GEOLOGIC HAZARD EVALUATION AND GEOTECHNICAL INVESTIGATION ARROWSWEST APARTMENTS GARDEN OF THE GODS AND 30TH STREET COLORADO SPRINGS, COLORADO

Prepared for:

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SCOPE

This report presents the results of our Geologic Hazard Evaluation and Geotechnical Investigation for the proposed Arrowswest Apartments to be constructed south of the intersection of Garden of the Gods Road and 30th Street in Colorado Springs, Colorado (Fig. 1). The purpose of our investigation was to evaluate general geologic and surficial subsoil conditions for their potential influence on development and construction and to provide geotechnical recommendations and criteria for design and construction of foundations, floor systems, as well as surface drainage precautions. The scope was described in our proposal (CS-20-0090) dated July 17, 2020. Evaluation of the property for the possible presence of potentially hazardous materials (Environmental Site Assessment) was beyond the scope of this investigation.

This report is based on conditions found in our exploratory borings, results of field and laboratory tests, engineering analysis, and our experience with similar conditions and projects. It includes our recommendations for design criteria and construction details for foundations and floor slabs, lateral earth loads, and drainage precautions, as well as a geologic hazards evaluation. The design criteria presented in the report are based on our understanding of the planned construction. It is noted that site grading plans, finish floor elevations, and structural information were not available at the time of this report. Once design data for the project become available, we should be retained to review these plans and update geotechnical recommendations and our slope stability analyses included in this report as necessary. The following section summarizes the report. More detailed descriptions of subsurface conditions, as well as our design and construction recommendations, are presented in the report.

SUMMARY

1. Subsurface conditions encountered in our exploratory borings drilled for the proposed buildings consisted of areas of surficial clay fill up to 16.5 feet thick and areas of natural clay and sand at the surface. The surficial material was underlain by steeply dipping claystone bedrock at depths ranging between 2 and 23 feet below existing grade. Samples of the clay

fill and claystone bedrock tested in our laboratory exhibited moderate to very high measured swell values when wetted under estimated overburden pressures (the weight of the overlying soils). Steeply dipping shale bedrock was encountered underlying claystone in portions of the site at depths ranging from 13 to 26 feet below existing grade.

2. At the time of drilling, groundwater was encountered in one of the borings at a depth of 24 feet below the existing ground surface. When checked eleven days after the completion of drilling, groundwater was measured in nine of the borings at depths of 11 to 24 feet below the existing ground surface. Groundwater levels will fluctuate seasonally and rise in response to precipitation and landscape irrigation.

3. The presence of expansive soil and bedrock constitutes a geologic hazard. There is risk that the expansive material will heave and damage slabs-on-grade and foundations. We believe the recommendations presented in this report will help to control risk of damage; they will not eliminate that risk. Slabs-on-grade and possibly the foundation may be damaged.

4. Potentially unstable slopes are considered a geologic hazard. Our analysis indicates the slope present on this site is stable in the existing condition. We believe the proposed construction will not destabilize the slope.

5. We believe the proposed buildings can be constructed with a minimum dead load spread footing underlain by sub-excavation fill consisting of a 10-foot thick layer of moisture conditioned and densely compacted on-site materials. The proposed swimming pool should also be underlain by a 10-foot thick layer of sub-excavation fill.

6. Based on our understanding of the proposed construction, subsurface conditions encountered in our borings, and the results of laboratory testing, we believe a low risk of differential movement and damage will exist for slab-on-grade floor systems if the slabs are underlain by a layer of newly moisture conditioned and densely compacted fill. The risk of damage is discussed in the report.

7. We believe the parking areas can be paved with 4 inches of asphalt over 6 inches of aggregate base course and access driveways can be paved with 5 inches of asphalt over 6 inches of aggregate base. 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 for the rapid removal of runoff away from the buildings 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 for foundations and slabs-on-grade included 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 structures, use prudent irrigation practices, and maintain surface drainage. It is critical that all recommendations in this report are followed.

SITE CONDITIONS AND BACKGROUND INFORMATION

Google Earth® Aerial Image October, 2019

The site is located south of the intersection of Garden of the Gods Road and 30th Street in Colorado Springs, Colorado. The general vicinity of the property is shown in Fig. 1. The site encompasses approximately 9.5 acres of the Arrowswest Development and partially surrounds an existing gas station and convenience store. An existing asphalt paved driveway divides the site into northern and southern portions. The site has experienced previous grading activity associated with the realignment of Garden of the Gods Road as well as overall development of the Arrowswest project. Garden of the Gods Road originally extended directly east to west across the north end of the site and was realigned around 1984 to its current configuration. The site generally exhibits a gentle slope descending to the northeast, with the exception of the southwest end of the site where a moderately steep slope ascends southwest to a mesa. Original slopes leading to the top of the mesa varied between approximately 3:1 (horizontal to vertical) and 6:1. Site development activities associated with the Arrowswest project performed in late 1983 and early 1984 resulted in cuts at the toe of the slope, steepening the lower portion of the slope to a 2:1 ratio.

At the time of drilling, the site area was primarily covered with grasses and scattered yucca plants. The site is bordered to the northeast by Garden of the Gods Road, to the east by Arrowswest Drive and undeveloped land, to the south by undeveloped land, and to northwest by 30th Street.

PROPOSED CONSTRUCTION

Based on the preliminary development sketch provided to us, we understand the apartment project will include seven low-rise apartment buildings. The preliminary configuration of the apartment complex is shown in Fig. 1.

The apartment buildings are expected to be 2 to 3-story, wood-framed structures with some buildings along the southwest portion of the site stepping down with the uphill side having 2-stories and the downhill side having 3-stories. Based on conversations with you and the project architect we understand the sides of the buildings nearest the slope (2-story side) will be filled about 4 feet while the downhill side (3-story side) of the building will be cut, and a below-grade retaining wall will extend longitudinally through the center of the buildings.

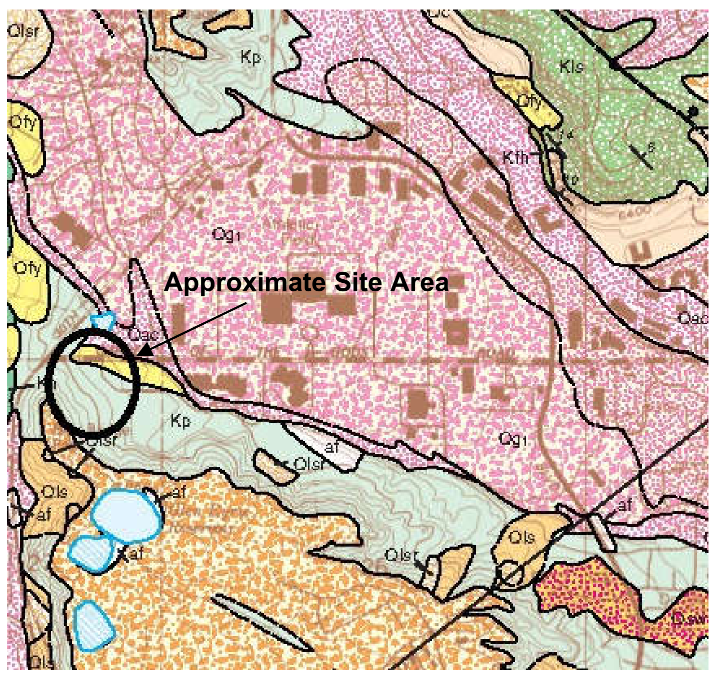
A single-story clubhouse building with an outdoor swimming pool is planned in the eastern portion of the development. Foundation loads are expected to be light to moderate. We assume slab-on-grade floors are the preferred floor systems for this project. Paved access driveways and automobile parking stalls are planned throughout the complex and will extend from the existing driveway that extends from North 30th Street to Arrowswest Drive.

PREVIOUS INVESTIGATIONS

CLT|Thompson has been involved with the overall Arrowswest Development since 1981. Numerous previous investigations have been completed at this site and in the general vicinity of this site. In January 1999, CLT | Thompson, Inc. prepared a Geologic Hazards Investigation (CTL Job No. CS-9300, report dated January 1999) that included the southeastern portion of the site. In May 2000, CLT|Thompson prepared a Pre-Purchase Geotechnical Investigation (CTL Job No. CS-10,378) for the site. In April 2002, CLT|Thompson, Inc. prepared a Geotechnical Investigation (CTL Job No. CS-12,228) for the site for a proposed project called Shops at Arrowswest which was to occupy the entire site including the area currently occupied by the existing gas station and convenience store. In May 2008 CLT | Thompson prepared an additional Geotchnical Investigation for the proposed Shops at Arrowswest (CLT | Thompson Project No. CS16862-125) The project as proposed was never constructed. The results of the previous investigations were utilized to supplement the data for this evaluation and the summary logs from exploratory borings advanced for these previous projects are attached in Appendix D.

GEOLOGY

Geologic conditions were evaluated through the review of published geologic maps, field reconnaissance, and our subsurface investigations. Information from these sources was used to produce our interpretation of site geology, as shown in Fig. 2. A list of references is included at the end of this report.



Geologic Map

Based on geologic mapping by Thorson and others (Thorson, 2001), the site is mapped predominantly as Kp, Pierre Shale Formation. The Pierre Shale is an Upper Cretaceous aged marine shale interbedded with bentonite, sandstone, and siltstone beds. This formation typically weathers near the ground surface to materials classified by engineering terms of clay, weathered claystone, and claystone as well as shale. The bedding within the Pierre Shale Formation in the general site area is nearly vertical. We have previously measured bedrock dip attitudes in the Arrowswest Development between 82 degrees from the horizontal to nearly vertical. The bedding strike is nearly north to south.

Holocene-age fan deposits, Qfy, are present in the northern portion of the site near the old alignment of Garden of the Gods Road. The fan deposits consist of sediments deposited by debris flows, hyper-concentrated flows, and sheetwash; comprised of poorly sorted, occasional clast-supported, pebble, cobble, and boulder gravel in a sandy matrix to matrix-supported, clayey sand and gravel. Unit is frequently cobbly and boulder, particularly near the heads of fans. Deposits tend to be finer grained in the distal ends of fans that include debris flow and sheetwash deposition.

Holocene-age, undivided alluvium and colluvium is also mapped at the far north end of the site. The conditions encountered in our borings generally match the mapped units. However, we found surficial alluvial deposits overlying the Pierre Shale formation and also large areas of the site have experienced previous grading activity and are overlain with fill material that was placed as part of the site development activities in late 1983 and early 1984. Colluvium derived from in-place weathering of bedrock deposited by gravity and sheetwash was found near the toe of the slope in the southwest portion of the site.

SUBSURFACE INVESTIGATION

We investigated subsurface conditions for the proposed apartment project by recently drilling and sampling twenty (20) exploratory borings within the proposed building footprints and five shallow pavement borings in the proposed pavement areas, as shown on Fig. 1. The borings within the building areas were advanced to depths ranging between 20 and 30 feet. The pavement borings were advanced to depths of 5 feet. All borings were drilled using 4-inch diameter, continuous-flight, solid-stem auger and a truck-mounted drilling rig.

Samples of the soils were obtained at 2 to 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 of the borings, log the soils encountered, and obtain samples for laboratory tests. A graphical log of the exploratory borings, including the results of field penetration resistance tests and a portion of the laboratory data, are presented in Fig. 2.

Soil samples obtained during drilling were returned to our laboratory and visually classified. Laboratory testing was then assigned to representative samples and included moisture content and dry density, swell-consolidation, Atterberg limits, sieve analysis (percent passing the No. 200 sieve), and water-soluble sulfate concentration tests. The swell test samples were wetted under applied loads that approximated the overburden pressure (the weight of overlying soil). Results of the laboratory tests are presented in Appendix B and summarized in Table B-1.

SUBSURFACE CONDITIONS

Subsurface conditions encountered in the exploratory borings drilled within the proposed building footprints consisted of widespread areas of clay fill and/or natural sandy and clay soils at the surface underlain by claystone and shale bedrock. The claystone and shale bedrock exhibit near vertical bedding. Some pertinent engineering characteristics of the surficial soils and bedrock encountered, as well as groundwater conditions, are described in the following paragraphs.

Existing Fill

Approximately 3 to 16.5 feet of sandy to very sandy clay fill was encountered at the ground surface in eleven of our borings. The sandy to very sandy clay fill was very stiff, based on the results of field penetration resistance tests. Samples of the fill tested in our laboratory contained 56 to 87 percent clay and silt-sized particles (passing the No. 200 sieve). Two samples of the fill subjected to swell-consolidation testing in our laboratory exhibited measured swells of 9.7 and 10.8 percent when wetted under approximate overburden pressure indicative of very high expansion potential.

The fill within the site was placed as part of overall grading operations performed at the Arrowswest Industrial Park in late 1983 and early 1984. Representatives of our office were on-site periodically during grading to observe the contractor's methods and to perform field density testing of compacted fills. Grading included making cuts at the toe of the slope that ascends south to a mesa and placing the material as fill in the north and east portions of the site. Our borings indicate up to approximately 16.5 feet of fill was placed in the northern portion of the site with lesser amounts placed in the eastern portion of the site adjacent to Arrowswest Drive. Because of the lack of development and irrigation on the site after site grading and fill placement activities, the fill has lost a significant amount of moisture and therefore exhibited very high measured swell when tested in our laboratory.

Natural Clay

Natural slightly sandy to sandy clay with occasional gravelly lenses was encountered sporadically either beneath the existing fill or at the surface. The natural clay was very stiff based on the results of field penetration resistance testing. Four samples of the clay tested in our laboratory contained 83 to 91 percent clay and silt-sized particles (passing the No. 200 sieve). Three samples exhibited measured swells of between 2.8 and 7.0 percent when wetted under approximate overburden pressure.

Natural Sand

Natural clayey sand was encountered locally at the surface or beneath fill. The natural clayey sand clay was medium dense based on the results of field penetration resistance testing. One sample of the clayey sand tested in our laboratory contained 27 percent clay and silt-sized particles (passing the No. 200 sieve). Our experience indicates the clayey sand exhibits low expansion potential.

Bedrock

Steeply dipping claystone bedrock was encountered beneath existing fill or natural soils in eighteen of the twenty borings advanced in the building areas at depths between 2 and 23 feet below the existing ground surface. The claystone beds are near vertical, exhibit varying degrees of weathering, and was medium hard to very hard based on the results of field penetration resistance testing. Seven samples of the claystone exhibited measured swells ranging between 1.0 and 5.6 percent when wetted under approximate overburden pressure. Shale bedrock with near vertical beds was encountered in eight of our borings underlying claystone at depths ranging between 13 to 26 feet. The shale is very hard based on results of field penetration resistance testing.

Groundwater

At the time of drilling, groundwater was encountered in boring TH-2 at a depth of 24 feet below the existing ground surface. When checked ten days after the completion of drilling, groundwater was measured in nine of the borings at depths of 11 to 24 feet below existing ground surface. Groundwater levels will fluctuate seasonally and rise in response to precipitation and landscape irrigation.

POTENTIAL GEOLOGIC HAZARDS AND ENGINEERING CONSTRAINTS

We did not identify geologic conditions we believe preclude development of this site as planned. We identified several geologic and man-made conditions we believe will pose restrictions on construction and influence foundation designs. These are: the presence at shallow depths of steeply dipping expansive bedrock with very high expansion potential as well as clay fill materials with very high expansion potential; 2:1 grading cut-slopes; and the regional geologic conditions of radioactivity (radon) and seismicity. These conditions are discussed in greater detail in the following sections. We believe these conditions can be mitigated with engineering design and construction methods commonly employed in this area.

Expansive Claystone with Steeply Dipping Bedding Planes

Our investigation indicates expansive claystone with steeply dipping bedding planes is widespread across the site and often near anticipated foundation elevations. The area is included in the “Map of Areas Susceptible to Differential Heave in Expansive, Steeply Dipping Bedrock, City of Colorado Springs, Colorado” by Himmelreich Jr. and Noe, dated 1999. We have seen a higher instance of foundation movement and damage for structures constructed in areas with steeply dipping expansive bedrock along the Front Range area where sufficient mitigation measures are not taken. This is predominantly due to larger vertical differential movements over relatively short horizontal distances. The larger differential movements are caused by changes in the swelling characteristics of the subsoils that can occur over relatively short horizontal distances due to the angle of the bedding planes. We have also seen an increase in the depth of wetting in areas of steeply dipping bedrock that can increase the magnitude of movement of expansive soils. Mitigation is discussed in the **SITE DEVELOPMENT** section.

Potentially Unstable Slopes and Hillside Overlay Zone

The southwestern portion of the site lies within the Hillside overlay zone of Colorado Springs. The moderately steep slope that ascends up the mesa in the southwestern part of the site is considered potentially unstable due to the steepness. The Landslide Susceptibility Map of Colorado Springs (by White & Wait) indicates the slope is in 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 structures, only that a higher risk exists compared to areas not mapped as susceptible. We did not observe indications of land sliding or slope instability at this property. A number of factors at this site suggest a low risk of a deep seated slope failure including: the slope orientation descends to the northeast while the bedrock dips at an angle in excess of 80 degrees, groundwater lies at a depth that is unlikely to impact materials in the slope, and the bedrock is well-indurated and very hard. In addition, we have extensively evaluated the slopes of the mesa associated with work we have performed for single-family residence construction occurring on top of the mesa as part of the North Point at Kissing Camels development. We performed numerous slope stability analyses for evaluation of the single-family lots and results indicate stable slope conditions.

