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GEOLOGIC HAZARDS EVALUATION AND GEOTECHNICAL INVESTIGATION THE LAUNCHPAD – YOUTH HOUSING 810 NORTH 19th STREET COLORADO SPRINGS, COLORADO

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SCOPE

This report presents the results of our Geologic Hazards Evaluation and Geotechnical Investigation for an approximate 1.4-acre site located at 810 North 19th Street in Colorado Springs, Colorado. The investigated parcel is planned for development of a youth housing facility. Our purpose was to evaluate the property for the occurrence of geologic hazards and assess their potential effect on the planned development. This report includes descriptions of our interpretation of site geology, our engineering analysis of the potential impact of geologic conditions on development, a summary of subsurface and groundwater conditions found in our exploratory borings, a description of our engineering analysis of the geologic conditions at the site, and geotechnical design and construction recommendations.

The report was prepared based on conditions interpreted from field reconnaissance of the site, conditions found in our exploratory borings, results of laboratory tests, engineering analysis, and our experience. The criteria presented are for the development as described. Revision in the scope of the project could influence our recommendations. If changes occur, we should be retained to review these plans and update geotechnical recommendations and our slope stability analyses included in this report, as necessary. Evaluation of the property for the possible presence of potentially hazardous materials (Environmental Site Assessment) was beyond the scope of this investigation. The following section summarizes the report. A more complete description of the conditions found, our interpretations, and our recommendations are included in the report.

SUMMARY

1. We did not identify geologic hazards we believe will preclude development of the site for its intended use. The most significant conditions identified during the study that will affect the proposed development include a potentially unstable slope at the west side of the site, existing fill and very highly expansive clay and claystone bedrock.
2. There is a very high risk that the expansive material will heave and damage slabs-on-grades, foundations, and exterior site improvements. Our

analysis indicates the proposed construction should maintain suitable factors of safety for the slope at the west side of the site. We believe the recommendations presented in this report will help to control risk of damage; they will not eliminate that risk.

3. Subsurface conditions encountered in our borings drilled at the site consisted of areas of surficial clay fill up to approximately 12 feet thick and areas of natural, very clayey sand and slightly sandy to sandy clay. The existing fill is of suspect quality and highly expansive in its current condition. The fill should be removed from the building area to expose the natural materials. The surficial soils were underlain by claystone bedrock at depths ranging between 6 and 19 feet. Samples of the clay fill and claystone bedrock exhibited moderate to very high measured swell values.

4. At the time of drilling, groundwater was encountered in one of the borings at a depth of 29 feet below the existing ground surface. When checked several days after the completion of drilling, groundwater was measured in one of the borings at a depth of 23 feet. Groundwater levels will fluctuate seasonally and rise in response to precipitation and landscape irrigation.

5. We understand a post-tensioned slab-on-grade foundation is the desired foundation type. The post-tensioned slab-on-grade should be underlain by at least 6 feet of moisture conditioned, sub-excavation fill measured from the lowest exterior footing elevation.

6. Based on subsurface conditions encountered in our borings and the results of laboratory testing, we believe a very high risk of differential movement and damage will exist for slabs-on-grade underlain by the onsite soils in their present condition. Sub-excavation and construction of a moisture conditioned fill prism beneath the structure will reduce the risk.

7. We believe the parking areas and drive aisles can be paved with 6.5 inches of asphalt or 4 inches of asphalt over 8 inches of aggregate base course. Further discussions of the pavements including pavement sub-excavation and subgrade preparation are included in the report.

8. Surface drainage should be designed and maintained to provide the rapid removal of runoff away from the building to reduce potential subsurface wetting. Water should not be allowed to pond adjacent to the structure or over flatwork and pavement areas. Conservative irrigation practices should be employed to avoid excessive subsurface wetting.

9. The design and construction criteria presented in this report were compiled with the expectation that all other recommendations presented related to surface and subsurface drainage, landscaping irrigation, backfill compaction, etc. will be incorporated into the project and that the property manager will maintain the structure, use prudent irrigation practices, and maintain surface drainage. It is critical that all recommendations in this report are followed.

SITE CONDITIONS

The investigated parcel is located immediately west of the intersection of North 19th Street and Dale Street in Colorado Springs, Colorado. The general vicinity of the property is shown in Fig. 1. The site consists of approximately 1.4 acres of vacant land. The site is bordered to the north, south, and west by existing single-family residences and to the east by North 19th Street.

The site is dominated by a steep slope within the western half that descends from the adjacent residential community to the west. Photographs of the slope are presented below. The high point of the slope is located in the northwest corner of the site at approximately 6174 feet above mean sea level (msl). The slope descends steeply to the east and southeast at gradients between approximately 1:1 (horizontal to vertical) and 3:1. The slope appears to have been partially excavated. Aerial photography and previous mapping confirm the slope formerly descended at a gentler gradient and extended further into the parcel. Below the slope, a generally level area is present before becoming gently sloping as the site approaches North 19th Street. Vegetation consists of scattered shrubs and grasses and a couple trees around the perimeter.

Photo 1 - View to the west toward steep slope

Photo 2 - View to the north toward steep slope

1966 Aerial Photo

The 1966 aerial photo shown above indicates a structure was present at the site. We believe it was likely a single-family residence. The structure is not shown on the September 1999 Google Earth® aerial image.

PROPOSED DEVELOPMENT

Based on the architectural concept plans prepared by Shop Works Architecture and meetings with the project team, the project will consist of a 4-story, youth housing building with a total of 50 dwelling units and an interior amenity and office area. The building will be constructed along the east side of the parcel adjacent to North 19th Street. We understand the planned ground level finish floor elevation will be approximately 6136 feet msl in the northern portion of the building and 6135 feet in the southern portion of the building. A site retaining wall is planned along the lower portion of the slope. Other exterior improvements will include a paved entrance driveway and parking area, exterior concrete flatwork, underground utilities, and a water quality pond.

SUBSURFACE INVESTIGATION

Subsurface conditions at the site were investigated by drilling three exploratory borings on April 14, 2022, to depths of 20 to 30 feet using 4-inch diameter, continuous-flight, solid-stem auger and a truck-mounted drilling rig. Two additional borings were drilled on July 30, 2022, to depths of 25 and 40 feet using a track-mounted drill rig. Following the change to the site plan, we returned to the site again on October 22, 2022, to advance two more borings. The approximate locations of our borings are shown in Fig. 1.

Samples of the soils were obtained at 5-foot intervals using a 2.5-inch diameter (O.D.) modified California barrel sampler driven by blows from a 140-pound hammer falling 30 inches. Our field representative was present to observe drilling operations, log the soils and bedrock encountered, and obtain samples for laboratory tests. Graphical logs of the conditions found in our exploratory borings, the results of field penetration resistance tests, and some laboratory data are shown in Appendix A. Swell-consolidation and gradation test results are presented in Appendix B. Laboratory test data are summarized in Table B-1. Results of direct shear testing are presented in Appendix C.

Soil samples obtained during this study were returned to our laboratory and visually classified. Laboratory testing was then assigned to representative samples. Testing included moisture content and dry density, swell-consolidation, direct shear, gradation analysis, Atterberg limits, and water-soluble sulfate content tests. The swell test samples were wetted under applied pressures that approximated the overburden pressure (the weight of overlying soil).

SUBSURFACE CONDITIONS

The soils encountered in the seven exploratory borings drilled at the site consisted of areas of surficial, undocumented fill material and natural, very clayey sand and slightly sandy to very sandy clay. Claystone bedrock was found in each of the seven borings. Some of the pertinent engineering characteristics of the soils encountered and groundwater conditions are discussed in the following paragraphs.

Undocumented Fill

Existing fill consisting of sandy to very sandy, gravelly clay was encountered in five of our borings extending from the surface to depths of about 3 to 12 feet. Based on field penetration resistance testing the fill was very stiff. Three samples of the fill contained 64 to 75 percent silt and clay-sized particles (passing the No. 200 sieve). Two samples exhibited measured swell of 2.7 and 15.4 percent swell when wetted under estimated overburden pressures.

Natural Soils

Natural soils were encountered beneath existing fill material in TH-3 and at the surface in TH-4 and TH-5 and extended to depths of 7.5 to 19 feet. The natural soils consisted of very clayey sand and slightly sandy to very sandy clay. The sand was medium dense to dense and the clay was very stiff based on field penetration resistance testing. Three samples of the sand contained 36 to 46 percent silt and clay-sized particles. Two samples of the clay contained 65 and 90 percent silt and clay-sized particles. Our experience indicates the very clayey sand exhibits low to moderate expansion potential and the slightly sandy to very sandy clay exhibits moderate to very high expansion potential.

Bedrock

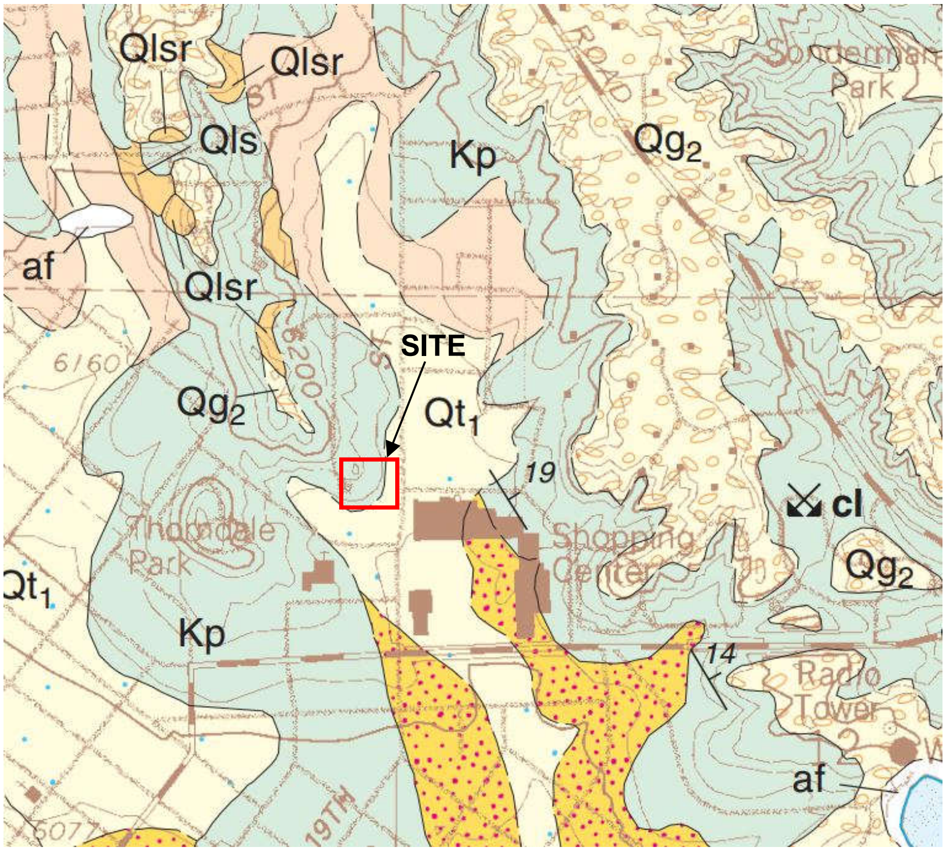
Claystone bedrock was encountered in each of our borings underlying fill and/or natural soils at depths ranging from 6 to 19 feet. The claystone was hard to very hard based on results of field penetration resistance testing. Four samples of the claystone exhibited 4.9 to 9.5 percent measured swell when wetted under estimated overburden pressures. Three samples of the claystone tested in our laboratory contained 99 percent silt and clay sized particles. A sample of the claystone was subjected to direct shear testing and results indicated a peak friction angle of 21 degrees and cohesion of 1120 psf.

Groundwater

At the time of drilling, groundwater was found in TH-4 at a depth of 29 feet. When checked several days after drilling groundwater was measured in TH-5 at a depth of 23 feet. Groundwater levels will fluctuate seasonally in response to seasonal precipitation and irrigation of landscaping. A seasonal rise of 3 to 5 feet is considered typical for this area.

SITE GEOLOGY

The geology of the site was evaluated by reviewing geologic maps, aerial photographs, conditions found in our borings, and observing field conditions. The native conditions have been disturbed by excavation into the slope and placement of fill material. The site has been mapped by Carroll and Crawford (2000) as part of the Colorado Springs Quadrangle for the Colorado Geological Survey. The mapping indicates the majority of the site is underlain by Pierre Shale (map unit “Kp”). The Pierre Shale consists of gray to dark-gray shale that weathers to brown and olive-green gray. The formation has a high potential for shrink-swell and heaving bedrock problems due to the presence of smectite claystone and bentonite beds.



Excerpt from Geologic Map

The eastern portion of the site adjacent to North 19th Street is mapped as Qt_1, denoted as Terrace alluvium one. This map unit is comprised of Holocene-age, poorly to moderately sorted, unconsolidated, matrix-supported cobble gravel in a sandy, silty, or clayey matrix.

POTENTIAL GEOLOGIC HAZARDS AND ENGINEERING CONSTRAINTS

We did not identify geologic hazards we believe preclude development of the site for its intended use. Conditions we identified at the site that pose constraints to development include the occurrence of expansive soils and a potentially unstable slope. Regional geologic conditions that may affect the site include seismicity and radioactivity (radon). We believe these conditions can be mitigated with engineering design and construction methods commonly employed in this area. The engineering conditions map presented in Fig. 2 uses a modified version of the system developed by Robinson (1977). These conditions are discussed in greater detail in the following sections.

Potentially Unstable Slopes

The Landslide Susceptibility Map of Colorado Springs (by White & Wait) indicates the western half of the site lies within a landslide susceptible zone. The Colorado Geological Survey notes that for locations that lie within the susceptible area, the designation does not imply that landslides will occur during the life of the proposed structure, only that a higher risk exists compared to areas not mapped as susceptible.

The site lies outside of the Hillside Overlay Zone of Colorado Springs as shown on zoning layers on the Cityview webpage as prepared by the GIS Division of the Planning, Development, and Finance Department; however, a steep slope is present in the western portion of the site. The slope descends steeply to the east and southeast at gradients between approximately 1:1 (horizontal to vertical) and 3:1. The slope appears to have been partially excavated.

Our analysis (Section A-A') indicates the current condition of the slope is stable from the standpoint of global stability; however, the slope is susceptible to erosion and surficial sloughing that may cause the slope to become unstable. Our analysis (Section B-B') indicates the proposed grading and retaining walls will result in an acceptable factor of safety. Further discussion of our analysis is included in the slope stability section of our report.

Expansive Soil and Bedrock

The existing fill material and natural soils are judged to have a low to very high potential for swell. Samples of these surficial materials exhibited 2.7 to 15.4 percent measure swell when wetted under estimated overburden pressure. Claystone bedrock lies beneath the natural soils. Test results and our experience indicates the claystone exhibits moderate to very high expansion potential.

Shallow Groundwater

Groundwater was found in two of our borings at depths of 23 and 29 feet. Our experience indicates the groundwater is likely flowing through fractures in the bedrock.

We understand no below-grade levels are currently planned. If plans change, we should be contacted to provide recommendations for subsurface drainage.

Steeply Dipping Bedrock

The site is not located within the Steeply Dipping Bedrock Overlay District outlined by the Colorado Geological Survey in their Map Series 32.

Mine Subsidence

The site is not included in the Colorado Springs Subsidence Investigation, State of Colorado, Division of Mined Land Reclamation, prepared by Dames & Moore, dated April 1985. The study was conducted specifically for known or suspected areas of underground coal mining in Colorado Springs. We did not observe evidence of subsurface mining at the site.

Hard Bedrock

The claystone within the Pierre Shale Formation is medium hard to very hard and may require aggressive excavation techniques including rock teeth and rock buckets.

Seismicity

This area, like most of central Colorado, is subject to a degree of seismic activity. Geologic evidence indicates that movement along some Front Range faults has occurred during the last two million years (Quaternary). This includes the Rampart Range and Ute Pass Faults, which are located about 1 and 3 miles west of the site, respectively. We believe the soils on the property classify as Site Class C (soft rock profile) according to the 2015 International Building Code (2015 IBC).

Radioactivity / Radon

We believe no unusual hazard exists from naturally occurring sources of radioactivity on this site. However, the materials found in our borings are often associated with the production of radon gas and concentrations in excess of those currently accepted by the EPA can occur. Passive and active mitigation procedures are commonly employed in this region to effectively reduce the buildup of radon gas. Measures that can be taken after a structure is enclosed during construction include installing a blower connected to the foundation drain and sealing the joints and cracks in concrete floors and foundation walls. If the occurrence of radon is a concern, we recommend the structure be tested after it is enclosed and commonly utilized techniques be employed to minimize the risk.

Flooding and Streamside Overlay Zone

The site is outside of the areas mapped as the Streamside Overlay as shown on zoning layers on the Cityview webpage as prepared by the GIS Division of the Planning, Development, and Finance Department. The site is outside of the 500-year flood plain, as shown on Flood Insurance Rate Map Number 08041C0513G with an effective date of December 7, 2018.

SLOPE STABILITY ANALYSIS

We conducted slope stability analyses for the steeper portion of the slope located at the west side of the site. Our analyses are based on the grading plan prepared by Ware Malcomb, dated December 2, 2022. Cross-section (A-A') considers what we interpret as the critical cross section extending down the steep portion of the slope. We evaluated the existing conditions, as well as an excavation condition for the proposed retaining wall. We also conducted slope stability analyses extending through the proposed retaining wall and building (B-B'). The approximate locations of the cross-sections are presented on Fig. 1. Our analysis also considered the temporary excavations into the slope and in the building pad area that will be necessary during construction.

The on-site materials were assigned unit weights and conservative shear strength parameters based on the results of our laboratory testing and our experience with similar materials in the site vicinity. The relevant shear strength parameters used in our slope stability analysis for different material types are presented in the analyses in Appendix D.

Groundwater was encountered at depths of 23 and 29 feet. Groundwater was modeled in the slope at a higher elevation (depth of around 10 feet) to evaluate impacts of a hypothetical and conservative increased groundwater level of the post-construction condition due to heavy precipitation. Factors of safety of 1.15 and 1.5 are usually considered appropriate by Geotechnical Engineers for temporary and permanent slopes, respectively. The results of our slope stability analysis indicate a factor of safety of about 1.5 for the existing slope in its current condition and 1.4 for the temporary cut condition at section A-A’. A factor of safety of 2.1 for a temporary construction condition and 1.5 for the post-construction condition were calculated for section B-B’. The results of our slope stability analyses are presented in Appendix D. This slope stability evaluation is limited to the specific portion of the site. Based on the results of our analysis and our site reconnaissance, we believe the slope should exhibit an acceptable factor of safety during and after construction. Global stability should be evaluated for critical wall sections during design.

SITE DEVELOPMENT CONSIDERATIONS

The project is technically feasible from a geotechnical standpoint; however, design and construction will be significantly complicated due to the presence of a steep slope adjacent to the outdoor amenity area, highly expansive soils and bedrock, and undocumented fill material.

Existing Fill

Existing fill was identified throughout the proposed building area and extended to depths of up to 12 feet. The fill is undocumented and highly expansive. The fill is not suitable for construction of slab-on-grade floors or shallow foundations in its current condition. The existing fill should be over-excavated to natural materials.

Sub-Excavation

The site is underlain by up to 12 feet of undocumented fill, consisting of very highly expansive, sandy to very sandy clay with occasional gravel as wells as expansive claystone. These materials are well below optimum moisture content. We calculated potential ground heave of up to about 11 inches with normal post-construction wetting.

We believe sub-excavation below the building and placement of the excavated soils as new, moisture conditioned, compacted fill should be performed to reduce settlement and/or heave and enhance performance of foundations, slabs-on-grade, and flatwork. Sub-excavation to a depth of at least 6 feet below the lowest exterior footing elevation will allow the use of a post-tension, slab-on-grade (PTS) foundation with reduced potential for differential movement. Removal of the undocumented fill below the minimum sub-excavation depth will result in greater depths of moisture conditioned fill beneath portions of the building, as indicated in TH-7, where the undocumented fill extended to a depth of about 12 feet.

Excavation and fill placement should be completed per the Excavation and Fill Placement sections of this report. The extent and depth of sub-excavation should be mapped by a surveyor and an “as-built” plan of the sub-excavated area is recommended. The sub-excavation area should extend at least 6 feet outside the building limits.

