ Standard Split Spoon
▨ Bag Sample

Water Level
▼ Water Level During Drilling
▽ Water Level After Drilling

Depth in feet	DESCRIPTION	USCS	GRAPHIC	Samples	Blow Count	Water Level	REMARKS
0 1 2 3 4 5	SANDY LEAN CLAY WITH GRAVEL; few organics, stiff to very stiff, slightly moist, dark brown.	CL	▨	▨ ■ ▨	12/6 16/6		
6 7 8	CLAYEY, SANDY GRAVEL WITH COBBLES; dense, slightly moist, brown.	GC	░	▨			
9	FRACTURED SANDSTONE FORMATION; very hard, dry, tan to grey.	SS	▤				
10	Practical auger drilling refusal on sandstone formation at 10 feet.						

TRAUTNER GEOTECH LLC

Field Engineer : Jacob Vaughn
Hole Diameter : 4" Solid
Drilling Method : Continuous Flight Auger
Sampling Method : Mod. California Sampler
Date Drilled : 12/11/2024
Total Depth (approx.) : 21 feet
Location : See Figure in Report

LOG OF BORING TB-4

Cascade Village Townhomes-South
Durango, Colorado
Lauren Davis, AIA,AICP.
ldavis@ra-ae.com

58656 GE

Sample Type
- ■ Mod. California Sampler
- ▦ Standard Split Spoon
- ▨ Bag Sample

Water Level
- ▼ Water Level During Drilling
- ▽ Water Level After Drilling

Depth in feet	DESCRIPTION	USCS	GRAPHIC	Samples	Blow Count	Water Level	REMARKS
0	SANDY SILTY LEAN CLAY WITH ORGANICS; few gravels, soft to medium stiff, moist, dark brown.			▨			
1							
2		CL					
3							
4	CLAYEY, SANDY GRAVEL WITH COBBLES; medium dense to dense, slightly moist, brown.			■	4/6 9/6		
5				▨			
6							intermittent CL seams from 6 feet to 11 feet.
7							
8							
9		GC		▦	6/6 14/6		
10					12/6		
11							dense at 11.5 feet
12							
13							
14							
15	FRACTURED SANDSTONE FORMATION; hard, dry, brown to grey.						
16							
17							
18		SS					
19							
20	SANDSTONE FORMATION; very hard, dry, brown to grey.	SS					
21	Practical auger drilling refusal on sandstone formation at 21 feet.						

LOG OF BORING TB-5

Field Engineer : Tom Harrison
Hole Diameter : 4" Solid
Drilling Method : Continuous Flight Auger
Sampling Method : Mod. California Sampler
Date Drilled : 12/11/2024
Total Depth (approx.) : 16.5 feet
Location : See Figure in Report

Cascade Village Townhomes-South
Durango, Colorado
Lauren Davis, AIA,AICP.
ldavis@ra-ae.com

58656 GE

Sample Type
- ■ Mod. California Sampler
- ▦ Standard Split Spoon
- ▨ Bag Sample

Water Level
- ▼ Water Level During Drilling
- ▽ Water Level After Drilling

Depth in feet	DESCRIPTION	USCS	GRAPHIC	Samples	Blow Count	Water Level	REMARKS
0	SANDY LEAN CLAY WITH ORGANICS; medium stiff, slightly moist, dark brown.	CL		▨			
2	CLAYEY, SANDY GRAVEL WITH COBBLES; medium dense to dense, moist, brown.	GC		▨			

Practical auger drilling refusal on sandstone formation or boulder at 16.5 feet.

TRAUTNER GEOTECH LLC

Field Engineer : Jacob Vaughn
Hole Diameter : 4" Solid
Drilling Method : Continuous Flight Auger
Sampling Method : Mod. California Sampler
Date Drilled : 12/11/2024
Total Depth (approx.) : 21 feet
Location : See Figure in Report

LOG OF BORING TB-6

Cascade Village Townhomes-South
Durango, Colorado
Lauren Davis, AIA,AICP.
ldavis@ra-ae.com

58656 GE

Sample Type
- ■ Mod. California Sampler
- ▦ Standard Split Spoon
- ▨ Bag Sample

Water Level
- ▼ Water Level During Drilling
- ▽ Water Level After Drilling

Depth in feet	DESCRIPTION	USCS	GRAPHIC	Samples	Blow Count	Water Level	REMARKS
0	SANDY SILTY LEAN CLAY WITH ORGANICS; medium stiff, moist, dark brown.			▨			
1				▨			
2		CL		▨			
3				▨			
4	CLAYEY, SANDY GRAVEL WITH COBBLES; dense to very dense, moist to slightly moist, brown.			▨			
5				▨			
6				▨			
7				▨			
8				▨			
9				■	15/6 24/6		
10							
11		GC					dense, slightly moist at 11 feet
12							
13							
14							
15							
16							
17							
18	FRACTURED SANDSTONE FORMATION; hard, dry, brown to grey.						
19		SS					
20							
21	Practical auger drilling refusal on sandstone formation at 20.5 feet.						

TRAUTNER GEOTECH LLC

Field Engineer : Jacob Vaughn
Hole Diameter : 4" Solid
Drilling Method : Continuous Flight Auger
Sampling Method : Mod. California Sampler
Date Drilled : 12/12/2024
Total Depth (approx.) : 11 feet
Location : See Figure in Report

LOG OF BORING TB-7

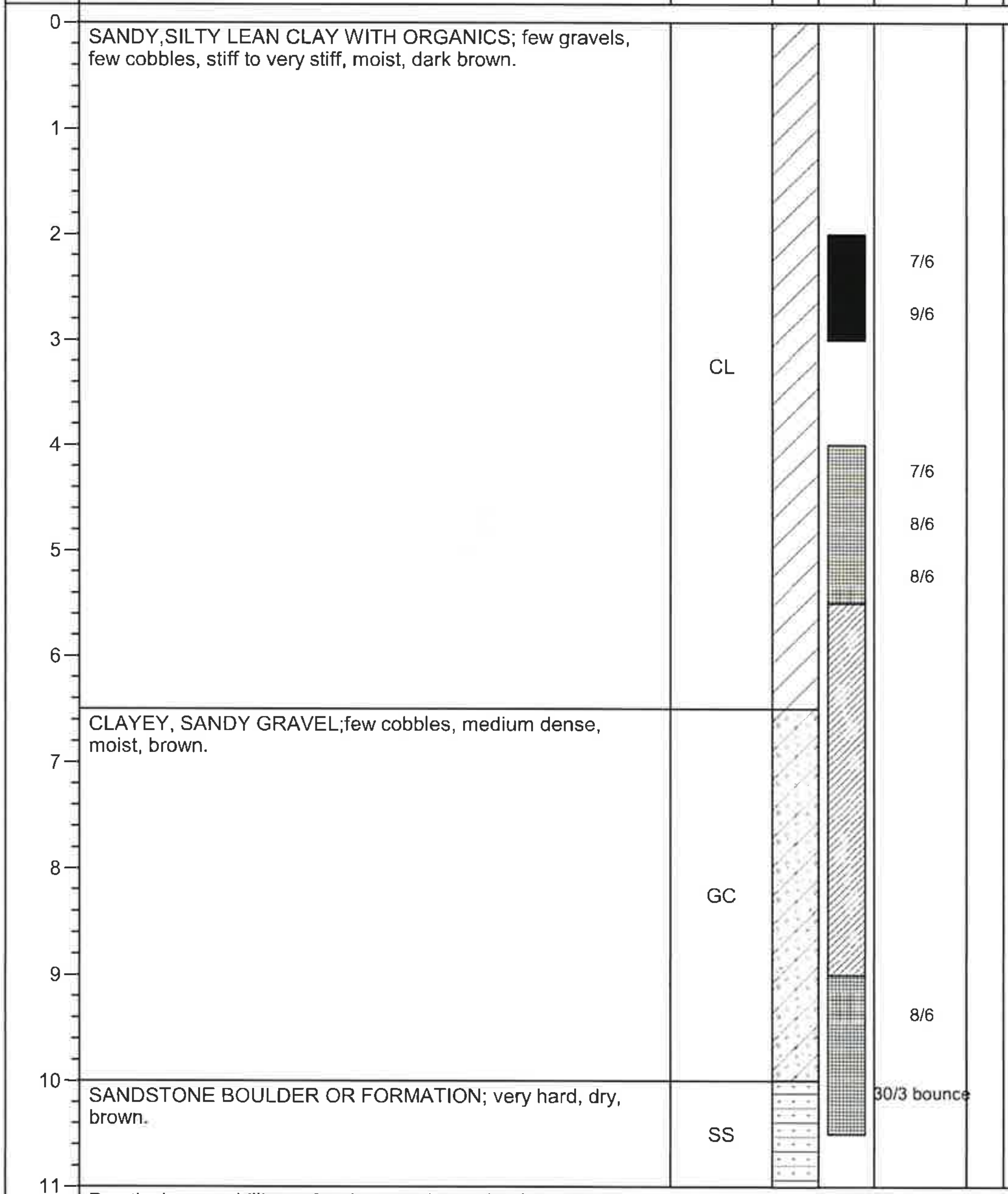
Cascade Village Townhomes-South
Durango, Colorado
Lauren Davis, AIA,AICP.
ldavis@ra-ae.com

58656 GE

Sample Type
- ■ Mod. California Sampler
- ▦ Standard Split Spoon
- ▨ Bag Sample

Water Level
- ▼ Water Level During Drilling
- ▽ Water Level After Drilling

Depth in feet	DESCRIPTION	USCS	Blow Count	Water Level	REMARKS
0	SANDY,SILTY LEAN CLAY WITH ORGANICS; few gravels, few cobbles, stiff to very stiff, moist, dark brown.	CL	7/6 9/6 7/6 8/6 8/6		
	CLAYEY, SANDY GRAVEL;few cobbles, medium dense, moist, brown.	GC	8/6		
10	SANDSTONE BOULDER OR FORMATION; very hard, dry, brown.	SS	30/3 bounce		
11	Practical auger drilling refusal on sandstone boulder or formation at 11 feet.				



Field Engineer : Tom Harrison
Hole Diameter : 4" Solid
Drilling Method : Continuous Flight Auger
Sampling Method : Mod. California Sampler
Date Drilled : 12/12/2024
Total Depth (approx.) : 34 feet
Location : See Figure in Report

LOG OF BORING TB-8

Cascade Village Townhomes-South
Durango, Colorado
Lauren Davis, AIA,AICP.
ldavis@ra-ae.com

58656 GE

Sample Type
- ■ Mod. California Sampler
- ▦ Standard Split Spoon
- ▨ Bag Sample

Water Level
- ▼ Water Level During Drilling
- ▽ Water Level After Drilling

Depth in feet	DESCRIPTION	USCS	GRAPHIC	Samples	Blow Count	Water Level	REMARKS
0	SUSPECTED MAN PLACED FILL, SANDY LEAN CLAY WITH ORGANICS AND GRAVEL; soft, moist, dark brown.	CL					Rebar found in suspected man placed fill at .5 feet.
2	CLAYEY, SANDY GRAVEL WITH COBBLES;few boulders, medium dense to dense, moist, brown.	GC					
4				■	18/6 16/6		Boring was offset by 2 feet to the north after refusal on boulder at 4 feet in first boring.
9	SANDY LEAN CLAY; few gravels, soft to medium stiff, moist to wet, dark brown.	CL		▦	4/6 4/6 4/6		
18						▽	Subsurface water measure at 18 feet after drilling. Wet and soft at 18 feet.
24				▦	1/6 1/6 1/6		
33	SHALE AND SANDSTONE FORMATION; hard, dry, tan.	SH-SS					
34	Bottom of boring at 34 feet in shale/sanstone formation.						

TRAUTNER GEOTECH LLC

Field Engineer : Jacob Vaughn
Hole Diameter : 4" Solid
Drilling Method : Continuous Flight Auger
Sampling Method : Mod. California Sampler
Date Drilled : 12/12/2024
Total Depth (approx.) : 31 feet
Location : See Figure in Report

LOG OF BORING TB-9

Cascade Village Townhomes-South
Durango, Colorado
Lauren Davis, AIA,AICP.
ldavis@ra-ae.com

58656 GE

Sample Type
- ■ Mod. California Sampler
- ▦ Standard Split Spoon
- ▨ Bag Sample

Water Level
- ▼ Water Level During Drilling
- ▽ Water Level After Drilling

Depth in feet	DESCRIPTION	USCS	GRAPHIC	Samples	Blow Count	Water Level	REMARKS
0	SANDY LEAN CLAY WITH GRAVEL; few organics, medium stiff to stiff, moist to very moist, dark brown.	CL	▨	▨			
4				■	7/6 7/6		
4.5				▨			
8							Soft and very moist at 8 feet.
9				▦	2/6 2/6 2/6		
16						▽	Subsurface water measure at 16 feet after drilling.
20	CLAYEY, SANDY GRAVEL WITH COBBLES; loose to medium dense , wet, brown.	GC	⁘				
24				▦	3/6 5/6 7/6		
30	SANDSTONE FORMATION; hard, dry, tan.	SS	⁘				
31	Bottom of boring at 31 feet in sandstone formation.						

TRAUTNER GEOTECH LLC

Field Engineer : Tom Harrison
Hole Diameter : 4" Solid
Drilling Method : Continuous Flight Auger
Sampling Method : Mod. California Sampler
Date Drilled : 12/12/2024
Total Depth (approx.) : 28 feet
Location : See Figure in Report

LOG OF BORING TB-10

Cascade Village Townhomes-South
Durango, Colorado
Lauren Davis, AIA,AICP.
ldavis@ra-ae.com

58656 GE

Sample Type
- ■ Mod. California Sampler
- ▦ Standard Split Spoon
- ▨ Bag Sample

Water Level
- ▼ Water Level During Drilling
- ▽ Water Level After Drilling

Depth in feet	DESCRIPTION	USCS	GRAPHIC	Samples	Blow Count	Water Level	REMARKS
0	SANDY LEAN CLAY WITH ORGANICS; few gravels,very soft to stiff, moist to wet, dark brown to brown.	CL	▨	▨			
3							Stiff and brown at 3 feet.
4				■	5/6 7/6		
5				▨			
6							Very soft to soft and moist to very moist at 6 feet.
9				▦	W.O.H./6 2/12		W.O.H.= weight of hammer.
10						▽	Wet at 10 feet. Subsurface water measure at 10 feet after drilling.
20	CLAYEY, SANDY GRAVEL WITH COBBLES; medium dense , wet, brown.	GC					
27	SANDSTONE FORMATION; hard, dry, grey.	SS					
28	Bottom of boring at 28 feet in sandstone formation.						

TRAUTNER GEOTECH LLC

Field Engineer : Tom Harrison
Hole Diameter : 4" Solid
Drilling Method : Continuous Flight Auger
Sampling Method : Mod. California Sampler
Date Drilled : 12/12/2024
Total Depth (approx.) : 29 feet
Location : See Figure in Report

LOG OF BORING TB-11

Cascade Village Townhomes-South
Durango, Colorado
Lauren Davis, AIA,AICP.
ldavis@ra-ae.com

58656 GE

Sample Type

- ■ Mod. California Sampler
- ▦ Standard Split Spoon
- ▨ Bag Sample

Water Level

- ▼ Water Level During Drilling
- ▽ Water Level After Drilling

Depth in feet	DESCRIPTION	USCS	GRAPHIC	Samples	Blow Count	Water Level	REMARKS
0	ORGANIC LEAN CLAY; very soft, very moist to wet, dark brown to black.	OL					
1							Wetlands soil - Peat to 21.5 feet
2						▽	Subsurface water measure at 2 feet after drilling.
4				■	W.O.H./6 W.O.H./6		W.O.H.= weight of hammer
21.5	CLAYEY, SANDY GRAVEL WITH COBBLES; medium dense, wet, brown to red.	GC	▨				Possible Molas Formation
26	SHALE FORMATION; hard, wet, grey.	SH	┄				
29	Bottom of boring at 29 feet in shale formation.						