We understand site retaining walls are not planned adjacent to the slope behind the buildings, however based on the preliminary development sketch a site retaining wall may be necessary in the south corner of the site adjacent to the planned parking area. Installation of retaining walls at the toe of the 2:1 slope is not suggested. If site retaining walls are necessary, we recommend raising the grades and setting walls back sufficiently from the slope to provide a slope no steeper than 3:1 adjacent to any site retaining walls.

During construction, provided the soils do not move laterally and moisture contents remain near current conditions (soils do not become saturated), cut slopes for foundations exposed for short periods during construction that are laid back at 1.5:1 (horizontal to vertical) or flatter in natural soils and 0.75:1 for claystone and sandstone bedrock should be stable. Cut material should not be stockpiled adjacent to the excavation.

In all circumstances, surface drainage must be planned to eliminate ponding or potential ponding of water and provide for rapid removal of runoff. Berms, ditches, and similar means could be used to decrease the potential for stormwater entering the work area and to efficiently convey it away from the building site. Failure to control surface drainage during construction could result in construction delays, require reworking of materials, and/or cause slope instability.

Shallow Groundwater

We measured groundwater in nine of our recent borings at depths of 11 to 24 feet. Our experience indicates the groundwater is likely flowing through cracks and fissures in the bedrock. Groundwater may be encountered sporadically in grading where deep sub-excavations are performed into the bedrock. Perimeter drains for structures with below-grade areas are recommended. Drains near the toe of the slopes may be appropriate to help mitigate groundwater.

Radioactivity/Radon

In our opinion, there are no unusual hazards from naturally occurring sources of radioactivity on this site. However, the type of materials found often are associated with the production of radon gas, and concentrations in excess of those currently accepted by the EPA are possible. Buildup of radon gas to unacceptable levels often occurs in residential structures that are sealed to minimize air exchange. Passive and active mitigation procedures are commonly employed in this region to effectively reduce the buildup of radon gas. Passive mitigation includes installing vents connected to the foundation drain and increasing ventilation of crawl spaces. More active measures that can be taken after dwelling construction include installing a blower connected to the foundation drain vent and sealing the joints and cracks in concrete floors and foundation walls. Because of the numerous variables involved, we recommend the dwelling be tested after construction to evaluate radon levels, and then explore more active measures of mitigation, if needed.

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. The shale is very hard bedrock and will be very difficult to excavate; however, based on our understanding of the project and the depth to shale encountered in our borings we do not expect significant excavation into shale bedrock.

Seismicity

This area, like most of central Colorado, is subject to a degree of seismic activity. The soil is not expected to respond unusually to seismic activity. According to the 2015 International Building Code (2015 IBC) and based on the results of our investigation, we judge the site classifies as Seismic Site Class D over most of the site. The southern third of the site likely classifies as Site Class C due to the relatively shallow depth of claystone and shale.

Flooding and Streamside Overlay Zone

This site lies within Zone X as shown on Flood Insurance Rate Maps prepared by the Federal Emergency Management Agency. Zone X is outside the 500-year flood plain with no base flood determined. Based on the topography at the site the potential for a flood to impact the building area of the site is very low. There are no streamside overlay zones within the site shown on the Streamside Overlay Map available on the City of Colorado Springs web page.

Subsurface and Surface Mining

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.

SLOPE STABILITY ANALYSIS

We conducted slope stability analyses along one cross-section for the steeper portion of the slope located at the southwest end of the site. The approximate location of the cross-section is presented on Fig. 1 as cross-section A to A'. The cross-section considered what we interpret as the critical cross section extending down the steeper portion of the slope and through a proposed 2 to 3-story split building. Our analysis also considered the excavation into the slope that will be necessary during sub-excavation of expansive soils and bedrock.

The on-site materials were assigned unit weights and shear strength parameters based on the results of our laboratory testing and on 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 Table 1.

Table 1 – Strength Parameters Used in Slope Stability Analysis

Earth Material	Total Unit Weight (pcf)	Cohesion (psf)	Friction Angle (degrees)
Compacted Fill	125	200	24
Natural Sand	125	50	34
Claystone	130	500	24
Shale	130	5,000	15
Notes: pcf – pounds per cubic foot, psf – pounds per square foot			

We performed a slope stability analysis considering the excavated condition of the slope during sub-excavation and the final slope condition with the an apartment building constructed. Although not expected to be present near the surface within the slope, groundwater was modeled in the slope above the bedrock to evaluate impacts of an increased groundwater level. 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 adequate factors of safety (i.e., about 1.3 for the temporary construction slope and 1.7 for the permanent slope condition). The results of our slope stability analyses are presented in Appendix C. 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 slope stability during and after construction will be stable. Once site grading plans and finish floor elevations are available, we should be contacted to review the plans and determine if additional analysis is required.

SITE DEVELOPMENT

Based on existing site grades and our understanding of the planned construction, we anticipate cuts and fills on the order of 10 feet or less will be necessary to achieve the proposed grades. Highly expansive clay soils and steeply dipping claystone bedrock are present at this site. Sub-excavation and re-compaction as described below should be completed to allow the use of spread footings and slabs-on-grade.

Sub-excavation

Our investigation indicates predominately highly expansive clay and moderately to highly expansive, steeply dipping, claystone bedrock is present at depths likely to affect the foundation performance. We estimate total potential ground heave of up to about 7.5 inches based on a 15-foot depth of wetting. To reduce the impact of the expansive materials on foundation and slab performance and create a more uniform layer of support, the clay fill and steeply-dipping claystone bedrock should be removed throughout the entire building footprint of each structure to a uniform depth of 10-feet below the lowest exterior footing elevation. The sub-excavation zone should extend outward at least 5 feet beyond outer edges of the footings. The boring advanced within the pool area also indicates highly expansive soils are present. Accordingly, the area of the pool should also be sub-excavated to a uniform depth of 10-feet below the lowest pool bottom elevation and extending at least 5 feet beyond the pool walls on all sides. The clay fill that has been sub-excavated may be moisture conditioned, and placed back into the excavation as densely compacted fill as described in the **Fill Placement** section of the report.

After sub-excavation, it is important that measures be planned to reduce excessive drying or wetting of the near-surface soils. If the sub-excavation fill dries excessively or softens due to wetting prior to building construction, it may be necessary to rework the upper, drier or softened materials prior to construction of the proposed improvements.

Excavation

We believe the soils encountered in the exploratory borings can be excavated with conventional, heavy-duty excavation equipment. Zones of very 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

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 should exhibit a Liquid Limit of less than 30 and a Plasticity Index of less than 15. Soils similar to the on-site soils may be suitable. A sample of any potential imported fill material should be submitted to our office for testing, prior to its use at the site.

Prior to fill placement, existing pavement, organic materials, and topsoil must be removed from the ground surface. Areas to receive fill should be scarified to a depth of 8 inches, moisture conditioned, and compacted to provide a firm subgrade surface prior to fill placement. Fill placed within the building footprints and swimming pool area should be placed in thin (8 inches or less), loose lifts that have been moisture conditioned to within 1 to 4 percent above optimum moisture content and then compacted to at least 95 percent of maximum standard Proctor dry density (ASTM D 698). Fill placed outside of the building footprints and swimming pool area should be in thin, loose lifts that have been moisture conditioned to within 2 percent of optimum moisture and then compacted to at least 95 percent of maximum standard Proctor dry density (ASTM D 698). Placement and compaction of the fill should be observed and tested by a representative of our firm during construction.

At the time of our investigation the onsite materials were relatively dry and well below optimum moisture content. The addition of significant moisture will be needed to raise the moisture contents of the surficial clay and excavated bedrock. Processing of these materials will be needed to break down the bedrock to 3-inch or small fragments prior to placement. Disking through each lift during moisture conditioning will aid in processing to achieve consistent moisture control through the fill. Moisture conditioning of soils being stockpiled will improve distribution of the moisture in the soils. It should be understood that sub-excavation and moisture conditioning will reduce but not eliminate swell potential.

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.

To reduce the potential for pumping and rutting of pavement subgrade soils, fill placed within the pavement areas and as backfill in trenches located in pavement areas should be moisture conditioned to within 2 percent of optimum moisture content.

FOUNDATIONS

In our opinion, the proposed buildings can be constructed on spread footing foundations designed to apply a minimum deadload pressure if sub-excavation is performed as described in this report. Following sub-excavation, ground heave of about 1.5 inches may still be possible; however, the total heave is driven by soils below the 10-foot sub-excavation. Due to the separation and layer of new fill, we anticipate structures will perform satisfactorily, with a reduced potential for differential movements. Design criteria for spread footing foundations developed from analysis of field and laboratory data and our experience are presented below.

Spread Footings

1. Spread footings for buildings should be constructed on new moisture conditioned, and densely compacted fill materials as described previously in the **SITE DEVELOPMENT** section of the report.
2. Footings should be designed for a maximum allowable soil pressure of 3,000 psf and a minimum deadload pressure of 1,000 psf. If interrupted footings are necessary to maintain the recommended deadload, a 8-inch (or thicker) void should be constructed below grade beams or foundation walls, between the pads.
3. Footings should have a minimum width of 16 inches. Foundations for isolated columns should have minimum dimensions of 24 inches by 24 inches. Larger sizes may be required depending on the loads and structural systems used.
4. Foundation walls should be well-reinforced. We recommend reinforcement sufficient to span an unsupported distance of at least 10 feet. Reinforcement should be designed by the structural engineer considering lateral earth pressure on wall performance.
5. We recommend designs consider total movement of 1-1/2 inches and differential movement of 3/4-inch.
6. Exterior footings must be protected from frost action. Normally, 30 inches of frost cover is assumed in the area.

7. A representative of our firm should observe the completed foundation excavations (bottom of the sub-excavation zone) to confirm the exposed conditions are similar to those encountered in our exploratory borings. The placement and compaction of below-footing fill and footing subgrade preparation should be observed and tested by a representative of our firm during construction.
8. Excessive wetting of foundation soils during and after construction can cause heave or softening and settlement of foundation soils and result in footing and slab movements. Proper surface drainage around the structures is critical to control wetting.

FLOOR SYSTEMS

We recommend the floor slabs be underlain by new, sub-excavation fill as described in the **SITE DEVELOPMENT** section of this report. In our opinion, a low risk of poor, long-term slab performance (movement and damage) will exist for conventional floor slabs that are underlain by a layer of soil that has been modified using the recommended sub-excavation procedure. We anticipate the slab movement to be less than about 1.5-inches. Our representative should observe and test compaction of fill during construction.

Differential movements of the slabs can deform and crack the floor slabs and can damage interior partitions. To our knowledge, there are no soil treatments combined with a slab-on-grade floor that will result in the same reduction in risk of floor movement as would be provided by a structural floor (crawl space construction).

All parties must realize that even small movements of the floor slabs (less than 1-inch) can damage comparatively brittle floor treatments (if any), such as ceramic tile. If the owner elects to use slab-on-grade construction and accepts the risk of movement and associated damage, we recommend the following precautions for slab-on-grade construction at this site. These recommendations will not prevent movement. Rather, they tend to reduce damage if movement occurs.

1. We recommend floor slabs be underlain by new, moisture conditioned and densely compacted fill as discussed in the **SITE DEVELOPMENT** section.

New fill placed below floor slabs should consist of the on-site soils or an approved, import material. The placement and compaction of the below-slab fill should be observed and tested by a representative of our office during construction, and should be controlled per the **<u>Fill Placement</u>** section of this report.

2. We recommend slab-on-grade floors be separated from exterior walls and interior bearing members with joints that allow for independent vertical movements of the slab relative to the foundation. Provision of a 2-inch thick slip joint in slab-bearing partitions can reduce the risk of drywall cracking that results from slab movements. If the “float” is provided at the top of the partitions, the connection between interior, slab-supported partitions and exterior, foundation-supported walls should be detailed to allow differential movement. These architectural connections are critical to help reduce cosmetic damage should foundations and floor slabs move relative to each other. We have seen instances where these architectural connections were not designed and constructed properly and resulted in moderate cosmetic damage, even though the movement experienced was well within the anticipated range. The architect should pay special attention to these issues and detail the connections accordingly. Masonry block partitions (load bearing or not) should be constructed on their own independent foundations and not a thickened floor slab.

3. Underslab plumbing should be avoided as much as possible. If underslab plumbing is necessary, service lines should be pressure tested for leaks during construction. Any utility line that penetrates a slab should be isolated from the slab with a joint to allow for free vertical movement.

4. Slab-supported mechanical systems should have flexible connections to allow for vertical movement without rupturing supply lines.

5. From a geotechnical viewpoint, we believe the floor slabs can be placed directly on the subgrade soils. The 2015 International Building Code (IBC) requires a vapor retarder be placed between base course or subgrade soils and the concrete slab-on-grade floor, unless the designer of the floor waives this requirement. The merits of installation of a vapor retarder below a floor slab depend on the sensitivity of floor coverings and building use to moisture. A properly installed vapor retarder (10 mil minimum) is more beneficial below concrete slab-on-grade floors where floor coverings, painted floor surfaces or products stored on the floor will be sensitive to moisture. The vapor retarder is most effective when concrete is placed directly on top of it, rather than placing a sand or gravel leveling course between the vapor retarder and the floor slab. The placement of concrete on the vapor retarder may increase the risk of shrinkage cracking and curling. Use of concrete with reduced shrinkage characteristics including minimized water content, maximized coarse aggregate content, and reasonably low slump will reduce the risk of shrinkage cracking and curling. Considerations and recommendations for the installation of vapor retarders below concrete slabs are outlined in Section 3.2.3 of 2006 report of the

American Concrete Institute (ACI) Committee 302, "Guide for Concrete Floor and Slab Construction (ACI 302.R-96)

6. Frequent control joints should be provided in the slab to reduce the effects of curling and help reduce shrinkage cracking. Our experience indicates a joint spacing of not greater than 30 times the slab thickness is effective in this area.
7. Exterior flatwork and sidewalks should be separated from the structure. These slabs should be designed to function as independent units. Movement of these slabs should not be transmitted directly to the foundation of the buildings.

RETAINING WALLS

Based on our understanding of the project, cast-in-place basement walls will be constructed through the center of the two buildings nearest the slope. No site retaining walls are currently planned for the project, however based on the preliminary development sketch a wall may be necessary adjacent to the parking area at the far south end of the project.

Lateral Earth Pressures

Retaining walls will be subject to lateral earth loads which are dependent on the height of the wall, soil type, and backfill configuration. Backfill behind the building retaining walls will be level for the adjacent slab-on-grade construction of the two-story side of the buildings. Backfill behind site retaining walls may be sloped particularly in cut areas. Slopes behind retaining walls should not exceed 3:1. For walls that are not free to rotate, such as the foundation walls within the two to three-story split buildings, we recommend they be designed to resist "at-rest" earth pressures. 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. 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, with an angle of internal friction at least (ϕ) of 30 degrees, a unit weight of 125 pounds per cubic foot (pcf), and static conditions.

Table 2 – Lateral Earth Pressure Values (Equivalent Fluid Density)

Retained Slope	Static Conditions		
	Active	**At-Rest**	**Passive**
Level	40	60	375
3:1	50	75	145*
Notes: pcf – pounds per cubic foot, psf – pounds per square foot * Value Reduced to limit deflection			

Retaining Wall Backfill

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 0.5:1 horizontal to vertical line drawn upward from the bottom of the wall. Thinner lifts should be used when utilizing smaller compaction equipment.

Based on our understanding of the proposed project, structural improvements including adjacent foundations and slabs-on-grade will be located above the retaining wall backfill zone within the two to three-story split buildings, it is critically important that compaction of these backfill materials be diligently testing and inspected to minimize any undesirable differential movement.

Retaining Wall Drainage

Measures should be taken so that moisture does not build up behind retaining walls. Retaining wall foundation drainage must be installed along the below-grade retaining wall foundations. A typical drain detail for the proposed 2 to 3-story split buildings is shown on Fig. 3. The drain pipe should outlet to a storm drain or a swale directed away from buildings. For site retaining walls drainage measures could include free-draining granular backfill and perforated drain pipes leading to a positive gravity outlet or granular backfill with weep holes.

Waterproofing Walls

Cast-in-place below-grade walls should be waterproofed in accordance with the recommendations of the project architect. To reduce the potential for water and sulfate/salt related damage or efflorescence to the retaining walls, particular care should be taken in selection of the appropriate type of waterproofing material to be utilized and in application of this material. Basement seepage can be extremely costly to repair; therefore, the wall drainage and waterproofing systems for basement retaining walls must be well designed and properly installed.

EXTERIOR FLATWORK

Exterior flatwork, including sidewalks and porch slabs, is normally constructed as a slab-on-grade. Various properties of the soils and environmental conditions influence the magnitude of movement and other performance characteristics of slabs. Exterior flatwork should be designed and constructed to move independently relative to the proposed building foundations.