The earth work contractor should be chosen based on experience with moisture conditioning, processing, and compacting clay fill and have the necessary compaction equipment. Special attention should be paid to compaction in the corners and along the edges of the excavation, as large equipment cannot easily access these areas. In order for the procedure to perform properly, close control of fill placement is required. Sub-excavation fill should be moisture conditioned and compacted to the specifications contained in the Fill Placement section. Our representative should observe the bottom of the excavation and observe and test compaction of fill during placement.

Once fill is placed, it is important that measures be planned to reduce drying of near-surface materials. If fill dries excessively prior to building construction, it may be necessary to rework the upper, drier materials just prior to construction of foundations.

Excavation

We believe the soils encountered in the exploratory borings can be excavated with conventional, heavy-duty excavation equipment. Zones of hard bedrock may require more aggressive excavation techniques. We recommend the contractor become familiar with applicable local, state, and federal safety regulations, including the current Occupational Safety and Health Administration (OSHA) Excavation and Trench Safety Standards, to determine appropriate excavation slopes. We anticipate the near-surface clays and clayey sands will classify as Type B materials. Temporary excavations in Type B materials require a maximum slope inclination of 1:1 (horizontal to vertical), unless the excavation is shored or braced. The claystone bedrock will classify as Type A materials. Temporary excavations in Type A materials require a maximum slope inclination of 0.75:1, unless the excavation is shored or brace. If groundwater seepage occurs, flatter slopes will likely be required. The contractor's "competent person" should review excavation conditions and refer to OSHA standards when worker exposure is anticipated. Stockpiles and equipment should not be placed within a horizontal distance equal to one-half the excavation depth, from the edge of the excavation. Excavations deeper than 20 feet should be designed by a registered professional engineer.

Fill Placement

The on-site, natural soils and excavated bedrock are suitable for use during site grading. The existing fill material may be suitable provided it is free of debris and deleterious material. Prior to placement of new fill, topsoil, vegetation, and organic materials should be removed from the ground surface. The ground surface in areas to receive fill should be scarified, moisture conditioned to near optimum moisture content, and compacted to a high density to establish a stable subgrade for fill placement.

Imported fill, if needed, should ideally consist of soil having a maximum particle size of 2 inches and 30 to 50 percent passing the No. 200 sieve. The import material should exhibit a Liquid Limit of less than 30 and a Plasticity Index of less than 15. A sample of any potential imported fill material should be submitted to our office for testing, prior to its use at the site.

The properties of the fill will affect the performance of surficial improvements such as pavements and concrete flatwork. As stated in the Sub-Excavation section, the existing undocumented fill soils are below optimum moisture content. Significant mixing, addition of water, and mellowing (allowing water to be absorbed) will be required to achieve uniform moisture contents above optimum. We recommend water be added to soils during stockpiling to help incorporate the water into the soil matrix. This process is complicated by freezing temperatures in winter months.

We recommend site grading fill be placed in thin, loose lifts, moisture conditioned to between 1 and 4 percent above optimum moisture content and compacted to at least 95 percent of maximum standard Proctor dry density (ASTM D 698). We recommend the moisture content be reduced to within 2 percent of optimum in the upper two feet of pavement areas, to reduce problems associated with unstable subgrade materials. Fill should not be placed on top of frozen soils. The frozen soils should be removed or allowed to thaw prior to the placement of fill. Placement and compaction of the grading fill should be observed and tested by our representative during construction.

During construction, the site should be graded such that surface water can drain readily away from the building area. Water that accumulates in excavations should be promptly pumped out or otherwise removed before resuming construction. Berms, ditches, and similar means should be used to decrease stormwater entering the work area and to efficiently convey it off site. Failure to control surface drainage during construction could result in construction delays, require reworking of materials, and/or cause slope instability of excavations.

Buried Utilities

Water and sewer lines are often constructed beneath slabs and pavements. Compaction of utility trench backfill can have a significant effect on the life and serviceability of floor slabs, pavements, and exterior flatwork. We recommend utility trench backfill be placed in compliance with City of Colorado Springs specifications. Our experience indicates the use of a self-propelled compactor results in more reliable performance compared to trench backfill compacted by a sheepsfoot wheel attachment on a backhoe or trackhoe. The upper portion of the trenches should be widened to allow the use of a self-propelled compactor. Personnel from our firm should periodically observe utility trench backfill placement and test the density of the backfill materials during construction.

FOUNDATIONS

Our investigation indicates existing fill material, natural sand and clay soils, and claystone bedrock are present at depths that will impact construction of shallow foundations. We understand a post-tensioned, slab-on-grade (PTS) foundation is the preferred foundation system. PTS foundations may be utilized for support of the proposed structure, provided the remedial grading and sub-excavation recommendations set forth previously are performed. Design and construction criteria are presented below.

PTS foundation design is based on a method developed by the Post-Tensioning Institute (PTI, 3rd Edition, 2004 with 2008 Supplement). Various climate and relevant soil factors are required to evaluate the PTI design criteria. The PTI slab design includes evaluation of two mechanisms of soil movement (edge lift and center lift) based on assumptions that the wetting and drying of the foundation soils are primarily affected by the climate. These values were calculated using software titled VOLFLO 1.5 and the parameters presented below.

PTI Parameters	
Parameter	**Design Value**
Thornthwaite Moisture Index	-20
Constant Soil Suction	3.6 pF
Depth of Seasonal Moisture Variation	15 feet
Percent Finer than 2 Microns	40 for fill and 60 for claystone
Soil Fabric Factor	1.0

Our experience indicates that the foundation soils will normally undergo a significant increase in moisture content due to covering of the ground surface by the buildings and pavements, and irrigation around the structures. Depending on the surface drainage or the amount of available water, the movement mechanism, which controls the design, could be as high as total heave. The edge moisture variation distance can also be more than the design values provided in the PTI manual. Considering the limitations of the current PTI design, we believe a conservative approach with reasonable engineering judgment is necessary to prepare geotechnical recommendations for PTS foundation design.

The PTI design method estimates movements for a depth of wetting of 9 feet below the ground surface. Based on our experience in the area, field data, and experience with the proposed development the depth of wetting will likely be 12 to 15 feet below the ground surface. It is possible wetting will not penetrate this deep; however, we believe it is a reasonable design assumption. The PTI design procedure does not predict soil movements that result from site conditions such as irrigation or poor surface drainage that may lead to deeper wetting. If deeper wetting of the foundation soils occurs, the foundation movement may exceed the design movements predicted in the PTI procedure. If surface drainage is properly designed and maintained, it is unlikely the total calculated heave would occur and manifest at the surface. We expect total heave or settlement will be on the order of 1 to 2 inches. PTS foundation design is based on the potential differential movement of the slabs due to both settlement and heave of the subsoils. The estimated differential soil movement (y_m) were evaluated for two cases: post-equilibrium and post-construction. In our opinion, the post-construction case is considered more appropriate due to the magnitude of potential movement and our experience with the local climate.

Design criteria for PTS foundations developed from analysis of field and laboratory data and our experience are presented below.

<u>Post-Tensioned, Slabs-On-Grade (PTS)</u>

1. PTS foundations should be constructed on newly placed, sub-excavation fill that is moisture conditioned and compacted according to the recommendations provided.
2. The foundations should be designed for a maximum allowable soil pressure of 2,000 psf.

3. For the PTI design method, we recommend a differential movement (y_m) of 2.0 inches for the edge lift condition and -1.3 inches for the center lift condition.
4. Based on the Thornthwaite Moisture Index (PTI Manual, Edition 3), an edge moisture variation distance (e_m) of 4.1 feet for the edge lift condition and 8.0 feet for the center lift condition should be used in design.
5. We understand the PTI design method assumes the slab is somewhat flexible. Some of the above-grade construction may not be flexible, such as drywall, brick, and stucco. We are aware of situations where minor differential slab movement has caused distress in finish materials. One way to enhance performance would be to place reinforcing steel in the bottoms of the stiffening beams. The structural engineer should evaluate the merits of this approach and other potential alternatives.
6. Stiffening beams may be poured “neat” into trenches excavated in the building pads. Soils may cave or slough during trench excavation for the stiffening beams. Disturbed soils should be removed from trench bottoms prior to placement of concrete. Formwork or other methods may be required for proper stiffening beam installation.
7. Exterior stiffening beams must be protected from frost action. Normally, 30 inches of frost cover is provided in the Colorado Springs area.
8. For slab tensioning design, a coefficient of friction value of 0.75 or 1.0 can be assumed for slabs on polyethylene sheeting or a sand layer, respectively. A coefficient of friction of 2 should be used for slabs on clay or clay fill.
9. A representative of our firm should observe the completed excavations. A representative of the structural engineer should observe the placement of the reinforcing tendons and reinforcement prior to placing the slabs and beams.

FLOOR SYSTEMS

Our investigation indicates the materials near the anticipated first floor levels of the proposed apartment buildings will consist predominantly of natural sandy clay and silty to clayey sand, new, grading fill and sandstone. For the PTS system, the foundations are structurally integrated with the floor slab and should exhibit more reliable long-term performance provided the remedial grading and sub-excavation recommendations set forth previously are performed. Underslab utilities such as water and sewer lines should be pressure tested prior to installing slabs. Utilities that penetrate slabs should be provided with sleeves and flexible connections that allow for independent movement of the slab and reduce likelihood of damaging buried pipes. We recommend these details allow at least 2 inches of differential movement between the slabs and pipes.

SITE RETAINING WALLS

Site retaining walls are currently planned at the slope in the western portion of the site and also along the south and east sides of the building. The walls vary in height with a maximum height of about 7 feet. Site retaining walls may be designed for a maximum allowable bearing pressure of 2,000 psf. For gravity block wall design or MSE wall design, we recommend an angle of friction of 24 degrees and a moist unit weight of 125 pcf be assigned to the retained soils.

For cast-in-place walls, design should consider lateral earth loads which are dependent on the height of the wall, soil type, and backfill configuration. Backfill behind site retaining walls is expected to be sloped in portions of the site. We expect site retaining walls will be subject to “active” earth pressures where walls are free to rotate and the soil moves toward the wall away from the soil mass. The active pressures are fully mobilized at horizontal movements of about 0.5 percent of the wall height for cohesionless soils, such as sands and gravels. Passive stresses exist when the wall moves toward the soil mass. Passive resistance requires relatively more movement than active, at-rest, or base friction to generate resistance. Passive pressures should only be used when movement is tolerable, and the soils is well compacted and will not be removed. For frictional resistance to lateral loads, we recommend that a coefficient of friction of 0.3 be used between soil and concrete contacts. The recommended equivalent fluid densities provided in the following table assume no surcharge loads next to the top of the wall and free-draining, granular backfill such as CDOT Class 1, with an angle of internal friction (ϕ) of 34 degrees, and a unit weight of 125 pounds per cubic foot (pcf).

Lateral Earth Pressure Values (Equivalent Fluid Density), pcf			
Retained Slope	**Active**	**At-Rest**	**Passive**
Level	35	55	400
1.5H:1V	77	-	250
2H:1V	52	-	250
3H:1V	44	-	250

Care must be exercised when compacting backfill against retaining walls. To reduce temporary construction loads on the walls, heavy equipment should not be used for placing and compacting fill within a region as determined by a 0.5H:1V line drawn upward from the bottom of the wall. Granular backfill behind any new site retaining walls should be compacted to 90 percent of the maximum modified Proctor dry density (ASTM D 1557) but should not be compacted to more than approximately 92 percent to minimize pressure against the walls. Thinner lifts should be used when utilizing smaller compaction equipment.

Adequate drainage is essential to the performance of retaining walls. New walls should include installation of a drainpipe that discharges away from the wall. For site retaining walls drainage measures could include free-draining granular backfill and perforated drainpipes leading to a positive gravity outlet and granular backfill.

PAVEMENTS

Our exploratory borings and understanding of the proposed construction suggest the subgrade soils within the planned access driveway and parking area will consist predominantly of sandy clay and very clayey sand. A sample of the sandy clay tested in our laboratory classified as A-7-6 materials according to the American Association of State Highway Transportation Officials (AASHTO) classification system. This type of material generally exhibits poor pavement support characteristics. Based on our laboratory classification testing (Atterberg Limits and gradation analysis), a Hveem Stabilometer ("R") value of 5 was assigned to the subgrade materials for design purposes.

We anticipate the access driveways could be subjected to occasional heavy vehicle loads such as trash trucks and moving vans. We considered a daily traffic number (DTN) of 5, which corresponds to an 18-kip Equivalent Single-Axle Loads (ESAL) of 36,500, for a 20-year pavement design life. Based on the estimated design traffic and pavement design input parameters, we recommend a pavement section of 6.5 inches of asphalt or 4 inches of asphalt over 8 inches of aggregate base. A plain portland cement concrete pavement section of 6 inches can also be considered.

It should be understood that the pavements may perform poorly due to the presence of moderately to very highly expansive soils. If the owner is unwilling to accept the high risk of damaging heave to pavements caused by swelling soils, the more reliable alternative to mitigate the risk is sub-excavation. We recommend at least 4 feet of the subgrade soils be sub-excavated and recompacted to at least 95 percent of maximum standard Proctor (ASTM D 698) at moisture contents between 0 and 4 percent above the optimum moisture content. Placement of the sub-excavation fills at high moisture contents results in soils that are more prone to pumping and rutting. We recommend the moisture content be reduced to within 2 percent of optimum in the upper two feet of pavement areas, to reduce problems associated with unstable subgrade materials. Due to high water-soluble sulfate contents, chemical stabilization with cement or lime are not recommended.

We recommend a concrete pad be provided at the trash dumpster site. The pad should be at least 8 inches thick and long enough to support the entire length of the trash truck and dumpster. Joints between concrete and asphalt pavements should be sealed with a flexible compound.

Our design considers pavement construction will be completed in accordance with the City of Colorado Springs “Standard Specifications” and the Pikes Peak Region Asphalt Paving Specifications. The specifications contain requirements for the pavement materials (asphalt, base course, and concrete) as well as the construction practices used (compaction, materials sampling, and proof-rolling). Of particular importance are those recommendations directed toward subgrade and base course compaction and proof-rolling. During proof-rolling, attention should be directed toward the areas of confined backfill compaction. Soft or loose subgrade or areas that pump excessively should be stabilized prior to pavement construction. A representative of our office should be present at the site during placement of fill and construction of pavements to perform density testing.

CONCRETE

Concrete in contact with soil can be subject to sulfate attack. We measured water-soluble sulfate concentration 0.4 percent in a sample from this site. As indicated in our tests and ACI 318-19, the sulfate exposure class is *Severe* or *S2.*

SULFATE EXPOSURE CLASSES PER ACI 318-19

Exposure Classes		Water-Soluble Sulfate (SO_4) in Soil [A] (%)
Not Applicable	**S0**	**< 0.10**
Moderate	**S1**	**0.10 to 0.20**
Severe	**S2**	**0.20 to 2.00**
Very Severe	**S3**	**> 2.00**

A) Percent sulfate by mass in soil determined by ASTM C1580

For this level of sulfate concentration, ACI 318-19 *Code Requirements* indicates there are special cement type requirements for sulfate resistance as indicated in the table below.

CONCRETE DESIGN REQUIREMENTS FOR SULFATE EXPOSURE PER ACI 318-19

<table>
<tr><th colspan="2" rowspan="2">Exposure Class</th><th rowspan="2">Maximum Water/ Cement Ratio</th><th rowspan="2">Minimum Compressive Strength (psi)</th><th colspan="3">Cementitious Material Types [A]</th><th rowspan="2">Calcium Chloride Admixtures</th></tr>
<tr><th>ASTM C150/ C150M</th><th>ASTM C595/ C595M</th><th>ASTM C1157/ C1157M</th></tr>
<tr><td colspan="2">S0</td><td>N/A</td><td>2500</td><td>No Type Restrictions</td><td>No Type Restrictions</td><td>No Type Restrictions</td><td>No Re-strictions</td></tr>
<tr><td colspan="2">S1</td><td>0.50</td><td>4000</td><td>II[B]</td><td>Type with (MS) Desig-nation</td><td>MS</td><td>No Re-strictions</td></tr>
<tr><td colspan="2">S2</td><td>0.45</td><td>4500</td><td>V [B]</td><td>Type with (HS) Desig-nation</td><td>HS</td><td>Not Permit-ted</td></tr>
<tr><td>S3</td><td>Option 1</td><td>0.45</td><td>4500</td><td>V + Pozzo-lan or Slag Cement [C]</td><td>Type with (HS) Desig-nation plus Pozzolan or Slag Ce-ment [C]</td><td>HS + Pozzo-lan or Slag Cement [C]</td><td>Not Permit-ted</td></tr>
<tr><td>S3</td><td>Option 2</td><td>0.4</td><td>5000</td><td>V [D]</td><td>Type with (HS) Desig-nation</td><td>HS</td><td>Not Permit-ted</td></tr>
</table>

A) Alternate combinations of cementitious materials shall be permitted when tested for sulfate resistance meeting the criteria in section 26.4.2.2(c).
B) Other available types of cement such as Type III or Type I are permitted in Exposure Classes S1 or S2 if the C3A contents are less than 8 or 5 percent, respectively.
C) The amount of the specific source of pozzolan or slag to be used shall not be less than the amount that has been determined by service record to improve sulfate resistance when used in concrete containing Type V cement. Alternatively, the amount of the specific source of the pozzolan or slab to be used shall not be less than the amount tested in accordance with ASTM C1012 and meeting the criteria in section 26.4.2.2(c) of ACI 318.
D) If Type V cement is used as the sole cementitious material, the optional sulfate resistance requirement of 0.040 percent maximum expansion in ASTM C150 shall be specified.

Superficial damage may occur to the exposed surfaces of highly permeable concrete. To control this risk and to resist freeze-thaw deterioration, the water-to-cementitious materials ratio should not exceed 0.50 for concrete in contact with soils that are likely to stay moist due to surface drainage or high-water tables. Concrete should have a total air content of 6 percent ± 1.5 percent. We advocate damp-proofing of all foundation walls and grade beams in contact with the subsoils.

SURFACE DRAINAGE AND IRRIGATION

Proper surface drainage cannot be overemphasized for this project. The performance of structures, flatwork, and pavements within the development will be influenced by surface drainage. When developing an overall drainage scheme, consideration should be given to drainage around the structure and pavement area. Drainage should be planned such that surface runoff is directed away from foundations and is not allowed to pond adjacent to or between structures, over pavements, or at the crest of permanent slopes. Ideally, slopes of at least 6 inches in the first 10 feet should be planned for the landscaped areas surrounding buildings, where possible. Roof downspouts and other water collection systems should discharge onto paved surfaces or well beyond the limits of all backfill around the structures.

Proper control of surface runoff is also important to prevent the erosion of surface soils. Concentrated flows must not be directed over unprotected slopes. Permanent slopes, including the existing steep slope in the western portion of the site, should be seeded or mulched to reduce the potential for erosion. Backfill soils behind the curb and gutter adjacent to access roads and parking lots, and in utility trenches should be compacted. If surface drainage is neglected, performance of the pavements, flatwork, and foundations may be compromised.

Landscaping concepts should consider a xeriscape scheme and concentrate on plantings that require little or no supplemental irrigation after the vegetation is established. Irrigated sod, irrigation mainlines, above-surface spray heads, rotors, and other above-surface irrigation spray devices should not be located or discharge within 10 feet of the building. Irrigation should be set at the lowest level necessary for plant growth.

CONSTRUCTION OBSERVATIONS

We recommend that CTL|Thompson, Inc. provide observation and testing services during construction to allow us the opportunity to verify whether soil conditions are consistent with those found during our investigation. If others perform these observations, they must accept responsibility to judge whether the recommendations in this report remain appropriate.

GEOTECHNICAL RISK

The concept of risk is an important aspect with any geotechnical evaluation primarily because the methods used to develop geotechnical recommendations do not comprise an exact science. We never have complete knowledge of subsurface conditions. Our analysis must be tempered with engineering judgment and experience. Therefore, the recommendations presented in any geotechnical evaluation should not be considered risk-free. Our recommendations represent our judgment of those measures that are necessary to increase the chances that the structure will perform satisfactorily. It is critical that all recommendations in this report are followed during design and construction.