TRAUTNER GEOTECH LLC

Field Engineer : Tom Harrison
Hole Diameter : 4" Solid
Drilling Method : Continuous Flight Auger
Sampling Method : Mod. California Sampler
Date Drilled : 12/13/2024
Total Depth (approx.) : 23 feet
Location : See Figure in Report

LOG OF BORING TB-12

Cascade Village Townhomes-South
Durango, Colorado
Lauren Davis, AIA,AICP.
ldavis@ra-ae.com

58656 GE

Sample Type
- ■ Mod. California Sampler
- ▦ Standard Split Spoon
- ▨ Bag Sample

Water Level
- ▼ Water Level During Drilling
- ▽ Water Level After Drilling

Depth in feet	DESCRIPTION	USCS	GRAPHIC	Samples	Blow Count	Water Level	REMARKS
0	ORGANIC LEAN CLAY; very soft, very moist to wet, dark brown.	OL		▨			
2						▽	Wetlands Soil - Peat to 8 feet Subsurface water measure at 2 feet after drilling.
4				■	W.O.H./6 W.O.H./6		W.O.H.= weight of hammer
8	CLAYEY, SANDY GRAVEL WITH ORGANICS; few cobbles, loose, wet, brown.	GC					
13	WEATHERED LIMESTONE FORMATION; hard, wet, red to brown.	LS					
21	LIMESTONE FORMATION; very hard, dry, red.	LS					
23	Practical auger drilling refusal on limestone formation at 23 feet.						

TRAUTNER GEOTECH LLC

Field Engineer : Tom Harrison
Hole Diameter : 4" Solid
Drilling Method : Continuous Flight Auger
Sampling Method : Mod. California Sampler
Date Drilled : 12/13/2024
Total Depth (approx.) : 14 feet
Location : See Figure in Report

LOG OF BORING TB-13

Cascade Village Townhomes-South
Durango, Colorado
Lauren Davis, AIA,AICP.
ldavis@ra-ae.com

58656 GE

Sample Type
- ■ Mod. California Sampler
- ▦ Standard Split Spoon
- ▨ Bag Sample

Water Level
- ▼ Water Level During Drilling
- ▽ Water Level After Drilling

Depth in feet	DESCRIPTION	USCS	GRAPHIC	Samples	Blow Count	Water Level	REMARKS
0	LEAN CLAY WITH ORGANICS; very soft, very moist to wet, dark brown.	CL	▨				
2						▽	Subsurface water measure at 2 feet after drilling.
				■	W.O.H./6 W.O.H./6		W.O.H.= weight of hammer
	WEATHERED LIMESTONE FORMATION; hard, wet, red to brown.	LS	▤				
				▦	15/6 19/6 29/6		
9	LIMESTONE FORMATION; very hard, dry, red.	LS	▤				
14	Bottom of boring at 14 feet in limestone formation.						

TRAUTNER GEOTECH LLC

Field Engineer : Tom Harrison
Hole Diameter : 4" Solid
Drilling Method : Continuous Flight Auger
Sampling Method : Mod. California Sampler
Date Drilled : 12/13/2024
Total Depth (approx.) : 12 feet
Location : See Figure in Report

LOG OF BORING TB-14

Cascade Village Townhomes-South
Durango, Colorado
Lauren Davis, AIA,AICP.
ldavis@ra-ae.com

58656 GE

Sample Type
- ■ Mod. California Sampler
- ▦ Standard Split Spoon
- ▨ Bag Sample

Water Level
- ▼ Water Level During Drilling
- ▽ Water Level After Drilling

Depth in feet	DESCRIPTION	USCS	GRAPHIC	Samples	Blow Count	Water Level	REMARKS
0	SANDY LEAN CLAY; medium stiff to stiff, moist, brown.	CL	▨	▨			
3.5				■	14/6		
4	SANDY GRAVEL AND COBBLE; slightly clayey, dense, moist to very moist, red to brown.	GC	▨	■	26/6		
5							Possible weathered Molas formation.
12	Bottom of boring at 12 feet in limestone formation.						

TRAUTNER GEOTECH LLC

Field Engineer : Tom Harrison
Hole Diameter : 4" Solid
Drilling Method : Continuous Flight Auger
Sampling Method : Mod. California Sampler
Date Drilled : 12/13/2024
Total Depth (approx.) : 14 feet
Location : See Figure in Report

LOG OF BORING TB-15

Cascade Village Townhomes-South
Durango, Colorado
Lauren Davis, AIA,AICP.
ldavis@ra-ae.com

58656 GE

Sample Type
- ■ Mod. California Sampler
- ▦ Standard Split Spoon
- ▨ Bag Sample

Water Level
- ▼ Water Level During Drilling
- ▽ Water Level After Drilling

Depth in feet	DESCRIPTION	USCS	GRAPHIC	Samples	Blow Count	Water Level	REMARKS
0	LEAN CLAY WITH ORGANICS; soft, moistto very moist, dark brown.	CL	▨	▨			
				■	8/6 11/6		
4	CLAYEY, SANDY GRAVEL AND COBBLE; medium dense to dense, very moist, brown.	GC					
12	LIMESTONE FORMATION; hard, dry, red to brown.	LS					
14	Bottom of boring in limestone formation at 14 feet.						

TRAUTNER GEOTECH LLC

Field Engineer : Tom Harrison
Hole Diameter : 4" Solid
Drilling Method : Continuous Flight Auger
Sampling Method : Mod. California Sampler
Date Drilled : 12/13/2024
Total Depth (approx.) : 11 feet
Location : See Figure in Report

LOG OF BORING TB-16

Cascade Village Townhomes-South
Durango, Colorado
Lauren Davis, AIA,AICP.
ldavis@ra-ae.com

58656 GE

Sample Type
- ■ Mod. California Sampler
- ▦ Standard Split Spoon
- ▨ Bag Sample

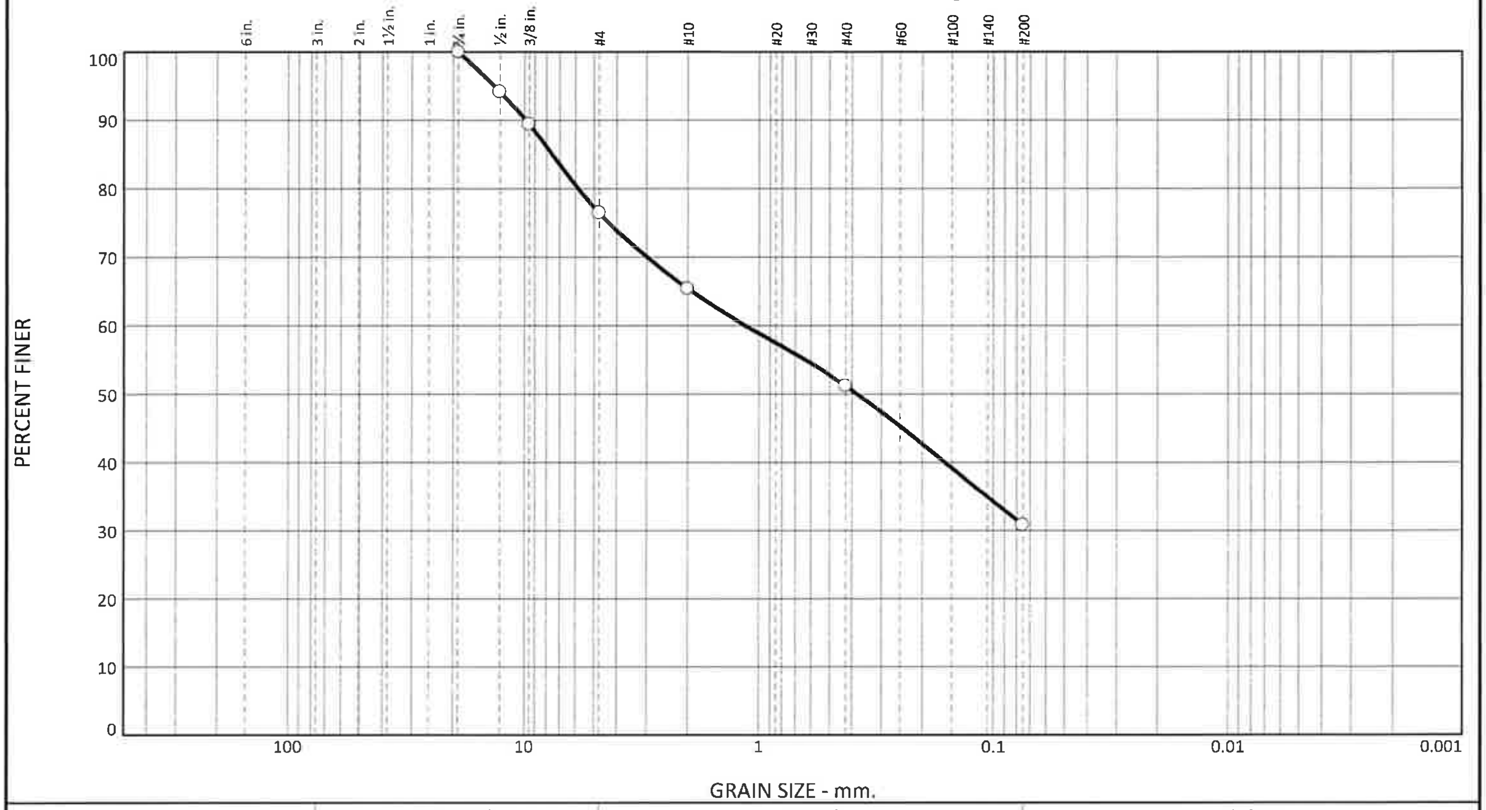
Water Level
- ▼ Water Level During Drilling
- ▽ Water Level After Drilling

Depth in feet	DESCRIPTION	USCS	GRAPHIC	Samples	Blow Count	Water Level	REMARKS
0	LEAN CLAY WITH ORGANICS; slightly sandy, soft, very moist to wet, dark brown.	CL	▨	▨ ■	3/6 2/6	▽	Subsurface water measure at 4 feet after drilling.
8	CLAYEY, SANDY GRAVEL AND COBBLE; medium dense, wet, brown.	GC	░	▦	10/6 11/6 11/6		
10	LIMESTONE FORMATION; hard, dry, red to brown.	LS	▦				
11	Bottom of boring in limestone formation at 11 feet.						

APPENDIX B

Laboratory Test Results

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	23.6	11.0	14.2	20.4	30.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100.0		
1/2"	94.1		
3/8"	89.4		
#4	76.4		
#10	65.4		
#40	51.2		
#200	30.8		

* (no specification provided)

Soil Description

SC - clayey sand with gravel

Atterberg Limits

PL= 13 LL= 23 PI= 10

Coefficients

D_{90}= 9.8722 D_{85}= 7.5522 D_{60}= 1.1420
D_{50}= 0.3805 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SC AASHTO= A-2-4(0)

Remarks

Source of Sample: Test Boring 2 **Depth:** 5' - 9'
Sample Number: 13335-F

Date: 12/13/2024

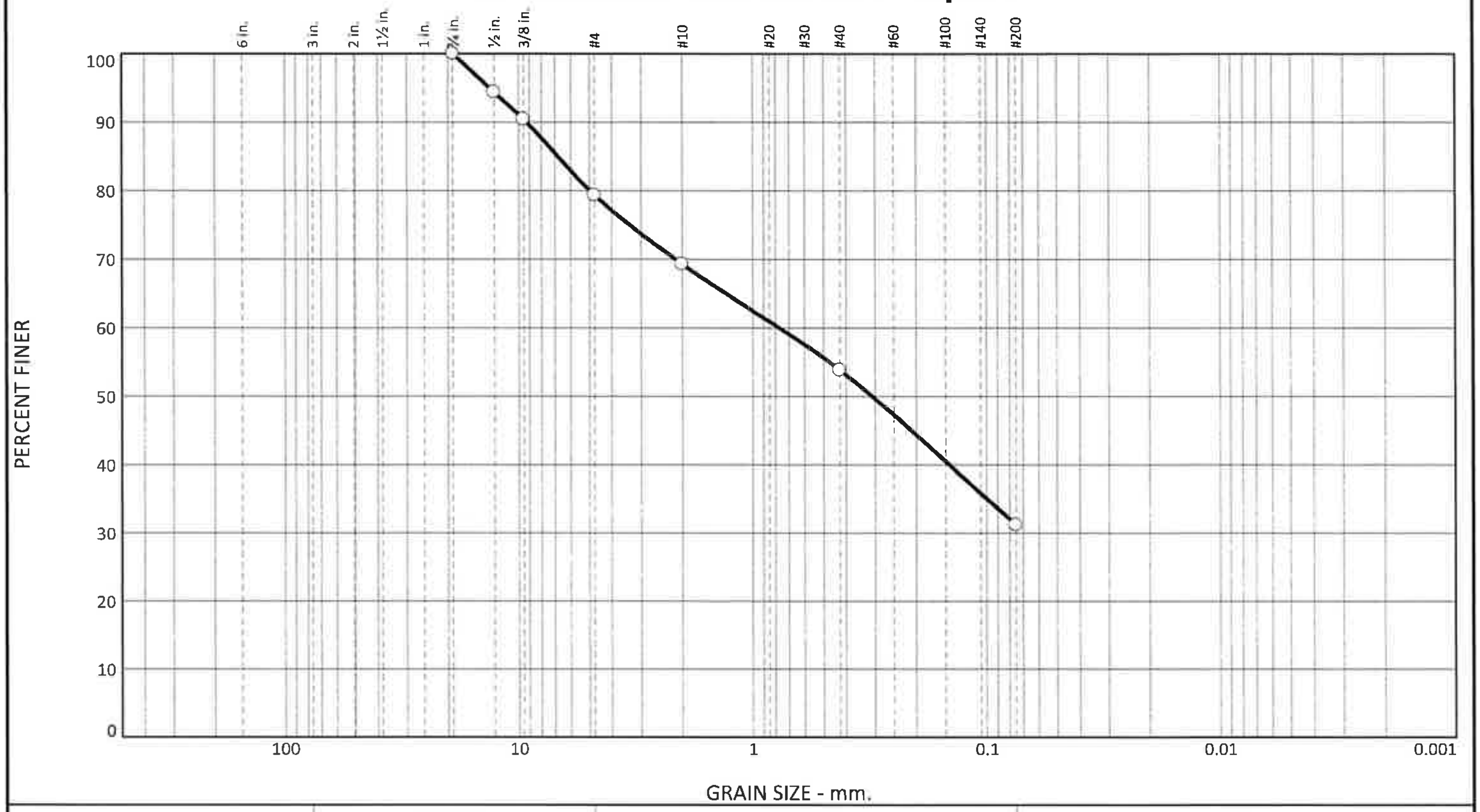
TRAUTNER GEOTECH LLC

Client: REYNOLDS ASH + ASSOCIATES
Project: Cascade Village Townhomes South

Project No: 58656GE **Figure** B.1

Tested By: N. Granda **Checked By:** J. Vaughn

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	20.7	10.0	15.5	22.6	31.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100.0		
1/2"	94.3		
3/8"	90.4		
#4	79.3		
#10	69.3		
#40	53.8		
#200	31.2		

* (no specification provided)

Soil Description

SC - clayey sand with gravel

Atterberg Limits

PL= 14 LL= 22 PI= 8

Coefficients

D_{90}= 9.2634 D_{85}= 6.8118 D_{60}= 0.7699
D_{50}= 0.3090 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SC AASHTO= A-2-4(0)