SWIMMING POOL

We understand a swimming pool associated pool deck area is planned south of the proposed clubhouse. No plans were available at the time of this investigation. We anticipate the pool structure may consist of spray-applied gunite against the on-site soils, or possibly a steel or a fiberglass shell. If gunite methods are used, the cement slurry should be properly reinforced.

The area of the proposed swimming pool is underlain by highly expansive clay soils. As such, we recommend the pool be placed on sub-excavated fill material in accordance with the recommendations set forth in the **Sub-Excavation** section.

The walls and based of the pool, as well as all connections/penetrations, should be watertight to prevent leaks and buildup of hydrostatic pressure behind or below the pool shell. Plumbing fixtures should be pressure tested following installation.

We recommend the pool be underlain by a drain system that collects water leakage and provides for discharge of the water to a sump or gravity outfall. The drain system should consist of free-draining gravel covering the bottom of the pool excavation. The excavation should slope to a 3 to 4-inch diameter, perforated or slotted pipe placed within the gravel layer. The drain should lead to a positive gravity outlet, such as a subdrain located beneath the sewer, or to a sump where water can be removed by pumping. A conceptual pool drain system is presented in Fig. 4. Overall surface drainage patterns should be planned to provide for the rapid removal of storm runoff and water that splashes over the edges of the pool. Drainage from the pool deck areas should be directed to area drains. The precautions described in the **Drainage, Irrigation, and Maintenance** section should be followed surrounding the swimming pool, as well as for all areas of the site.

The swimming pool structure may settle more than the flatwork surrounding the pool. To avoid damage to the pool structure, a slip joint should be used around the perimeter of the pool structure and adjacent to any other structural elements. Utility lines that penetrate the pool structure should be separated and isolated with joints to allow for free vertical movement. All ducts with connections between the pool structure and surrounding soil should be flexible or “crushable,” to allow some relative movement.

PAVEMENTS

Pavement subgrade soils across the site will likely consist predominantly of sandy clays that generally classify as A-7-6 soils and provide very poor subgrade support characteristics for pavement systems. Based on our laboratory testing, a Hveem stabilometer (“R”) value of 5 was assigned to the subgrade materials for design purposes.

We anticipate the parking stalls will be subject to relatively frequent vehicle passes. We considered a daily traffic number (DTN) of 2 for the parking stalls which corresponds to an 18-kip Equivalent Single-Axle Loads (ESAL) of 14,600 for a 20-year pavement design life. In our opinion, the parking stalls can be paved with 4 inches of asphalt concrete over 6 inches of aggregate base course. Access driveways are expected to see relatively frequent vehicle. We considered a daily traffic number (DTN) of 10 for the access driveways which corresponds to an 18-kip Equivalent Single-Axle Loads (ESAL) of 73,000 for a 20-year pavement design life. In our opinion, the access driveways can be paved with 5 inches of asphalt concrete over 6 inches of aggregate base course.

The pavement subgrade soils are well below optimum moisture content and based on our experience and the results of laboratory testing these soils will exhibit significant heave when wetted. It should be understood that the pavement sections provided above do not consider the effects on pavements of heaving subgrade soils which are likely to result in cracking. To reduce movement and enhance performance of project pavements, we recommend the subgrade soils be sub-excavated to provide a zone of moisture conditioned and compacted on-site soils below the pavement section. Due to the high measured swell potential of the existing fill, we recommend removing the clay fill to a depth of about 2 feet below pavement. The clay fill may be moisture conditioned to within 2 percent of optimum moisture and compacted to 95 percent of a standard proctor (ASTM 698). Over wetting the soils will result in higher moisture contents that tend to pump and deflect during proof rolling.

We recommend a concrete pad be provided at dumpster locations. The pad should be at least 8 inches thick and long enough to support the entire length of the trash truck and dumpster. The concrete pad should extend at least 5 feet outside of the anticipated truck dimensions. 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 Regional 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, particular 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 a water-soluble sulfate concentration of 0.25 percent in one sample from this site. Water-soluble sulfate concentrations between 0.2 and 2 percent indicate Class 2 exposure to sulfate attack, according to the American Concrete Institute (ACI) Guide To Durable Concrete (ACI 201.2R). For sites with Class 2 sulfate exposure, ACI 201 recommends using a cement meeting the requirements for Type V (sulfate resistant) cement or the equivalent, with a maximum water-to-cementitious material ratio of 0.45. As an alternative, ACI allows the use of cement that conforms to ASTM C 150 Type II requirements, if it meets the Type V performance requirements (ASTM C 452) of ASTM C 150 Table 4. ACI 201 also allows a blend of any type of portland cement and fly ash with an expansion of less than 0.05 percent at 6 months when tested in accordance with ASTM C 1012. ACI 318 indicates concrete in severe exposure should have a specified compressive strength of 4,500 psi. Concrete subjected to freeze-thaw cycles should be air entrained.

DRAINAGE, IRRIGATION AND MAINTENANCE

Performance of foundations, pavements, and flatwork is influenced by the moisture conditions existing within the foundation or subgrade soils. Overall surface drainage should be designed, constructed, and maintained to provide rapid removal of surface water runoff away from the proposed buildings, and off of pavements and flatwork. Final grading of pavement subgrade should be carefully controlled so that the designed slopes are maintained and low spots in the subgrade that could trap water are eliminated. We recommend the following precautions be observed during construction and maintained at all times after construction is completed.

1. Wetting or drying of open foundation, utility, and earthwork excavations should be avoided.
2. Positive drainage should be provided away from the buildings. We recommend a minimum slope of at least 5 percent in the first 5 to 10 feet away from the foundations in landscaped areas. In flatwork areas adjacent to the buildings, the slope may be reduced to grades that comply with ADA requirements. Paved surfaces should be sloped to drain away from the buildings. A minimum slope of 2 percent is suggested. More slope is desirable. Concrete curbs and sidewalks may “dam” surface runoff adjacent to the structures and disrupt proper flow. Use of “chase” drains or weep holes at low points in the curb should be considered to promote proper drainage.
3. Foundation wall backfill should be thoroughly compacted to decrease permeability and reduce the potential for irrigation and stormwater to migrate behind retaining walls or below floor slabs. Areas behind curb and gutter should be backfilled and well compacted to reduce ponding of surface water. Seals should be provided between the curb and pavement to reduce infiltration.
4. Landscaping should be carefully designed to minimize irrigation. Plants placed close to foundation walls should be limited to those with low moisture requirements. Irrigation should be limited to the minimum amount sufficient to maintain vegetation. Application of more water will increase likelihood of slab and foundation movements and associated damage. Landscaped areas should be adequately sloped to direct flow away from the structures and improvements. Area drains can be used to drain locations that cannot be provided with adequate slope.
5. Foundation wall backfill should be placed in thin, loose lifts, moisture conditioned to within 2 percent of optimum and compacted to at least 95 percent of maximum standard proctor dry density (ASTM D 698). Areas behind curb and gutter should be backfilled and well compacted to reduce ponding of surface water. Seals should be provided between the curb and pavement to reduce infiltration.
6. Impervious plastic membranes should not be used to cover the ground surface immediately surrounding foundations. These membranes tend to trap moisture and prevent normal evaporation from occurring. Geotextile fabrics can be used to control weed growth and allow evaporation.
7. Roof drains should be directed away from the building and discharge beyond backfill zones or into an appropriate storm sewer or detention area. Downspout extensions and splash blocks should be provided at all discharge points. Roof drains can also be connected to buried, solid pipe out-

lets. Roof drains should not be directed below slab-on-grade floors. Roof drain outlets should be maintained.

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 this 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 structures and improvements will perform satisfactorily. It is critical that all recommendations in this report are followed during construction. The owner must assume responsibility for maintaining the structures and use appropriate practices regarding drainage and landscaping. Improvements made after construction should be completed in accordance with recommendations provided in this report and may require additional soil investigation and consultation.

LIMITATIONS

This report has been prepared for the exclusive use of Weidner Apartment Homes for the purpose of providing geotechnical design and construction criteria for the proposed apartment development. The information, conclusions, and recommendations presented herein are based on consideration of many factors including, 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 evolve in the area of geotechnical engineering. The recommendations provided are appropriate for about three years. If the facility is not constructed within about three years, we should be contacted to determine if we should update this report.

The borings were located to obtain a reasonably accurate indication of subsurface conditions at this site. The boring is representative of conditions encountered only at the location that was drilled. Subsurface variations not indicated by our borings are possible.

We believe this investigation was conducted in a manner consistent with that level of care and skill ordinarily 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 subsurface conditions on the design of the structure or any other aspect of the proposed construction, please call.

CTL | THOMPSON, INC.

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JEFFREY MARK JONES
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9-14-20
PROFESSIONAL ENGINEER

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Senior Engineering Consultant

JMJ:WCH:cw

2 copies sent

Via e-mail: davida@weidner.com

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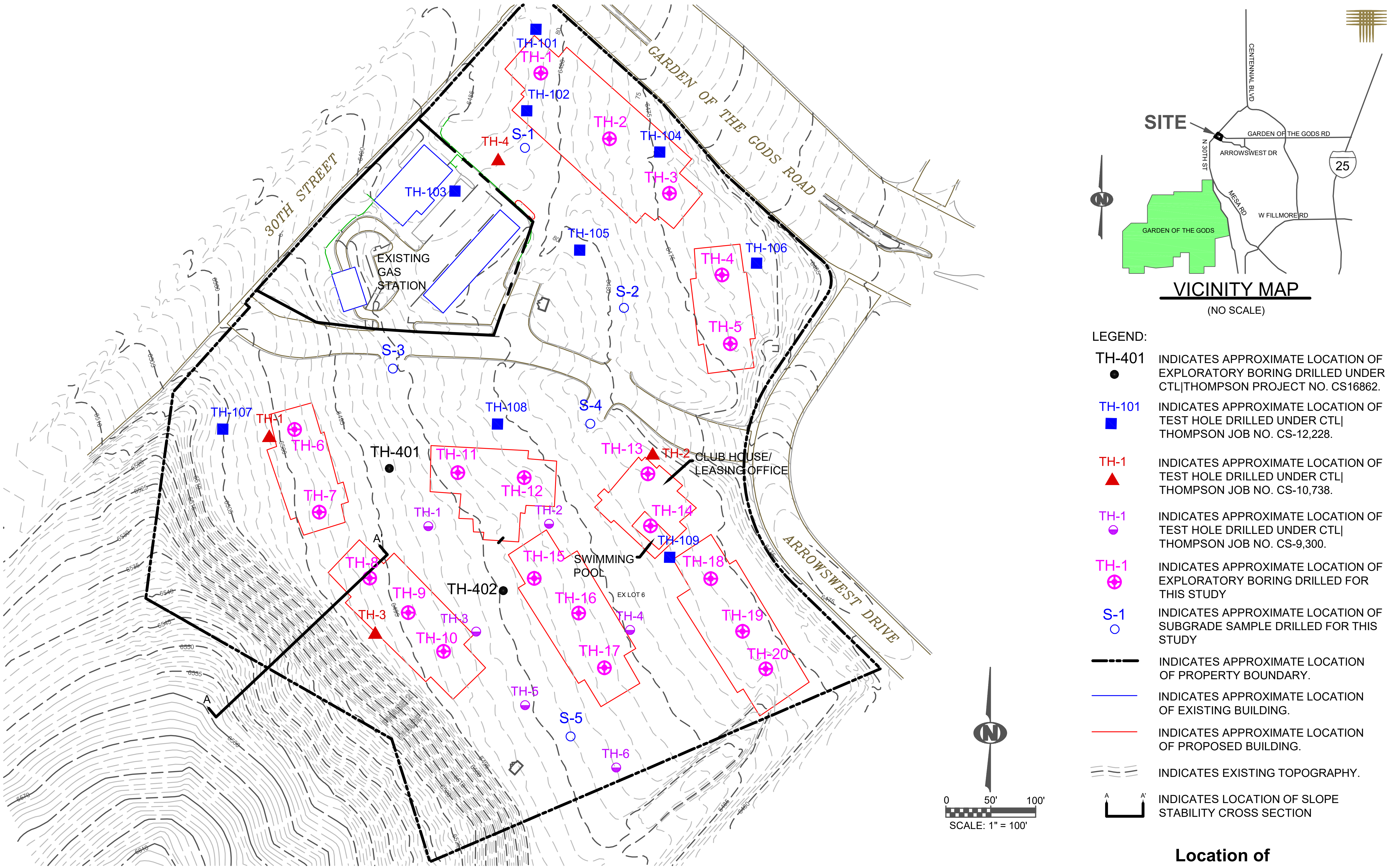

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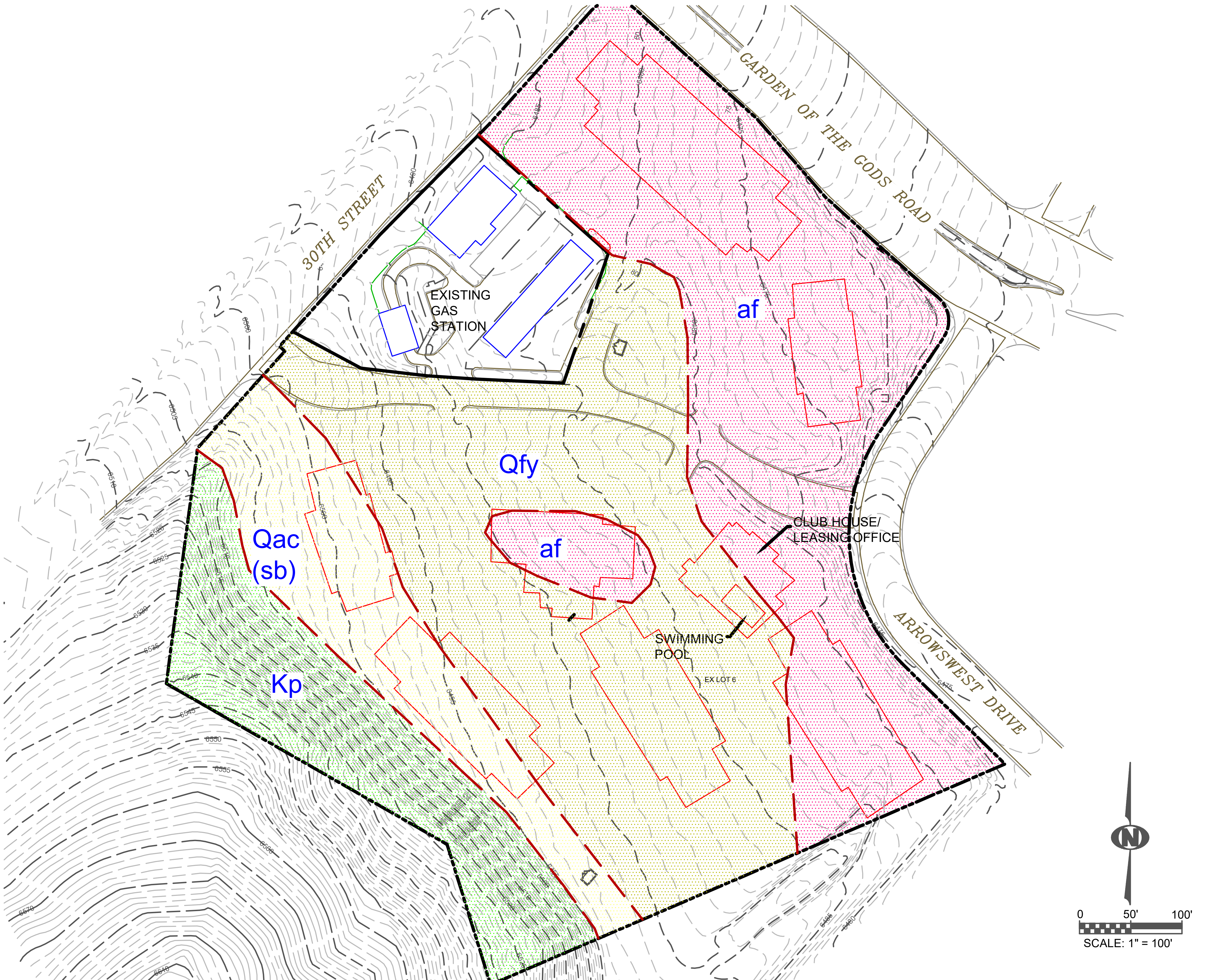
Location of Exploratory Borings

FIG. 1



LEGEND:

- INDICATES APPROXIMATE LOCATION OF PROPERTY BOUNDARY.
- INDICATES APPROXIMATE LOCATION OF EXISTING BUILDING.
- INDICATES APPROXIMATE LOCATION OF PROPOSED BUILDING.
- INDICATES EXISTING TOPOGRAPHY.
- INDICATES INTERPRETED GEOLOGIC CONTACT LINE
- af — ARTIFICAL FILL FILL PLACED DURING REALIGNMENT OF GARDEN OF THE GODS ROAD AND OVERLOT GRADING OF THE ARROWSWEST DEVELOPMENT
- Qfy — HOLOCENE AGE FAN DEPOSITS, CONSIST OF SAND, SILT, AND CLAY, MAY HAVE GRAVEL IN PLACES
- Qac — HOLOCENE AGE COLLUVIAL DEPOSITS, MOSTLY MATRIX SUPPORTED SILTY CLAY, CLAYEY SILT AND SAND
- Kp — PIERRE SHALE, GRAY TO DARK GRAY SHALE THAT WEATHERS TO BROWN AND OLIVE=GREEN CLAY
- (sb) — SHALLOW BEDROCK EXPECTED TO BE 10 OR LESS FEET BELOW THE GROUND SURFACE.