LIMITATIONS

This report has been prepared for the exclusive use of the Cohen Esrey and the project design team for the purpose of providing geotechnical design and construction criteria for the proposed building additions. The information, conclusions, and recommendations presented herein are based upon consideration of many factors, but not limited to, the type of structure proposed, the geologic setting, and the subsurface conditions encountered. The conclusions and recommendations contained in the report are not valid for use by others. Standards of practice continuously evolve in the area of geotechnical engineering. The recommendations are appropriate for about three years. If the project is not constructed within about three years, we should be contacted to determine if we should update this report.

Our borings were located to obtain a reasonably accurate indication of subsurface foundation conditions. The borings are representative of conditions encountered at the exact boring location only. Variations in subsurface conditions not indicated by the borings are possible. We recommend a representative of our office observe the completed foundation excavations. Representatives of our firm should be present during construction to perform construction observation and materials testing services.

We believe this investigation was conducted with that level of skill and care normally used by geotechnical engineers practicing under similar conditions. No warranty, express or implied, is made. If we can be of further service in discussing the contents of this report or in the analysis of the influence of the subsoil conditions on design of the structure from a geotechnical engineering point-of-view, please call.

CTL | THOMPSON, INC.

COLORADO LICENSED
JEFFREY MARK JONES
47539
12/20/22
PROFESSIONAL ENGINEER

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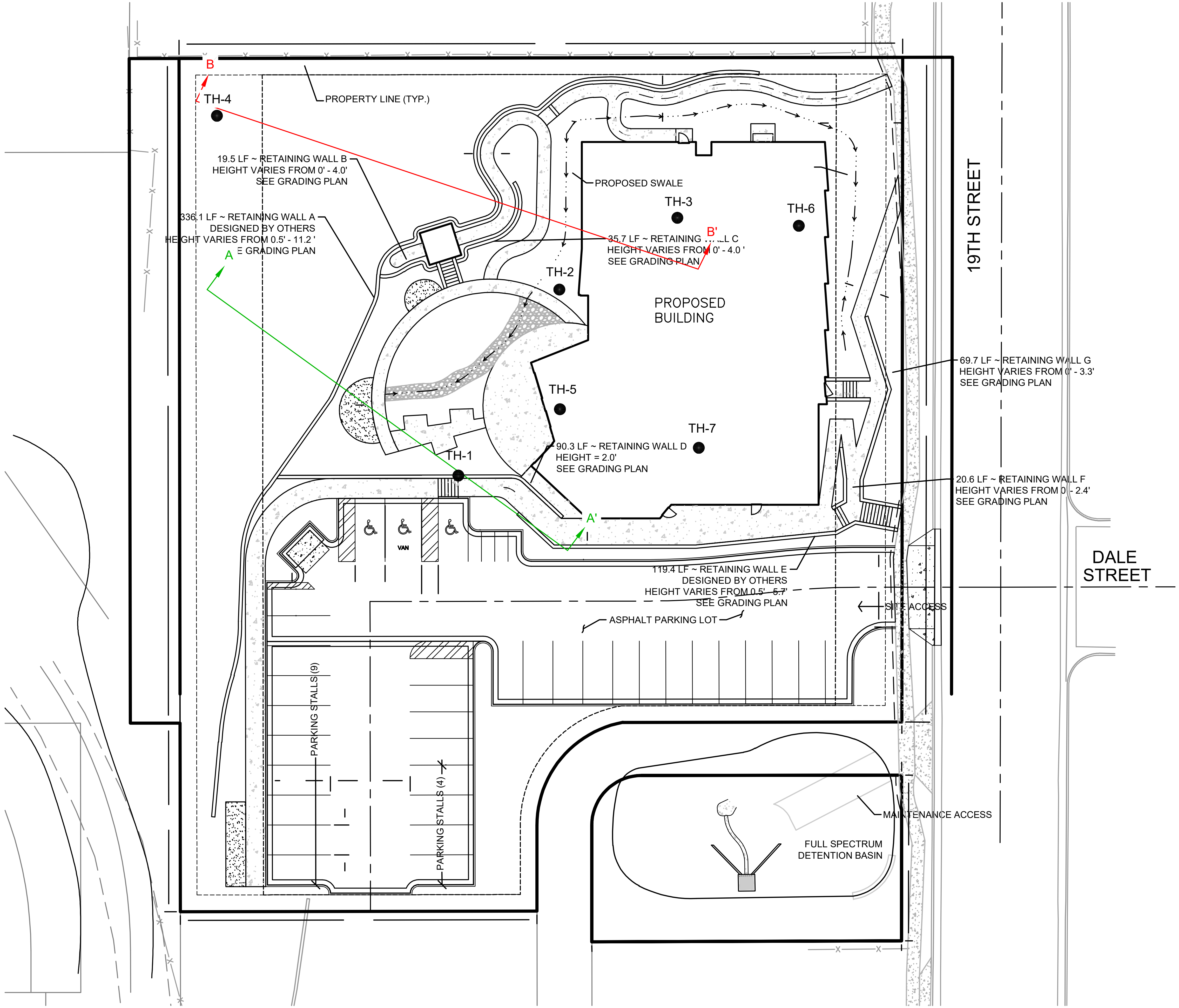

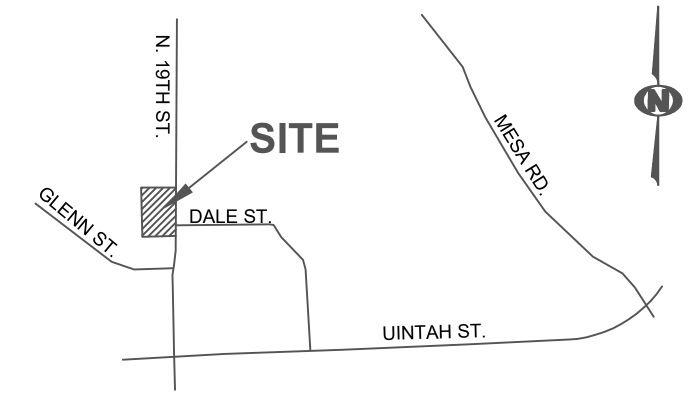

VICINITY MAP

(NOT TO SCALE)

LEGEND:

TH-1 ●	APPROXIMATE LOCATION OF EXPLORATORY BORING.
— — —	PROJECT BOUNDARY
A	SLOPE STABILITY CROSS-SECTION (EXISTING CONDITION)
B	SLOPE STABILITY CROSS-SECTION (PROPOSED CONSTRUCTION)

0 15' 30'

SCALE: 1" = 30'

NOTE:
BASE DRAWING WAS PROVIDED BY SHOP WORKS ARCHITECTURE (THE LAUNCHPAD - YOUTH HOUSING).

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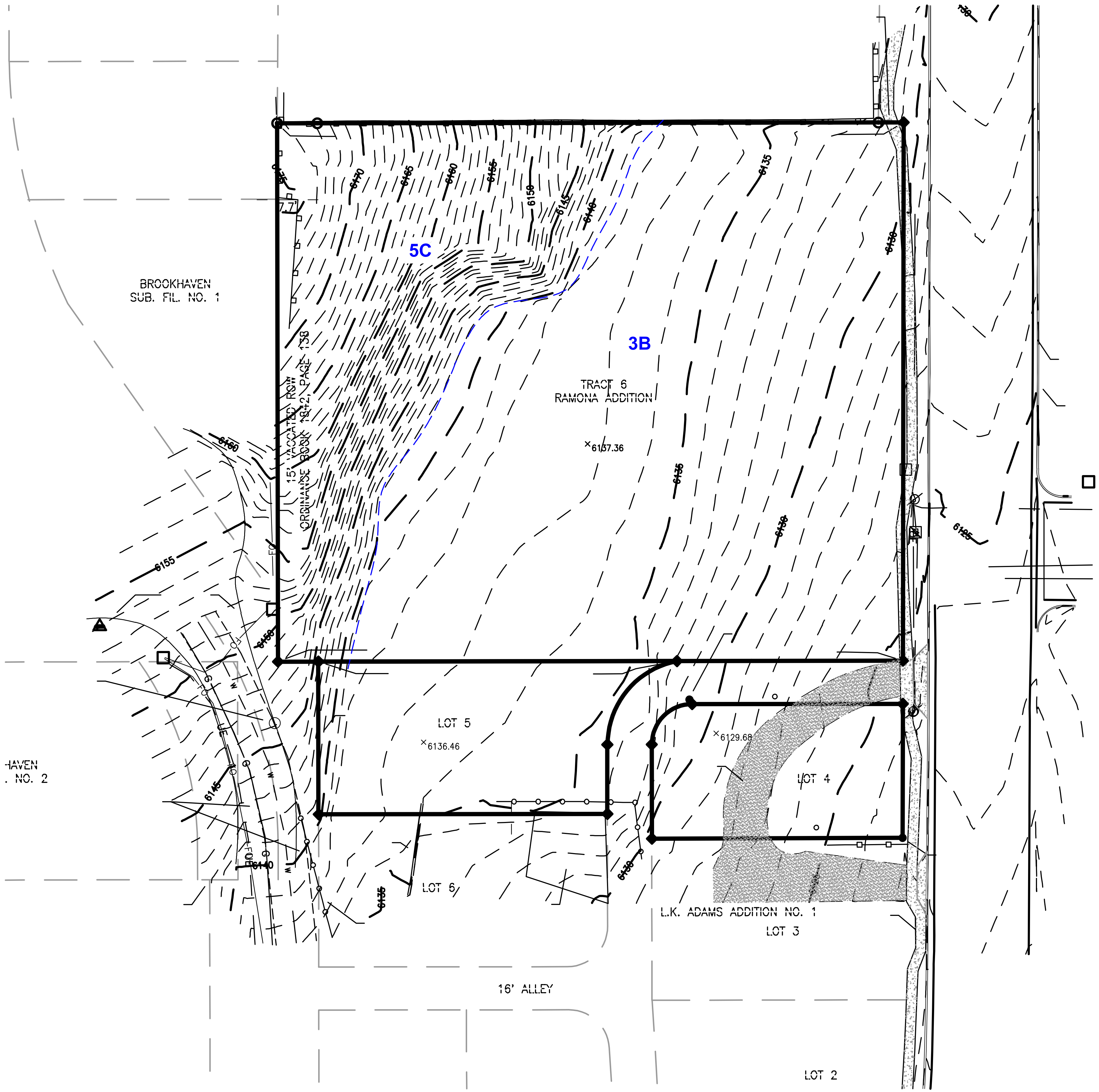
Location of Exploratory Borings

FIG. 1



LEGEND:

ENGINEERING GEOLOGIC CONTACTS

3B EXPANSIVE AND POTENTIALLY EXPANSIVE SOIL AND BEDROCK ON FLAT TO MODERATE SLOPES (0-12%). EMPHASIS ON POTENTIAL FOR SWELL, DEPTH OF BEDROCK, DESIGN OF FOUNDATION AND DRAINAGE.

5C UNSTABLE OR POTENTIALLY UNSTABLE SLOPES COLLUVIUM OR BEDROCK ON STEEP SLOPES. EMPHASIS ON SLOPE STABILITY, CONTROL OF CUTS AND SURFACE DRAINAGE.

0 20' 40'

SCALE: 1" = 40'

NOTE:
BASE DRAWING WAS PROVIDED BY SHOP WORKS ARCHITECTURE (THE LAUNCHPAD - YOUTH HOUSING).

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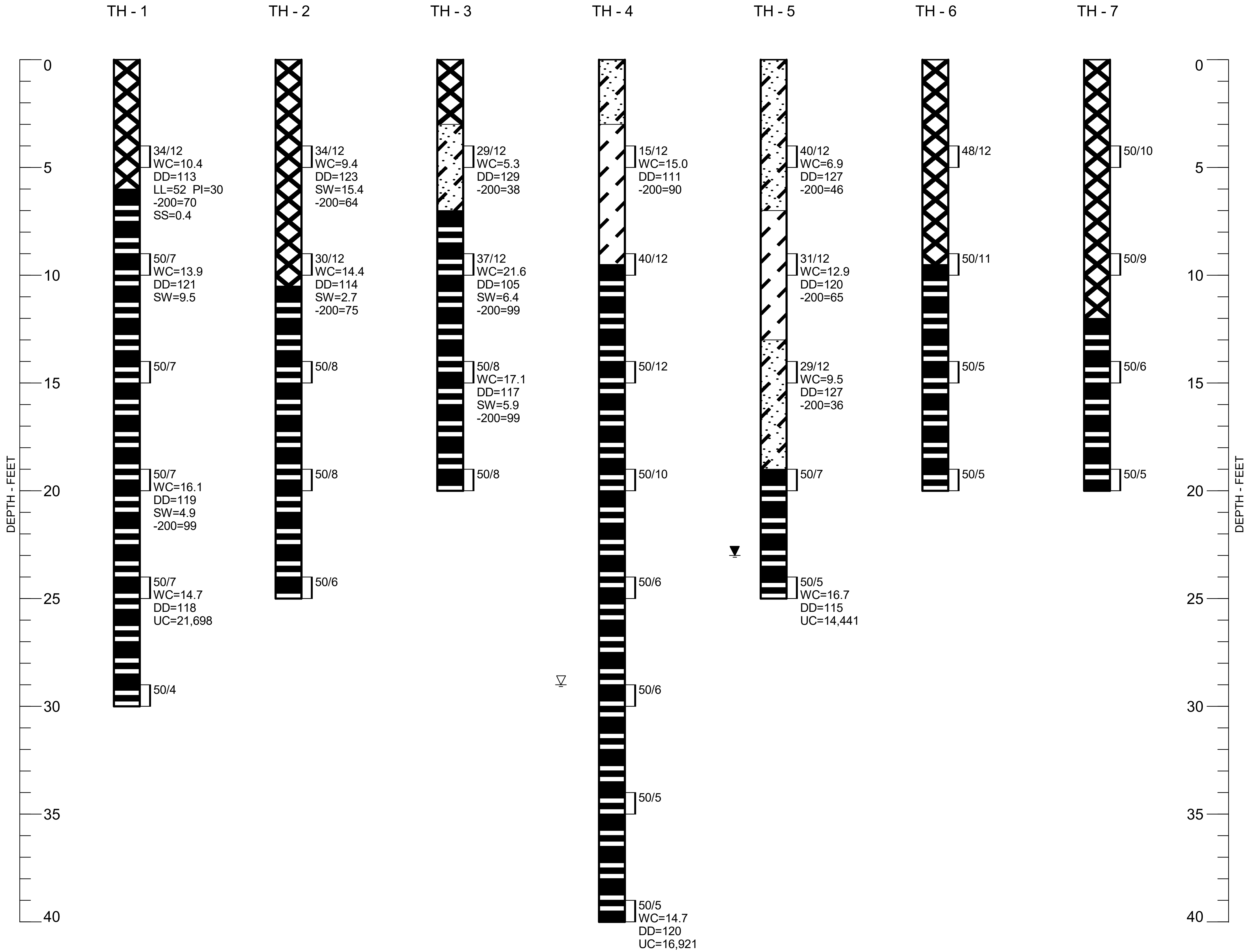
Engineering Geologic Conditions

FIG. 2

APPENDIX A

SUMMARY LOGS OF EXPLORATORY BORINGS



LEGEND:

FILL, CLAY, SANDY TO VERY SANDY, GRAVELLY, VERY STIFF, SLIGHTLY MOIST, LIGHT BROWN.

SAND, VERY CLAYEY, GRAVELLY, MEDIUM DENSE TO DENSE, SLIGHTLY MOIST, LIGHT TO DARK BROWN (SC).

CLAY, SLIGHTLY SANDY TO SANDY, VERY STIFF, MOIST, REDDISH BROWN TO LIGHT BROWN (CL).

BEDROCK, CLAYSTONE, SANDY, MEDIUM HARD TO VERY HARD, SLIGHTLY MOIST, MEDIUM BROWN.

DRIVE SAMPLE. THE SYMBOL 34/12 INDICATES 34 BLOWS OF A 140-POUND HAMMER FALLING 30 INCHES WERE REQUIRED TO DRIVE A 2.5-INCH O.D. SAMPLER 12 INCHES.

▽ GROUNDWATER LEVEL MEASURED AT TIME OF DRILLING.

▼ GROUNDWATER LEVEL MEASURED AFTER DRILLING ON AUGUST 11, 2022.

NOTES:

1. THE BORINGS WERE DRILLED APRIL 14, JULY 30, AND OCTOBER 21, 2022 USING A 4-INCH DIAMETER, CONTINUOUS-FLIGHT AUGER AND A CME-45, AND A DIEDRICH D-90, TRUCK-MOUNTED DRILL RIGS.
2. THESE LOGS ARE SUBJECT TO THE EXPLANATIONS, LIMITATIONS, AND CONCLUSIONS AS CONTAINED IN THIS REPORT.
3. WC - INDICATES MOISTURE CONTENT. (%)
   DD - INDICATES DRY DENSITY. (PCF)
   SW - INDICATES SWELL WHEN WETTED UNDER APPROXIMATE OVERBURDEN PRESSURE. (%)
   LL - INDICATES LIQUID LIMIT. (NV : NO VALUE)
   PI - INDICATES PLASTICITY INDEX. (NP : NON-PLASTIC)
   -200 - INDICATES PASSING NO. 200 SIEVE. (%)
   SS - INDICATES WATER-SOLUBLE SULFATE CONTENT. (%)
   UC - INDICATES UNCONFINED COMPRESSIVE STRENGTH. (PSF)

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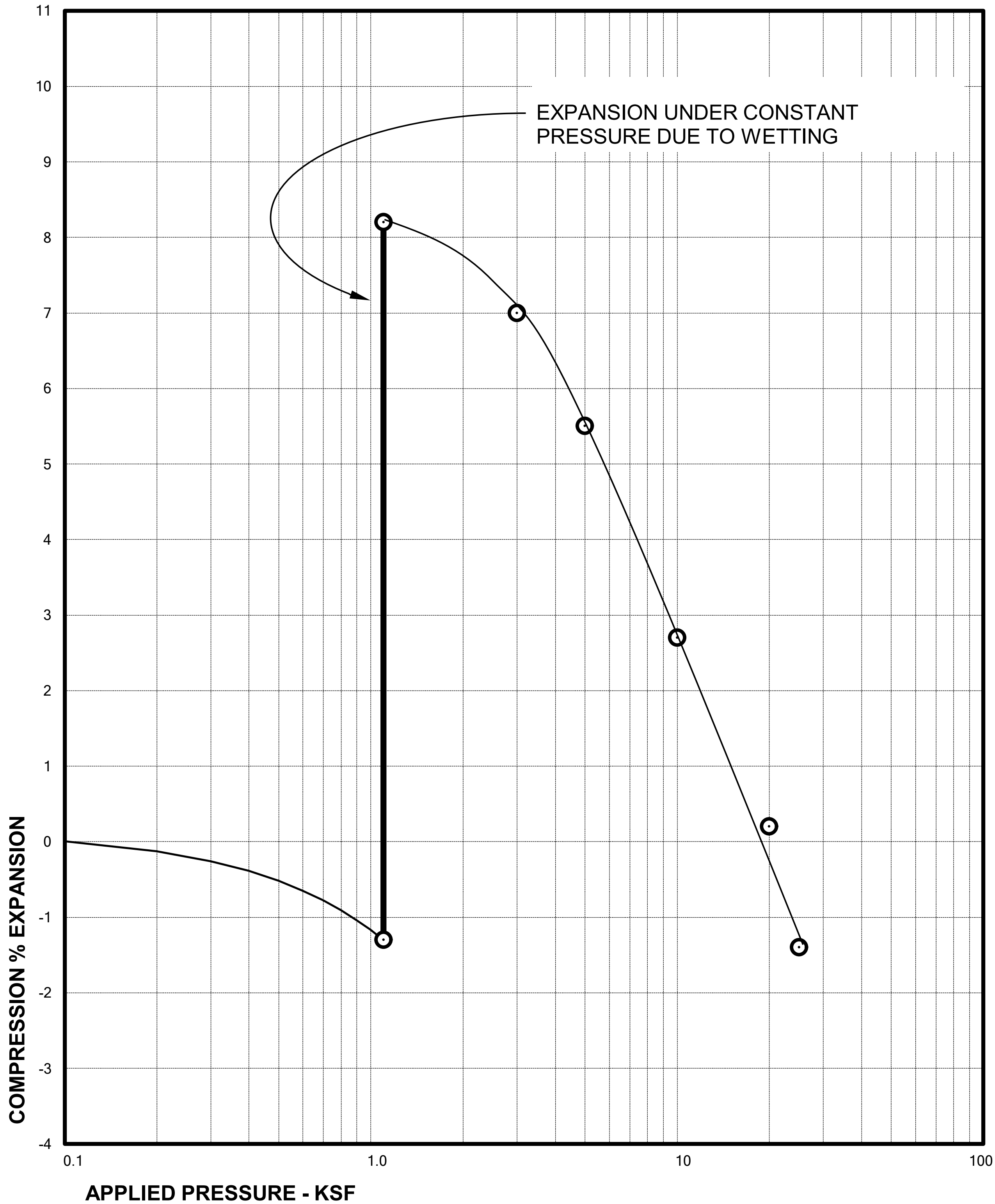
Summary Logs of Exploratory Borings