Remarks

Source of Sample: Test Boring 5 **Depth:** 3.5' - 8.5'
Sample Number: 13335-Q

Date: 12/13/2024

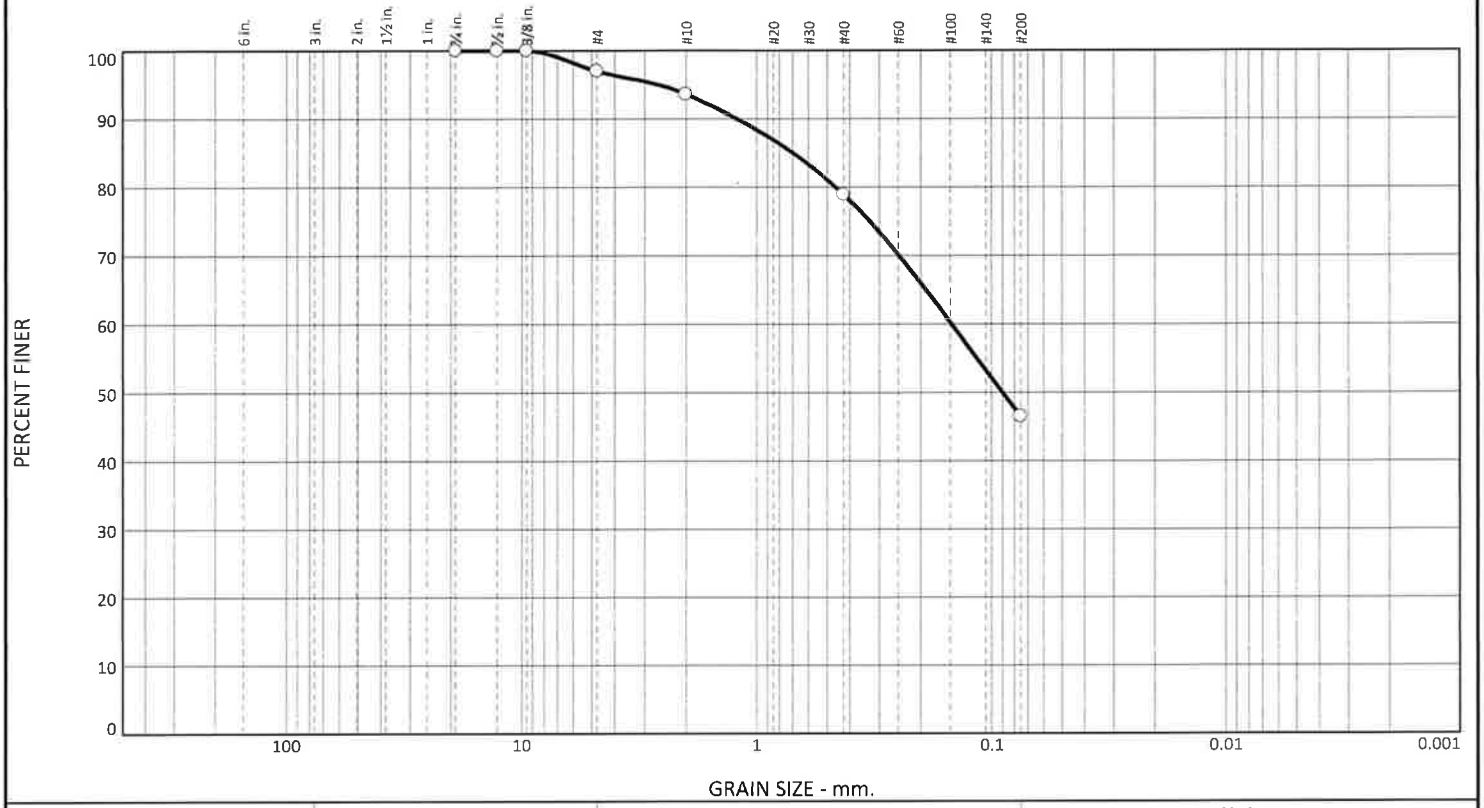
TRAUTNER GEOTECH LLC

Client: REYNOLDS ASH + ASSOCIATES
Project: Cascade Village Townhomes South

Project No: 58656GE **Figure** B.2

Tested By: N. Granda **Checked By:** J. Vaughn

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	3.0	3.4	14.7	32.4	46.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100.0		
1/2"	100.0		
3/8"	100.0		
#4	97.0		
#10	93.6		
#40	78.9		
#200	46.5		

* (no specification provided)

Soil Description

SC - clayey sand

Atterberg Limits

PL= 19 LL= 32 PI= 13

Coefficients

D_{90}= 1.2149 D_{85}= 0.7007 D_{60}= 0.1485
D_{50}= 0.0901 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SC AASHTO= A-6(3)

Remarks

Source of Sample: Test Boring 9 **Depth:** 5' - 9'
Sample Number: 13335-GA

Date: 12/13/2024

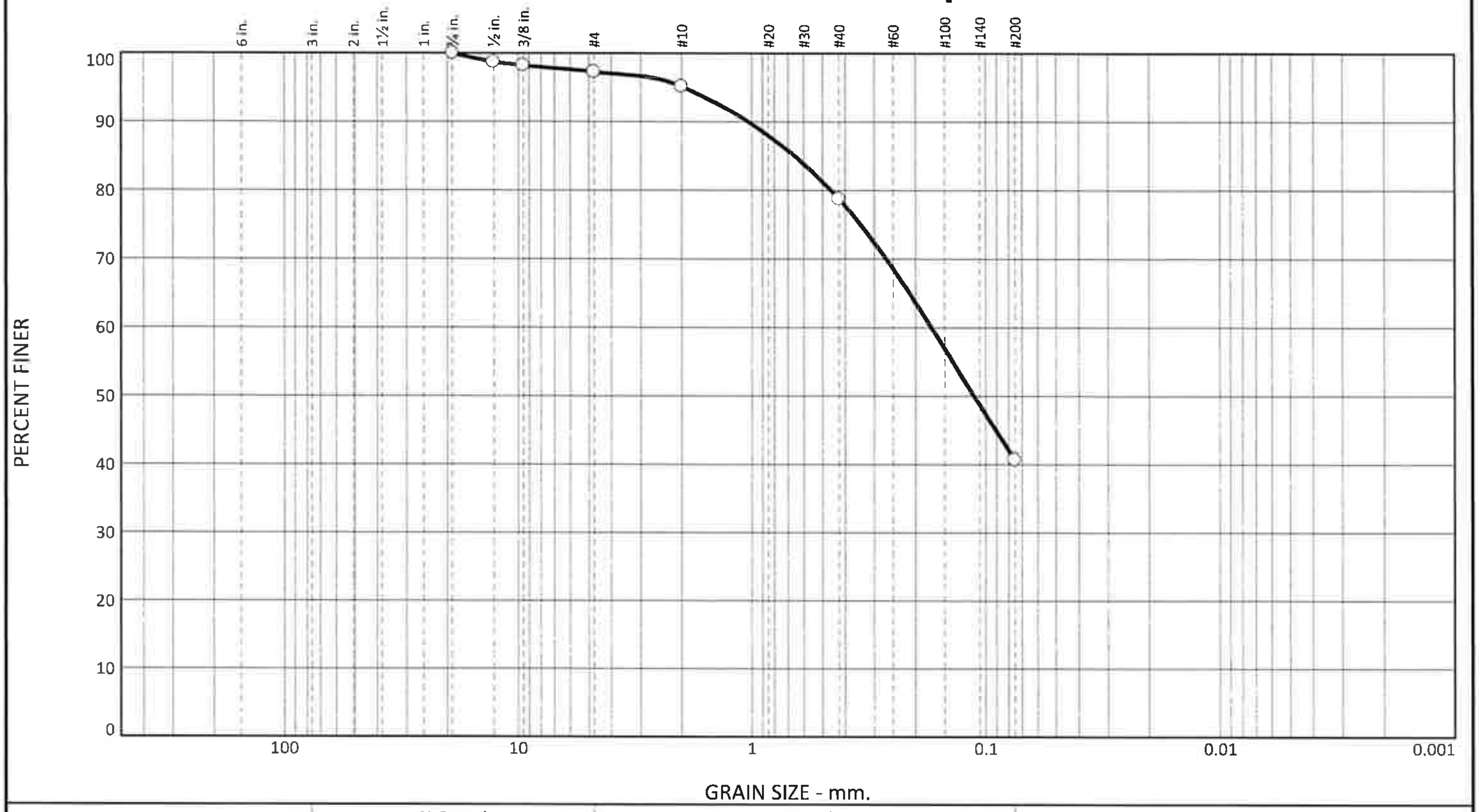
TRAUTNER GEOTECH LLC

Client: REYNOLDS ASH + ASSOCIATES
Project: Cascade Village Townhomes South

Project No: 58656GE **Figure** B.3

Tested By: N. Granda/N. Ellis **Checked By:** J. Vaughn

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	2.7	2.2	16.4	38.0	40.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100.0		
1/2"	98.7		
3/8"	98.2		
#4	97.3		
#10	95.1		
#40	78.7		
#200	40.7		

* (no specification provided)

Soil Description

SC - clayey sand

Atterberg Limits

PL= 22 LL= 38 PI= 16

Coefficients

D_{90}= 1.0389 D_{85}= 0.6577 D_{60}= 0.1714
D_{50}= 0.1125 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SC AASHTO= A-6(3)

Remarks

Source of Sample: Test Boring 10 **Depth:** 4.5' - 8.5'
Sample Number: 13335-JA

Date: 12/13/2024

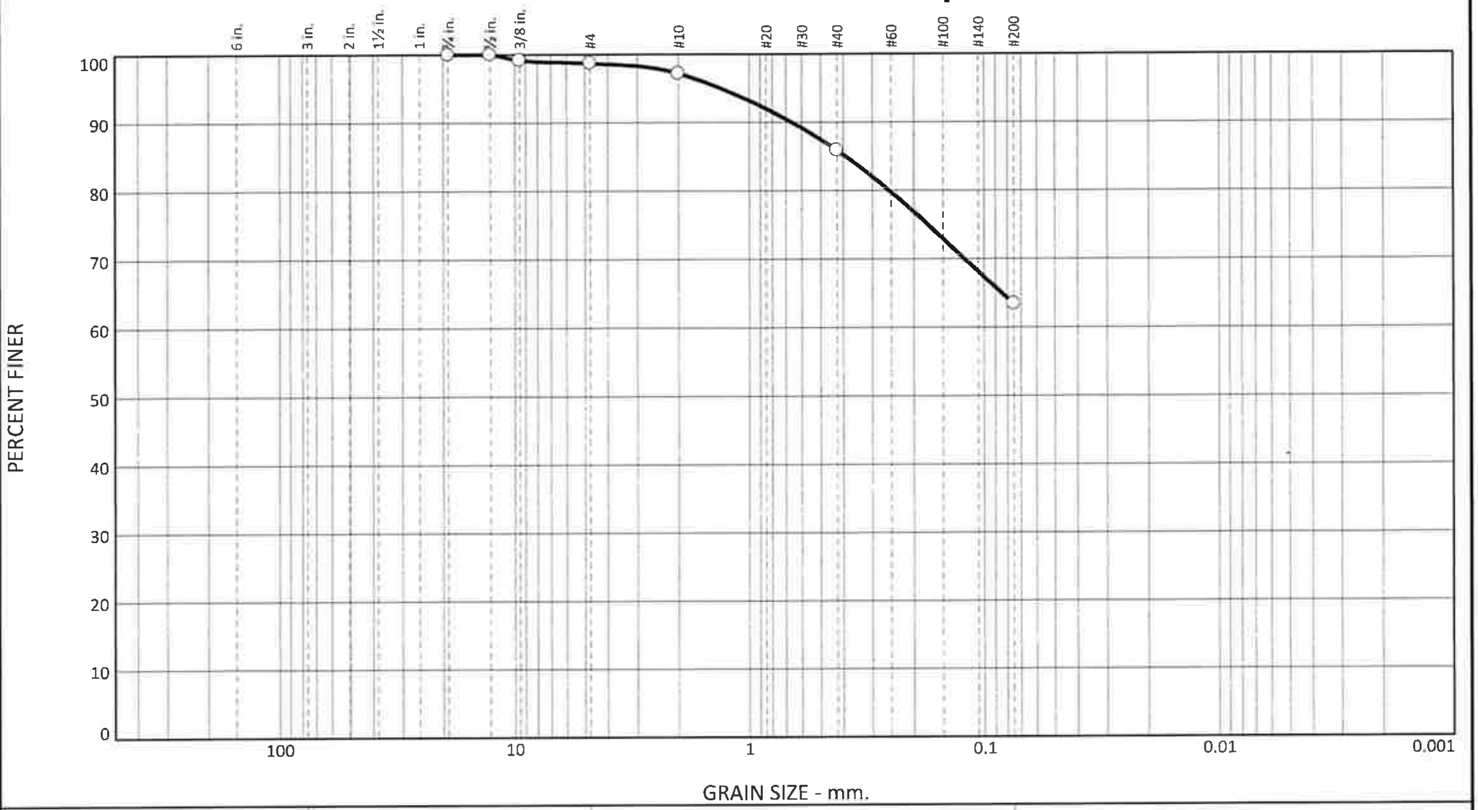
TRAUTNER GEOTECH LLC

Client: REYNOLDS ASH + ASSOCIATES
Project: Cascade Village Townhomes South

Project No: 58656GE **Figure** B.4

Tested By: N. Granda **Checked By:** J. Vaughn

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	1.3	1.5	11.3	22.5	63.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100.0		
1/2"	100.0		
3/8"	99.2		
#4	98.7		
#10	97.2		
#40	85.9		
#200	63.4		

* (no specification provided)

Soil Description

CL - sandy lean clay

Atterberg Limits

PL= 20 LL= 42 PI= 22

Coefficients

D_{90}= 0.6618 D_{85}= 0.3907 D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO= A-7-6(12)

Remarks

Source of Sample: Test Boring 15 **Depth:** 0' - 3.5'
Sample Number: 13335-QA

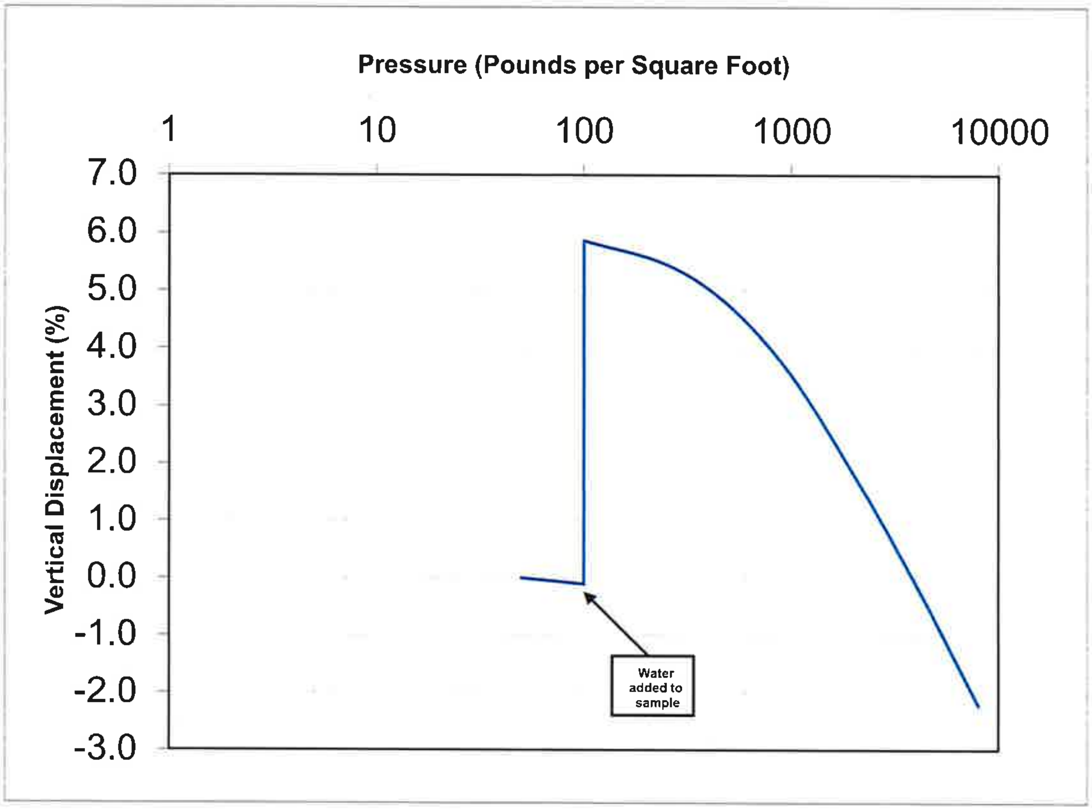
Date: 12/13/2024

TRAUTNER GEOTECH LLC

Client: REYNOLDS ASH + ASSOCIATES
Project: Cascade Village Townhomes South
Project No: 58656GE **Figure** B.6