NOTES: THE CONTACTS BETWEEN GEOLOGIC UNITS WAS BASED ON A SUBJECTIVE INTERPOLATION OF CONDITION FOUND IN TEST HOLES REVISED OF AVAILABLE MAPPING, A SITE RECONNAISSANCE AND OUR EXPERIENCE. LOCAL VARIATIONS SHOULD BE EXPECTED.

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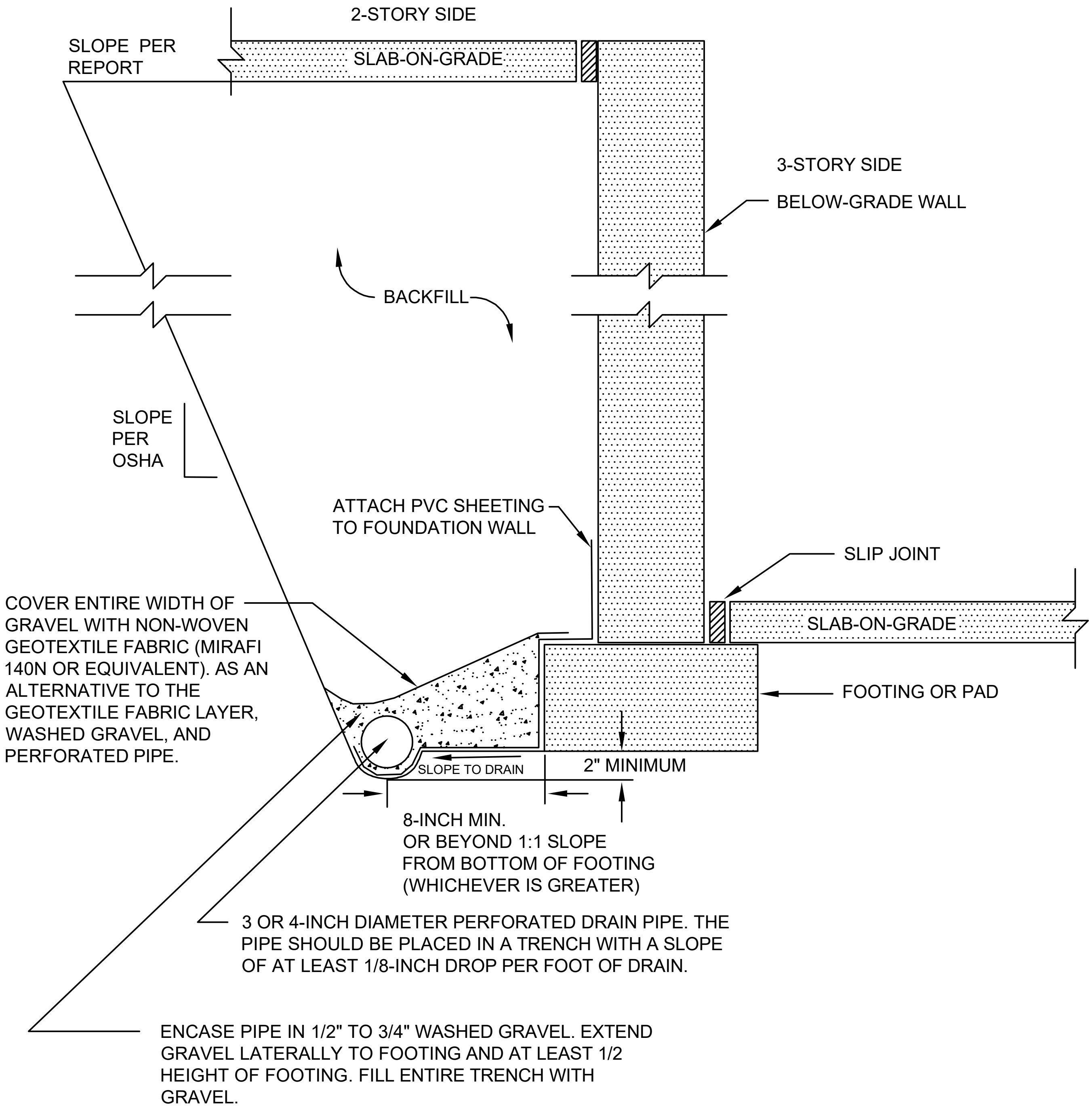
Surficial Geology

FIG. 2



NOTE:
THE BOTTOM OF THE DRAIN SHOULD BE AT LEAST 2 INCHES BELOW BOTTOM OF FOOTING AT THE HIGHEST POINT AND SLOPE DOWNWARD TO A POSITIVE GRAVITY OUTLET OR TO A SUMP WHERE WATER CAN BE REMOVED BY PUMPING. SUMP MAY BE DISCHARGED TO UNDERDRAIN SYSTEM, DOWN GRADIENT OF THE FOUNDATION DRAIN CONNECTION.

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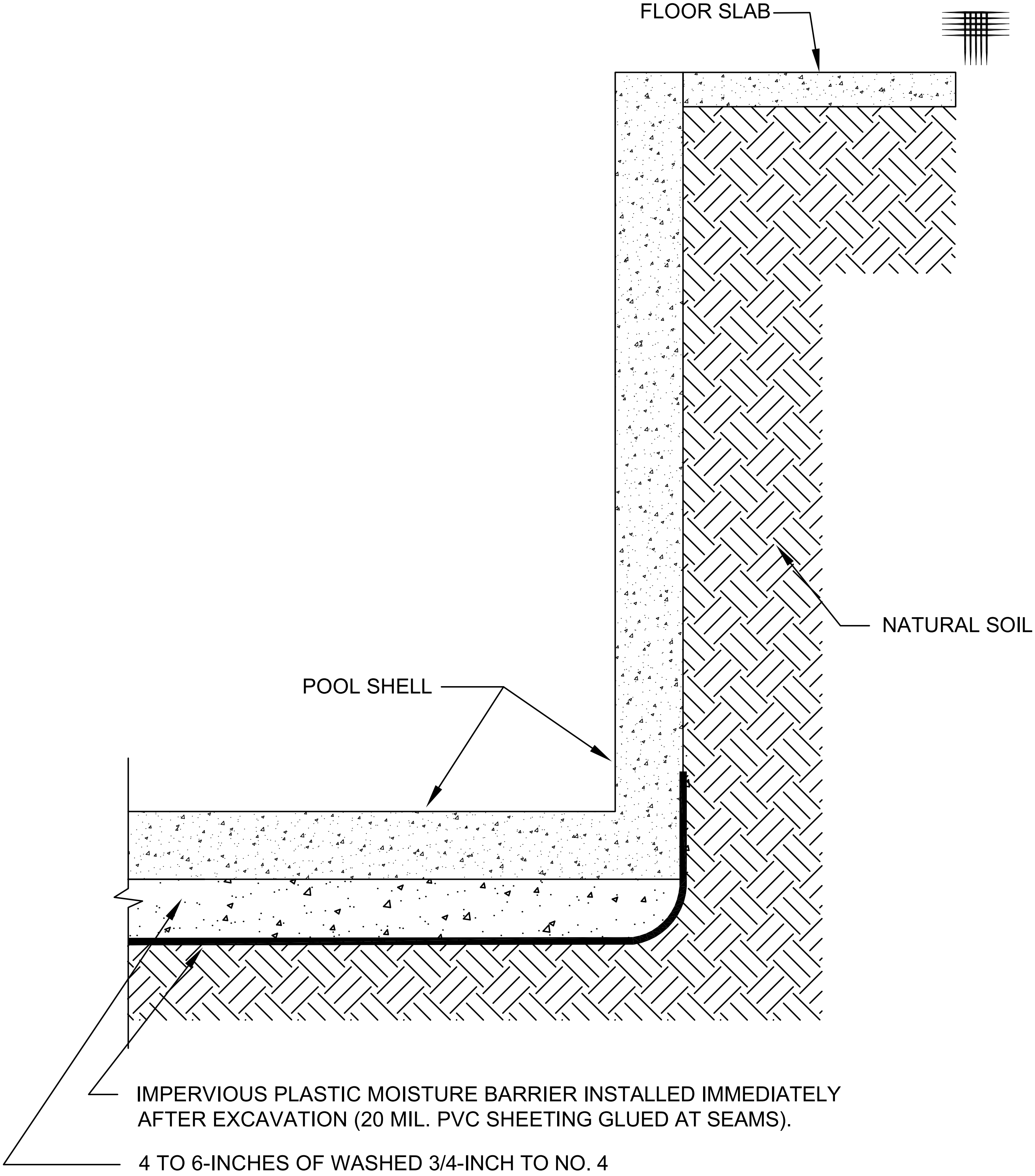
2-3 Story Split Foundation Wall Drain

Fig. 3



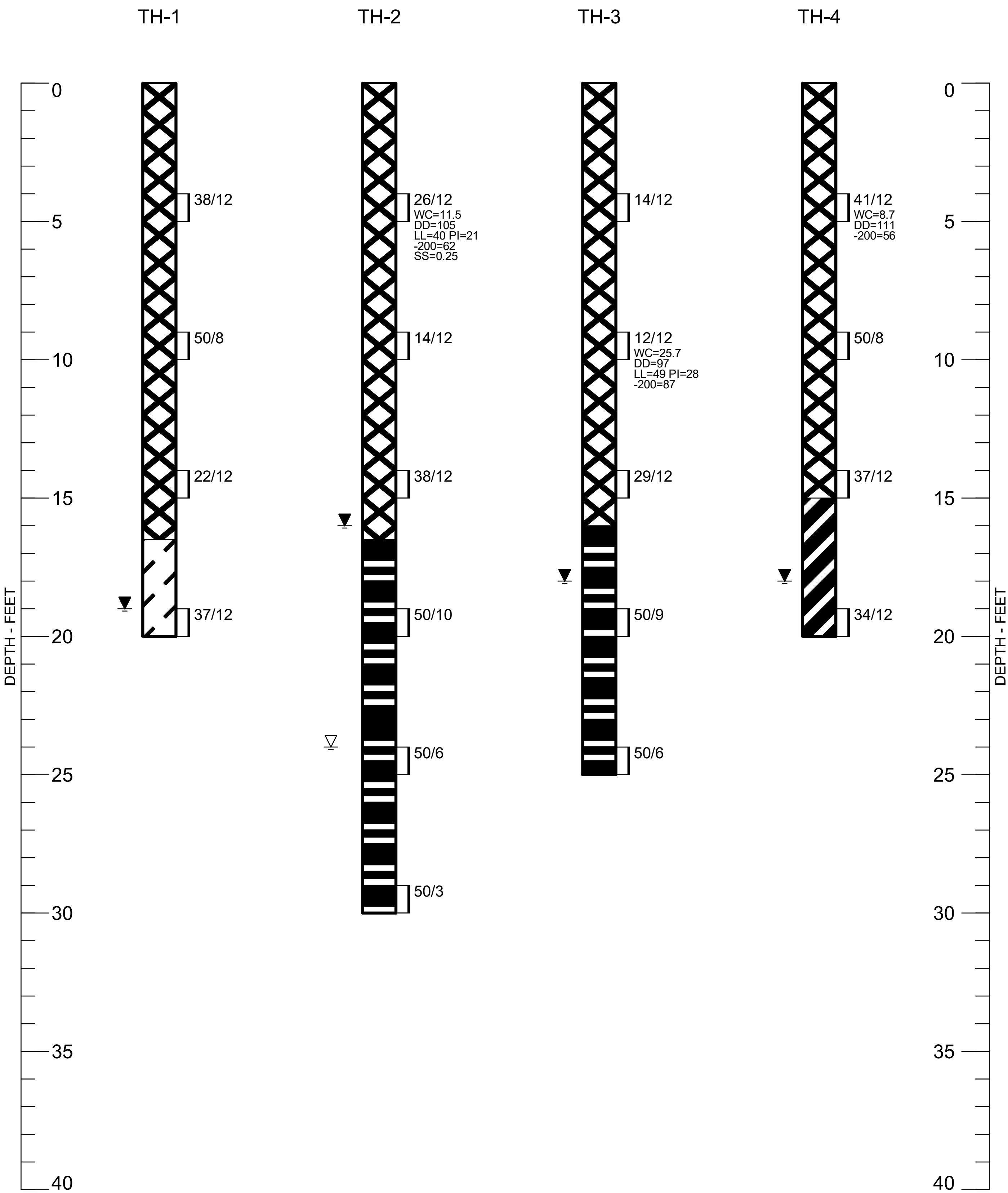
NOTE:
DRAIN PIPE SHOULD CONSIST OF A 3 OR 4-INCH DRAIN PIPE WITH A MINIMUM SLOPE OF 1/8 INCH DROP PER FOOT, TO A POSITIVE GRAVITY OUTLET OR TO A SUMP WHERE WATER CAN BE REMOVED BY PUMPING.

Recommended Pool Drain Detail

Fig. 4

APPENDIX A

SUMMARY LOGS OF EXPLORATORY BORINGS



SUMMARY LOGS OF EXPLORATORY BORINGS

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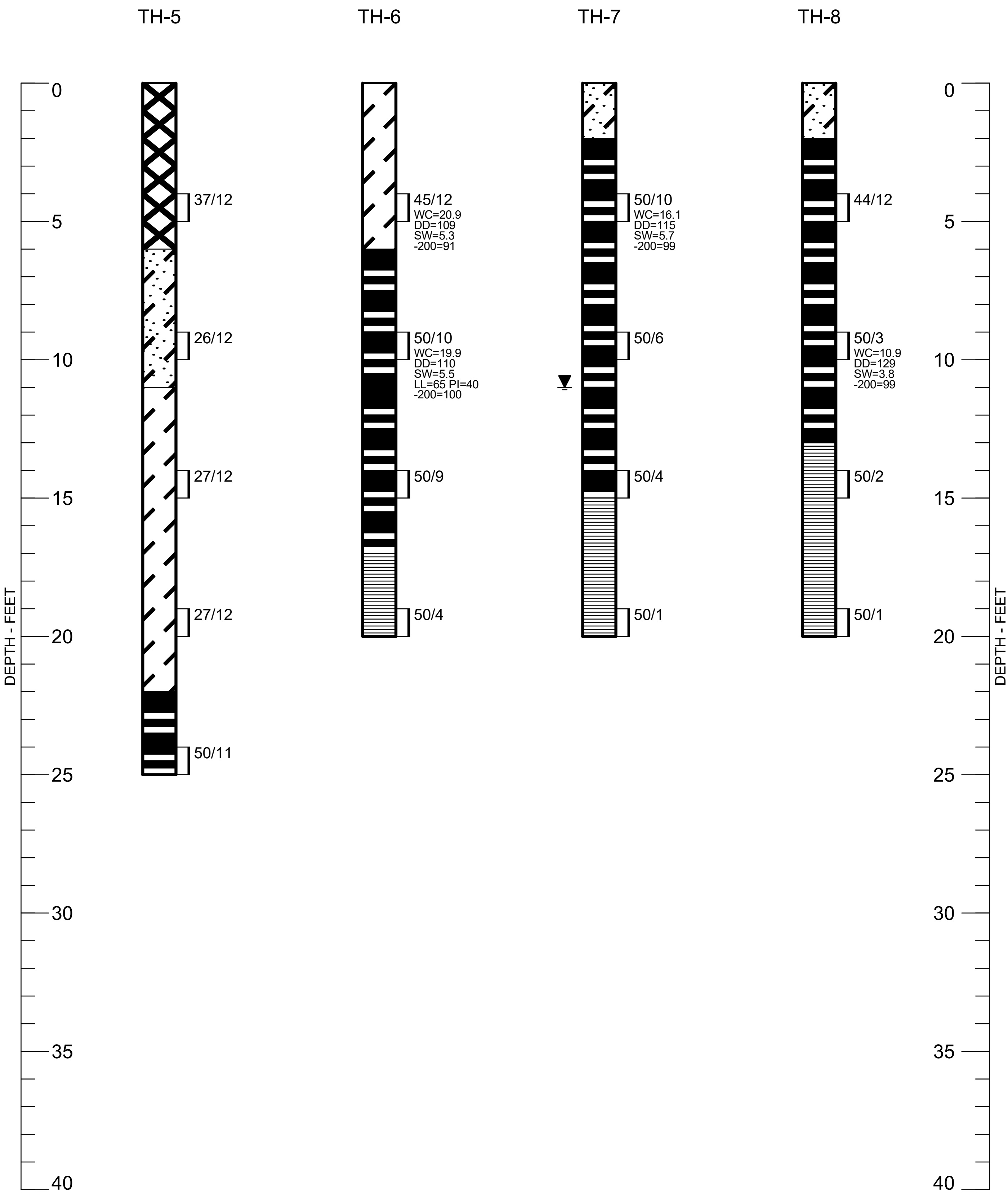
FIG. A- 1