FIG. A-1

APPENDIX B

LABORATORY TEST RESULTS
TABLE B-1: SUMMARY OF LABORATORY TESTING



Sample of	CLAYSTONE	DRY UNIT WEIGHT=	121	PCF
From	TH-1 AT 9 FEET	MOISTURE CONTENT=	13.9	%

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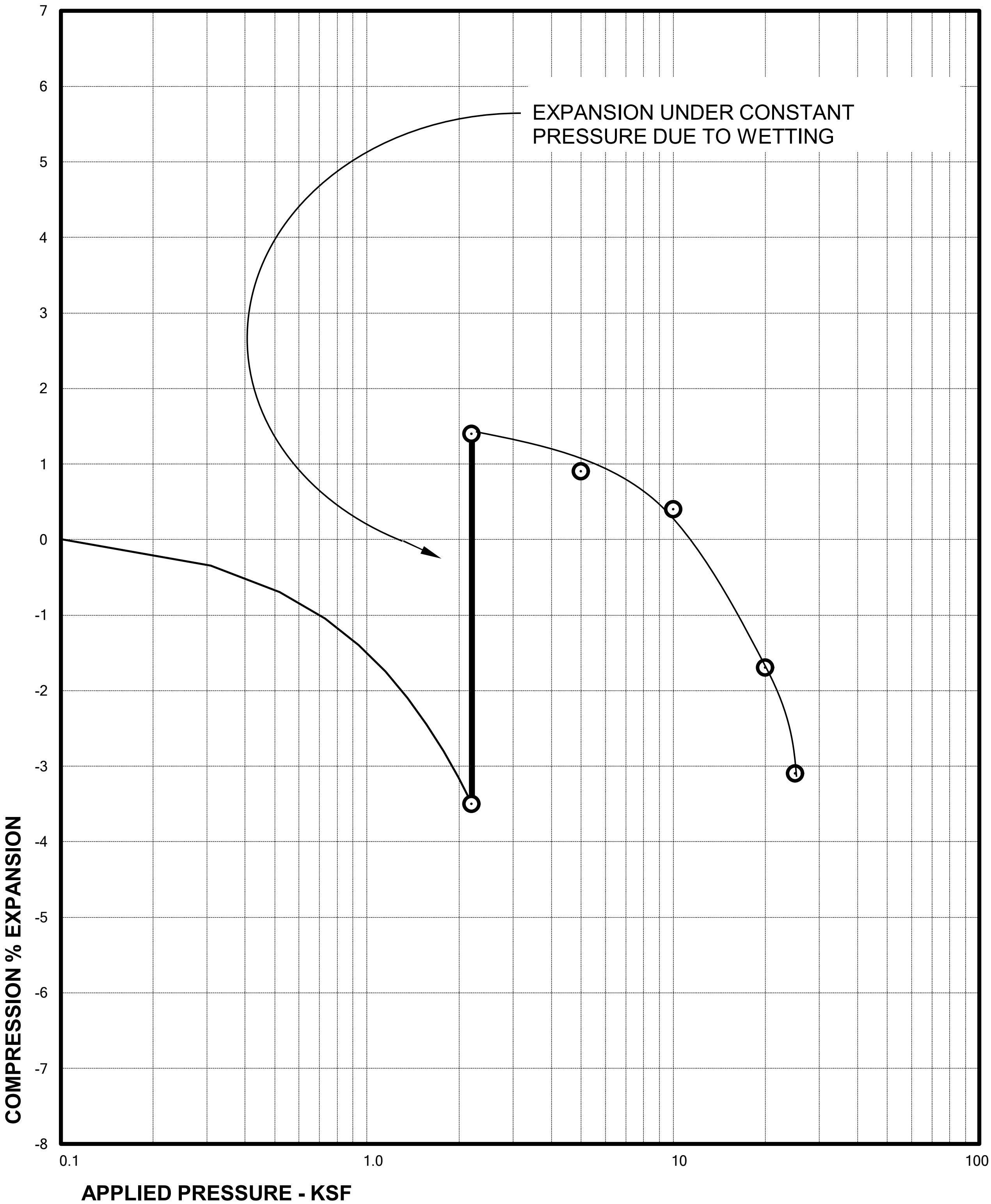
Swell Consolidation Test Results

FIG. B-1



Sample of CLAYSTONE
From TH-1 AT 19 FEET

DRY UNIT WEIGHT= 119 PCF
MOISTURE CONTENT= 16.1 %

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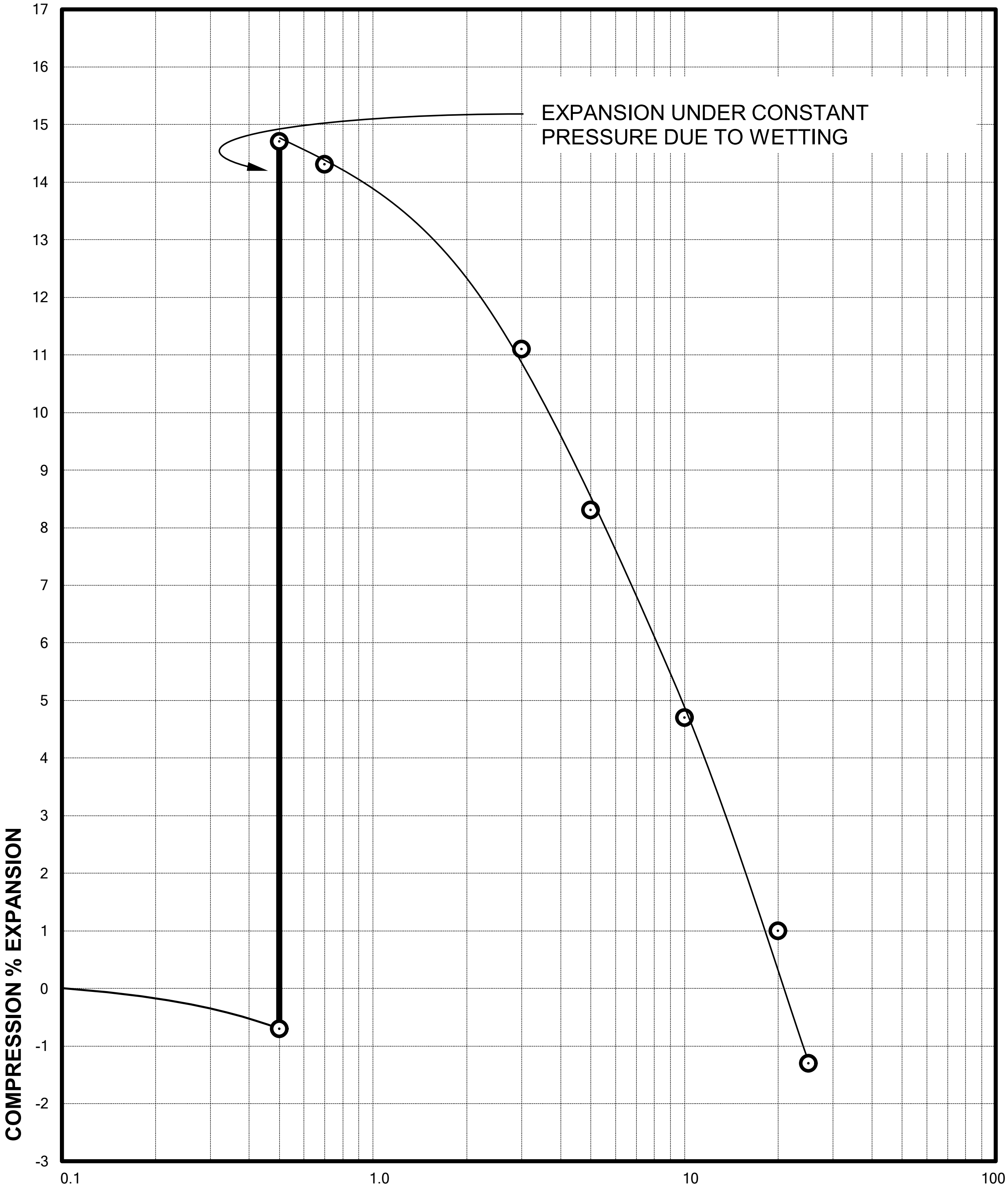
Swell Consolidation Test Results

FIG. B-2



Sample of FILL, CLAY, VERY SANDY
From TH-2 AT 4 FEET

DRY UNIT WEIGHT= 123 PCF
MOISTURE CONTENT= 9.4 %

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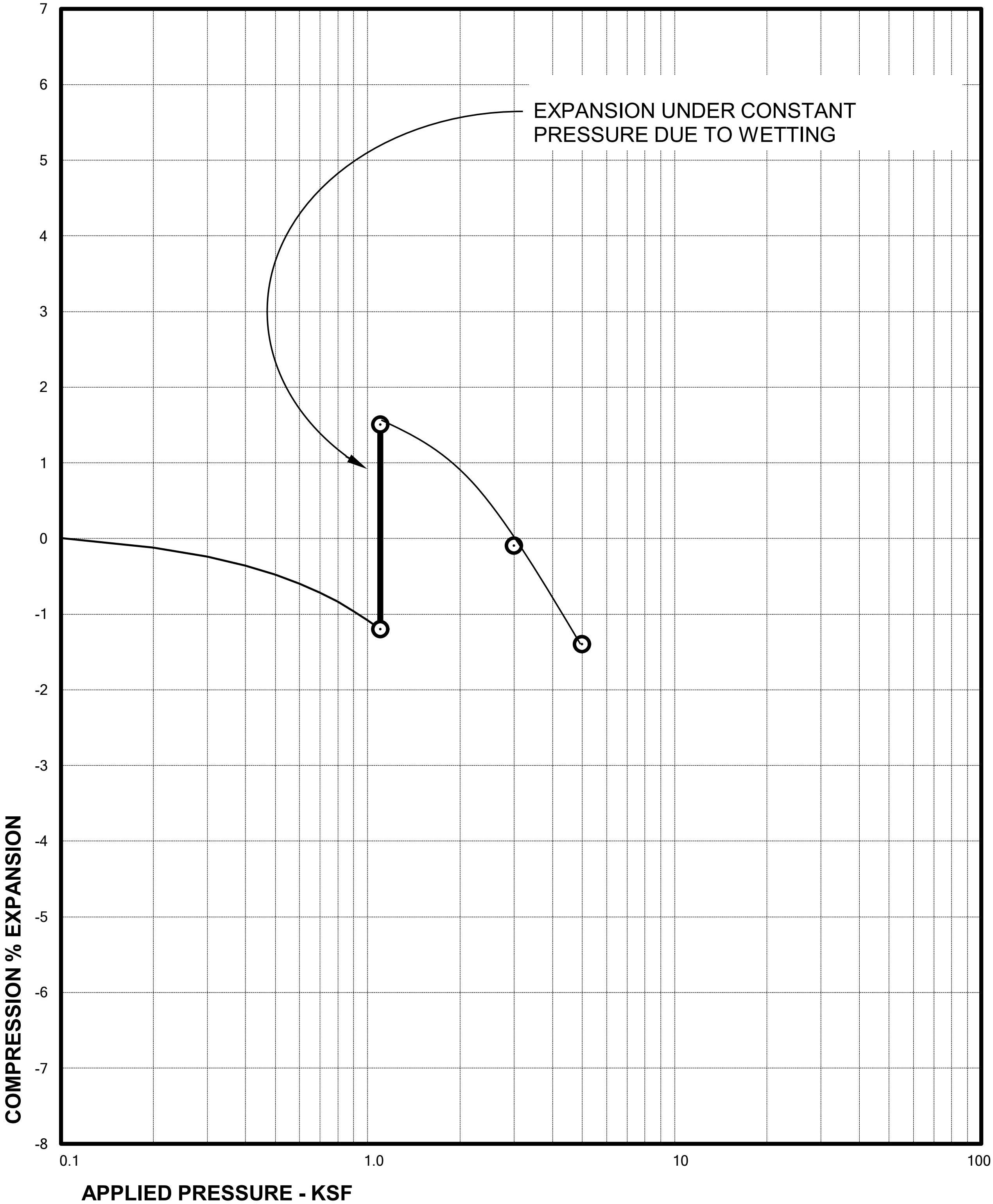
Swell Consolidation Test Results

FIG. B-3



Sample of FILL, CLAY, SANDY
From TH-2 AT 9 FEET

DRY UNIT WEIGHT= 114 PCF
MOISTURE CONTENT= 14.4 %

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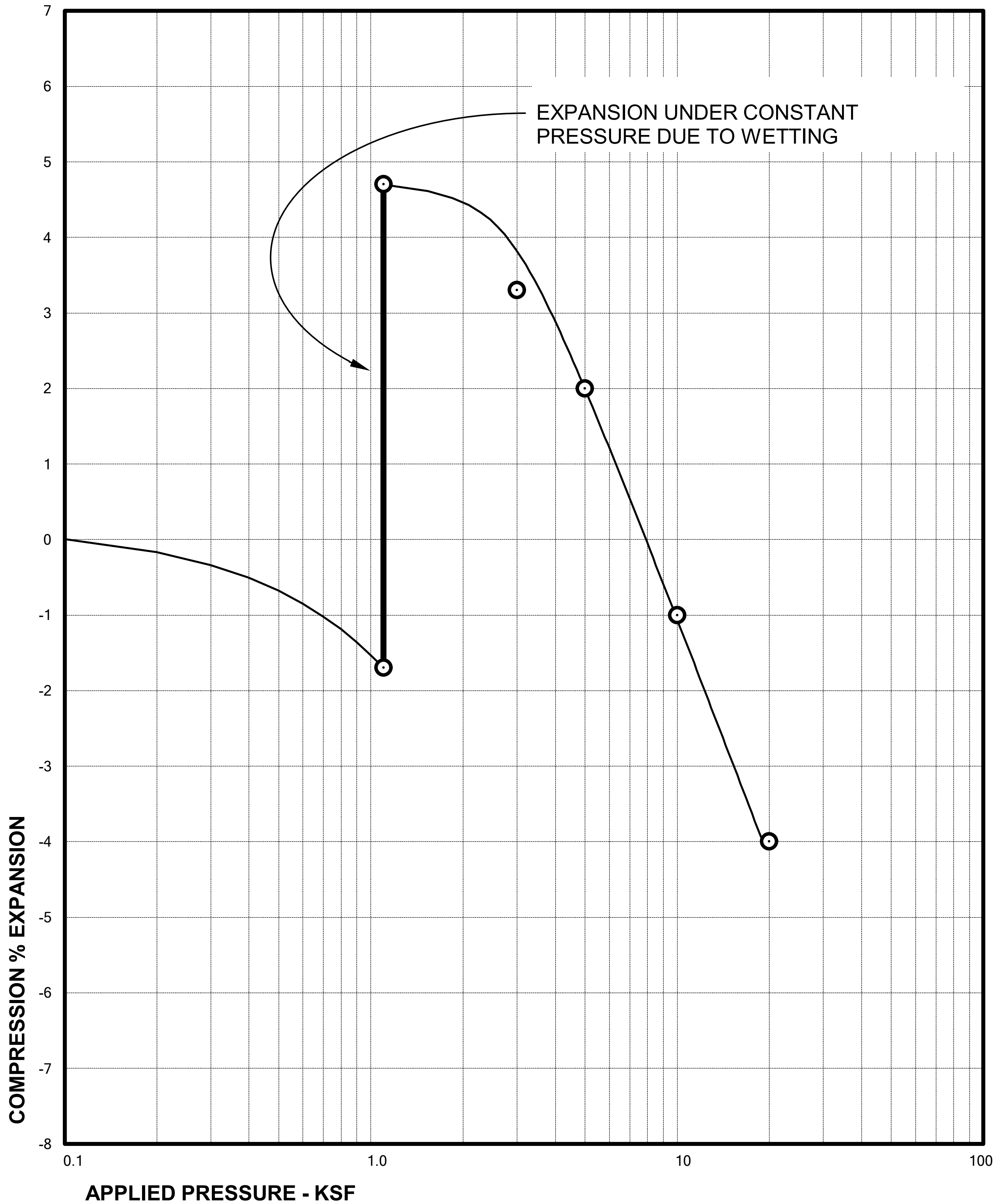
Swell Consolidation Test Results

FIG. B-4



Sample of CLAYSTONE
From TH-3 AT 9 FEET

DRY UNIT WEIGHT= 105 PCF
MOISTURE CONTENT= 21.6 %

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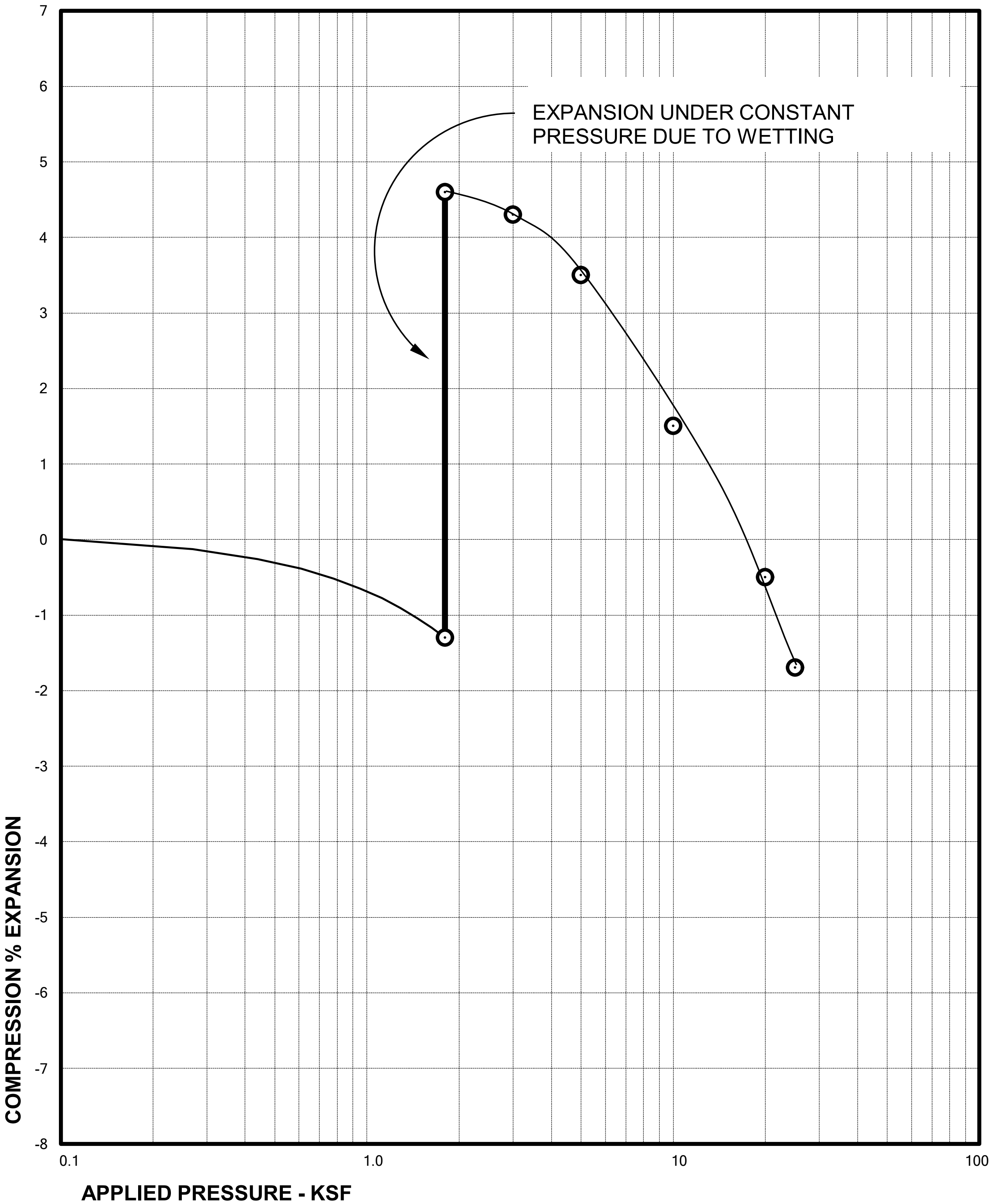
Swell Consolidation Test Results