Tested By: N. Granda **Checked By:** J. Vaughn

SWELL - CONSOLIDATION TEST

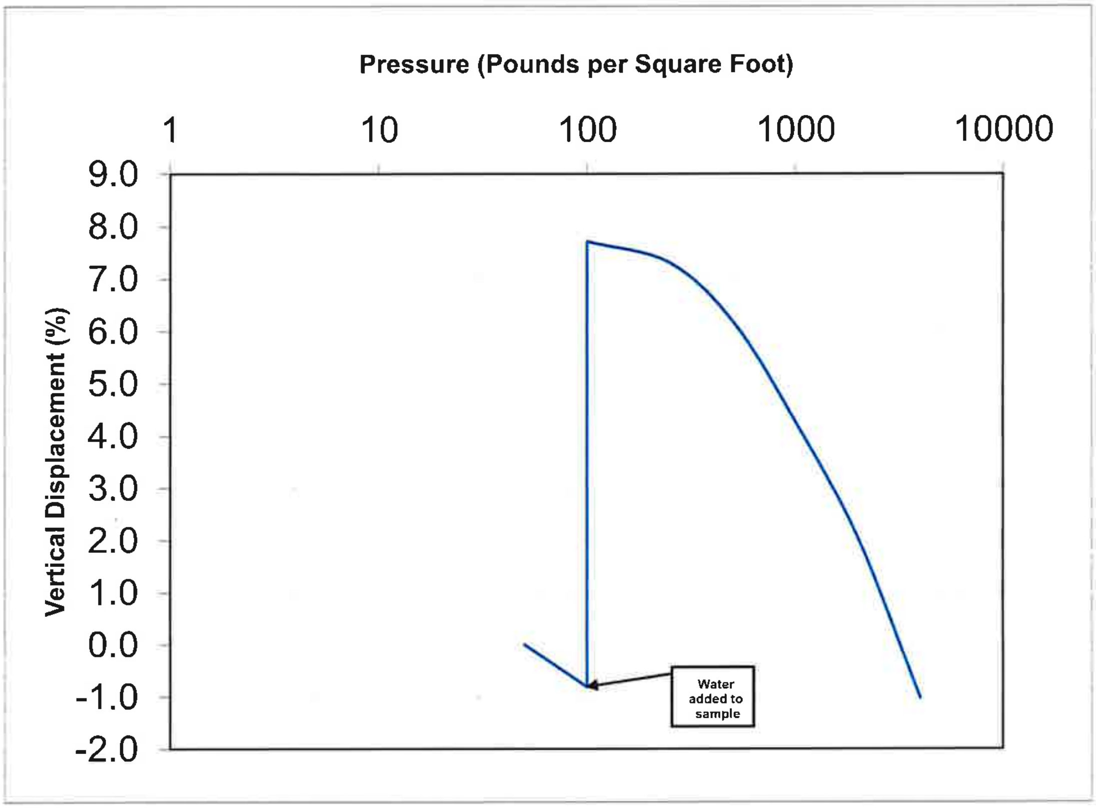


SUMMARY OF TEST RESULTS		
Sample Source:	TB-1 @ 3'	
Visual Soil Description:	CL	
Swell Potential (%)	6.0%	
Estimated Load-Back Swell Pressure (lb/ft^2):	4,000	
	Initial	**Final**
Moisture Content (%):	8.2	18.7
Dry Density (lb/ft^3):	112.7	114.5
Height (in.):	0.989	0.967
Diameter (in.):	1.94	1.94

Note: Remolded Sample; Molded from the portion of sample passing a #10 sieve. Consolidated under 500 PSF prior to initiating load sequence and wetting. Initial values represent the conditions under 50 PSF following the pre-consolidation under 500 PSF.

Project Number:	58656 GE
Sample ID:	13335-B
Figure:	B.8

SWELL - CONSOLIDATION TEST

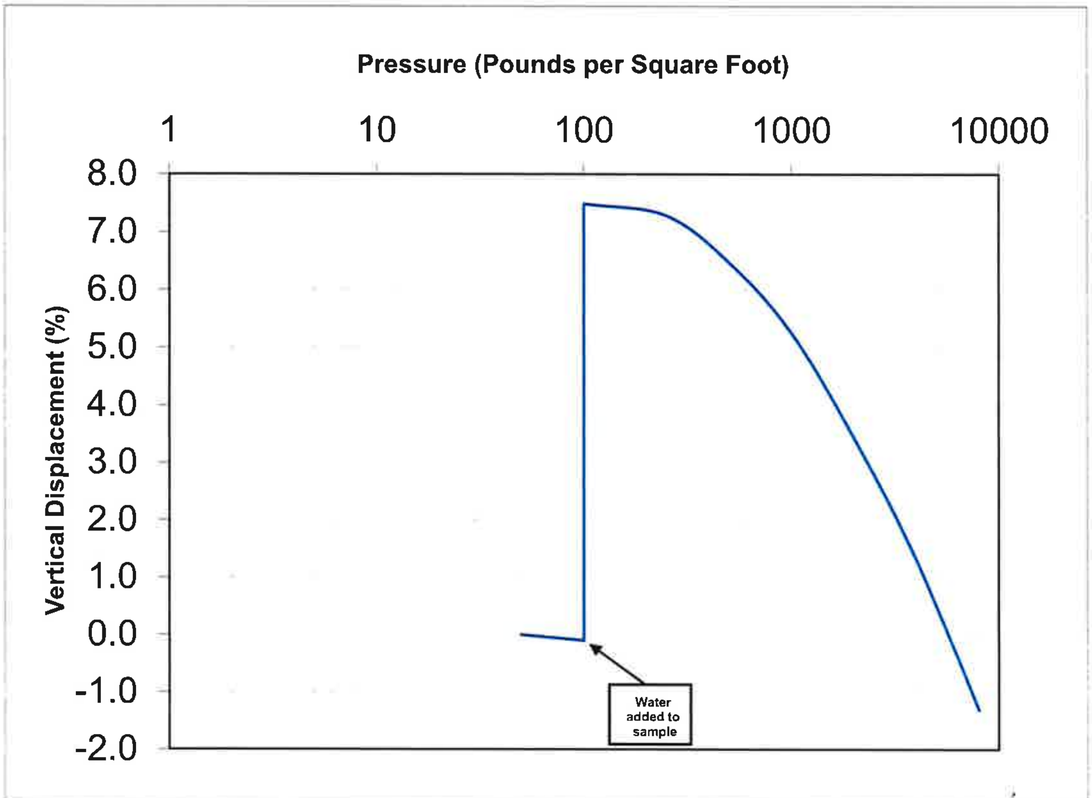


SUMMARY OF TEST RESULTS		
Sample Source:	TB-2 @ 2'	
Visual Soil Description:	CL-ML	
Swell Potential (%)	8.5%	
Estimated Load-Back Swell Pressure (lb/ft^2):	3,370	
	Initial	**Final**
Moisture Content (%):	12.9	34.9
Dry Density (lb/ft^3):	86.8	87.3
Height (in.):	0.985	0.975
Diameter (in.):	1.94	1.94

Note: Remolded Sample; Molded from the portion of sample passing a #10 sieve. Consolidated under 500 PSF prior to initiating load sequence and wetting. Initial values represent the conditions under 50 PSF following the pre-consolidation under 500 PSF.

Project Number:	58656 GE
Sample ID:	13335-D
Figure:	B.9

SWELL - CONSOLIDATION TEST

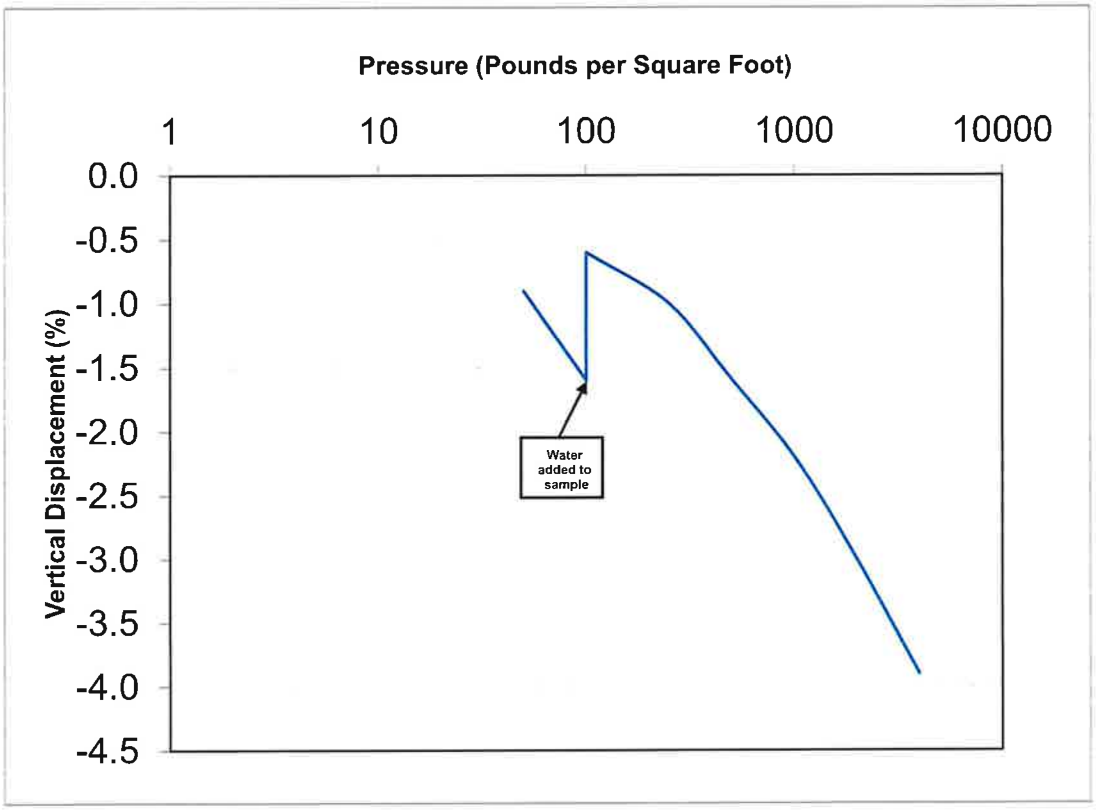


SUMMARY OF TEST RESULTS		
Sample Source:	TB-3 @ 3.5	
Visual Soil Description:	CL	
Swell Potential (%)	7.6%	
Estimated Load-Back Swell Pressure (lb/ft^2):	5,000	
	Initial	**Final**
Moisture Content (%):	10.0	19.2
Dry Density (lb/ft^3):	112.8	114.3
Height (in.):	0.988	0.975
Diameter (in.):	1.94	1.94

Note: Remolded Sample; Molded from the portion of sample passing a #10 sieve. Consolidated under 500 PSF prior to initiating load sequence and wetting. Initial values represent the conditions under 50 PSF following the pre-consolidation under 500 PSF.

Project Number:	58656 GE
Sample ID:	13335-I
Figure:	B.10

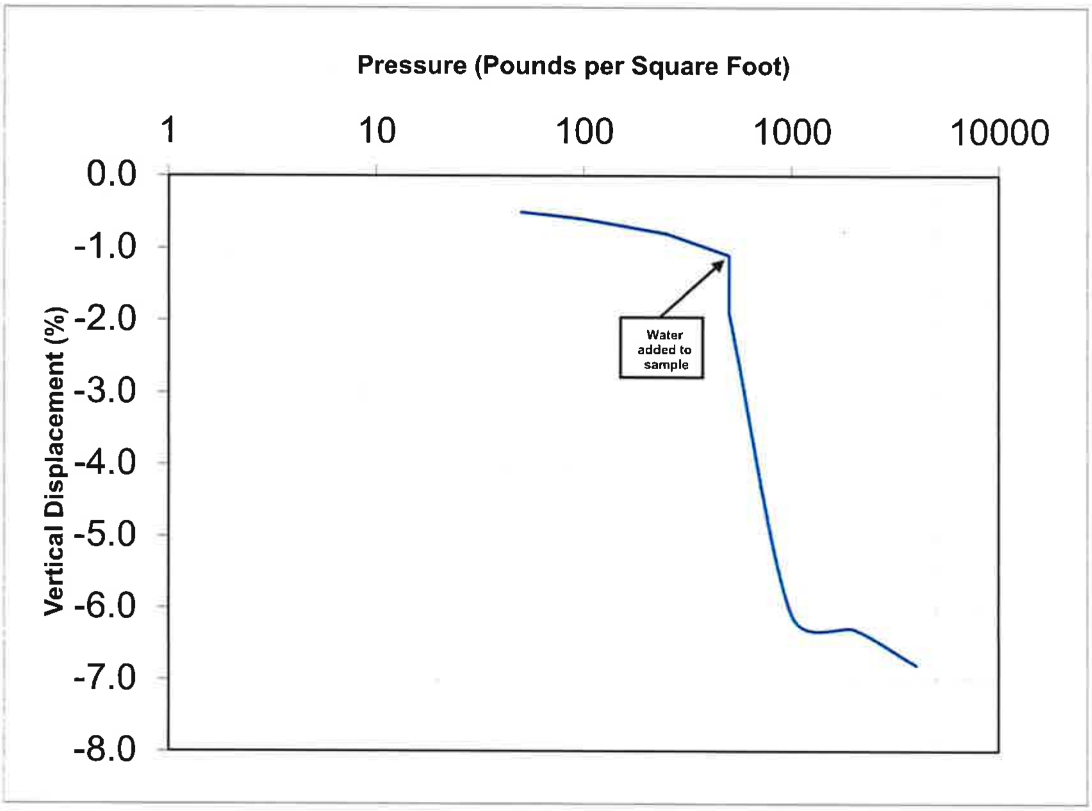
SWELL - CONSOLIDATION TEST



SUMMARY OF TEST RESULTS		
Sample Source:	TB-4 @ 4'	
Visual Soil Description:	GC	
Swell Potential (%)	1.0%	
Estimated Load-Back Swell Pressure (lb/ft^2):	640	
	Initial	**Final**
Moisture Content (%):	11.1	17.4
Dry Density (lb/ft^3):	118.6	117.8
Height (in.):	1.000	0.961
Diameter (in.):	1.94	1.94