SUMMARY LOGS OF EXPLORATORY BORINGS

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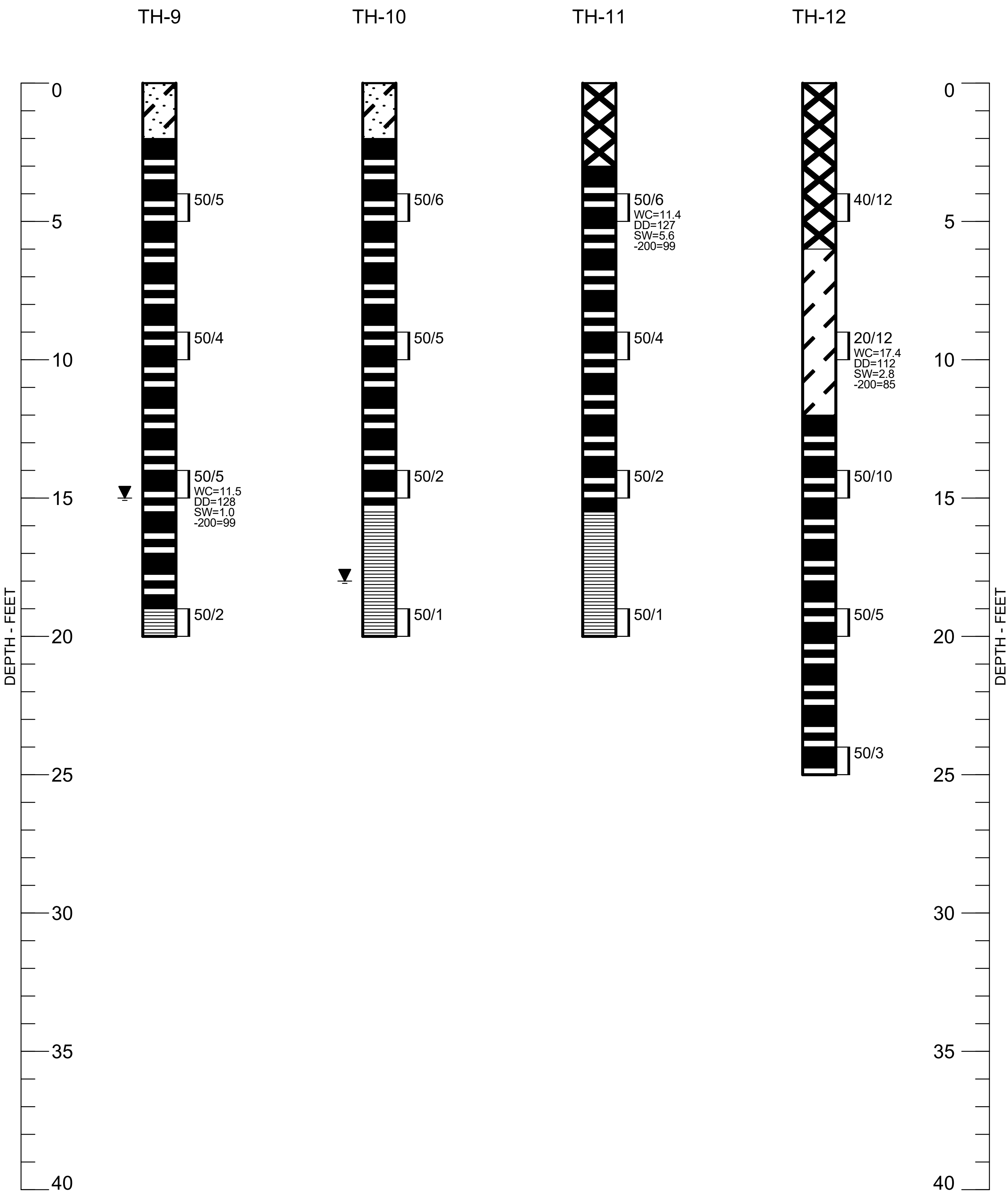
FIG. A- 2



SUMMARY LOGS OF EXPLORATORY BORINGS

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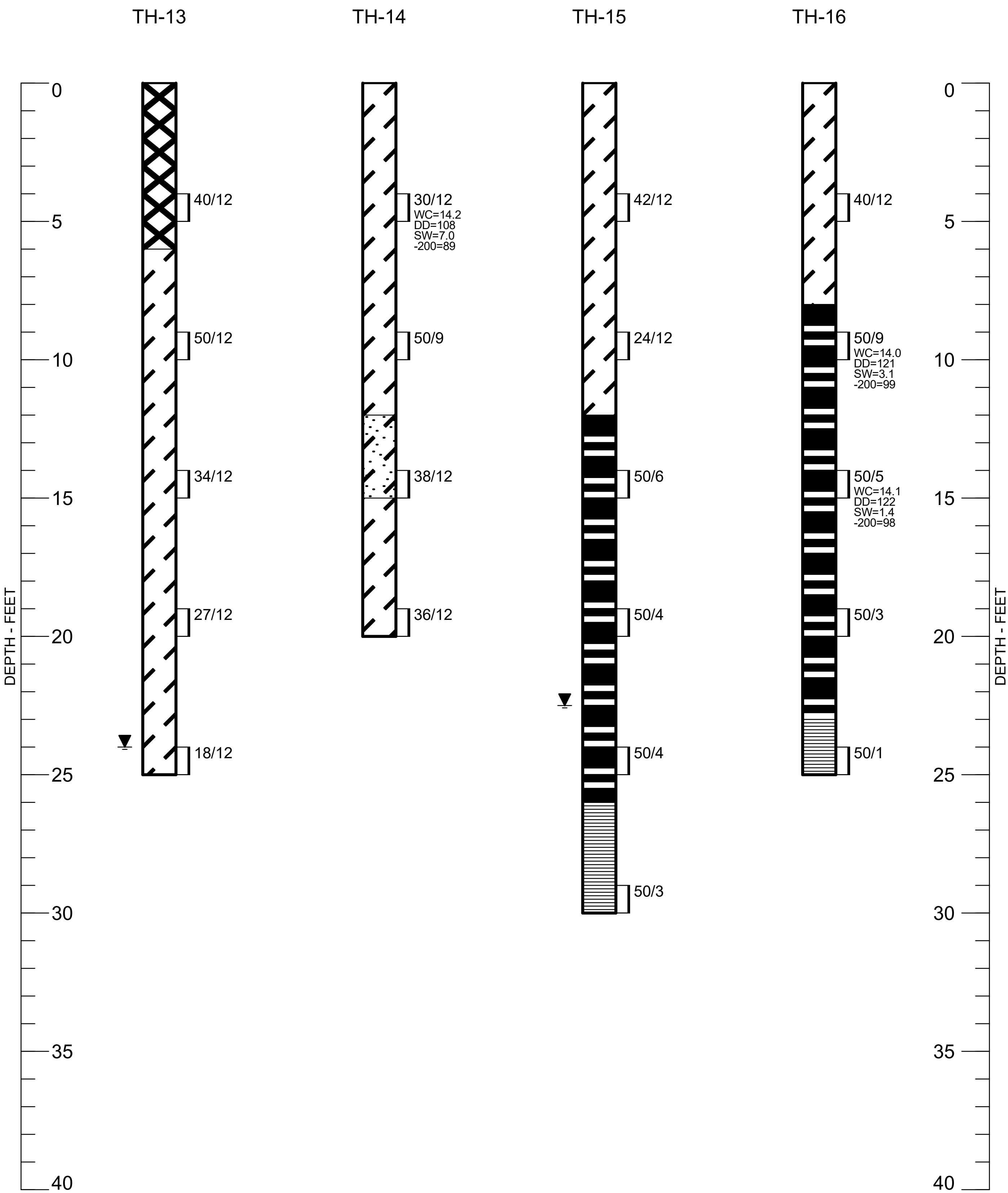
FIG. A- 3



SUMMARY LOGS OF EXPLORATORY BORINGS

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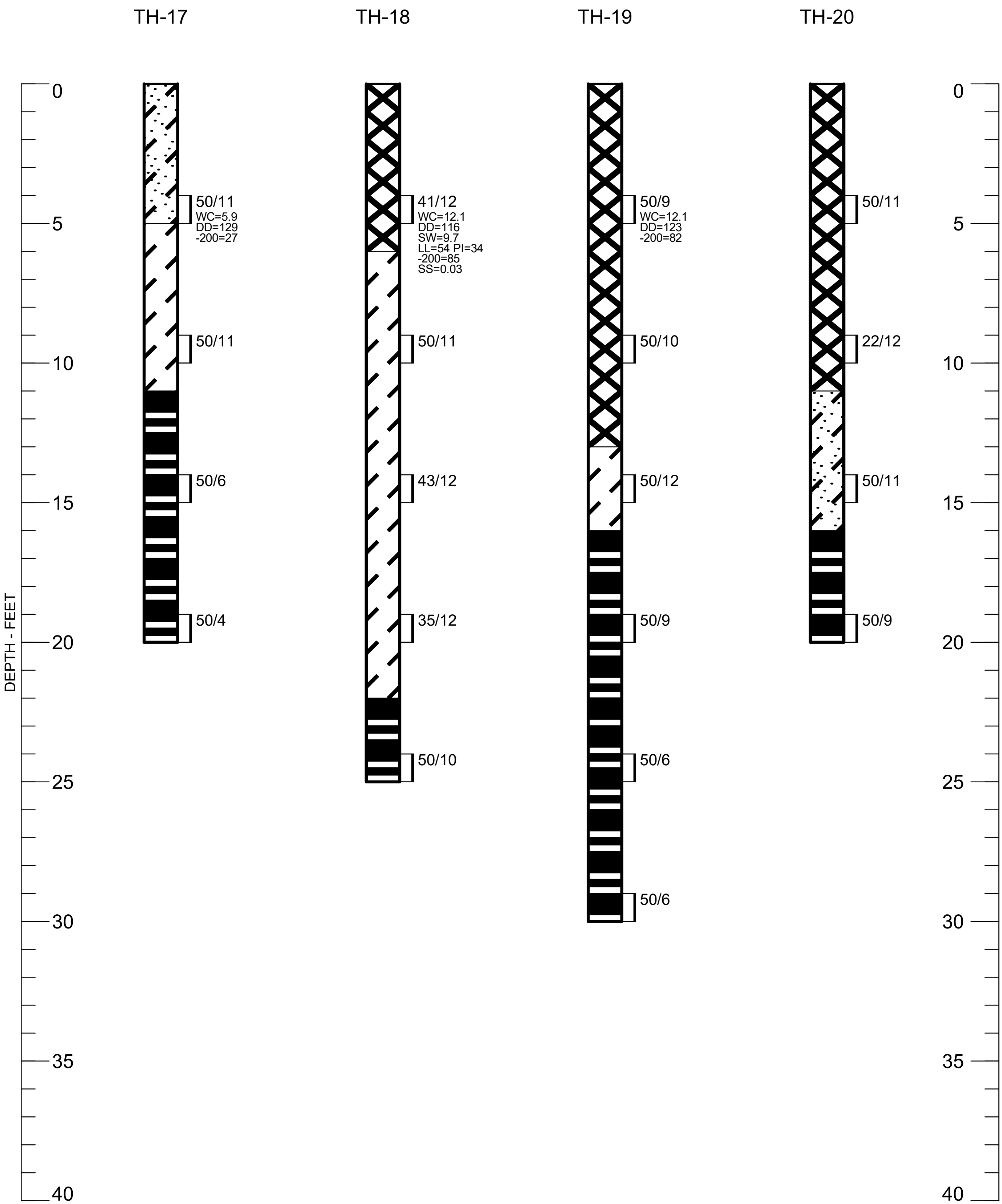
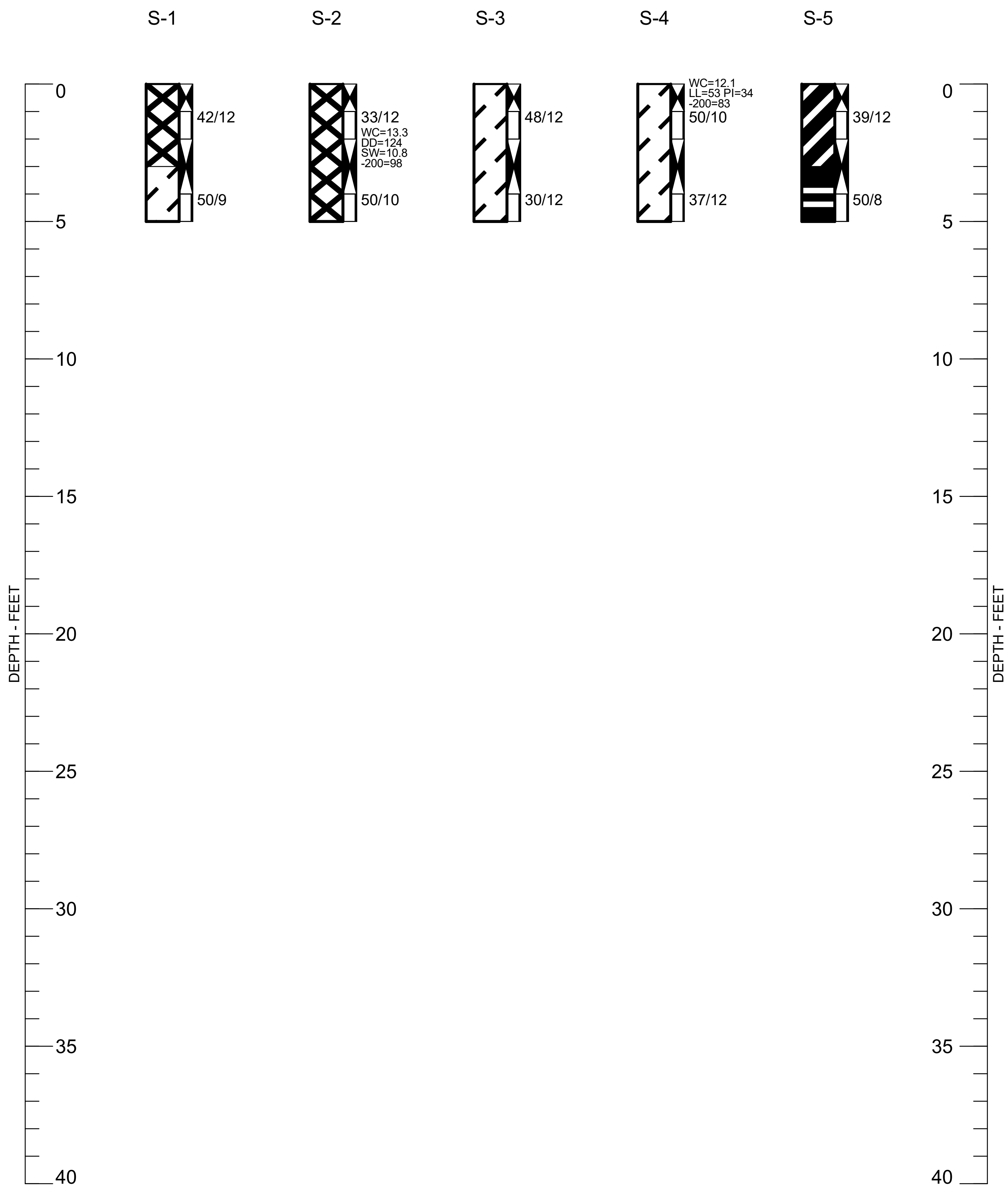
FIG. A- 4



SUMMARY LOGS OF EXPLORATORY BORINGS



SUMMARY LOGS OF EXPLORATORY BORINGS

LEGEND:

FILL, CLAY, SANDY TO SAND, CLAYEY, SLIGHTLY GRAVELLY, VERY STIFF/MEDIUM DENSE TO VERY DENSE, SLIGHTLY MOIST, BROWN, OLIVE.

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

SAND, CLAYEY, GRAVELLY, MEDIUM DENSE, SLIGHTLY MOIST, REDDISH BROWN (SC).

WEATHERED CLAYSTONE, NEAR VERTICAL BEDDING, MEDIUM HARD TO HARD, SLIGHTLY MOIST, LIGHT BROWN.

BEDROCK, CLAYSTONE, NEAR VERTICAL BEDDING, HARD TO VERY HARD, SLIGHTLY MOIST, BROWN TO GRAY.

SHALE, NEAR VERTICAL BEDDING, VERY HARD, FISSILE, SLIGHTLY MOIST, DARK GRAY.

DRIVE SAMPLE. THE SYMBOL 38/12 INDICATES 38 BLOWS OF AN AUTOMATIC 140-POUND HAMMER FALLING 30 INCHES WERE REQUIRED TO DRIVE A 2.5-INCH O.D. SAMPLER 12 INCHES.

BULK SAMPLE COLLECTED FROM AUGER CUTTINGS.

WATER LEVEL MEASURED AT TIME OF DRILLING.