FIG. B-5



Sample of CLAYSTONE
From TH-3 AT 14 FEET

DRY UNIT WEIGHT= 117 PCF
MOISTURE CONTENT= 17.1 %

Swell Consolidation Test Results

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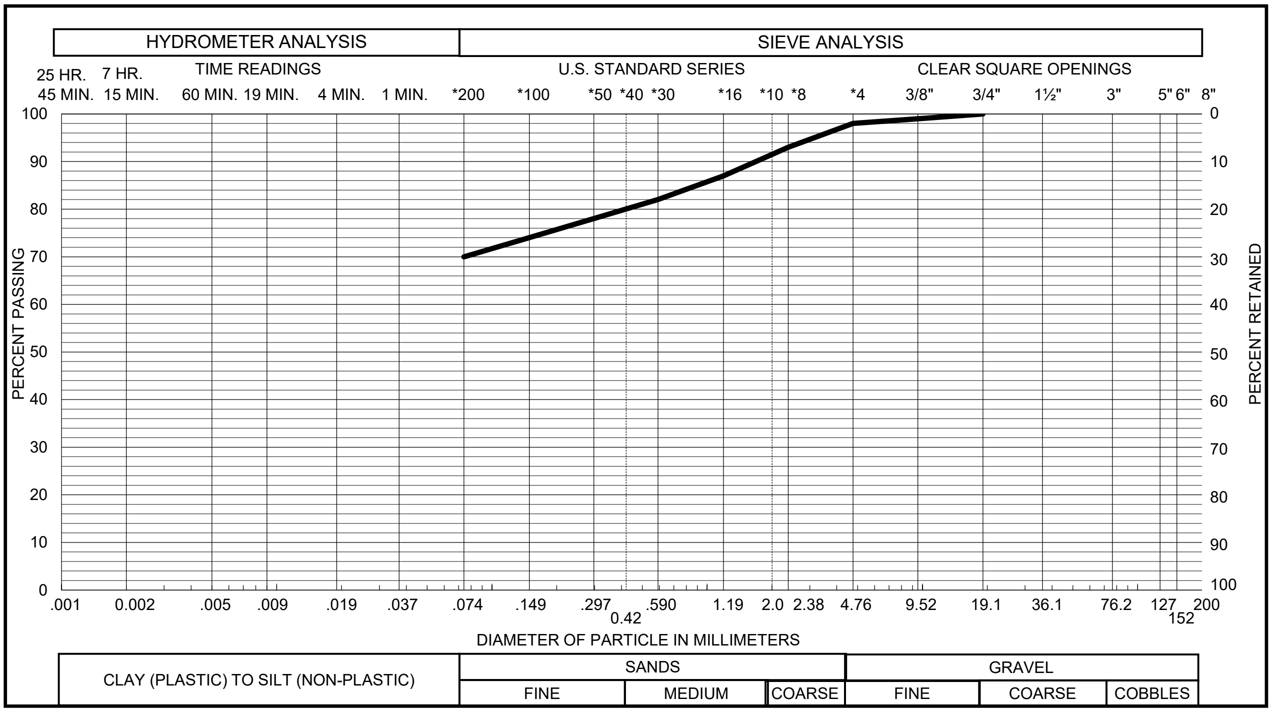
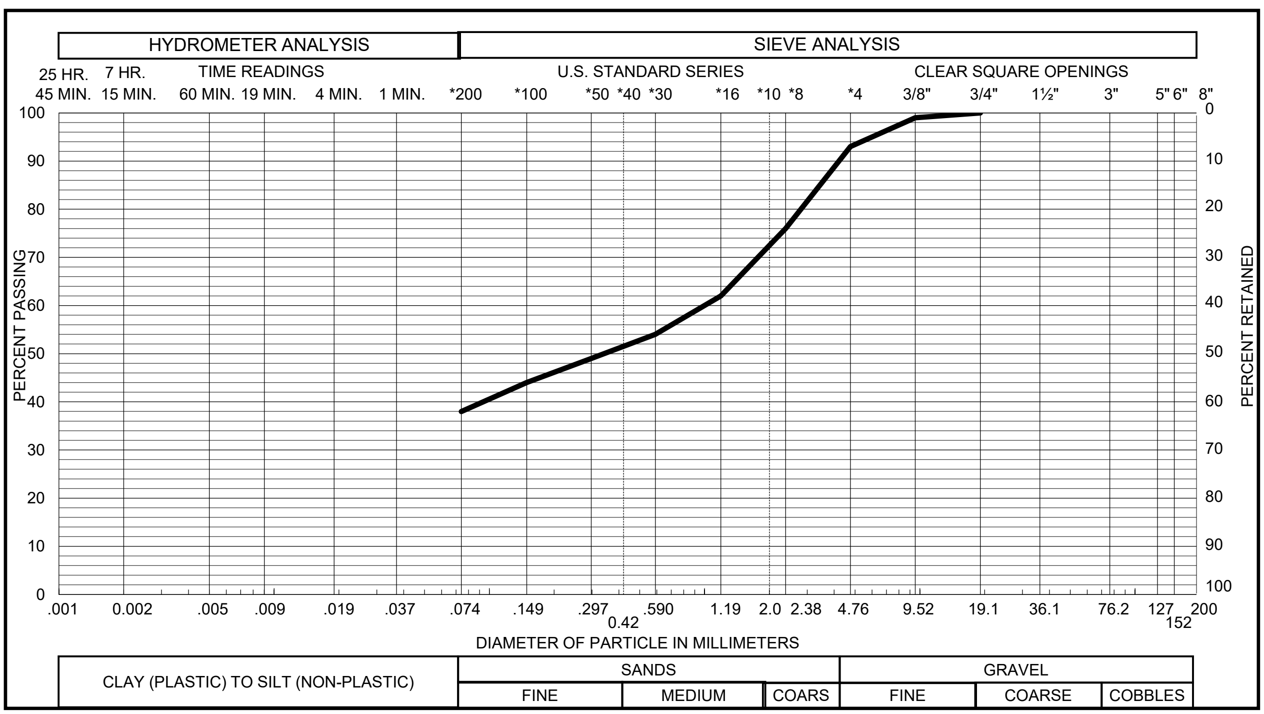
CTL|T PROJECT NO. CS19543-125

FIG. B-6



Sample of FILL, CLAY, SANDY
From TH - 1 AT 4 FEET

GRAVEL 2 %   SAND 28 %
SILT & CLAY 70 %   LIQUID LIMIT 52 %
PLASTICITY INDEX 30 %



Sample of SAND, VERY CLAYEY (SC)
From TH - 3 AT 4 FEET

GRAVEL 7 %   SAND 55 %
SILT & CLAY 38 %   LIQUID LIMIT %
PLASTICITY INDEX %

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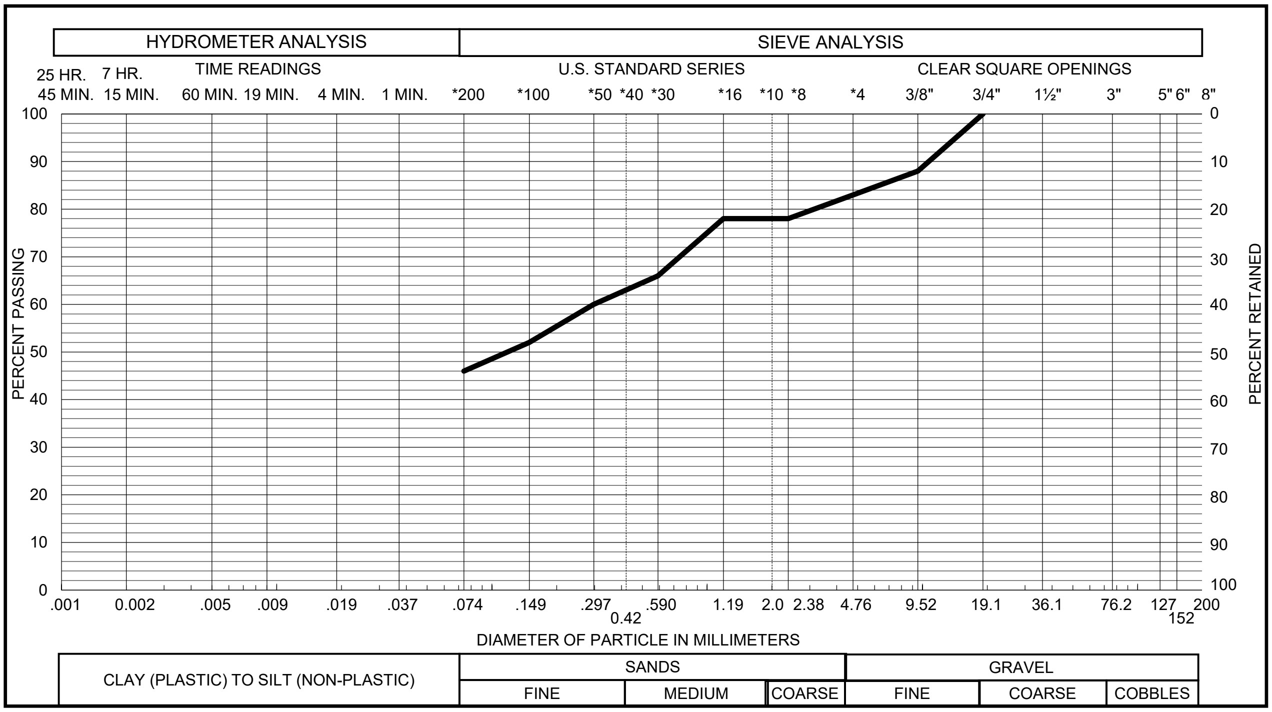
CTL|T PROJECT NO. CS19543-125

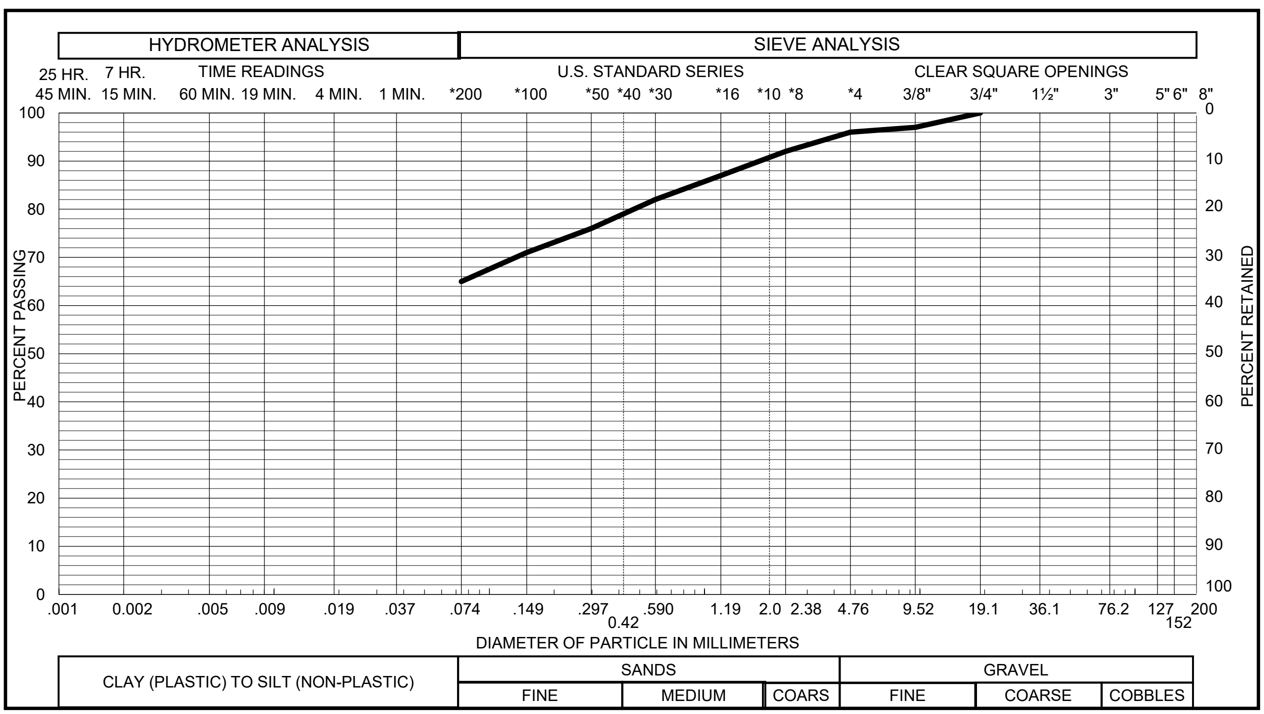
Gradation Test Results

FIG. B-7



Sample of SAND, VERY CLAYEY (SC)
From TH - 5 AT 4 FEET

GRAVEL 17 % SAND 37 %
SILT & CLAY 46 % LIQUID LIMIT %
PLASTICITY INDEX %



Sample of CLAY, SANDY (CL)
From TH - 5 AT 9 FEET

GRAVEL 4 % SAND 31 %
SILT & CLAY 65 % LIQUID LIMIT %
PLASTICITY INDEX %

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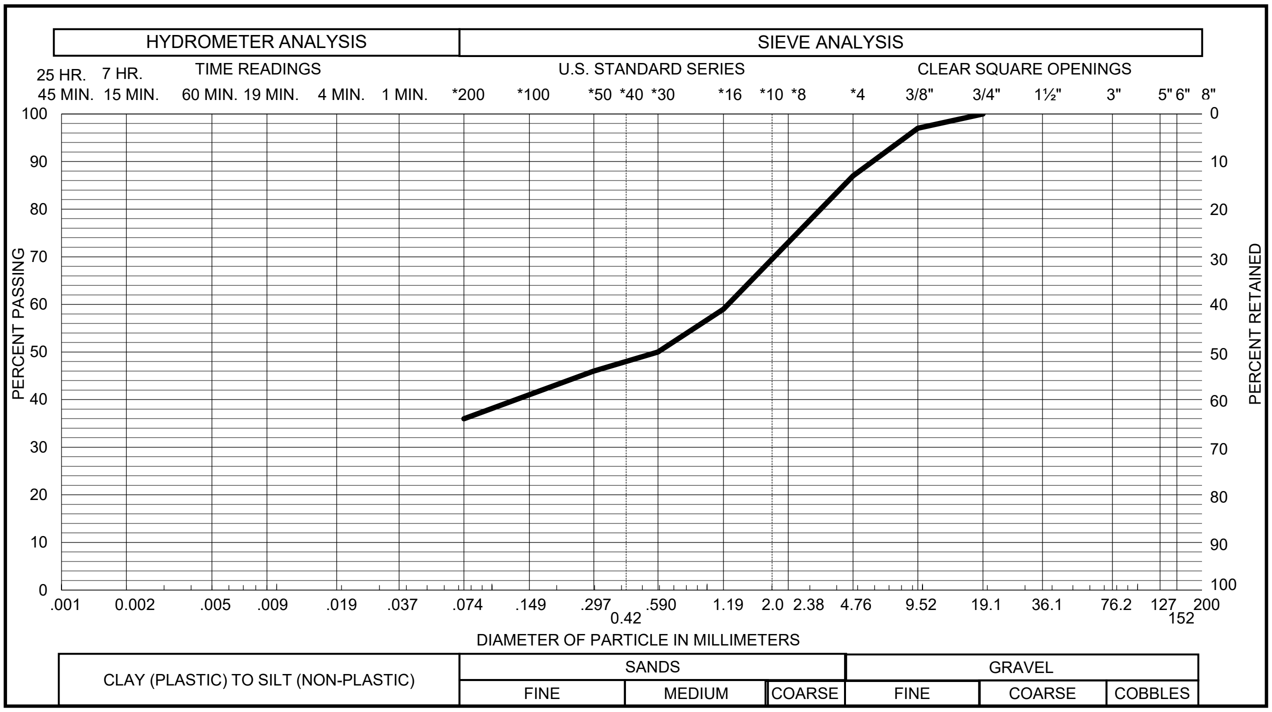
THE LAUNCHPAD - YOUTH HOUSING
CTL|T PROJECT NO. CS19543-125

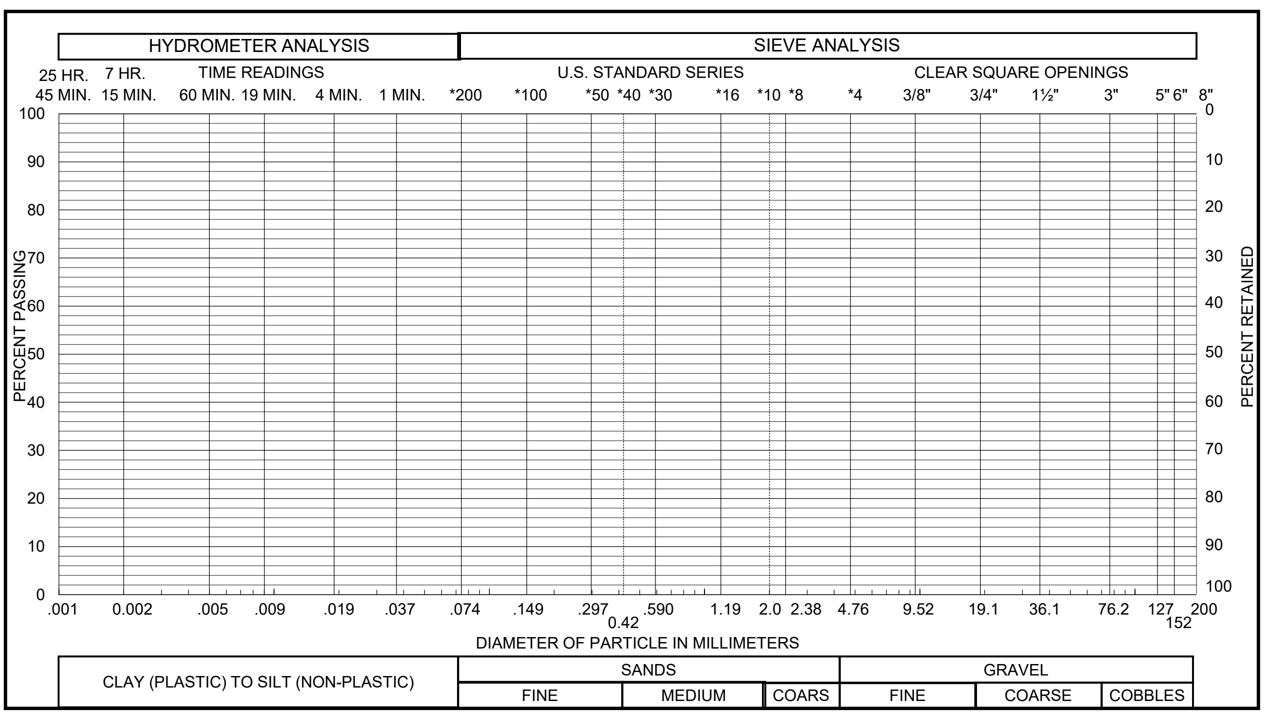
Gradation Test Results

FIG. B-8



Sample of SAND, VERY CLAYEY (SC)
From TH - 5 AT 14 FEET

GRAVEL 13 % SAND 51 %
SILT & CLAY 36 % LIQUID LIMIT %
PLASTICITY INDEX %



Sample of
From

GRAVEL % SAND %
SILT & CLAY % LIQUID LIMIT %
PLASTICITY INDEX %

COHEN ESREY
THE LAUNCHPAD - YOUTH HOUSING
CTL|T PROJECT NO. CS19543-125

Gradation Test Results

FIG. B-9

TABLE B-1

SUMMARY OF LABORATORY TESTING
CTL|T PROJECT NO. CS19543-125

BORING	DEPTH (FEET)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	ATTERBERG LIMITS LIQUID LIMIT	ATTERBERG LIMITS PLASTICITY INDEX	SWELL TEST RESULTS* SWELL (%)	SWELL TEST RESULTS* APPLIED PRESSURE (PSF)	SWELL TEST RESULTS* SWELL PRESSURE (PSF)	PASSING NO. 200 SIEVE (%)	WATER SOLUBLE SULFATES (%)	DESCRIPTION
TH-1	4	10.4	113	52	30				70	0.4	FILL, CLAY, SANDY
TH-1	9	13.9	121			9.5	1100	26500			CLAYSTONE
TH-1	19	16.1	119			4.9	2400	31500	99		CLAYSTONE
TH-2	4	9.4	123			15.4	500	26500	64		FILL, CLAY, VERY SANDY
TH-2	9	14.4	114			2.7	1100	3500	75		FILL, CLAY, SANDY
TH-3	4	5.3	129						38		SAND, VERY CLAYEY (SC)
TH-3	9	21.6	105			6.4	1100	12000	99		CLAYSTONE
TH-3	14	17.1	117			5.9	1800	26500	99		CLAYSTONE
TH-4	4	15.0	111						90		FILL, CLAY, SLIGHTLY SANDY
TH-5	4	6.9	127						46		SAND, VERY CLAYEY (SC)
TH-5	9	12.9	120						65		CLAY, SANDY (CL)
TH-5	14	9.5	127						36		SAND, VERY CLAYEY (SC)

* SWELL MEASURED UNDER ESTIMATED IN-SITU OVERBURDEN PRESSURE.
NEGATIVE VALUE INDICATES COMPRESSION.