Project Number:	58656 GE
Sample ID:	13335-L
Figure:	B.11

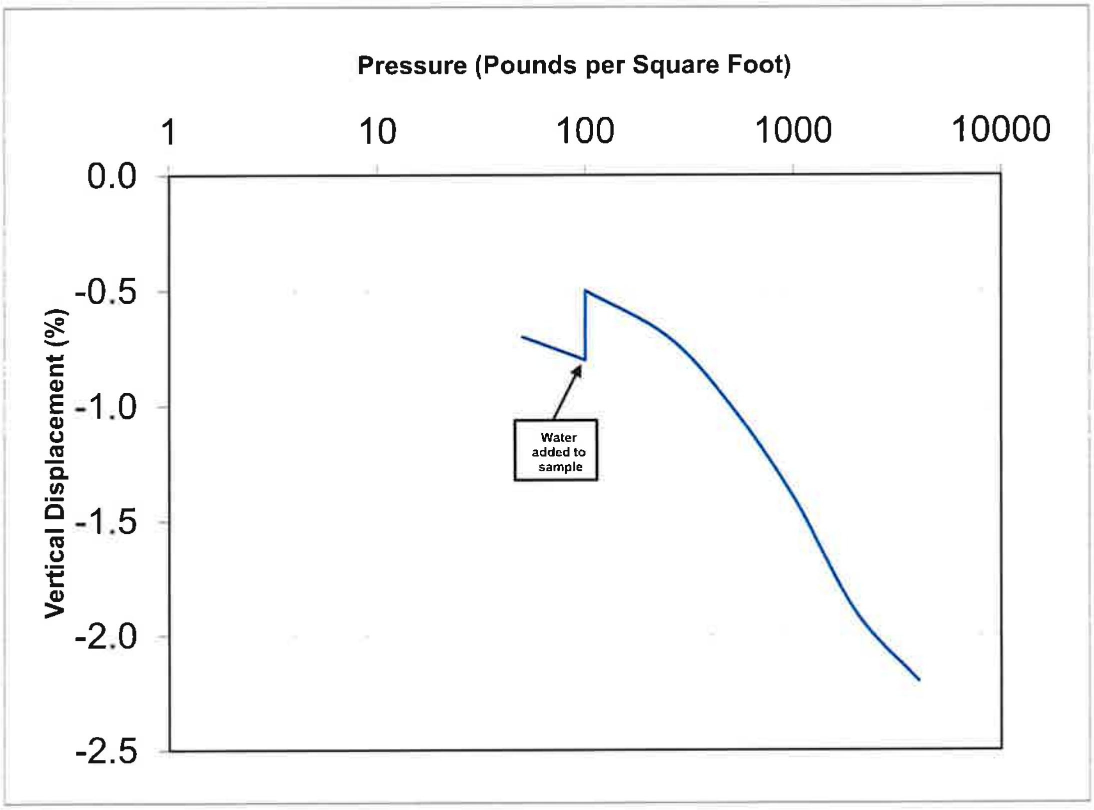
SWELL - CONSOLIDATION TEST



SUMMARY OF TEST RESULTS		
Sample Source:	TB-6 @ 8.5'	
Visual Soil Description:	GC	
Swell Potential (%)	-0.8%	
Estimated Load-Back Swell Pressure (lb/ft^2):	0	
	Initial	**Final**
Moisture Content (%):	6.4	7.1
Dry Density (lb/ft^3):	140.7	151.0
Height (in.):	1.000	0.932
Diameter (in.):	1.94	1.94

Project Number:	58656 GE
Sample ID:	13335-T
Figure:	B.12

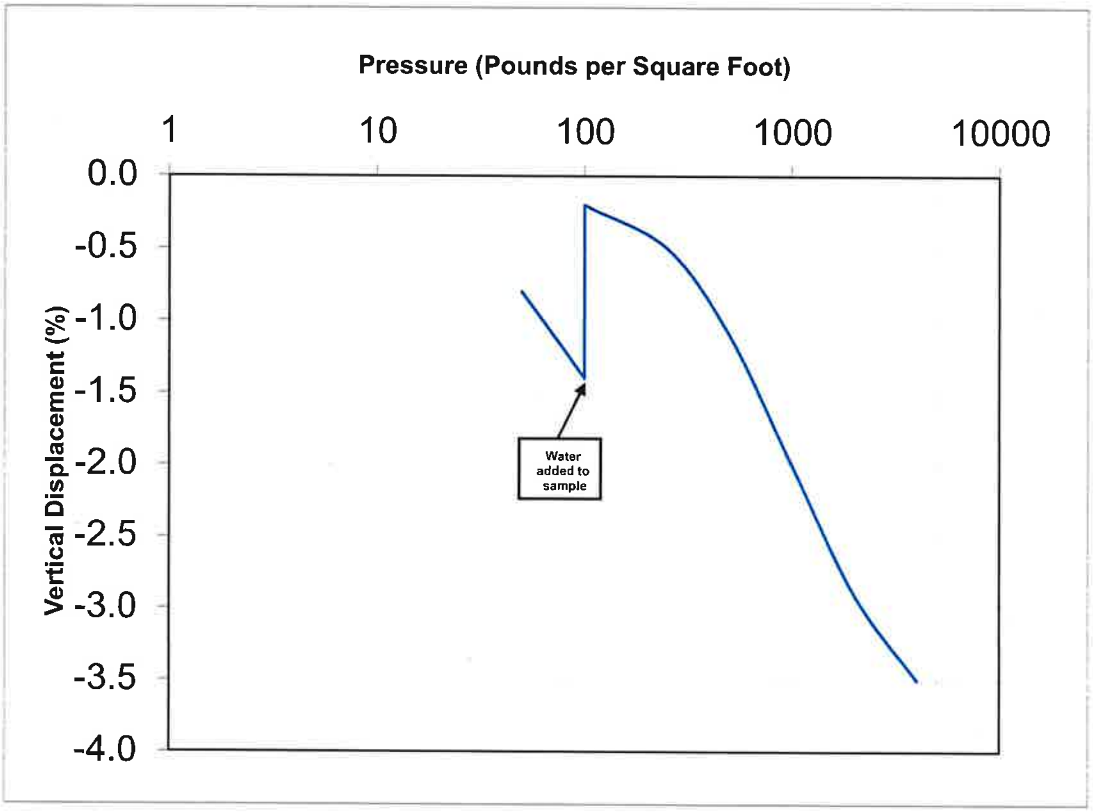
SWELL - CONSOLIDATION TEST



SUMMARY OF TEST RESULTS		
Sample Source:	TB-7 @ 2'	
Visual Soil Description:	SC	
Swell Potential (%)	0.3%	
Estimated Load-Back Swell Pressure (lb/ft^2):	350	
	Initial	**Final**
Moisture Content (%):	12.7	10.7
Dry Density (lb/ft^3):	125.7	132.9
Height (in.):	1.000	0.978
Diameter (in.):	1.94	1.94

Project Number:	58656 GE
Sample ID:	13335-U
Figure:	B.13

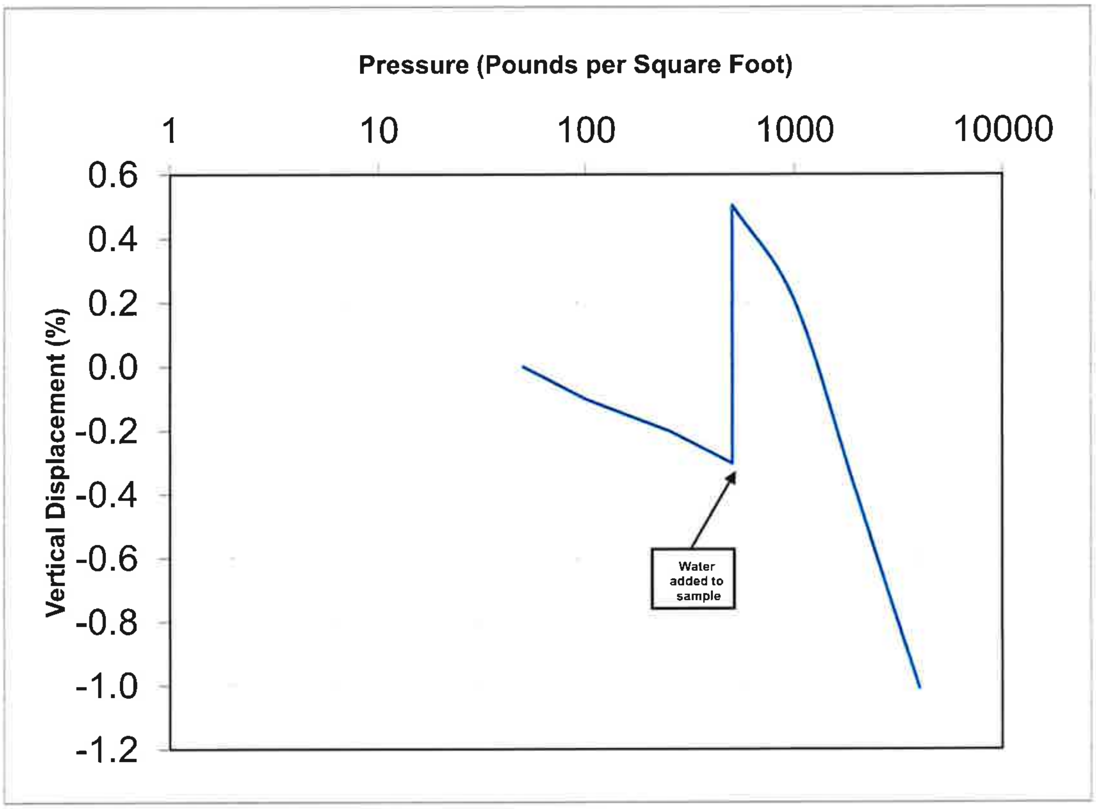
SWELL - CONSOLIDATION TEST



SUMMARY OF TEST RESULTS		
Sample Source:	TB-8 @ 3.5'	
Visual Soil Description:	GC	
Swell Potential (%)	1.2%	
Estimated Load-Back Swell Pressure (lb/ft^2):	720	
	Initial	**Final**
Moisture Content (%):	5.4	13.3
Dry Density (lb/ft^3):	128.6	129.8
Height (in.):	1.000	0.965
Diameter (in.):	1.94	1.94

Project Number:	58656 GE
Sample ID:	13335-Z
Figure:	B.14

SWELL - CONSOLIDATION TEST

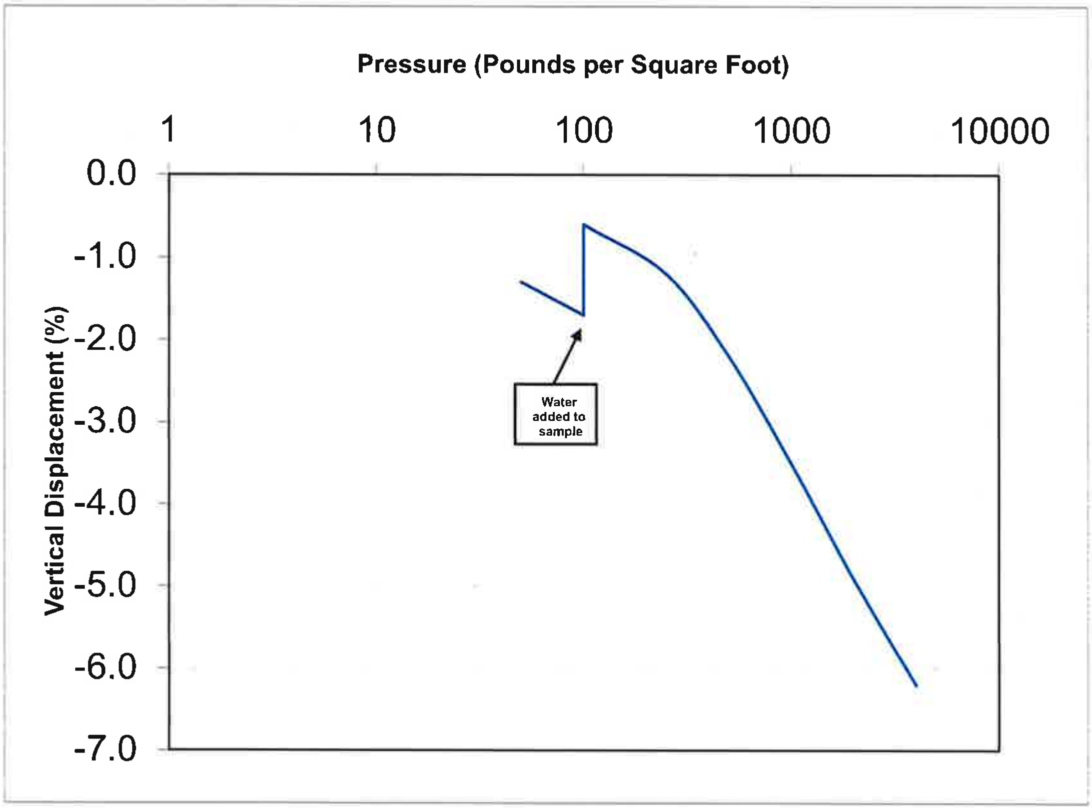


SUMMARY OF TEST RESULTS		
Sample Source:	TB-9 @ 3.5	
Visual Soil Description:	SC	
Swell Potential (%)	0.8%	
Estimated Load-Back Swell Pressure (lb/ft²):	1,860	
	Initial	**Final**
Moisture Content (%):	7.8	14.4
Dry Density (lb/ft³):	119.8	120.7
Height (in.):	0.992	0.982
Diameter (in.):	1.94	1.94

Note: Remolded Sample; Molded from the portion of sample passing a #10 sieve. Consolidated under 500 PSF prior to initiating load sequence and wetting. Initial values represent the conditions under 50 PSF following the pre-consolidation under 500 PSF.

Project Number:	58656 GE
Sample ID:	13335-FA
Figure:	B.15

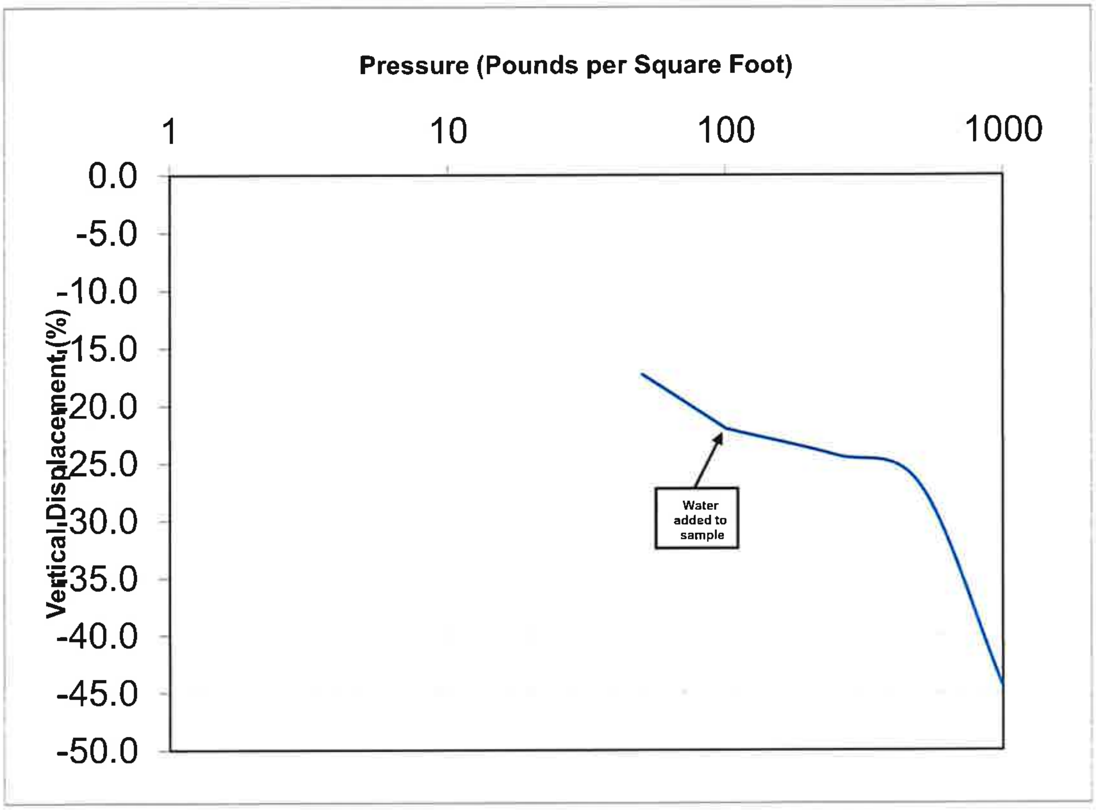
SWELL - CONSOLIDATION TEST



SUMMARY OF TEST RESULTS		
Sample Source:	TB-10 @ 3.5'	
Visual Soil Description:	SC	
Swell Potential (%)	1.1%	
Estimated Load-Back Swell Pressure (lb/ft^2):	360	
	Initial	**Final**
Moisture Content (%):	10.7	19.8
Dry Density (lb/ft^3):	111.0	116.6
Height (in.):	1.000	0.938
Diameter (in.):	1.94	1.94