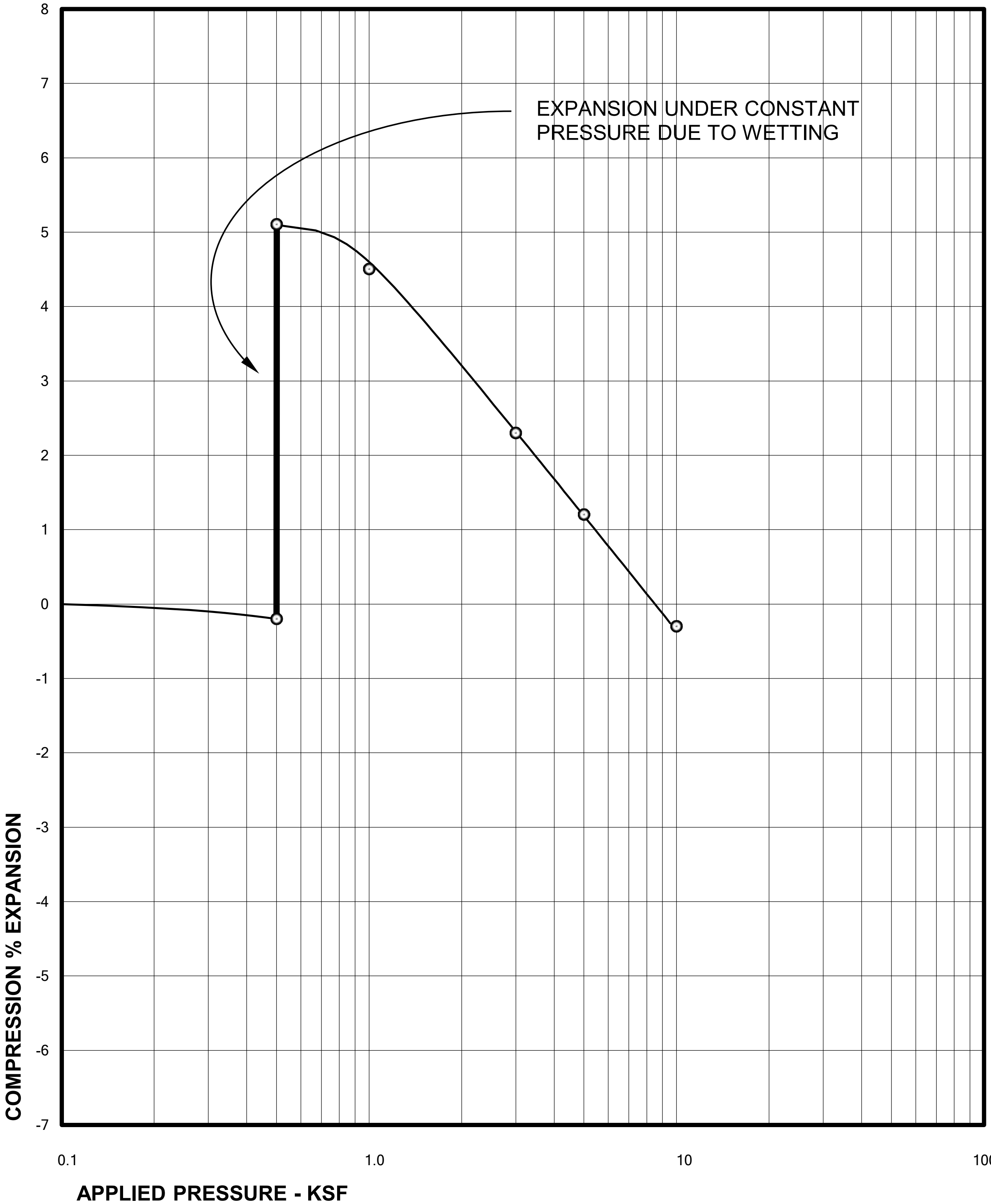
WATER LEVEL MEASURED AFTER DRILLING ON AUGUST 17, 2020.

NOTES:

1. THE BORINGS WERE DRILLED ON AUGUST 6 AND 7, 2020 USING 4-INCH DIAMETER, CONTINUOUS-FLIGHT SOLID-STEM AUGER AND TRUCK-MOUNTED CME-55 DRILL RIG.
2. WC - INDICATES MOISTURE CONTENT (%).
   DD - INDICATES DRY DENSITY (PCF).
   SW - INDICATES SWELL WHEN WETTED UNDER APPLIED PRESSURE (%).
   COM- INDICATES COMPRESSION WHEN WETTED UNDER APPLIED PRESSURE (%).
   LL - INDICATES LIQUID LIMIT.
   PI - INDICATES PLASTICITY INDEX.
   -200 - INDICATES PASSING NO. 200 SIEVE (%).
   SS - INDICATES WATER-SOLUBLE SULFATE CONTENT (%).
3. THESE LOGS ARE SUBJECT TO THE EXPLANATIONS, LIMITATIONS AND CONCLUSIONS CONTAINED IN THIS REPORT.

APPENDIX B

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



Sample of CLAY, SLIGHTLY SANDY (CL)

From TH-6 AT 4 FEET

DRY UNIT WEIGHT= 109 PCF

MOISTURE CONTENT= 20.9 %

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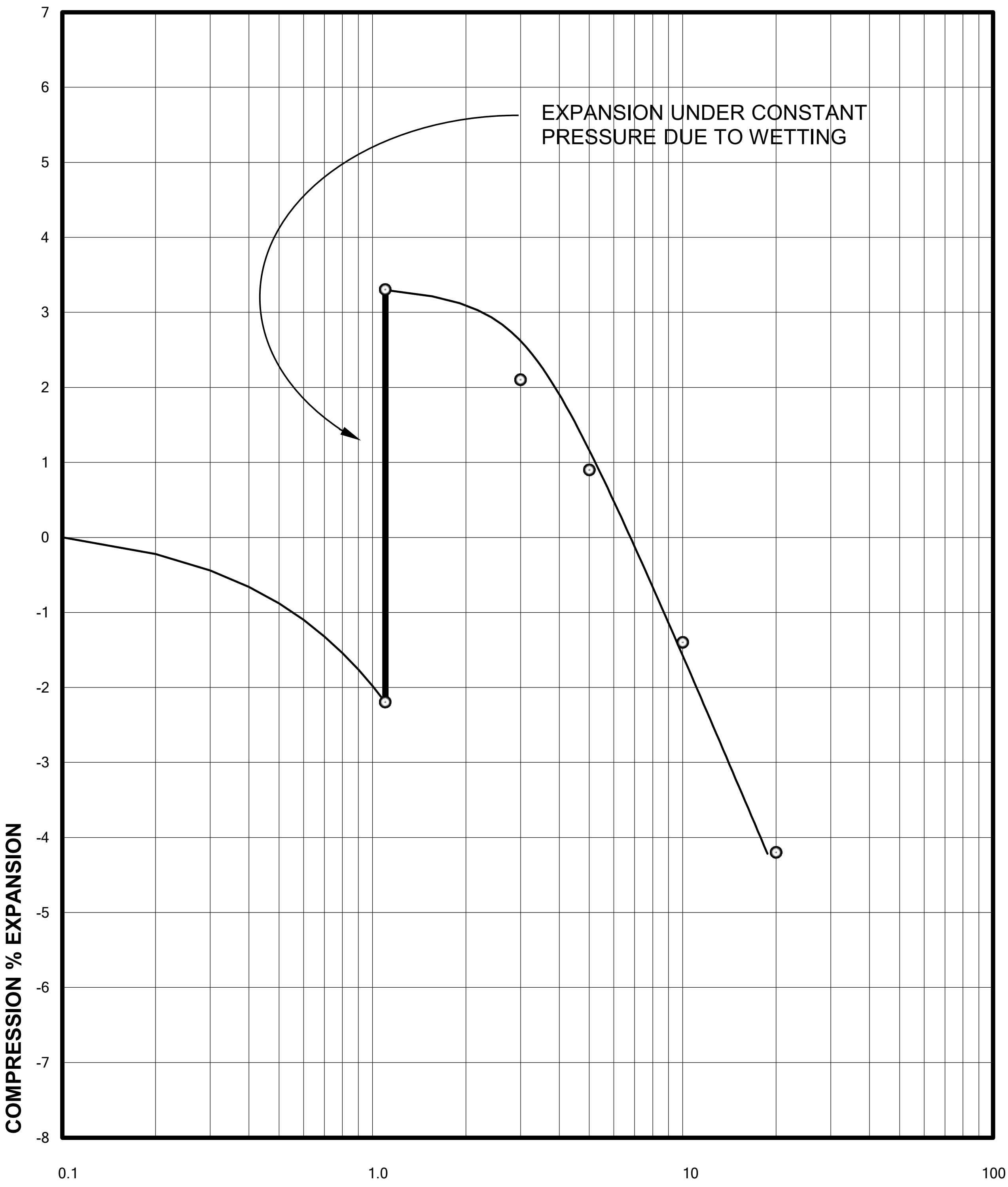
Swell Consolidation Test Results

FIG. B-1



Sample of	CLAYSTONE	DRY UNIT WEIGHT=	110	PCF
From	TH-6 AT 9 FEET	MOISTURE CONTENT=	19.9	%

Swell Consolidation Test Results

FIG. B-2

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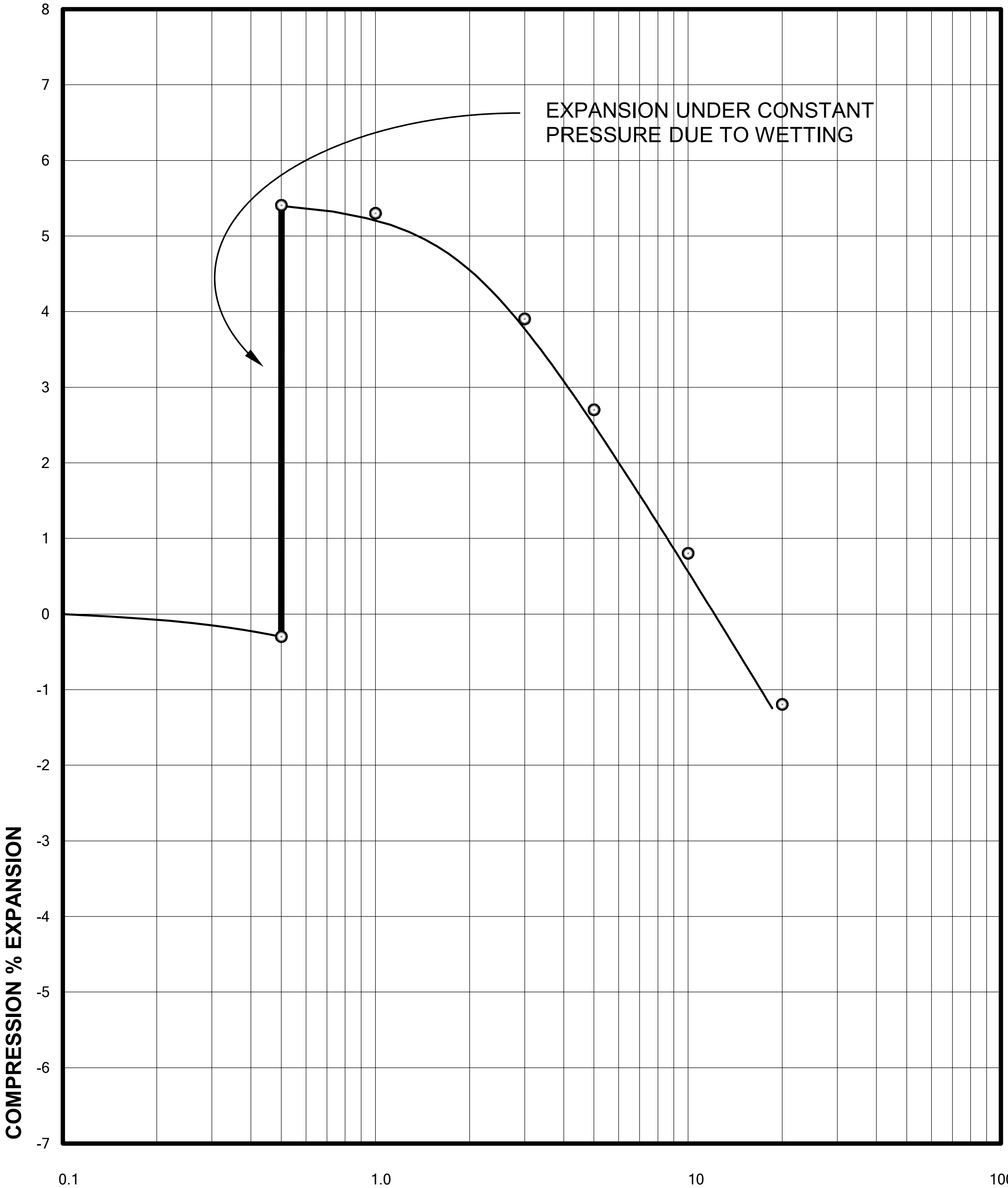
ARROWSWEST APARTMENTS
CTL|T PROJECT NO. CS19280-120