APPENDIX C

DIRECT SHEAR TEST RESULTS

ADVANCED TERRA TESTING

Direct Shear

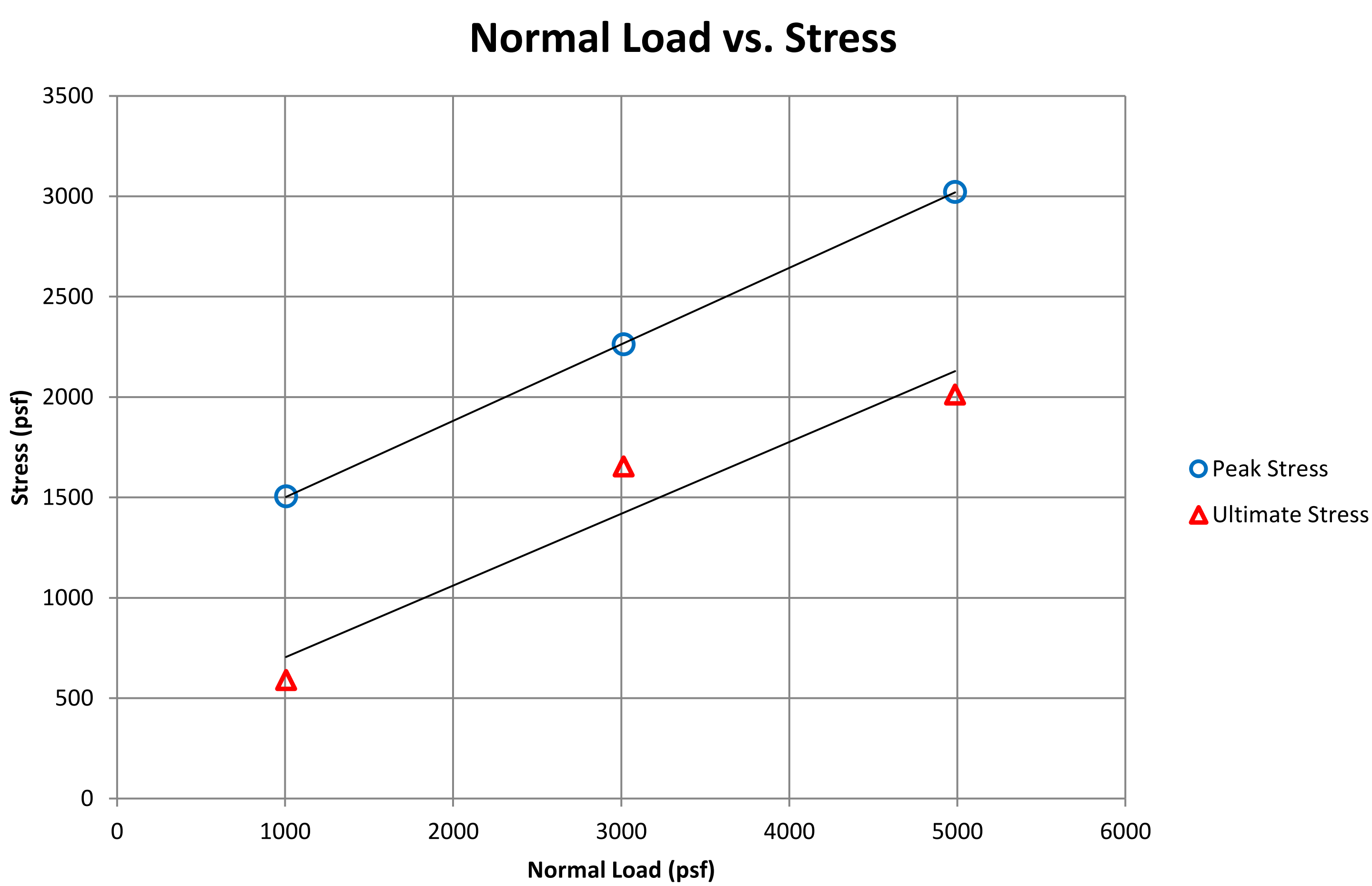
ASTM D 3080

CLIENT	CTL Thompson	BORING NO.	TH-4
JOB NO.	2843-014	DEPTH	14-19'
PROJECT	The Launchpad	SAMPLE NO.	--
PROJECT NO.	CS19243-125	DATE SAMPLED	--
LOCATION	--	DESCRIPTION	--
DATE TESTED	08/18/22		
TECHNICIAN	JL		

Direct Shear Results

Point:	A	B	C
Normal Load (psf):	4987	3015	1005
Normal Load (kPa):	238.8	144.4	48.1
Peak Strength (psf):	3022	2263	1505
Ultimate Strength (psf):	2013	1654	591
Peak Strength (kPa):	144.7	108.3	72.1
Ultimate Strength (kPa):	96.4	79.2	28.3

Peak Strength		Ultimate Strength	
Friction Angle:	20.9	Friction Angle:	19.7
Cohesion (psf):	1120	Cohesion (psf):	346
Cohesion (kPa):	53.7	Cohesion (kPa):	16.6



Data entry by:	AC	Date: 08/25/22
Checked by:	JL	Date: 08/25/22
File name:	2843014__Direct Shear ASTM D3080_0.xlsm	Page 1 of 2

Direct Shear

ASTM D 3080

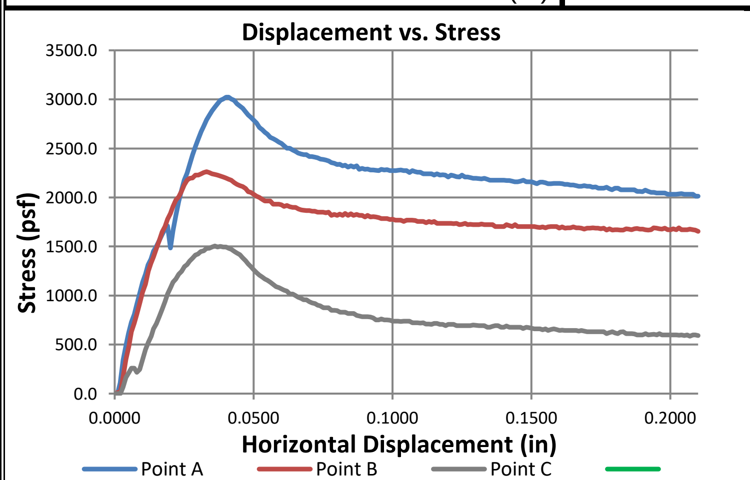
CLIENT	CTL Thompson	BORING NO.	TH-4
JOB NO.	2843-014	DEPTH	14-19'
PROJECT	The Launchpad	SAMPLE NO.	--
PROJECT NO.	CS19243-125	DATE SAMPLED	--
LOCATION	--	DESCRIPTION	--
DATE TESTED	08/18/22		
TECHNICIAN	JL		

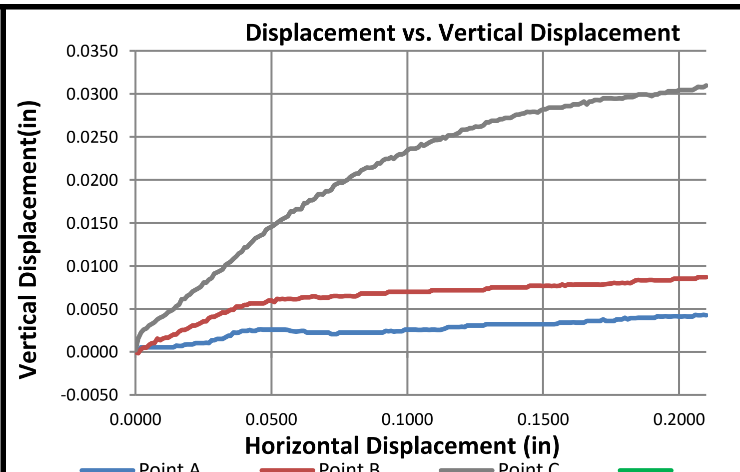
Test Parameters

Displacement Rate (in/min): 0.0001458 Displacement Rate (cm/min): 0.000370332

Raw Data Files: 2843-014_TH-4_DS_A_8-19.txt, 2843-014_TH-4_DS_B_8-22.txt, 2843-014_TH-4_DS_C_8-23.txt,

Before Test Mass of Wet Soil and Ring (g):	126.32	126.93	128.83
After Test Mass of Wet Soil and Pan (g):	101.69	102.78	106.55
Mass of Ring (g):	27.94	28.00	28.00
Mass of Wet Soil and Pan (g):	54.49	54.49	61.43
Mass of Dry Soil and Pan (g):	46.88	46.88	52.84
Mass of Pan (g):	6.89	6.89	6.75
Diameter (in):	1.94	1.94	1.94
Initial Height (in):	1.00	1.00	1.00
Height Change (in):	0.00496	-0.0088	-0.0462
Area (in^2):	2.95	2.95	2.95
Initial Wet Density (pcf):	127.1	127.8	130.2
Initial Dry Density (pcf):	106.7	107.3	109.8
Initial Wet Density (kg/m^3):	2035	2047	2086
Initial Dry Density (kg/m^3):	1710	1719	1758
Initial Moisture (%):	19.0	19.0	18.6
Final Wet Density (pcf):	132.0	131.6	131.5
Final Dry Density (pcf):	107.3	106.4	104.9
Final Wet Density (kg/m^3):	2114	2108	2107
Final Dry Density (kg/m^3):	1718	1704	1681
Final Moisture (%):	23.0	23.7	25.4





NOTES: Sample swelled during consolidation, shear rate calculated for 24 hour shear per ASTM D3080. Single moisture content taken for A and B points.

File name: 2843014__Direct Shear ASTM D3080_0.xlsm

Direct Shear Data
ASTM D3080

CLIENT	CTL Thompson	BORING NO.	TH-4
JOB NO.	2843-014	DEPTH	14-19'
PROJECT	The Launchpad	SAMPLE NO.	--
PROJECT NO.	CS19243-125	DATE SAMPLED	--
LOCATION	--	DESCRIPTION	--
DATE TESTED	08/18/22		
TECHNICIAN	JL		

Point A			Point B			Point C		
Displacement (in)	Stress (psf)	Vertical Displacment (in)	Displacement (in)	Stress (psf)	Vertical Displacment (in)	Displacement (in)	Stress (psf)	Vertical Displacment (in)
0.0000	0.6	0.0000	0.0000	0.0	0.0000	0.0000	0.6	0.0002
0.0010	0.9	0.0000	0.0010	10.4	-0.0002	0.0010	0.0	0.0016
0.0020	110.6	0.0005	0.0021	22.3	0.0003	0.0021	1.5	0.0023
0.0030	339.4	0.0005	0.0030	171.9	0.0005	0.0030	66.3	0.0026
0.0041	489.3	0.0005	0.0040	349.5	0.0005	0.0040	163.9	0.0027
0.0050	616.0	0.0005	0.0051	482.1	0.0008	0.0050	212.6	0.0031
0.0060	729.1	0.0005	0.0060	629.8	0.0010	0.0060	259.1	0.0032
0.0070	808.3	0.0005	0.0070	718.4	0.0010	0.0070	258.5	0.0034
0.0080	910.4	0.0005	0.0080	810.5	0.0015	0.0080	216.5	0.0038
0.0090	1020.7	0.0005	0.0091	919.9	0.0014	0.0090	246.0	0.0040
0.0101	1134.5	0.0005	0.0101	1035.2	0.0015	0.0100	349.0	0.0041
0.0111	1216.2	0.0005	0.0111	1115.6	0.0017	0.0111	449.9	0.0043
0.0120	1311.8	0.0005	0.0121	1249.2	0.0017	0.0121	524.3	0.0046
0.0131	1370.8	0.0005	0.0130	1330.0	0.0018	0.0131	585.2	0.0048
0.0140	1453.8	0.0005	0.0140	1392.5	0.0020	0.0140	661.3	0.0050
0.0150	1499.7	0.0007	0.0150	1478.0	0.0020	0.0150	712.4	0.0053
0.0160	1559.1	0.0007	0.0160	1559.1	0.0024	0.0160	783.4	0.0055
0.0170	1621.6	0.0007	0.0171	1647.4	0.0026	0.0170	860.1	0.0061
0.0180	1669.4	0.0008	0.0181	1701.8	0.0025	0.0181	938.1	0.0062
0.0190	1709.6	0.0009	0.0191	1777.2	0.0027	0.0190	1014.8	0.0065
0.0200	1485.5	0.0009	0.0200	1835.3	0.0029	0.0200	1078.4	0.0067
0.0210	1662.2	0.0009	0.0210	1907.0	0.0031	0.0210	1136.9	0.0070
0.0220	1817.1	0.0010	0.0220	1973.6	0.0031	0.0220	1168.8	0.0072
0.0230	1961.0	0.0010	0.0231	2005.0	0.0032	0.0231	1218.1	0.0074
0.0241	2079.2	0.0010	0.0240	2060.0	0.0034	0.0241	1246.5	0.0076
0.0251	2174.7	0.0010	0.0250	2126.6	0.0036	0.0251	1294.8	0.0081
0.0261	2248.3	0.0011	0.0260	2174.4	0.0036	0.0260	1316.9	0.0081
0.0270	2336.9	0.0010	0.0270	2193.9	0.0039	0.0270	1352.7	0.0084
0.0280	2431.8	0.0014	0.0280	2199.9	0.0041	0.0280	1379.6	0.0086
0.0290	2517.0	0.0014	0.0290	2226.9	0.0041	0.0290	1411.6	0.0091
0.0301	2597.7	0.0015	0.0300	2228.8	0.0043	0.0300	1423.2	0.0092
0.0311	2671.9	0.0015	0.0311	2240.7	0.0044	0.0310	1449.8	0.0094
0.0321	2727.5	0.0015	0.0321	2254.9	0.0046	0.0321	1456.6	0.0096
0.0331	2793.2	0.0017	0.0331	2262.7	0.0046	0.0331	1481.1	0.0101
0.0342	2841.3	0.0019	0.0340	2254.9	0.0047	0.0340	1481.7	0.0103
0.0350	2882.5	0.0019	0.0350	2241.4	0.0049	0.0350	1495.2	0.0105
0.0360	2921.7	0.0022	0.0360	2234.1	0.0049	0.0360	1505.3	0.0108
0.0370	2956.0	0.0022	0.0370	2227.2	0.0053	0.0370	1495.2	0.0111
0.0380	2989.3	0.0022	0.0380	2217.2	0.0053	0.0381	1501.7	0.0115
0.0391	3003.8	0.0024	0.0390	2207.1	0.0053	0.0390	1495.2	0.0116
0.0401	3022.3	0.0024	0.0400	2200.2	0.0054	0.0401	1495.2	0.0122
0.0410	3022.3	0.0024	0.0410	2184.2	0.0054	0.0410	1481.7	0.0122
0.0420	3004.1	0.0026	0.0420	2174.4	0.0056	0.0420	1470.7	0.0125

Direct Shear Data
ASTM D3080

CLIENT	CTL Thompson	BORING NO.	TH-4
JOB NO.	2843-014	DEPTH	14-19'
PROJECT	The Launchpad	SAMPLE NO.	--
PROJECT NO.	CS19243-125	DATE SAMPLED	--
LOCATION	--	DESCRIPTION	--
DATE TESTED	08/18/22		
TECHNICIAN	JL		

Point A			Point B			Point C		
Displacement (in)	Stress (psf)	Vertical Displacment (in)	Displacement (in)	Stress (psf)	Vertical Displacment (in)	Displacement (in)	Stress (psf)	Vertical Displacment (in)
0.0430	2989.3	0.0024	0.0431	2148.3	0.0056	0.0430	1449.8	0.0129
0.0440	2956.0	0.0024	0.0441	2133.9	0.0056	0.0441	1431.9	0.0132
0.0451	2932.1	0.0026	0.0451	2119.1	0.0056	0.0450	1415.1	0.0133
0.0461	2908.2	0.0026	0.0461	2111.6	0.0056	0.0460	1386.5	0.0135
0.0470	2874.9	0.0026	0.0471	2089.9	0.0056	0.0471	1354.2	0.0137
0.0480	2841.3	0.0026	0.0480	2059.7	0.0058	0.0480	1323.2	0.0142
0.0490	2817.1	0.0026	0.0490	2050.6	0.0060	0.0490	1295.1	0.0144
0.0500	2786.9	0.0026	0.0500	2031.1	0.0060	0.0500	1263.5	0.0145
0.0511	2760.5	0.0026	0.0511	2012.6	0.0058	0.0510	1233.6	0.0147
0.0521	2712.8	0.0026	0.0521	1998.4	0.0062	0.0521	1207.4	0.0150
0.0531	2689.2	0.0026	0.0531	1981.1	0.0061	0.0531	1187.1	0.0152
0.0541	2665.0	0.0026	0.0541	1963.9	0.0062	0.0541	1168.8	0.0154
0.0550	2644.6	0.0026	0.0550	1964.2	0.0061	0.0550	1155.7	0.0156
0.0560	2612.5	0.0026	0.0560	1964.5	0.0062	0.0560	1135.4	0.0158
0.0571	2598.4	0.0024	0.0571	1931.5	0.0062	0.0571	1113.6	0.0163
0.0580	2584.5	0.0024	0.0581	1930.5	0.0061	0.0580	1091.5	0.0163
0.0591	2565.1	0.0024	0.0591	1930.5	0.0061	0.0591	1081.4	0.0166
0.0600	2551.9	0.0024	0.0601	1923.9	0.0062	0.0600	1069.1	0.0166
0.0611	2524.2	0.0024	0.0610	1907.3	0.0063	0.0610	1056.0	0.0166
0.0620	2503.1	0.0024	0.0620	1917.3	0.0063	0.0620	1046.1	0.0173
0.0630	2503.1	0.0022	0.0630	1906.7	0.0063	0.0630	1015.1	0.0173
0.0640	2484.0	0.0022	0.0641	1897.5	0.0063	0.0641	1015.1	0.0176
0.0651	2469.8	0.0022	0.0651	1897.9	0.0064	0.0651	1000.8	0.0176
0.0661	2451.3	0.0022	0.0661	1882.8	0.0065	0.0660	984.0	0.0178
0.0671	2443.7	0.0022	0.0671	1874.0	0.0063	0.0671	969.7	0.0183
0.0681	2436.2	0.0022	0.0680	1869.6	0.0063	0.0680	955.4	0.0183
0.0690	2436.2	0.0022	0.0690	1865.2	0.0063	0.0690	955.7	0.0183
0.0700	2417.7	0.0022	0.0700	1865.2	0.0063	0.0700	938.4	0.0187
0.0710	2415.8	0.0022	0.0710	1861.1	0.0063	0.0711	924.3	0.0187
0.0721	2411.1	0.0021	0.0720	1859.2	0.0064	0.0721	917.5	0.0188
0.0730	2403.5	0.0021	0.0730	1850.1	0.0065	0.0731	899.8	0.0194
0.0740	2390.6	0.0021	0.0742	1850.1	0.0065	0.0740	894.5	0.0195
0.0750	2388.7	0.0022	0.0750	1850.1	0.0065	0.0751	874.2	0.0197
0.0760	2382.1	0.0022	0.0760	1843.2	0.0064	0.0760	874.5	0.0197
0.0770	2369.9	0.0022	0.0770	1850.1	0.0065	0.0770	874.2	0.0198
0.0780	2355.7	0.0022	0.0781	1816.8	0.0065	0.0781	847.9	0.0200
0.0790	2348.5	0.0022	0.0791	1826.2	0.0065	0.0791	847.9	0.0204
0.0801	2336.3	0.0022	0.0801	1817.4	0.0065	0.0800	845.5	0.0205
0.0810	2337.5	0.0022	0.0810	1830.9	0.0065	0.0812	829.1	0.0207
0.0820	2323.1	0.0022	0.0820	1817.4	0.0066	0.0820	829.1	0.0207
0.0830	2328.1	0.0022	0.0830	1835.3	0.0068	0.0830	829.1	0.0211
0.0840	2308.0	0.0022	0.0840	1817.1	0.0068	0.0840	815.6	0.0213
0.0851	2323.1	0.0022	0.0851	1832.8	0.0068	0.0851	815.9	0.0214