Project Number:	58656 GE
Sample ID:	13335-IA
Figure:	B.16

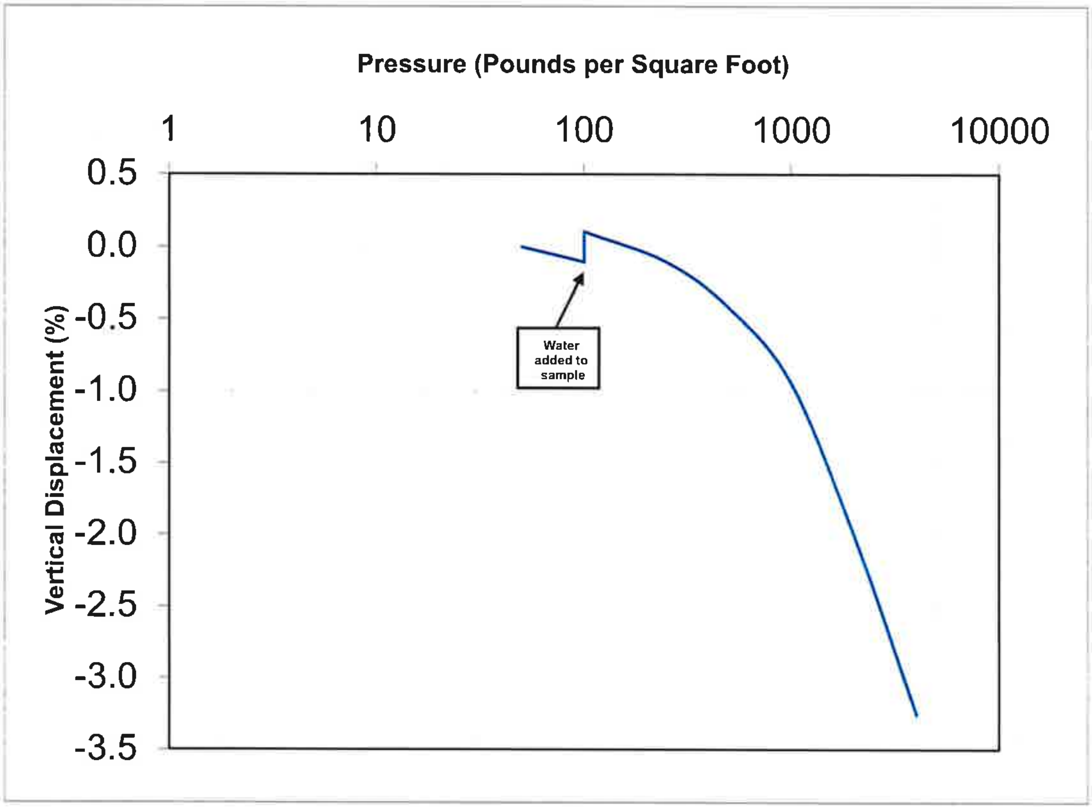
SWELL - CONSOLIDATION TEST



SUMMARY OF TEST RESULTS		
Sample Source:	TB-11 @ 4'	
Visual Soil Description:	OL	
Swell Potential (%)	0.0%	
Estimated Load-Back Swell Pressure (lb/ft^2):	0	
	Initial	**Final**
Moisture Content (%):	831.2	457.1
Dry Density (lb/ft^3):	7.0	13.0
Height (in.):	1.000	0.555
Diameter (in.):	1.94	1.94

Project Number:	58656 GE
Sample ID:	13335-Z
Figure:	B.17

SWELL - CONSOLIDATION TEST

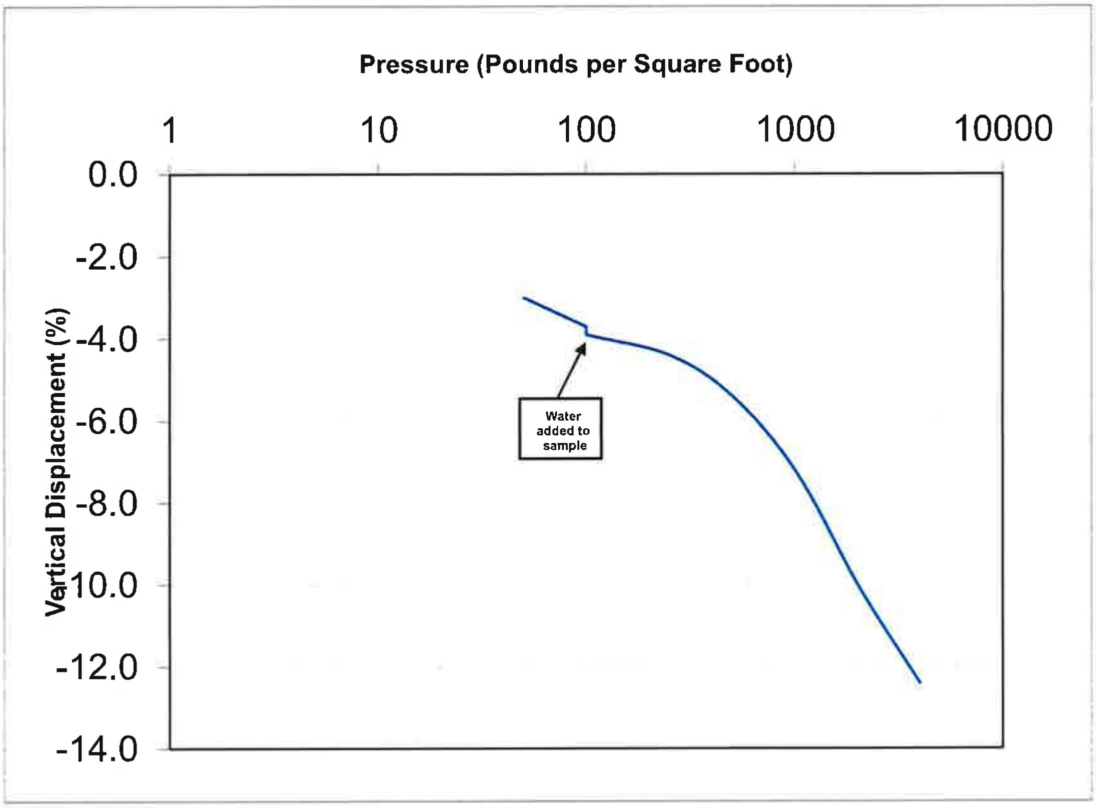


SUMMARY OF TEST RESULTS		
Sample Source:	TB-14 @ 3.5'	
Visual Soil Description:	SC	
Swell Potential (%)	0.2%	
Estimated Load-Back Swell Pressure (lb/ft^2):	270	
	Initial	**Final**
Moisture Content (%):	5.3	13.0
Dry Density (lb/ft^3):	127.0	130.4
Height (in.):	0.952	0.921
Diameter (in.):	1.94	1.94

Note: Remolded Sample; Molded from the portion of sample passing a #10 sieve. Consolidated under 500 PSF prior to initiating load sequence and wetting. Initial values represent the conditions under 50 PSF following the pre-consolidation under 500 PSF.

Project Number:	58656 GE
Sample ID:	13335-OA
Figure:	B.19

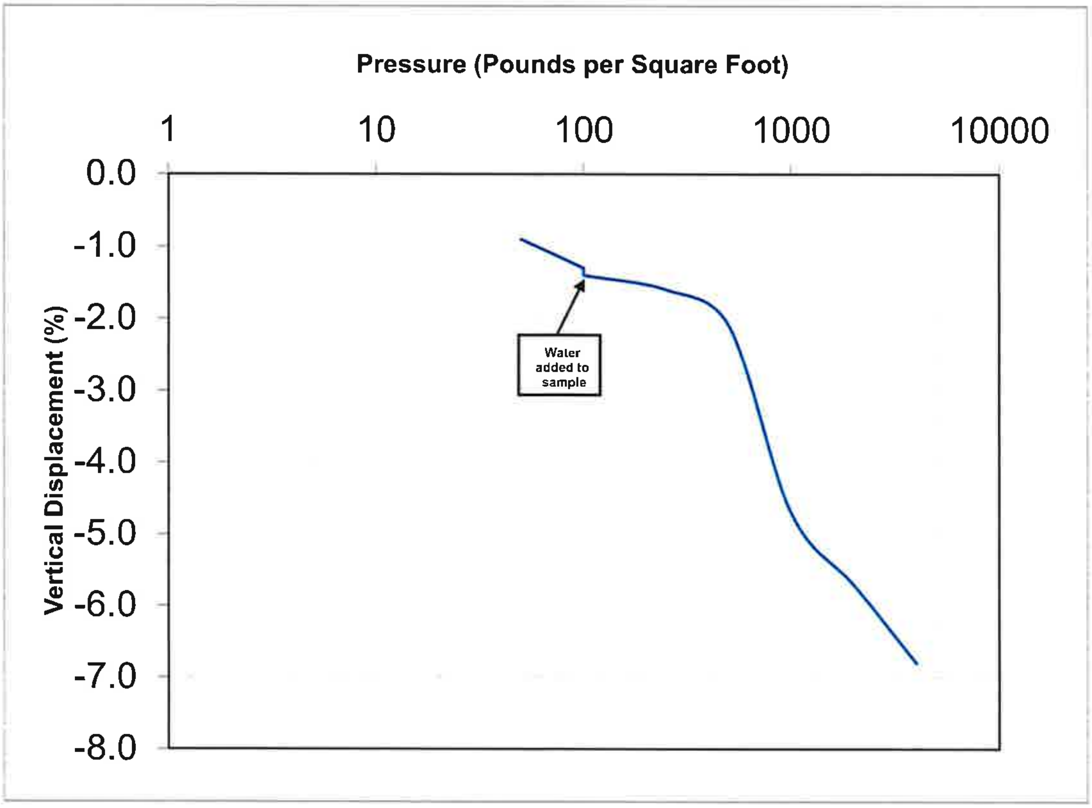
SWELL - CONSOLIDATION TEST



SUMMARY OF TEST RESULTS		
Sample Source:	TB-15 @ 3.5'	
Visual Soil Description:	CL	
Swell Potential (%)	-0.2%	
Estimated Load-Back Swell Pressure (lb/ft^2):	0	
	Initial	**Final**
Moisture Content (%):	23.4	23.0
Dry Density (lb/ft^3):	103.2	113.4
Height (in.):	1.000	0.876
Diameter (in.):	1.94	1.94

Project Number:	58656 GE
Sample ID:	13335-RA
Figure:	B.20

SWELL - CONSOLIDATION TEST



SUMMARY OF TEST RESULTS		
Sample Source:	TB-16 @ 3.5'	
Visual Soil Description:	SC	
Swell Potential (%)	-0.1%	
Estimated Load-Back Swell Pressure (lb/ft^2):	0	
	Initial	**Final**
Moisture Content (%):	26.0	22.7
Dry Density (lb/ft^3):	99.3	106.6
Height (in.):	1.000	0.932
Diameter (in.):	1.94	1.94

Project Number:	58656 GE
Sample ID:	13335-TA
Figure:	B.21

Residual Direct Shear Test Results:

Project: Cascade Village Townhomes-South

Project Number: 58656 GE

Laboratory Sample ID: 13335-M

Sample Date: 12/13/2024

Test Date: 12/23/2024

Technician: G. Jadrych

Sample Source: TB-4 5'-9'

Visual Soil Description: SC

Type of Specimen: Remolded Square Shear Box

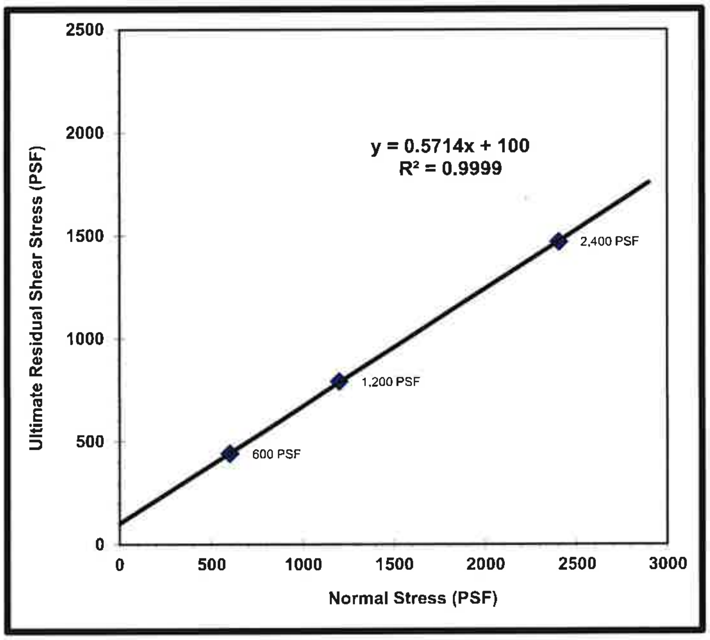
Diameter: 2.5 in

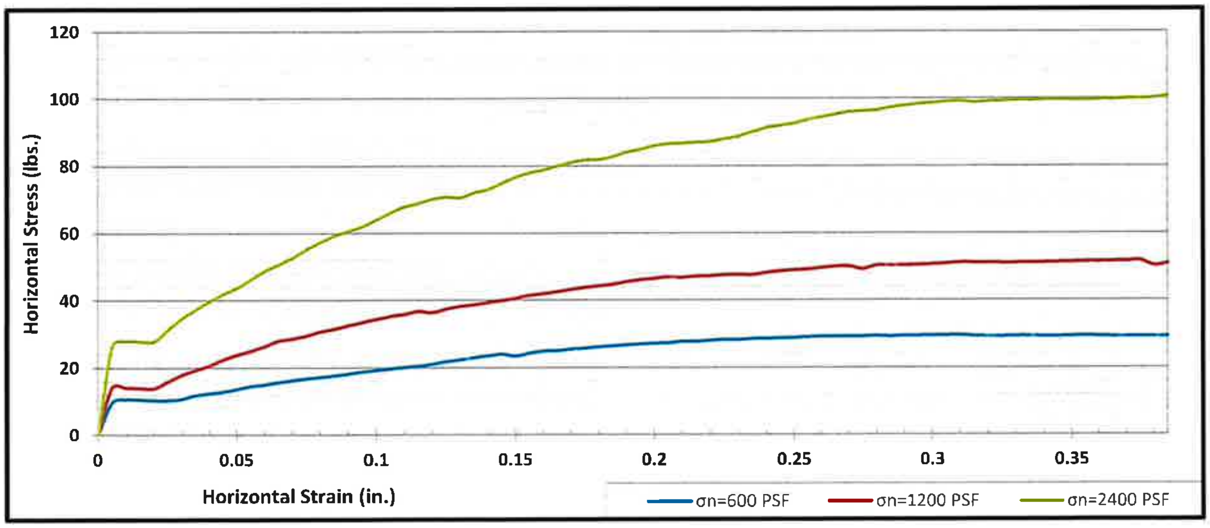
Height: 1.0 in

Residual Direct Shear Test Results:			
Normal Stress, σ_n (PSF):	2400	1200	600
Ultimate Shear Stress, τ_{ult} (PSF):	1470	790	440

Summary of Sample Data:	
Initial Moisture Content (%):	9.2
Intial Dry Density (PCF):	104.0
Final Moisture Content (%):	21.2
Final Dry Density (PCF):	93.9

ESTIMATED STRENGTH PARAMETERS	
Angle of Internal Friction, ϕ (°):	30
Cohesion (PSF):	100
Horizontal Displacement (in.)	0.1





TRAUTNER GEOTECH LLC
GEOTECHNICAL ENGINEERING, MATERIAL TESTING
AND ENGINEERING GEOLOGY

Residual Direct Shear Test Results:

Project: Cascade Village Townhomes- South

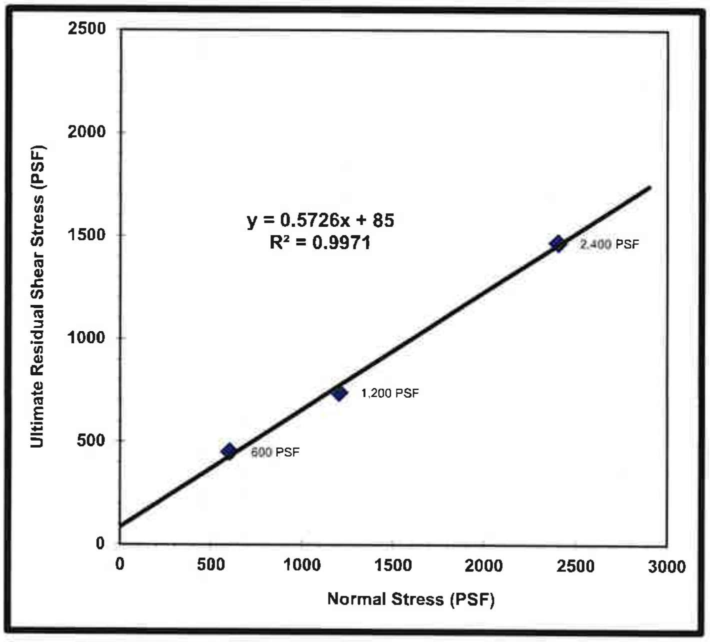
Project Number: 58656 GE
Laboratory Sample ID: 13335-DA
Sample Date: 12/13/2024
Test Date: 12/16/2024
Technician: G. Jadrych/ N. Granda

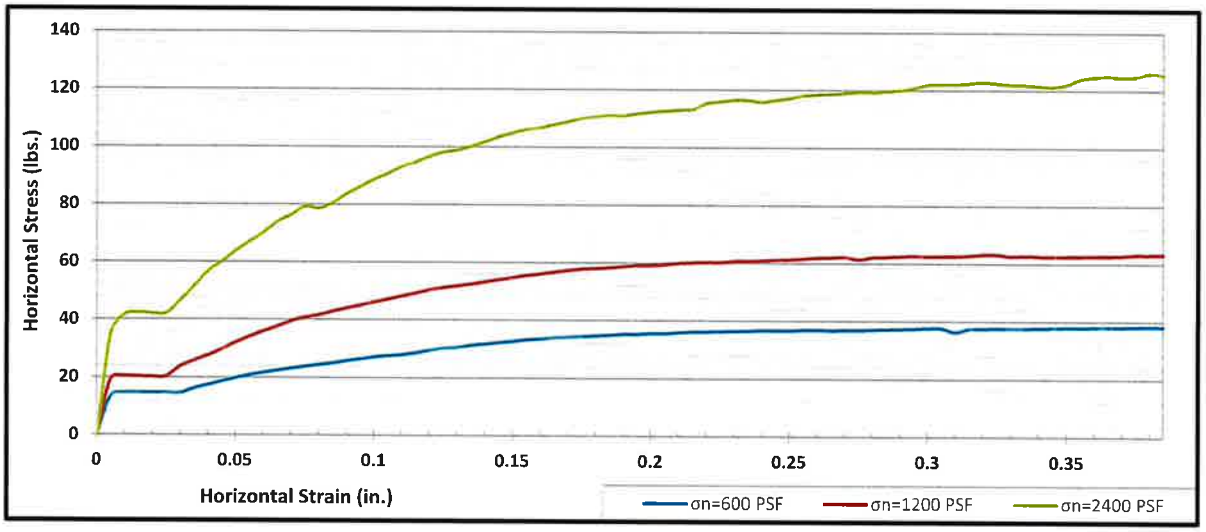
Sample Source: TB-8 14'-19'
Visual Soil Description: CL with sand
Type of Specimen: Remolded Square Shear Box
Diameter: 2.5 in
Height: 1.0 in

Residual Direct Shear Test Results:			
Normal Stress, σ_n (PSF):	2400	1200	600
Ultimate Shear Stress, τ_{ult} (PSF):	1470	740	450

Summary of Sample Data:	
Initial Moisture Content (%):	8.3
Intial Dry Density (PCF):	131.6
Final Moisture Content (%):	14.5
Final Dry Density (PCF):	124.8

ESTIMATED STRENGTH PARAMETERS	
Angle of Internal Friction, ϕ (°):	30
Cohesion (PSF):	85
Horizontal Displacement (in.)	0.05





TRAUTNER GEOTECH LLC
GEOTECHNICAL ENGINEERING, MATERIAL TESTING
AND ENGINEERING GEOLOGY

California Bearing Ratio Test Results

ASTM D-1883

Project Name: Cascade South
Project Number: 58656-GE
Sample I.D.: 13335-XA
Sample Date: 1/3/2025
Technician: G. Jadrych

Sample Source: Combined from all borings
Sample Description: Bulk Subgrade

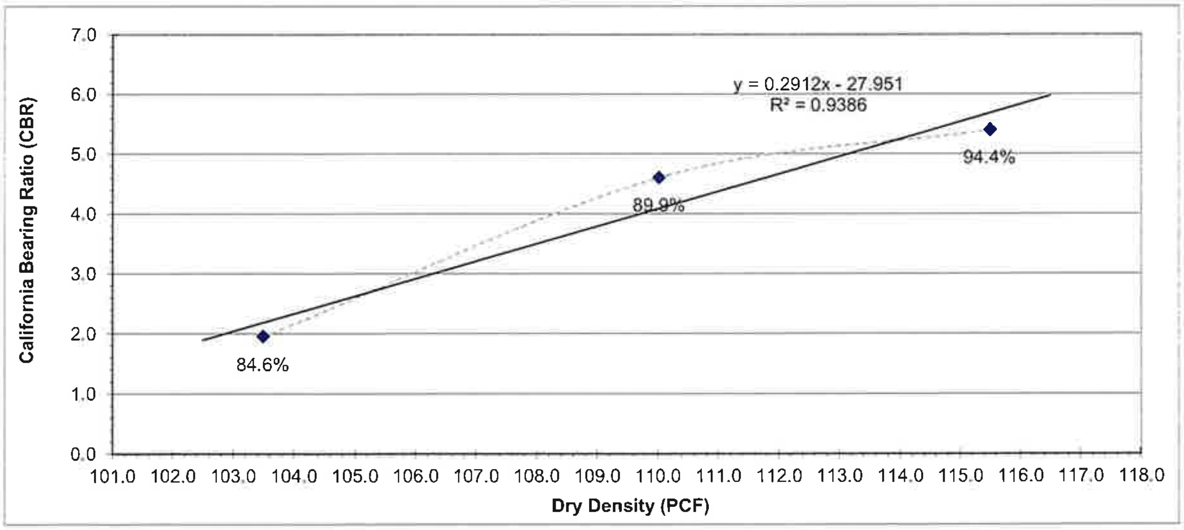
Proctor Method: D 1557 method A
Proctor Maximum Dry Density: 122.4 PCF
Optimum Moisture Content: 11.2 %
Start Soak: 12/30/2024
End Soak: 1/3/2025
Surcharge During Soak: 15 Lbs

Pre-Soak:

Dry Density (PCF)	Moisture Content (%)	Relative Compaction
103.5	10.4	84.6%
110.0	10.9	89.9%
115.5	11.2	94.4%

Post-Soak:

Dry Density (PCF)	Moisture Content of Top One (1) Inch (%)	Swell (%)	CBR (0.100" penetration)
97.5	25.5	3.2	2.0
103.3	22.8	3.3	4.6
108.8	20.9	2.6	5.4



California Bearing Ratio @ 90% of Proctor Density: 4.1

155917
Page 1 of 2
SAN JUAN COUNTY, COLORADO
LADONNA L. JARAMILLO, RECORDER
05-15-2025 02:31 PM Recording Fee $18.00
No Doc Fee

WHEN RECORDED RETURN TO:
Amy Rhyne
PO Box 34781
Charlotte, NC 28234

CORRECTION BARGAIN AND SALE DEED

This Deed is made this 12th day of May, 2025, between Morehead Property One, LLC, a North Carolina limited liability company having an address of 1355 Greenwood Cliff Suite 150, Charlotte, NC 28204 ("Grantor") for the consideration of ten dollars, ($10.00), in hand paid, hereby sells and conveys to Cascade Meadows, LLC, a Colorado limited liability company having a mailing address of PO Box 34781, Charlotte, NC 28234 ("Grantee"), the real property together with improvements, if any, situate and lying and being in the County of San Juan, State of Colorado described as follows:

See Exhibit A attached hereto and incorporated herein.

This deed corrects the name of the county and legal description of the real property identified in that certain Bargain and Sale Deed recorded on March 20, 2025 at Reception No. 155841 in the office of the clerk and recorder of San Juan County, Colorado.

With all appurtenances hereunto belonging.

IN WITNESS WHEREOF, the Grantor has executed this deed on the date set forth above.

GRANTOR:

Morehead Property One, LLC,
a North Carolina limited liability company

By: Charles Lindsey McAlpine, Manager

STATE OF COLORADO)
)ss.
COUNTY OF LA PLATA)

The foregoing instrument was acknowledged before me this 12th day of May 2025, by Charles Lindsey McAlpine, Manager of Morehead Property One, LLC, a North Carolina limited liability company.

Notary Public
My Commission expires: December 27, 2027

SARAH R VOGEL
NOTARY PUBLIC
STATE OF COLORADO
NOTARY ID 20074046267
MY COMMISSION EXPIRES 12/27/2027

Exhibit A - TRACT A-1

A parcel of land being a portion of that tract of land as shown on the Cascade Village Results of Survey plat, deposited in the Office of the San Juan County Clerk and Recorder under Reception No. 141, San Juan County, Colorado, and a portion of Tract A-1 of the Cascade Village Amended Master Plan recorded under Reception No. 137955, said parcel being more particularly described as follows:

Beginning at the CS1/16 Corner of Section 13, Township 39 North, Range 9 West, N.M.P.M.,

Thence S 89°18'44" W, along the south line of the NE1/4SW1/4 of said Section 13 a distance of 1327.94 feet to the SW1/16 Corner of Section 13, , Township 39 North, Range 9 West, N.M.P.M.;

Thence N 00°21'14" W, along the west line of the NE1/4SW1/4 of said Section 13, a distance of 1321.26 feet to the CW1/16 of said Section 13, Township 39 North, Range 9 West, N.M.P.M.;

Thence N 00°25'55" W, along the west line of the SE1/4NW1/4 of said Section 13, a distance of 2327.94 feet;

Thence East, along the south line of Cascade Village Phase 1, recorded in said Clerk and Recorder in Book 222 and on Pages 125, 126, and 27, a distance of 246.74 feet;

Thence S 15°00'00" E, along the west line of said Cascade Village Phase 1, a distance of 531.77 feet to the north line of the Twilight Meadow Subdivision Phase II at Cascade Village Final Plat, recorded in said Clerk and Recorder, Reception No. 140023;

Thence N 89°59'18" W, along said north line, a distance of 16.73 feet;

Thence S 13°30'56" E, along the west line of said the Twilight Meadow Subdivision Phase II and the west line of the Resubdivision of the Twilight Meadow Subdivision at Cascade Village recorded in said Clerk and Recorder, Reception No. 136239, a distance of 951.46 feet;

Thence S 89°03'40" E, along the south line of said Resubdivision of the Twilight Meadow Subdivision at Cascade Village, a distance of 360.48 feet, to a point on the easterly line of said Tract A-1 of the Cascade Village Amended Master Plan recorded under Reception No. 137955;

Thence S 07°33'00" E, along said easterly line of said Tract A-1, a distance of 699.29 feet to a point also being on the centerline of an aqueduct easement (twenty-five (25) feet on the westerly side and forty (40) feet on the easterly side) recorded in said San Juan County Clerk and Recorder in Book 222 on Page 101;

Thence S 10°14'00" E, along said easterly line of Tract A-1 and said centerline aqueduct,easement, a distance of 123.00 feet;

Thence S 32°49'00" E, along said easterly line of Tract A-1 and said centerline aqueduct easement, a distance of 454.00 feet;

Thence N 89°39'51" E, along said easterly line of Tract A-1 and departing said aqueduct easement, a distance of 68.32 feet to a point on the east line of the NE1/4SW1/4 of Section 13, Township 39 North, Range 9 West, N.M.P.M.;

Thence S 00°20'09" E, along said east line of the NE1/4SW1/4 of Section 13, a distance of 688.29 feet to the point of beginning;

Contains 66.450 acres, more or less.

Name and Address of Person Creating Newly Created Legal Description (§38-35-106.5, C.R.S.): Robert L. Trudeaux, P.L.S. of Goff Engineering & Surveying, Inc., PO Box 97, Durango CO 81302.

155918
Page 1 of 3
SAN JUAN COUNTY, COLORADO
LADONNA L. JARAMILLO, RECORDER
05-15-2025 02:31 PM Recording Fee $23.00
No Doc Fee

WHEN RECORDED RETURN TO:
Amy Rhyne
PO Box 34781
Charlotte, NC 28234

CORRECTION BARGAIN AND SALE DEED

This Deed is made this 12th day of May, 2025, between Morehead Property One, LLC, a North Carolina limited liability company having an address of 1355 Greenwood Cliff Suite 150, Charlotte, NC 28204 ("Grantor") for the consideration of ten dollars, ($10.00), in hand paid, hereby sells and conveys to Cascade Hospitality, LLC, a Colorado limited liability company having a mailing address of PO Box 34781, Charlotte, NC 28234 ("Grantee"), the real property together with improvements, if any, situate and lying and being in the County of San Juan, State of Colorado described as follows:

See Exhibit A attached hereto and incorporated herein.

This deed corrects the name of the county and legal description of the real property identified in that certain Bargain and Sale Deed recorded on March 20, 2025 at Reception No. 155843 in the office of the clerk and recorder of San Juan County, Colorado.

With all appurtenances hereunto belonging.

IN WITNESS WHEREOF, the Grantor has executed this deed on the date set forth above.

GRANTOR:

Morehead Property One, LLC,
a North Carolina limited liability company

By: Charles Lindsey McAlpine, Manager

STATE OF COLORADO)
)ss.
COUNTY OF LA PLATA)

The foregoing instrument was acknowledged before me this 12th day of May 2025, by Charles Lindsey McAlpine, Manager of Morehead Property One, LLC, a North Carolina limited liability company.

Notary Public
My Commission expires: December 27, 2027

SARAH R VOGEL
NOTARY PUBLIC
STATE OF COLORADO
NOTARY ID 20074046267
MY COMMISSION EXPIRES 12/27/2027

Exhibit A

TRACT B-1

A parcel of land being a portion of that tract of land as shown on the Cascade Village Results of Survey plat, deposited in the Office of the San Juan County Clerk and Recorder under Reception No. 141, San Juan County, Colorado, and Tract B-1 of the Cascade Village Amended Master Plan recorded under Reception No. 137955, said parcel being more particularly described as follows:

Beginning at a point on the east line of the NE1/4SW1/4 of Section 13, Township 39 North, Range 9 West, N.M.P.M., from which the CS1/16 Corner of Section 13 bears S 00°20'09" E, a distance of 688.29 feet;

Thence S 89°39'51" W, along the south line of said Tract B-1 of the Cascade Village Amended Master Plan recorded under Reception No. 137955, a distance of 68.32 feet, to a point on the westerly line of said Tract B-1 and the centerline of an aqueduct easement (twenty-five (25) feet on the westerly side and forty (40) feet on the easterly side) recorded in said San Juan County Clerk and Recorder in Book 222 on Page 101;

Thence N 32°49'00" W, along said westerly line of Tract B-1 and said centerline aqueduct easement, a distance of 454.00 feet;

Thence N 10°14'00" W, along said westerly line of Tract B-1 and said centerline aqueduct easement, a distance of 123.00 feet;

Thence N 07°33'00" W, along said westerly line of Tract B-1 and said centerline aqueduct easement, a distance of 699.29 feet;

Thence N 05°26'23" E, along said westerly line of Tract B-1 and departing said centerline aqueduct easement, a distance of 306.18 feet to the southerly line of the First Amendment of the Resubdivision of the Twilight Meadow Subdivision at Cascade Village, recorded in said Clerk and Recorder, Reception No. 136848;

Thence N 05°26'23" E, along said southerly line, a distance of 70.51 feet;

Thence S 76°00'00" E, along said southerly line, a distance of 144.57 feet;

Thence along said southerly line, along a non-tangent curve to the right with a delta angle of 64°53'40" and a radius of 69.05 feet, a distance of 78.21 feet, the long chord bears S 43°33'10" E, a distance of 74.09 feet;

Thence along said southerly line, along a non-tangent curve to the right with a delta angle of 62°26'57" and a radius of 20.00 feet, a distance of 21.80 feet, the long chord bears S 20°07'08" W, a distance of 20.74 feet;

Thence along said southerly line, along a non-tangent curve to the left with a delta angle of 68°03'01" and a radius of 35.00 feet, a distance of 41.57 feet, the long chord bears S 17°19'06" W, a distance of 39.17 feet;

Thence S 76°38'11" W, along said southerly line, a distance of 13.85 feet;

Thence S 09°40'48" W, along said southerly line, a distance of 76.62 feet;

Thence S 19°09'25" E, along said southerly line, a distance of 205.18 feet;

Thence N 85°00'00" E, along said southerly line, a distance of 172.74 feet a point on the east line of the SE1/4NW1/4 of Section 13, Township 39 North, Range 9 West, N.M.P.M.,

Thence S 00°19'52" E, along said east line of the SE1/4NW1/4 of Section 13, a distance of 535.81 feet to the C1/4 Corner, Thence S 00°20'09" E, along said NE1/4SW1/4 of Section 13, a distance of 531.60 feet to the point of beginning.