Sample of CLAYSTONE

From TH-7 AT 4 FEET

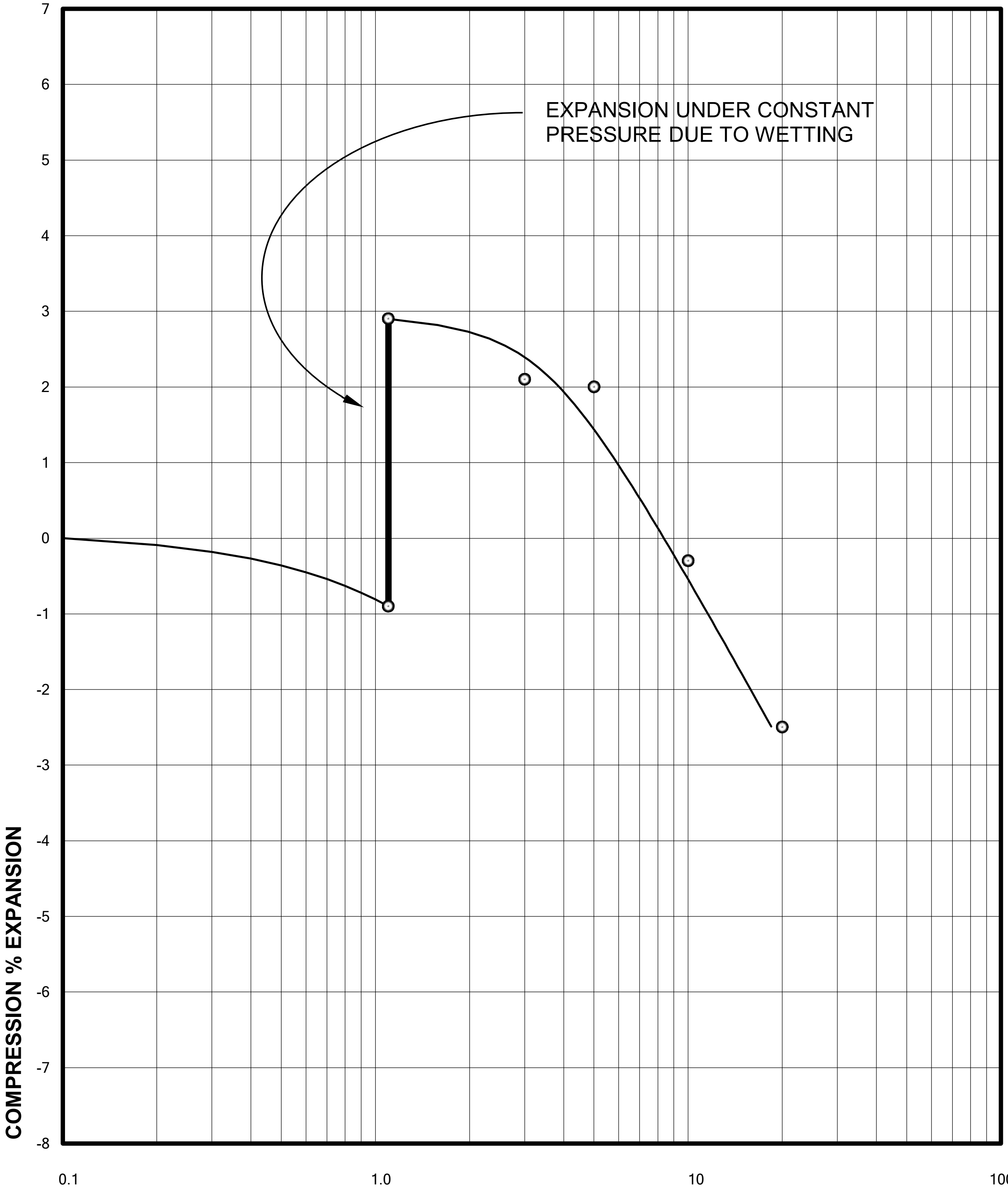
DRY UNIT WEIGHT= 115 PCF

MOISTURE CONTENT= 16.1 %

Swell Consolidation Test Results

FIG. B-3

WEIDNER APARTMENTS
ARROWSWEST APARTMENTS
CTL|T PROJECT NO. CS19280-120

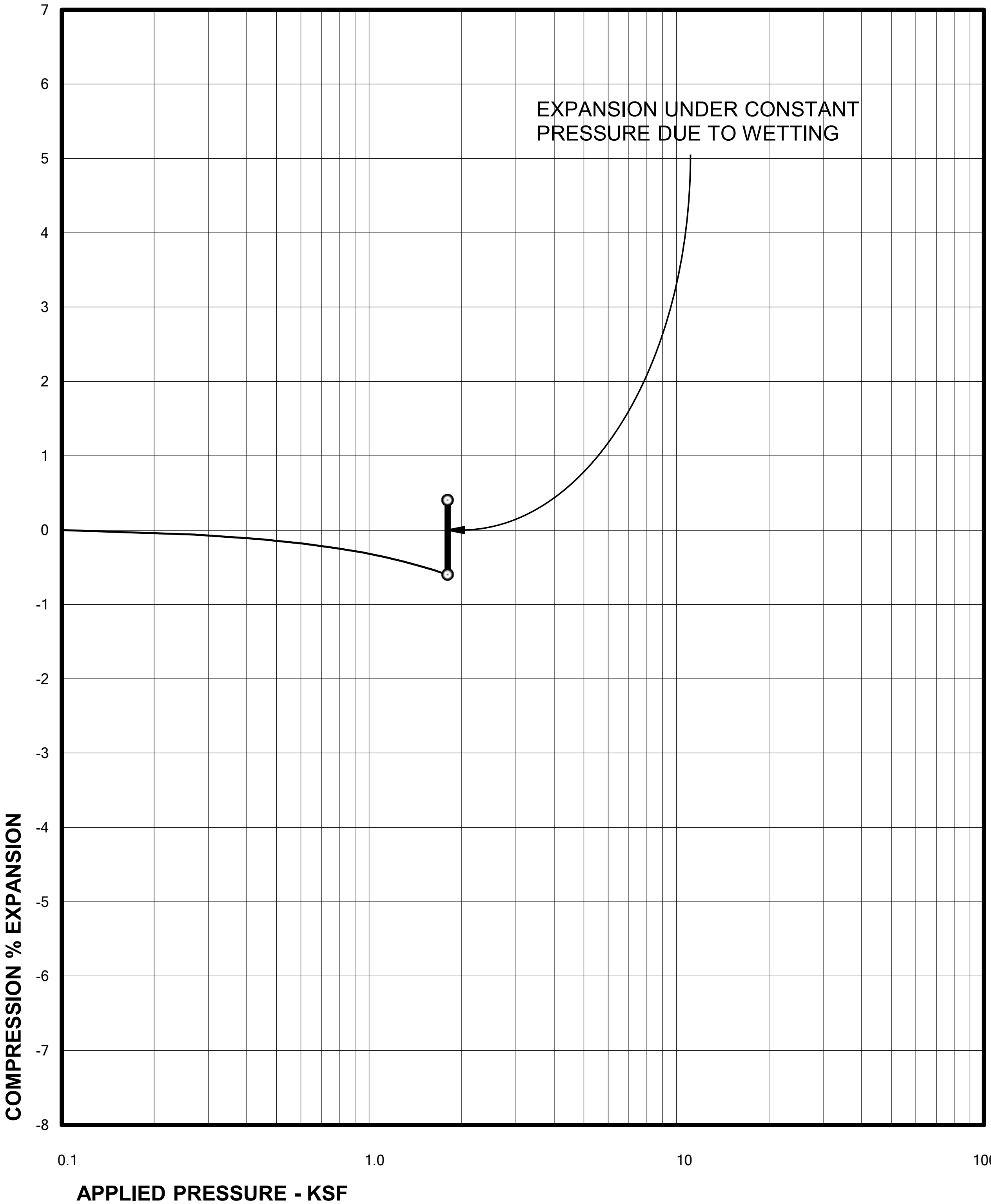


Sample of	CLAYSTONE	DRY UNIT WEIGHT=	129	PCF
From	TH-8 AT 9 FEET	MOISTURE CONTENT=	10.9	%

Swell Consolidation Test Results

FIG. B-4

WEIDNER APARTMENTS
ARROWSWEST APARTMENTS
CTL|T PROJECT NO. CS19280-120



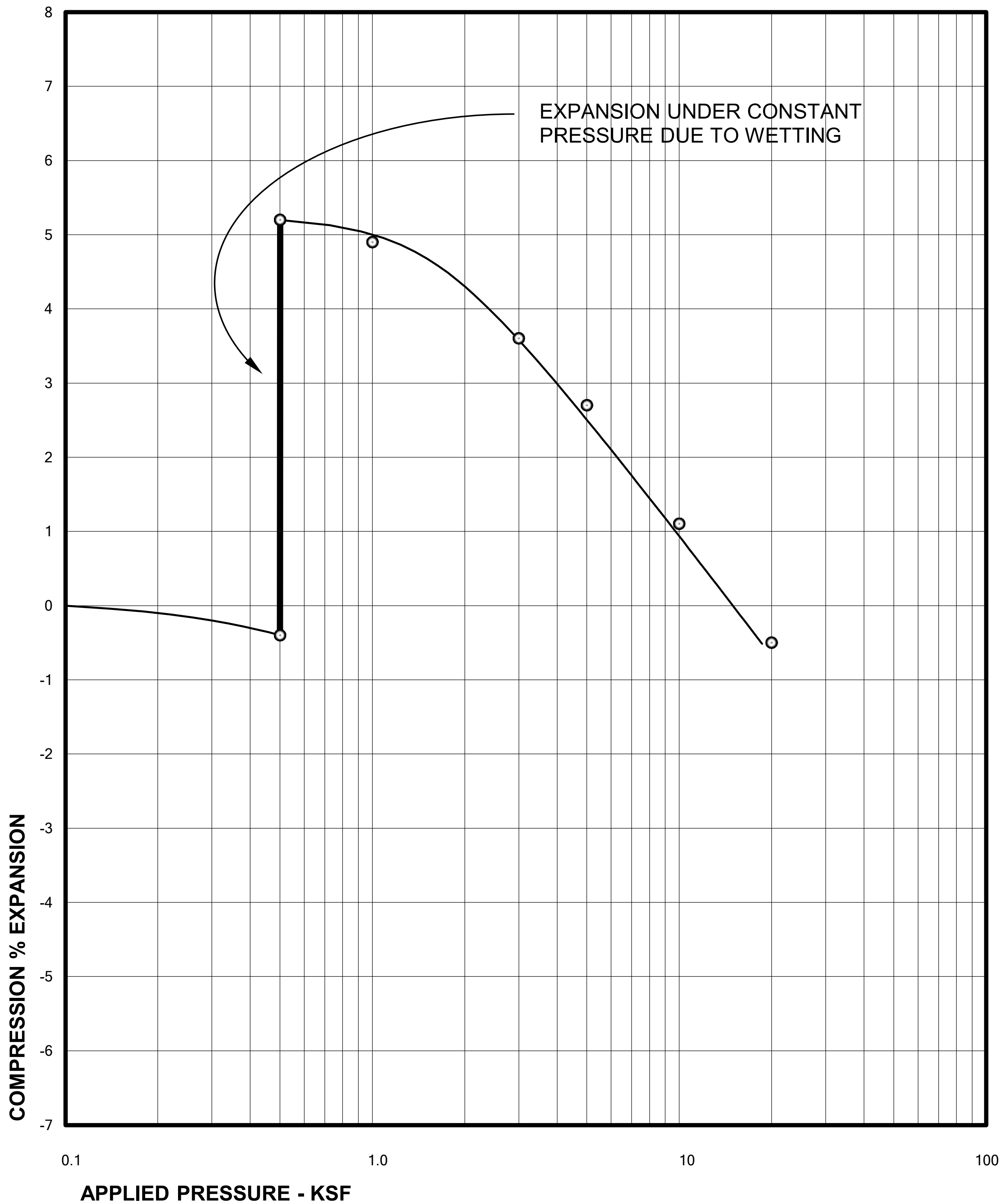
Sample of CLAYSTONE
From TH-9 AT 14 FEET

DRY UNIT WEIGHT= 128 PCF
MOISTURE CONTENT= 11.5 %

Swell Consolidation Test Results

FIG. B-5

WEIDNER APARTMENTS
ARROWSWEST APARTMENTS
CTL|T PROJECT NO. CS19280-120



Sample of CLAYSTONE

From TH-11 AT 4 FEET

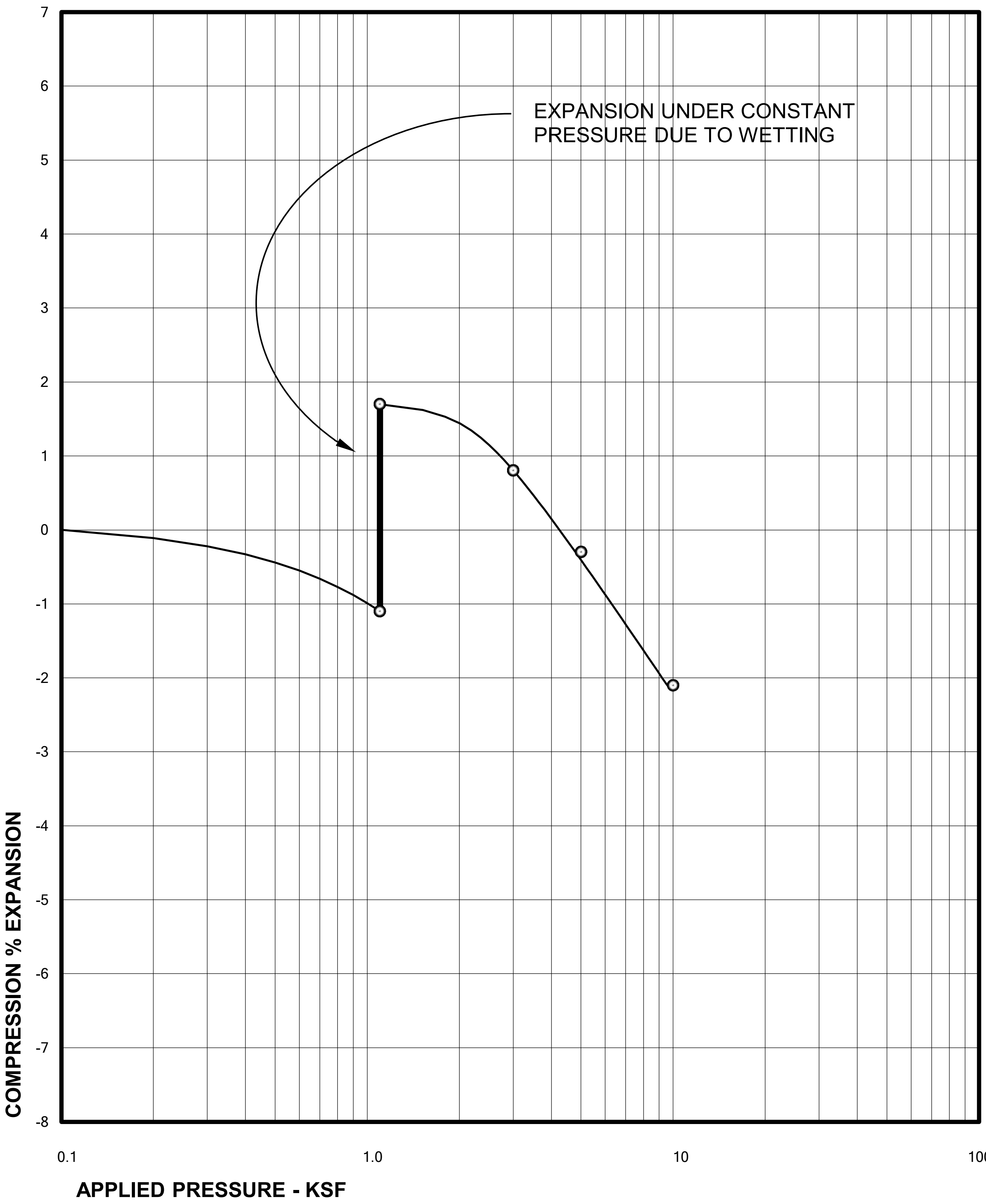
DRY UNIT WEIGHT= 127 PCF

MOISTURE CONTENT= 11.4 %

WEIDNER APARTMENTS
ARROWSWEST APARTMENTS
CTL|T PROJECT NO. CS19280-120

Swell Consolidation Test Results

FIG. B-6



Sample of CLAY, SANDY (CL)