Direct Shear Data
ASTM D3080

CLIENT	CTL Thompson	BORING NO.	TH-4
JOB NO.	2843-014	DEPTH	14-19'
PROJECT	The Launchpad	SAMPLE NO.	--
PROJECT NO.	CS19243-125	DATE SAMPLED	--
LOCATION	--	DESCRIPTION	--
DATE TESTED	08/18/22		
TECHNICIAN	JL		

Point A			Point B			Point C		
Displacement (in)	Stress (psf)	Vertical Displacment (in)	Displacement (in)	Stress (psf)	Vertical Displacment (in)	Displacement (in)	Stress (psf)	Vertical Displacment (in)
0.0861	2308.0	0.0022	0.0860	1816.8	0.0068	0.0861	815.9	0.0214
0.0871	2321.8	0.0022	0.0871	1826.2	0.0068	0.0871	797.7	0.0214
0.0880	2288.2	0.0022	0.0880	1817.1	0.0068	0.0881	790.6	0.0216
0.0890	2295.7	0.0022	0.0890	1817.1	0.0068	0.0890	784.0	0.0219
0.0900	2288.2	0.0022	0.0900	1802.3	0.0068	0.0900	784.3	0.0219
0.0910	2289.1	0.0022	0.0911	1817.1	0.0068	0.0910	783.4	0.0223
0.0921	2281.6	0.0024	0.0920	1802.3	0.0068	0.0921	780.4	0.0224
0.0931	2280.0	0.0024	0.0931	1802.6	0.0070	0.0931	771.2	0.0224
0.0940	2278.4	0.0024	0.0940	1802.3	0.0070	0.0940	752.4	0.0226
0.0951	2275.0	0.0024	0.0950	1795.7	0.0070	0.0950	752.4	0.0224
0.0960	2287.9	0.0024	0.0960	1784.1	0.0070	0.0960	761.9	0.0228
0.0970	2281.6	0.0024	0.0971	1784.1	0.0070	0.0970	752.4	0.0230
0.0981	2281.9	0.0024	0.0980	1783.2	0.0070	0.0980	752.4	0.0229
0.0991	2274.4	0.0026	0.0991	1775.9	0.0070	0.0991	749.4	0.0231
0.1000	2274.0	0.0026	0.1000	1779.1	0.0070	0.1000	738.9	0.0234
0.1011	2274.7	0.0026	0.1010	1769.6	0.0070	0.1010	738.0	0.0236
0.1020	2276.9	0.0026	0.1020	1769.6	0.0070	0.1020	738.3	0.0236
0.1030	2280.3	0.0026	0.1030	1769.3	0.0070	0.1030	734.1	0.0236
0.1040	2274.7	0.0026	0.1040	1756.4	0.0070	0.1040	738.6	0.0238
0.1051	2274.7	0.0025	0.1050	1769.3	0.0070	0.1051	738.0	0.0242
0.1061	2255.5	0.0026	0.1061	1769.6	0.0070	0.1061	738.0	0.0240
0.1071	2275.0	0.0026	0.1071	1769.6	0.0070	0.1071	720.7	0.0242
0.1081	2264.9	0.0026	0.1082	1763.7	0.0070	0.1081	720.7	0.0243
0.1090	2255.8	0.0026	0.1090	1754.6	0.0072	0.1090	721.0	0.0245
0.1100	2255.8	0.0026	0.1100	1754.6	0.0072	0.1100	719.8	0.0246
0.1110	2240.7	0.0025	0.1111	1754.6	0.0072	0.1110	721.0	0.0246
0.1121	2241.4	0.0025	0.1121	1742.6	0.0072	0.1120	708.5	0.0247
0.1130	2240.7	0.0026	0.1131	1754.6	0.0072	0.1131	710.6	0.0250
0.1141	2241.0	0.0027	0.1140	1745.1	0.0072	0.1140	707.0	0.0248
0.1150	2227.5	0.0029	0.1150	1754.6	0.0072	0.1150	714.4	0.0251
0.1160	2241.7	0.0029	0.1160	1736.0	0.0072	0.1160	717.1	0.0251
0.1170	2227.5	0.0029	0.1171	1735.4	0.0072	0.1170	707.0	0.0251
0.1181	2230.7	0.0029	0.1181	1735.4	0.0072	0.1181	707.0	0.0253
0.1191	2229.1	0.0029	0.1191	1736.0	0.0072	0.1191	692.9	0.0255
0.1200	2207.7	0.0029	0.1201	1735.4	0.0072	0.1200	707.0	0.0258
0.1211	2227.5	0.0029	0.1211	1736.6	0.0072	0.1211	707.0	0.0258
0.1220	2222.5	0.0031	0.1220	1732.2	0.0072	0.1220	707.3	0.0259
0.1230	2212.8	0.0031	0.1230	1735.7	0.0072	0.1230	692.6	0.0260
0.1240	2208.0	0.0031	0.1240	1721.9	0.0072	0.1240	693.5	0.0260
0.1250	2227.5	0.0031	0.1250	1727.2	0.0072	0.1251	693.8	0.0262
0.1260	2208.0	0.0031	0.1261	1736.0	0.0072	0.1260	693.5	0.0262
0.1271	2207.4	0.0031	0.1270	1723.8	0.0072	0.1271	692.9	0.0262
0.1281	2201.1	0.0031	0.1280	1726.0	0.0072	0.1280	693.5	0.0264

Direct Shear Data
ASTM D3080

CLIENT	CTL Thompson	BORING NO.	TH-4
JOB NO.	2843-014	DEPTH	14-19'
PROJECT	The Launchpad	SAMPLE NO.	--
PROJECT NO.	CS19243-125	DATE SAMPLED	--
LOCATION	--	DESCRIPTION	--
DATE TESTED	08/18/22		
TECHNICIAN	JL		

Point A			Point B			Point C		
Displacement (in)	Stress (psf)	Vertical Displacment (in)	Displacement (in)	Stress (psf)	Vertical Displacment (in)	Displacement (in)	Stress (psf)	Vertical Displacment (in)
0.1290	2195.8	0.0032	0.1290	1721.9	0.0073	0.1290	700.1	0.0267
0.1300	2194.2	0.0032	0.1301	1721.9	0.0073	0.1300	696.8	0.0267
0.1310	2193.6	0.0032	0.1310	1721.9	0.0075	0.1310	693.8	0.0269
0.1321	2187.6	0.0032	0.1320	1728.5	0.0075	0.1321	693.8	0.0269
0.1331	2193.9	0.0032	0.1330	1721.9	0.0075	0.1331	692.6	0.0269
0.1341	2186.0	0.0032	0.1341	1721.9	0.0075	0.1340	677.4	0.0270
0.1350	2177.2	0.0032	0.1350	1721.9	0.0075	0.1350	678.0	0.0270
0.1360	2174.7	0.0032	0.1360	1721.9	0.0075	0.1360	686.4	0.0272
0.1370	2174.7	0.0032	0.1370	1702.1	0.0075	0.1370	694.4	0.0272
0.1381	2174.7	0.0032	0.1381	1702.1	0.0075	0.1380	693.8	0.0272
0.1391	2174.7	0.0032	0.1391	1701.8	0.0075	0.1390	675.6	0.0274
0.1401	2174.7	0.0032	0.1401	1702.1	0.0075	0.1401	675.9	0.0276
0.1411	2171.0	0.0032	0.1410	1721.9	0.0075	0.1410	691.2	0.0276
0.1420	2173.2	0.0032	0.1420	1712.4	0.0075	0.1420	675.6	0.0277
0.1430	2165.3	0.0032	0.1430	1702.4	0.0075	0.1430	675.3	0.0277
0.1440	2159.6	0.0032	0.1441	1721.9	0.0075	0.1440	675.6	0.0279
0.1451	2160.0	0.0032	0.1450	1701.8	0.0077	0.1451	675.3	0.0279
0.1461	2167.2	0.0032	0.1461	1702.1	0.0077	0.1461	675.9	0.0279
0.1471	2174.7	0.0032	0.1470	1702.1	0.0077	0.1470	663.1	0.0279
0.1482	2159.6	0.0032	0.1480	1703.0	0.0077	0.1481	675.9	0.0279
0.1490	2159.6	0.0032	0.1490	1702.1	0.0077	0.1490	672.0	0.0281
0.1501	2159.6	0.0032	0.1500	1702.1	0.0077	0.1500	667.0	0.0282
0.1511	2150.8	0.0032	0.1511	1701.8	0.0077	0.1511	661.6	0.0282
0.1520	2141.4	0.0032	0.1521	1695.5	0.0077	0.1521	661.6	0.0284
0.1531	2157.1	0.0032	0.1531	1692.0	0.0077	0.1531	661.6	0.0284
0.1540	2155.2	0.0032	0.1540	1700.2	0.0077	0.1540	652.0	0.0284
0.1550	2150.5	0.0032	0.1550	1689.2	0.0077	0.1550	661.6	0.0284
0.1560	2141.4	0.0034	0.1560	1698.9	0.0077	0.1560	643.1	0.0284
0.1570	2141.4	0.0034	0.1570	1701.8	0.0078	0.1571	655.0	0.0284
0.1581	2141.7	0.0034	0.1581	1701.8	0.0077	0.1580	659.5	0.0286
0.1591	2142.0	0.0034	0.1591	1701.8	0.0078	0.1590	652.6	0.0286
0.1601	2141.7	0.0034	0.1601	1687.6	0.0078	0.1601	645.2	0.0286
0.1612	2141.1	0.0034	0.1610	1700.5	0.0078	0.1610	646.1	0.0288
0.1620	2134.5	0.0034	0.1620	1688.2	0.0078	0.1620	643.1	0.0288
0.1630	2134.2	0.0034	0.1630	1689.8	0.0078	0.1630	643.1	0.0288
0.1640	2127.0	0.0034	0.1640	1689.2	0.0078	0.1640	643.4	0.0289
0.1650	2127.0	0.0034	0.1651	1699.9	0.0078	0.1650	643.4	0.0291
0.1661	2121.0	0.0036	0.1661	1688.6	0.0078	0.1661	643.1	0.0288
0.1670	2112.8	0.0036	0.1670	1688.9	0.0079	0.1670	636.5	0.0291
0.1680	2112.5	0.0036	0.1680	1687.9	0.0078	0.1680	643.1	0.0291
0.1690	2119.1	0.0036	0.1690	1678.5	0.0078	0.1690	638.0	0.0293
0.1700	2112.5	0.0036	0.1700	1687.9	0.0078	0.1700	632.3	0.0293
0.1711	2112.5	0.0036	0.1710	1687.9	0.0078	0.1711	630.5	0.0293

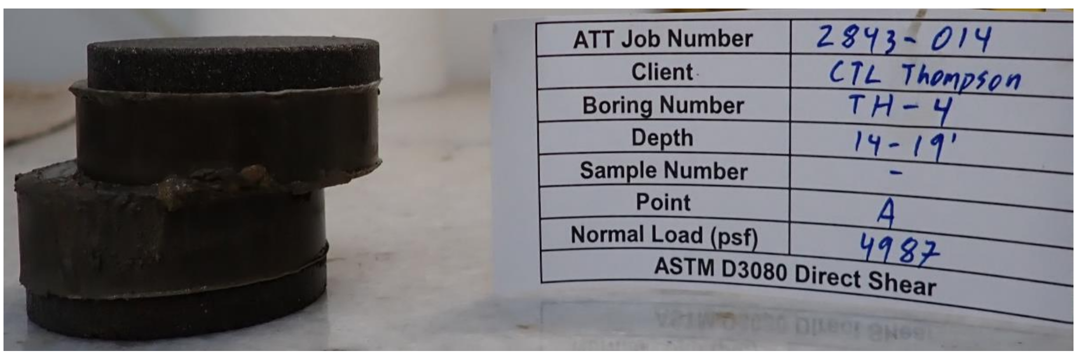
Direct Shear Data
ASTM D3080

CLIENT	CTL Thompson	BORING NO.	TH-4
JOB NO.	2843-014	DEPTH	14-19'
PROJECT	The Launchpad	SAMPLE NO.	--
PROJECT NO.	CS19243-125	DATE SAMPLED	--
LOCATION	--	DESCRIPTION	--
DATE TESTED	08/18/22		
TECHNICIAN	JL		

Point A			Point B			Point C		
Displacement (in)	Stress (psf)	Vertical Displacment (in)	Displacement (in)	Stress (psf)	Vertical Displacment (in)	Displacement (in)	Stress (psf)	Vertical Displacment (in)
0.1720	2103.1	0.0038	0.1721	1688.6	0.0078	0.1721	630.5	0.0295
0.1731	2106.5	0.0036	0.1731	1687.9	0.0079	0.1730	630.2	0.0295
0.1740	2093.3	0.0036	0.1740	1687.9	0.0079	0.1741	630.5	0.0295
0.1750	2094.3	0.0036	0.1750	1678.8	0.0080	0.1750	629.7	0.0295
0.1760	2094.3	0.0036	0.1760	1687.9	0.0080	0.1760	630.5	0.0294
0.1770	2103.1	0.0038	0.1771	1668.1	0.0080	0.1770	611.4	0.0294
0.1780	2086.4	0.0038	0.1781	1681.0	0.0080	0.1780	620.7	0.0295
0.1790	2079.2	0.0038	0.1791	1668.8	0.0080	0.1791	630.2	0.0295
0.1801	2094.0	0.0040	0.1801	1669.1	0.0080	0.1801	618.6	0.0296
0.1812	2094.3	0.0038	0.1810	1669.1	0.0080	0.1811	612.0	0.0296
0.1821	2079.2	0.0039	0.1820	1668.4	0.0080	0.1820	628.2	0.0296
0.1830	2079.2	0.0039	0.1830	1668.8	0.0081	0.1830	630.2	0.0296
0.1840	2079.2	0.0039	0.1840	1678.8	0.0083	0.1840	611.4	0.0298
0.1850	2079.2	0.0040	0.1850	1678.2	0.0083	0.1851	612.0	0.0299
0.1861	2079.2	0.0040	0.1860	1669.1	0.0083	0.1861	611.4	0.0299
0.1871	2079.2	0.0039	0.1871	1678.8	0.0083	0.1871	609.1	0.0299
0.1880	2062.5	0.0040	0.1880	1682.9	0.0083	0.1880	598.9	0.0299
0.1890	2060.3	0.0040	0.1890	1678.5	0.0083	0.1890	597.4	0.0298
0.1900	2060.0	0.0040	0.1900	1678.2	0.0083	0.1900	598.3	0.0297
0.1910	2074.2	0.0040	0.1911	1669.1	0.0083	0.1911	611.4	0.0299
0.1921	2060.3	0.0041	0.1921	1666.9	0.0083	0.1921	598.0	0.0299
0.1931	2052.5	0.0041	0.1931	1669.4	0.0083	0.1930	601.3	0.0301
0.1941	2053.4	0.0041	0.1940	1688.9	0.0083	0.1941	605.2	0.0301
0.1951	2045.6	0.0041	0.1950	1687.6	0.0083	0.1951	598.9	0.0301
0.1960	2045.3	0.0041	0.1960	1678.8	0.0083	0.1960	611.7	0.0303
0.1970	2046.5	0.0041	0.1970	1676.6	0.0083	0.1970	598.3	0.0303
0.1980	2045.3	0.0041	0.1981	1687.9	0.0085	0.1980	597.4	0.0303
0.1991	2031.4	0.0041	0.1991	1669.1	0.0085	0.1990	597.4	0.0303
0.2001	2031.4	0.0041	0.2001	1669.1	0.0085	0.2001	598.6	0.0305
0.2010	2031.7	0.0041	0.2010	1678.8	0.0085	0.2011	598.0	0.0305
0.2021	2031.4	0.0041	0.2022	1669.1	0.0085	0.2022	598.3	0.0305
0.2030	2037.1	0.0041	0.2030	1688.6	0.0085	0.2032	595.9	0.0305
0.2040	2038.3	0.0041	0.2041	1669.1	0.0085	0.2040	598.0	0.0305
0.2050	2031.4	0.0041	0.2050	1669.1	0.0085	0.2050	592.0	0.0305
0.2060	2031.4	0.0043	0.2061	1669.1	0.0085	0.2061	598.9	0.0306
0.2070	2031.4	0.0043	0.2070	1675.7	0.0087	0.2070	585.2	0.0308
0.2081	2031.4	0.0043	0.2081	1669.1	0.0087	0.2081	598.3	0.0308
0.2091	2013.2	0.0043	0.2090	1668.8	0.0087	0.2090	598.9	0.0308
0.2100	2012.9	0.0043	0.2100	1654.3	0.0087	0.2100	591.1	0.0310

Direct Shear
ASTM D3080
(Point A Picture 1)

CLIENT	CTL Thompson	BORING NO.	TH-4
JOB NO.	2843-014	DEPTH	14-19'
PROJECT	The Launchpad	SAMPLE NO.	--
PROJECT NO.	CS19243-125	DATE SAMPLED	--
LOCATION	--	DESCRIPTION	--
DATE TESTED	08/18/22		
TECHNICIAN	JL		



NOTES	Sample swelled during consolidation, shear rate calculated for 24 hour shear per ASTM D3080. Single moisture content taken for A and B points.

Picture File: P8220931.JPG
File name: 2843014__Direct Shear ASTM D3080_0.xlsm

ADVANCED TERRA TESTING

Direct Shear
ASTM D3080
(Point A Picture 2)

CLIENT	CTL Thompson	BORING NO.	TH-4
JOB NO.	2843-014	DEPTH	14-19'
PROJECT	The Launchpad	SAMPLE NO.	--
PROJECT NO.	CS19243-125	DATE SAMPLED	--
LOCATION	--	DESCRIPTION	--
DATE TESTED	08/18/22		
TECHNICIAN	JL		

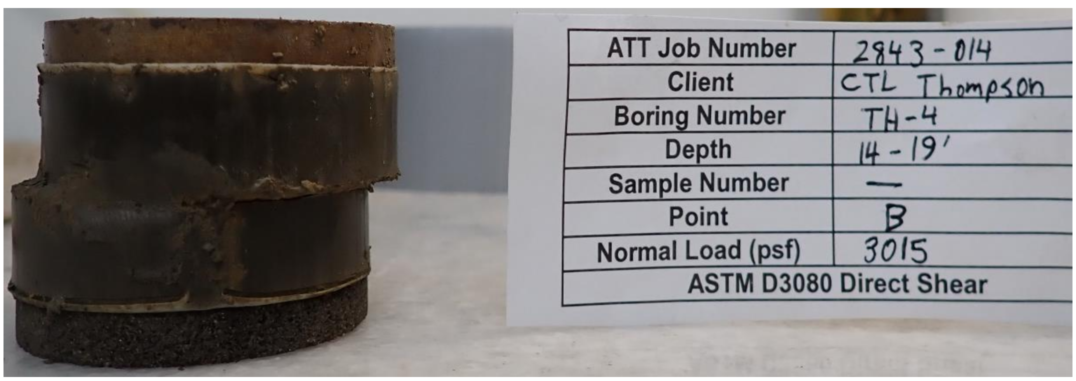
NOTES	Sample swelled during consolidation, shear rate calculated for 24 hour shear per ASTM D3080. Single moisture content taken for A and B points.