Contains 10.480 acres, more or less.

Name and Address of Person Creating Newly Created Legal Description (§38-35-106.5, C.R.S.) Robert L. Trudeaux, P.L.S. of Goff Engineering & Surveying, Inc., PO Box 97, Durango CO 81302.

155919
Page 1 of 2
SAN JUAN COUNTY, COLORADO
LADONNA L. JARAMILLO, RECORDER
05-15-2025 02:31 PM Recording Fee $18.00
No Doc Fee

WHEN RECORDED RETURN TO:
Amy Rhyne
PO Box 34781
Charlotte, NC 28234

CORRECTION DEED BARGAIN AND SALE DEED

This Deed is made this 12th day of May, 2025, between Morehead Property One, LLC, a North Carolina limited liability company having an address of 1355 Greenwood Cliff Suite 150, Charlotte, NC 28204 ("Grantor") for the consideration of ten dollars, ($10.00), in hand paid, hereby sells and conveys to Camp Meadows, LLC, a Colorado limited liability company having a mailing address of PO Box 34781, Charlotte, NC 28234 ("Grantee"), the real property together with improvements, if any, situate and lying and being in the County of San Juan, State of Colorado described as follows:

See Exhibit A attached hereto and incorporated herein.

This deed corrects the name of the county identified in that certain Bargain and Sale Deed recorded on March 20, 2025 at Reception No. 155839 in the office of the clerk and recorder of San Juan County, Colorado.

With all appurtenances hereunto belonging.

IN WITNESS WHEREOF, the Grantor has executed this deed on the date set forth above.

GRANTOR:

Morehead Property One, LLC,
a North Carolina limited liability company

By: Charles Lindsey McAlpine, Manager

STATE OF COLORADO)
)ss.
COUNTY OF LA PLATA)

The foregoing instrument was acknowledged before me this 12th day of May 2025, by Charles Lindsey McAlpine, Manager of Morehead Property One, LLC, a North Carolina limited liability company.

Notary Public
My Commission expires: December 27, 2027

SARAH R VOGEL
NOTARY PUBLIC
STATE OF COLORADO
NOTARY ID 20074046267
MY COMMISSION EXPIRES 12/27/2027

Exhibit A

TRACT G

A parcel of land being a portion of Parcel IV, a 17.879-acre tract as shown on the Cascade Village Results of Survey plat, deposited in the Office of the San Juan County Clerk and Recorder under Reception No. 141, San Juan County, Colorado, also commonly known as Tract G of the Cascade Village Amended Master Plan recorded under Reception No. 137955, and being more particularly described as follows:

Beginning at a point on the westerly right-of-way line of State Highway 550, from which the CS1/16 Corner of Section 12, Township 39 North, Range 9 West, N.M.P.M., bears S 89°39'58" W, a distance of 205.51 feet;

Thence S 33°55'00" E, a distance of 209.37 feet;

Thence along the arc of a non-tangent curve to the left with a delta angle of 8°03'24" and a radius of 1020.91 feet, a distance of 143.56 feet, the long chord bears S 37°56'42" E, a distance of 143.44 feet;

Thence S 00°05'44" W, a distance of 206.62 feet;

Thence S 89°51'32" W, a distance of 506.23 feet to the easterly right-of-way line of State Highway 550;

Thence N 20°46'08" W, along said easterly right-of-way line of State Highway 550, a distance of 13.74 feet;

Thence N 24°39'08" W, along said easterly right-of-way of State Highway 550, a distance of 99 01 feet;

Thence N 26°32'08" W, along said easterly right-of-way of State Highway 550, a distance of 70.63 feet;

Thence N 25°52'08" W, along said easterly right-of-way of State Highway 550, a distance of 99.91 feet;

Thence N 10°22'08" W, along said easterly right-of-way of State Highway 550, a distance of 49.95 feet;

Thence N 02°39'08" W, along said easterly right-of-way of State Highway 550, a distance of 46.96 feet;

Thence N 07°04'12" E, along said easterly right-of-way of State Highway 550, a distance of 46.64 feet;

Thence N 89°39'58" E, departing said easterly right-of-way line of State Highway 550, a distance of 462.76 feet to the point of beginning.

Contains 6.350 acres, more or less.

155920
Page 1 of 3
SAN JUAN COUNTY, COLORADO
LADONNA L. JARAMILLO, RECORDER
05-15-2025 02:31 PM Recording Fee $23.00
No Doc Fee

WHEN RECORDED RETURN TO:
Amy Rhyne
PO Box 34781
Charlotte, NC 28234

CORRECTION BARGAIN AND SALE DEED

This Deed is made this 12th day of May, 2025, between Morehead Property One, LLC, a North Carolina limited liability company having an address of 1355 Greenwood Cliff Suite 150, Charlotte, NC 28204 ("Grantor") for the consideration of ten dollars, ($10.00), in hand paid, hereby sells and conveys to Cascade Highlands I, LLC, a Colorado limited liability company having a mailing address of PO Box 34781, Charlotte, NC 28234 ("Grantee"), the real property together with improvements. if any, situate and lying and being in the County of San Juan, State of Colorado described as follows:

Any and all development rights of Grantor in the common interest community known as Cascade Village, including but not limited to:

1. Development rights described in that certain Quit Claim Deed recorded on July 9, 2012 at Reception No.148558. Said Quit Claim Deed contains a reference to Article No. 1.27 and Special Rights of Mill Creek in the declaration recorded at Reception No. 145763 which declaration has since been amended and restated in its entirety and replaced and superseded by the terms and conditions of that Amended and Restated Master Declaration of Cascade Village recorded on October 2, 2015 at Reception No. 1501929 (the "Master Declaration"); and

2. Any and all development rights as described in the Master Declaration, including but not limited to, all development rights in Unbuilt Units, Tracts, and the Original Tract (consisting of the Grizzly Tract and the Vermillion Tract) more particularly described in Article 13 of the Master Declaration and as set forth on Exhibit A attached hereto and incorporated herein; and

3. Any and all Tract Rights of a Tract Owner to develop and install improvements on a Tract as more particularly described in Article 14 of the Master Declaration.

This deed corrects the name of the county identified in that certain Bargain and Sale Deed recorded on March 20, 2025 at Reception No. 155842 in the office of the clerk and recorder of San Juan County, Colorado.

With all appurtenances hereunto belonging.

IN WITNESS WHEREOF, the Grantor has executed this deed on the date set forth above.

GRANTOR:

Morehead Property One, LLC,
a North Carolina limited liability company

By: Charles Lindsey McAlpine, Manager

STATE OF COLORADO)
)ss.
COUNTY OF LA PLATA)

The foregoing instrument was acknowledged before me this 13th day of May 2025, by Charles Lindsey McAlpine, Manager of Morehead Property One, LLC, a North Carolina limited liability company.

Notary Public
My Commission expires: December 27, 2027

SARAH R VOGEL
NOTARY PUBLIC
STATE OF COLORADO
NOTARY ID 20074046267
MY COMMISSION EXPIRES 12/27/202

EXHIBIT A

Legal Description of Original Tracts as set forth in the Master Declaration

Grizzly Tract
Tract "A":
Beginning at a point from which the Northwest corner of said Cascade Village Phase 1bears North 32°11'06" West, a distance of 493.21 feet;
Thence North 68°30'00" East, a distance of 40 feet; Thence South 21°30'00" East, a distance of288 feet;
Thence South 68°30'00" West, a distance of 40 feet;
Thence North 21°30'00" East, a distance of288 feet to the point of beginning;

Vermillion Tract
Tract "AA":
Beginning at a point from which the Northwest corner of said Cascade Village Phase 1 bears North 49°03'02" West, a distance of 169.58 feet;
Thence North 67°00'00" East, a distance of 40 feet; Thence South 23°00'00" East, a distance of 288 feet;
Thence South 67°00'00" West, a distance of 40 feet;
Thence North 23"00'00" East, a distance of288 feet to the point of beginning.

155921
Page 1 of 2
SAN JUAN COUNTY, COLORADO
LADONNA L. JARAMILLO, RECORDER
05-15-2025 02:31 PM Recording Fee $18.00
No Doc Fee

WHEN RECORDED RETURN TO:
Amy Rhyne
PO Box 34781
Charlotte, NC 28234

CORRECTION BARGAIN AND SALE DEED

This Deed is made this 12th day of May, 2025, between Morehead Property One, LLC, a North Carolina limited liability company having an address of 1355 Greenwood Cliff Suite 150, Charlotte, NC 28204 ("Grantor") for the consideration of ten dollars, ($10.00), in hand paid, hereby sells and conveys to 550 Estates, LLC, a Colorado limited liability company having a mailing address of PO Box 34781, Charlotte, NC 28234 ("Grantee"), the real property together with improvements, if any, situate and lying and being in the County of San Juan, State of Colorado described as follows:

See Exhibit A attached hereto and incorporated herein.

This deed corrects the name of the county identified in that certain Bargain and Sale Deed recorded on March 20, 2025 at Reception No. 155840 in the office of the clerk and recorder of San Juan County, Colorado.

With all appurtenances hereunto belonging.

IN WITNESS WHEREOF, the Grantor has executed this deed on the date set forth above.

GRANTOR:

Morehead Property One, LLC,
a North Carolina limited liability company

By: Charles Lindsey McAlpine, Manager

STATE OF COLORADO)
)ss.
COUNTY OF LA PLATA)

The foregoing instrument was acknowledged before me this 12th day of May 2025, by Charles Lindsey McAlpine, Manager of Morehead Property One, LLC, a North Carolina limited liability company.

Notary Public
My Commission expires: December 27, 2027

SARAH R VOGEL
NOTARY PUBLIC
STATE OF COLORADO
NOTARY ID 20074046267
MY COMMISSION EXPIRES 12/27/2027

Exhibit A

TRACT E

A parcel of land being a portion of Parcel IV, a 17.879-acre tract as shown on the Cascade Village Results of Survey plat, deposited in the Office of the San Jaun County Clerk and Recorder under Reception No. 141, San Juan Colorado, Colorado, also commonly known as Tract E of the Cascade Village Amended Master Plan recorded under Reception No. 137955, and being more particularly described as follows:

Beginning at a point from which the CS1/16 Corner of Section 12, Township 39 North, Range 9 West, N.M.P.M., bears N 00°05'44" E, a distance of 926.23 feet;

Thence S 00°05'44" W, a distance of 410.61 feet to the computed position of the S1/4 Corner of Section 12, Township 39 North, Range 9 West, N.M.P.M.;

Thence S 89°54'04" W, along the south line of the SE1/4SW1/4 of said Section 12, a distance of 399.24 feet to the easterly right-of-way line of State Highway 550;

Thence continuing along said easterly right-of-way line of State Highway 550, along a non-tangent curve to the right with a delta angle of 6°39'08" and a radius of 2763.38 feet, a distance of 320.83 feet, the long chord bears N 10°14'42" W, a distance of 320.65 feet;

Thence continuing along said easterly right-of-way line of State Highway 550, N 20°32'19" W, a distance of 103.18 feet;

Thence N 89°54'04" E, departing said easterly right of way line of State Highway 550, a distance of 399.24 feet to the point of beginning;

Contains 4.630 acres, more or less.

155922
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SAN JUAN COUNTY, COLORADO
LADONNA L. JARAMILLO, RECORDER
05-15-2025 02:31 PM Recording Fee $18.00
No Doc Fee

WHEN RECORDED RETURN TO:
Amy Rhyne
PO Box 34781
Charlotte, NC 28234

CORRECTION BARGAIN AND SALE DEED

This Deed is made this 12th day of May, 2025, between Morehead Property One, LLC, a North Carolina limited liability company having an address of 1355 Greenwood Cliff Suite 150, Charlotte, NC 28204 ("Grantor") for the consideration of ten dollars, ($10.00), in hand paid, hereby sells and conveys to East Cascade Commercial, LLC, a Colorado limited liability company having a mailing address of PO Box 34781, Charlotte, NC 28234 ("Grantee"), the real property together with improvements, if any, situate and lying and being in the County of San Juan, State of Colorado described as follows:

See Exhibit A attached hereto and incorporated herein.

This deed corrects the name of the county identified in that certain Bargain and Sale Deed recorded on March 20, 2025 at Reception No. 155838 in the office of the clerk and recorder of San Juan County, Colorado.

With all appurtenances hereunto belonging.

IN WITNESS WHEREOF, the Grantor has executed this deed on the date set forth above.

GRANTOR:

Morehead Property One, LLC,
a North Carolina limited liability company

By: Charles Lindsey McAlpine, Manager

STATE OF COLORADO)
)ss.
COUNTY OF LA PLATA)

The foregoing instrument was acknowledged before me this 12th day of May 2025, by Charles Lindsey McAlpine, Manager of Morehead Property One, LLC, a North Carolina limited liability company.

Notary Public
My Commission expires: December 27, 2027

SARAH R VOGEL
NOTARY PUBLIC
STATE OF COLORADO
NOTARY ID 20074046267
MY COMMISSION EXPIRES 12/27/2027

Exhibit A

TRACT C

A parcel of land being a portion of Parcel IV, a 17.879-acre tract as shown on the Cascade Village Results of Survey plat, deposited in the Office of the San Juan County Clerk and Recorder under Reception No. 141, San Juan County, Colorado, also commonly known as Tract C of the Cascade Village Amended Master Plan recorded under Reception No. 137955, and being more particularly described as follows:

Beginning at the computed position of the S1/4 Corner of Section 12, Township 39 North, Range 9 West, N.M.P.M., from which the 128.04 foot Witness Corner to the said S1/4 Corner of Section 12 bears S 89°27'20" W, a distance of 128.04 feet;

Thence S 00°19'52" E, along the east line of the NE1/4NW1/4 of Section 13, Township 39 North, Range 9 West, N.M.P.M., a distance of 1033.25 feet to the easterly right-of-way line of State Highway 550;

Thence N 19°07'44" W, along said easterly right-of-way line of State Highway 550, a distance of 811.80 feet;

Thence N 02°37'43" W, along said easterly right-of-way line of State Highway 550, a distance of 116.75 feet;

Thence along said easterly right-of-way line of State Highway 550, along a non-tangent curve to the right with a delta angle of 3°10'00" and a radius of 2763.38 feet, a distance of 152.72 feet, the long chord bears N 15°09'15" W, a distance of 152.70 feet;

Thence N 89°34'46" E, departing said easterly right-of-way line of State Highway 550, a distance of 305.33 feet to the point of beginning.

Contains 3.480 acres, more or less.