Picture File: P8220933.JPG
File name: 2843014__Direct Shear ASTM D3080_0.xlsm

Direct Shear
ASTM D3080
(Point B Picture 1)

ADVANCED TERRA TESTING

CLIENT	CTL Thompson	BORING NO.	TH-4
JOB NO.	2843-014	DEPTH	14-19'
PROJECT	The Launchpad	SAMPLE NO.	--
PROJECT NO.	CS19243-125	DATE SAMPLED	--
LOCATION	--	DESCRIPTION	--
DATE TESTED	08/18/22		
TECHNICIAN	JL		



NOTES	Sample swelled during consolidation, shear rate calculated for 24 hour shear per ASTM D3080. Single moisture content taken for A and B points.

Picture File: P8230942.JPG
File name: 2843014__Direct Shear ASTM D3080_0.xlsm

ADVANCED TERRA TESTING

Direct Shear
ASTM D3080
(Point B Picture 2)

CLIENT	CTL Thompson	BORING NO.	TH-4
JOB NO.	2843-014	DEPTH	14-19'
PROJECT	The Launchpad	SAMPLE NO.	--
PROJECT NO.	CS19243-125	DATE SAMPLED	--
LOCATION	--	DESCRIPTION	--
DATE TESTED	08/18/22		
TECHNICIAN	JL		

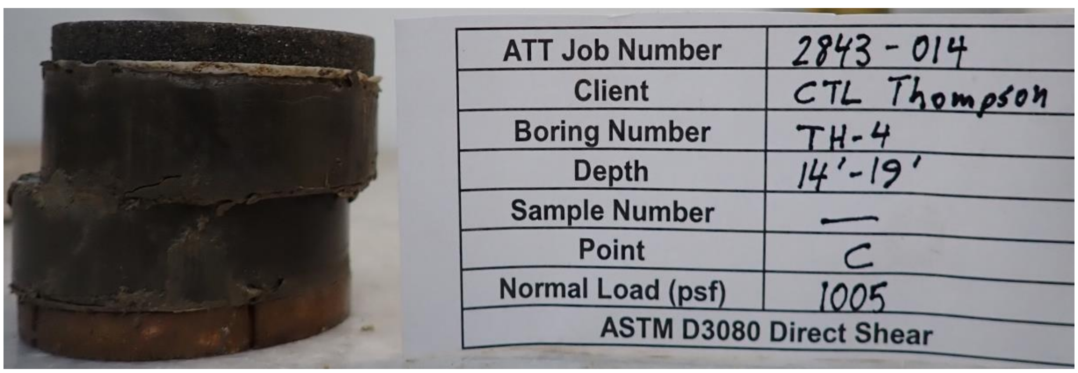
NOTES	Sample swelled during consolidation, shear rate calculated for 24 hour shear per ASTM D3080. Single moisture content taken for A and B points.

Picture File: P8230943.JPG
File name: 2843014__Direct Shear ASTM D3080_0.xlsm

Direct Shear
ASTM D3080
(Point C Picture 1)

ADVANCED TERRA TESTING

CLIENT	CTL Thompson	BORING NO.	TH-4
JOB NO.	2843-014	DEPTH	14-19'
PROJECT	The Launchpad	SAMPLE NO.	--
PROJECT NO.	CS19243-125	DATE SAMPLED	--
LOCATION	--	DESCRIPTION	--
DATE TESTED	08/18/22		
TECHNICIAN	JL		



NOTES	Sample swelled during consolidation, shear rate calculated for 24 hour shear per ASTM D3080. Single moisture content taken for A and B points.

Picture File: P8240957.JPG
File name: 2843014__Direct Shear ASTM D3080_0.xlsm

Direct Shear
ASTM D3080
(Point C Picture 2)

ADVANCED TERRA TESTING

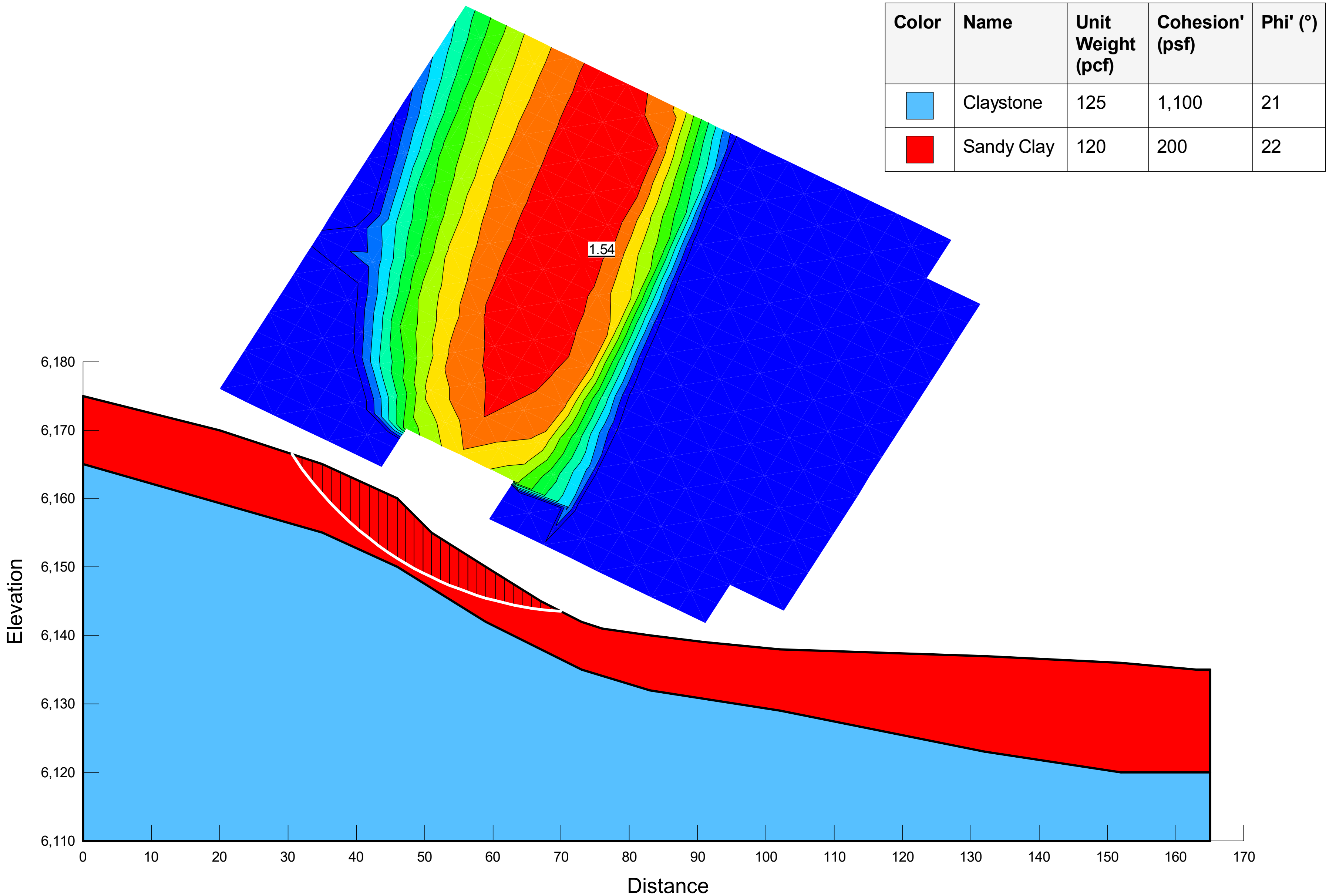
CLIENT	CTL Thompson	BORING NO.	TH-4
JOB NO.	2843-014	DEPTH	14-19'
PROJECT	The Launchpad	SAMPLE NO.	--
PROJECT NO.	CS19243-125	DATE SAMPLED	--
LOCATION	--	DESCRIPTION	--
DATE TESTED	08/18/22		
TECHNICIAN	JL		

NOTES	Sample swelled during consolidation, shear rate calculated for 24 hour shear per ASTM D3080. Single moisture content taken for A and B points.

Picture File: P8240958.JPG
File name: 2843014__Direct Shear ASTM D3080_0.xlsm

APPENDIX D

RESULTS OF SLOPE STABILITY ANALYSIS



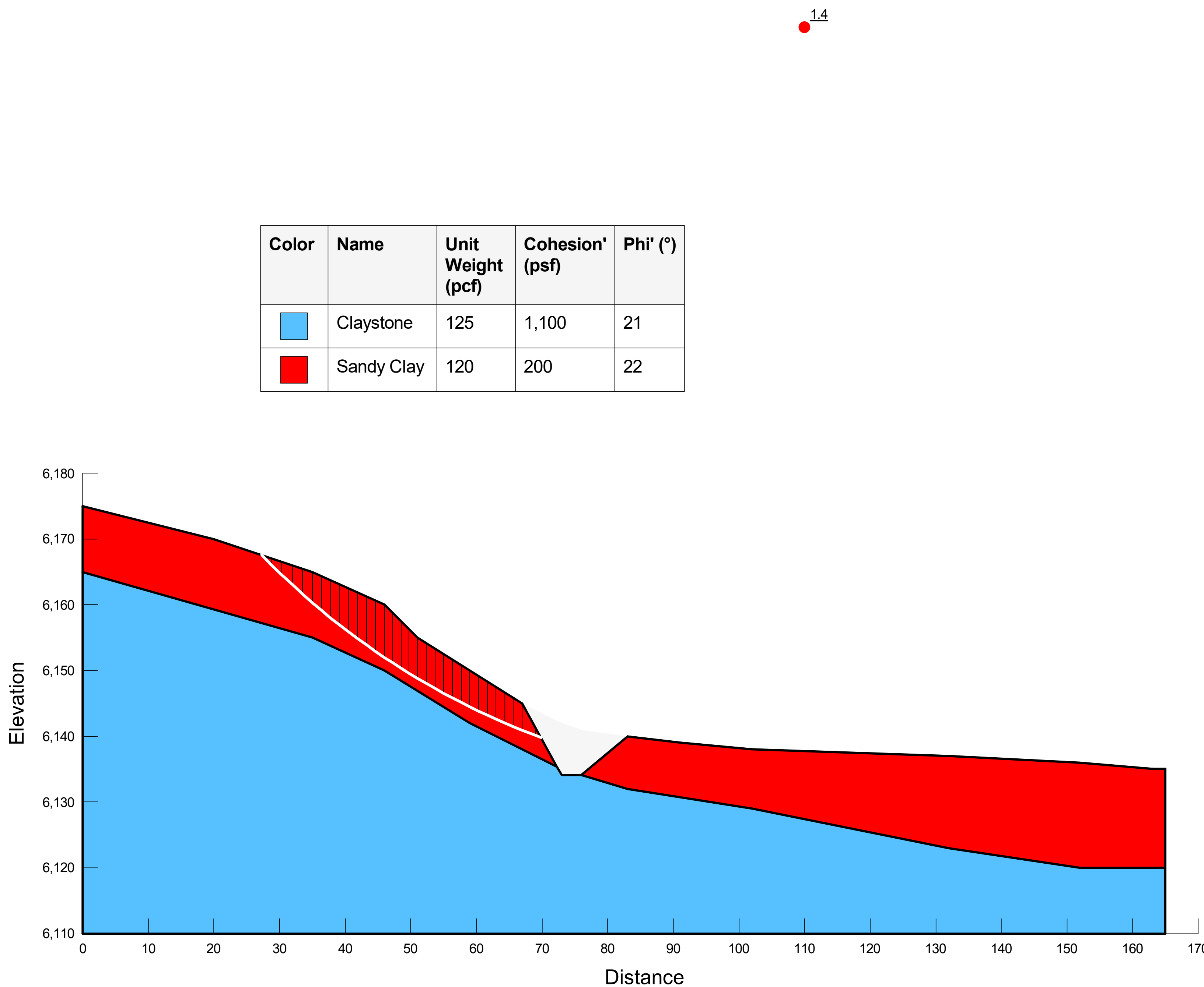
Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Claystone	125	1,100	21
	Sandy Clay	120	200	22

EXISTING CONDITIONS (SECTION A-A')

COHEN ESREY
THE LAUNCHPAD - YOUTH HOUSING
CS19543-125

FIG. D-1

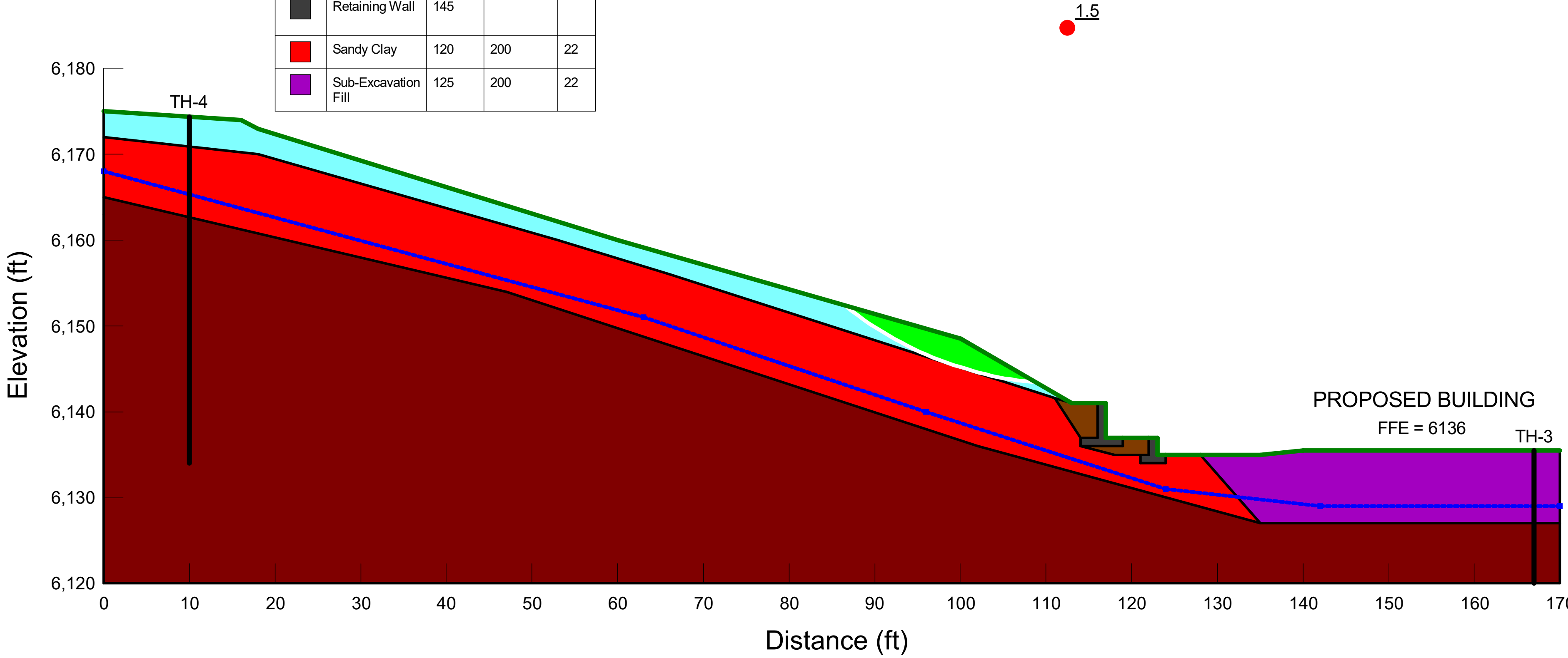
Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Claystone	125	1,100	21
	Sandy Clay	120	200	22



TEMPORARY CONSTRUCTION CONDITION (SECTION A-A')

COHEN ESREY
THE LAUNCHPAD - YOUTH HOUSING
CS19543-125

FIG. D-2



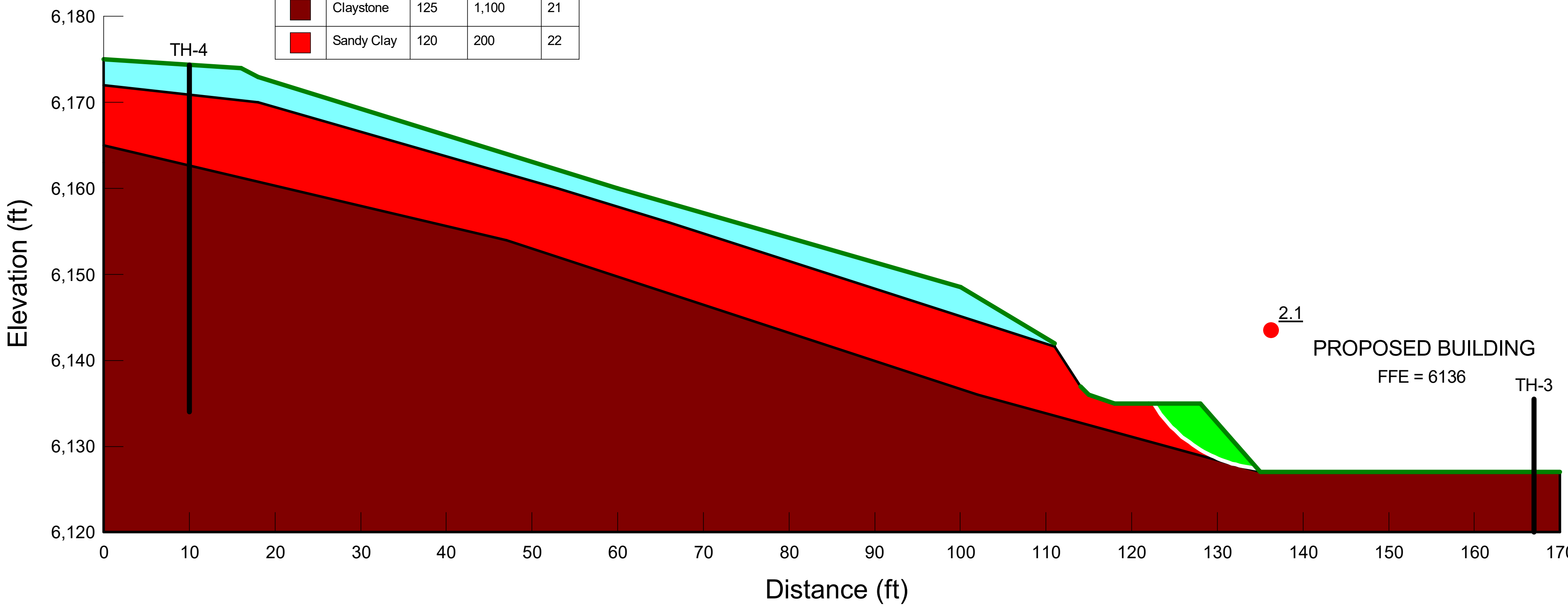
Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Backfill	125	0	34
	Clayey Sand	125	20	26
	Claystone	125	1,100	21
	Retaining Wall	145		
	Sandy Clay	120	200	22
	Sub-Excavation Fill	125	200	22

PROPOSED CONSTRUCTION (SECTION B-B')

COHEN ESREY
THE LAUNCHPAD - YOUTH HOUSING
CS19543-125

FIG. D-3

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Clayey Sand	125	20	26
	Claystone	125	1,100	21
	Sandy Clay	120	200	22



TEMPORARY CONSTRUCTION CONDITION (SECTION B-B')

COHEN ESREY
THE LAUNCHPAD - YOUTH HOUSING
CS19543-125

FIG. D-4