From TH-12 AT 9 FEET

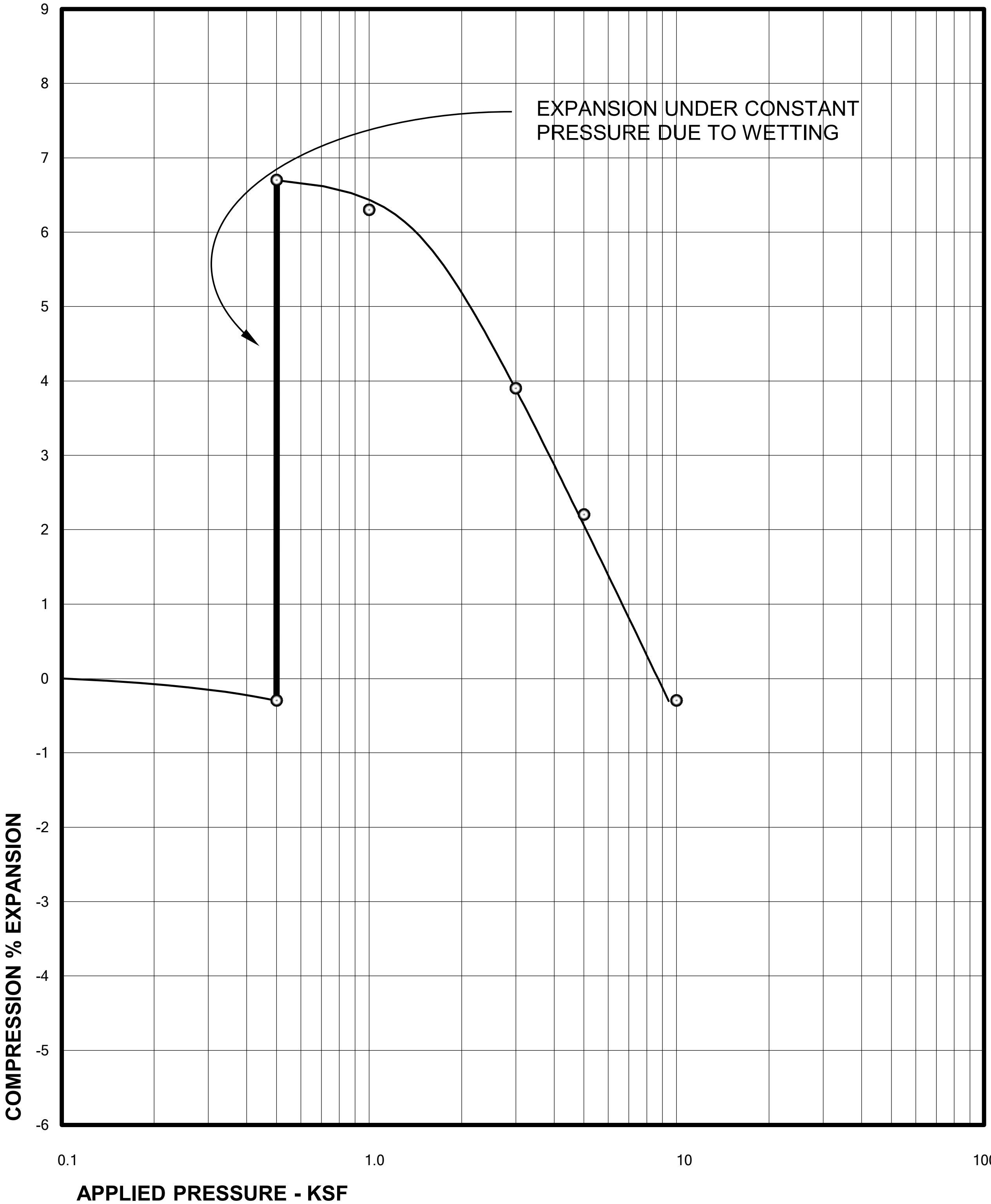
DRY UNIT WEIGHT= 112 PCF

MOISTURE CONTENT= 17.4 %

WEIDNER APARTMENTS
ARROWSWEST APARTMENTS
CTL|T PROJECT NO. CS19280-120

Swell Consolidation Test Results

FIG. B-7

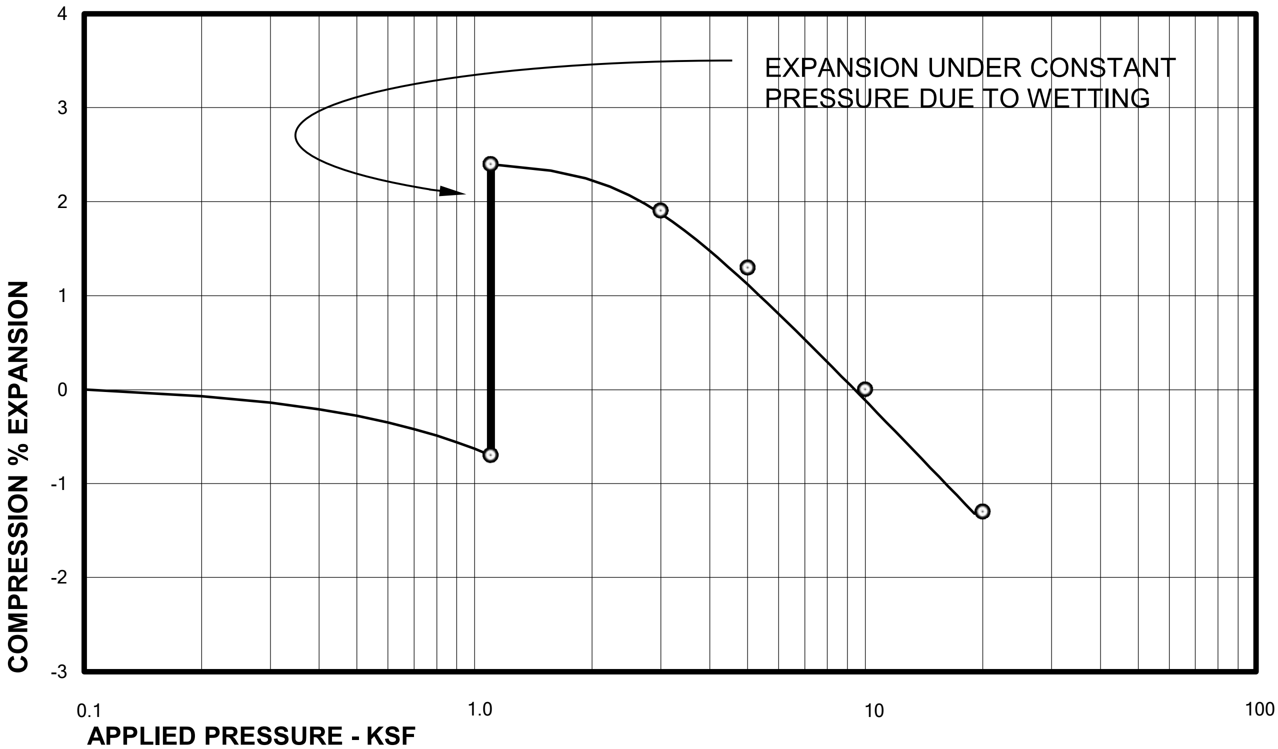


Sample of	CLAY, SLIGHTLY SANDY (CL)		DRY UNIT WEIGHT=	108	PCF
From	TH-14 AT 4 FEET		MOISTURE CONTENT=	14.2	%

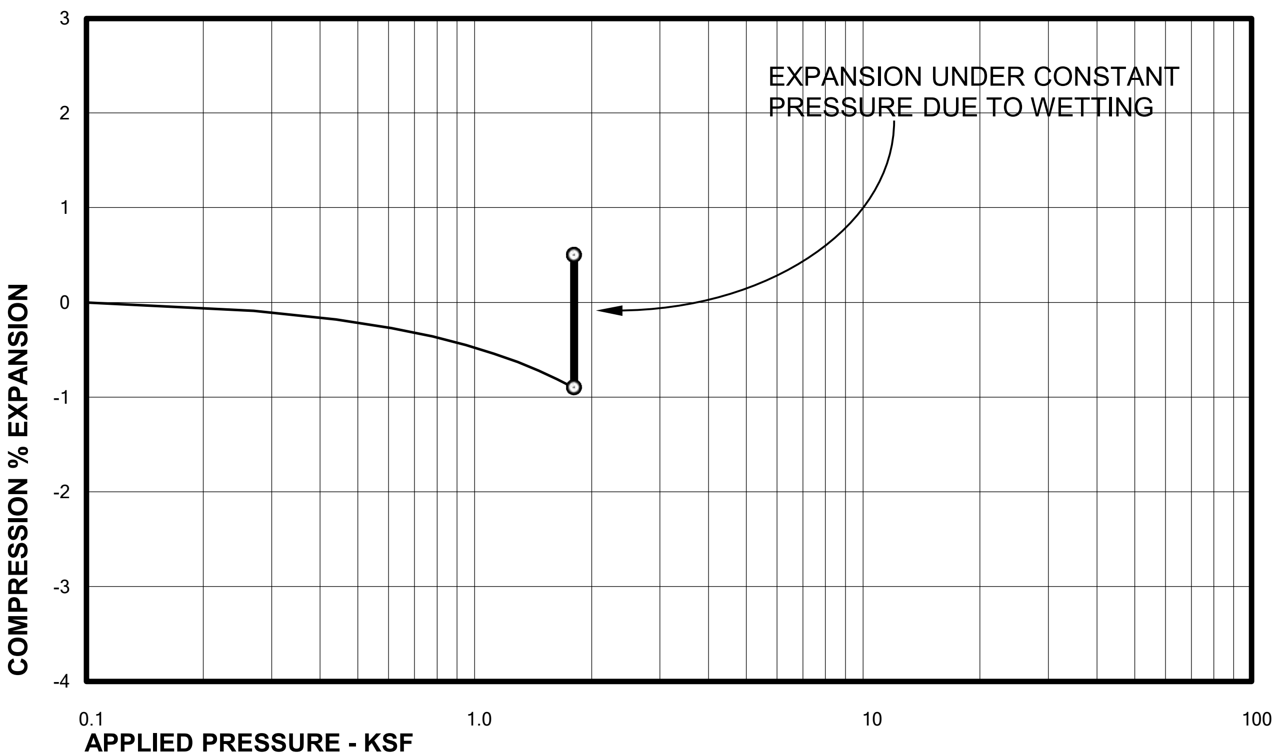
WEIDNER APARTMENTS
ARROWSWEST APARTMENTS
CTL|T PROJECT NO. CS19280-120

Swell Consolidation Test Results

FIG. B-8



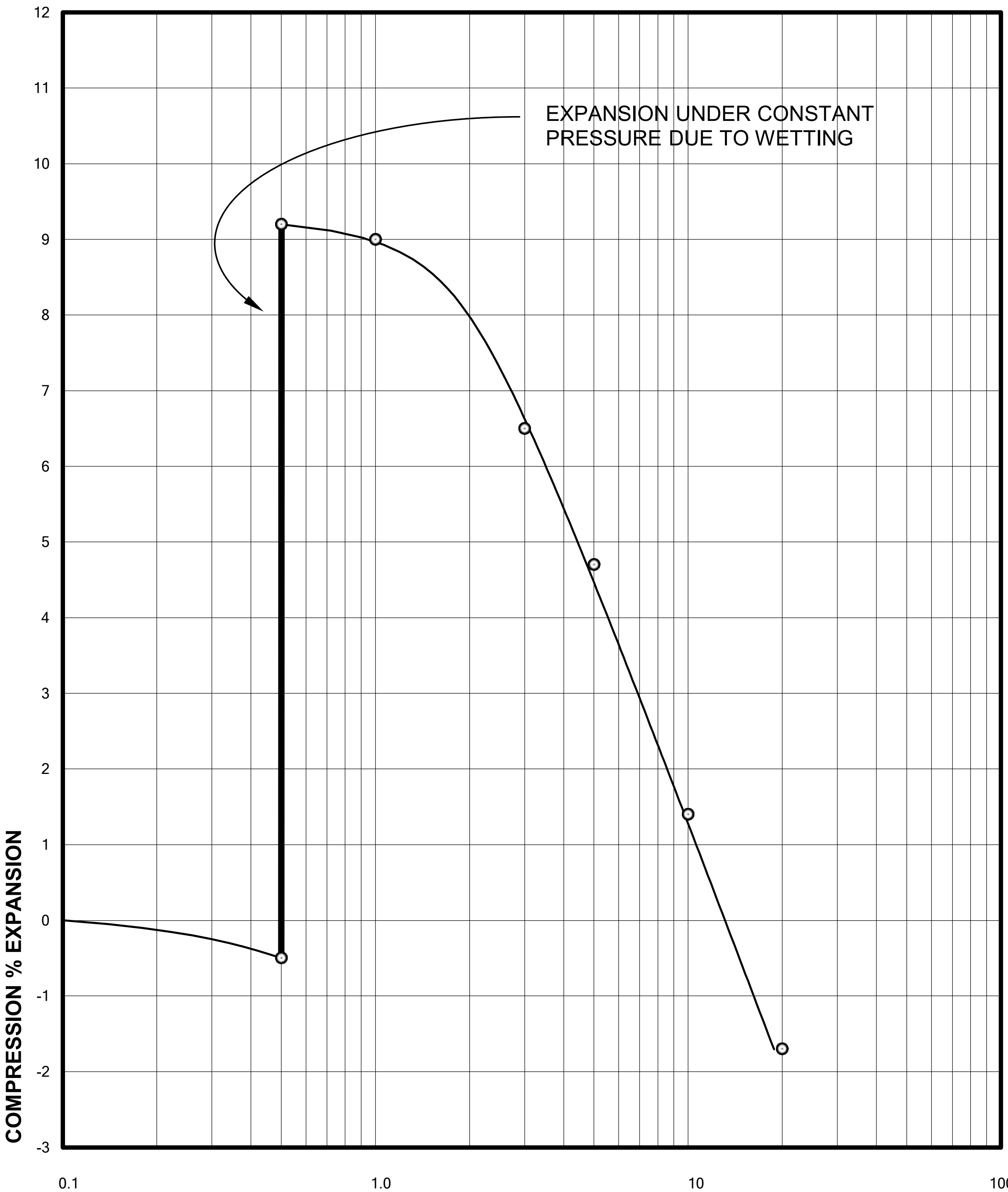
Sample of	CLAYSTONE	DRY UNIT WEIGHT=	121	PCF
From	TH-16 AT 9 FEET	MOISTURE CONTENT=	14.0	%



Sample of	CLAYSTONE	DRY UNIT WEIGHT=	122	PCF
From	TH-16 AT 14 FEET	MOISTURE CONTENT=	14.1	%

Swell Consolidation Test Results

FIG. B-9



Sample of FILL, CLAY, SANDY

From TH-18 AT 4 FEET

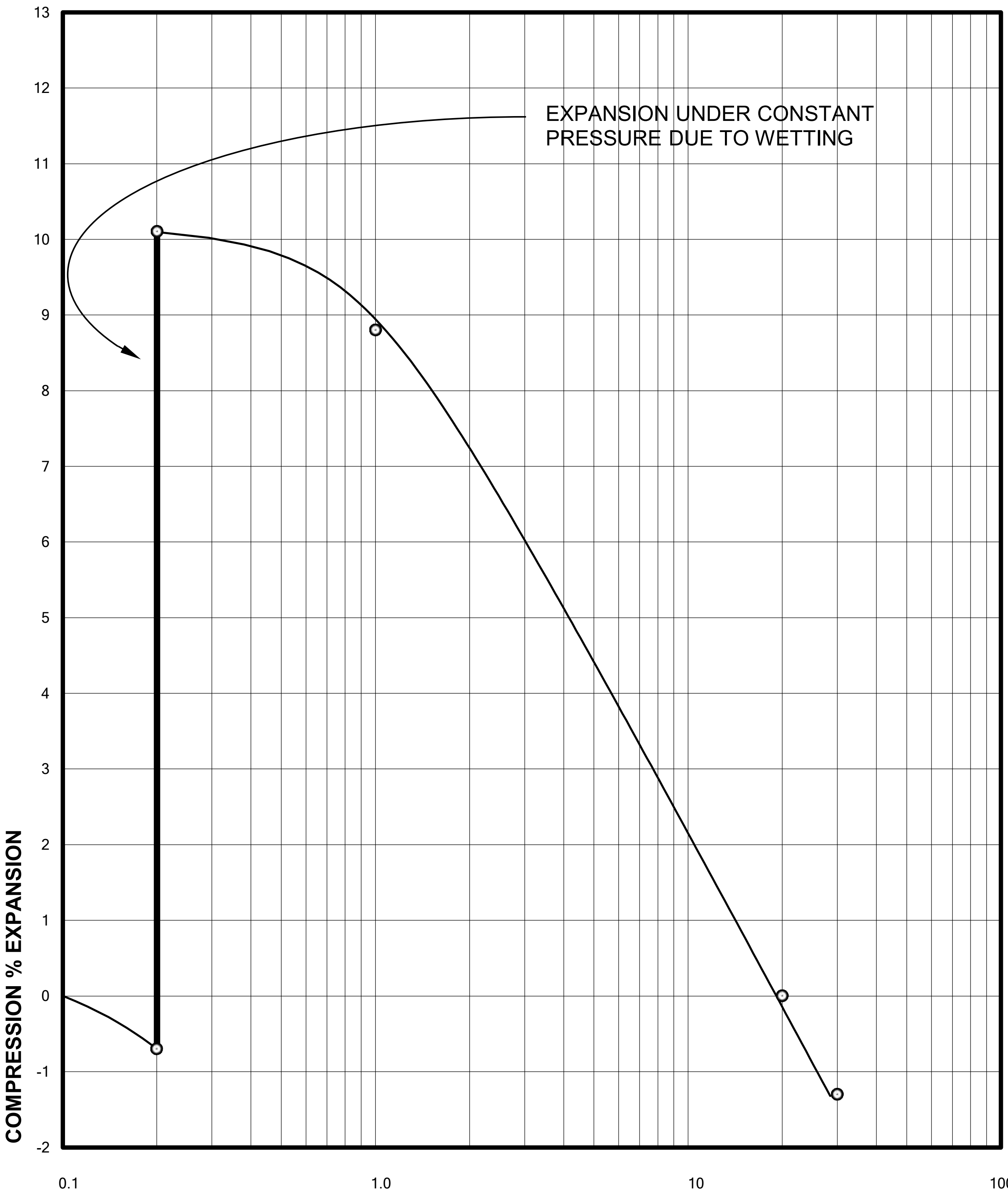
DRY UNIT WEIGHT= 116 PCF

MOISTURE CONTENT= 12.1 %

Swell Consolidation Test Results

FIG. B-10

WEIDNER APARTMENTS
ARROWSWEST APARTMENTS
CTL|T PROJECT NO. CS19280-120



Sample of FILL, CLAY, SANDY

From S-2 AT 1 FEET

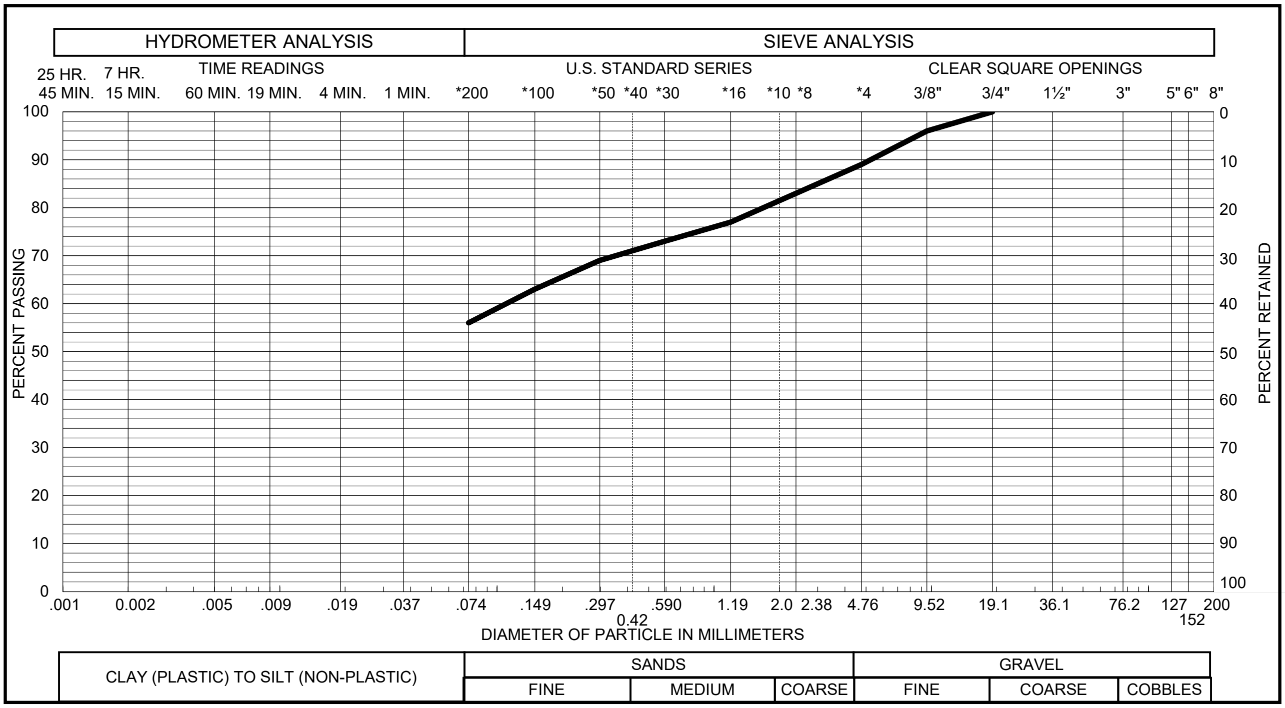
DRY UNIT WEIGHT= 124 PCF

MOISTURE CONTENT= 13.3 %

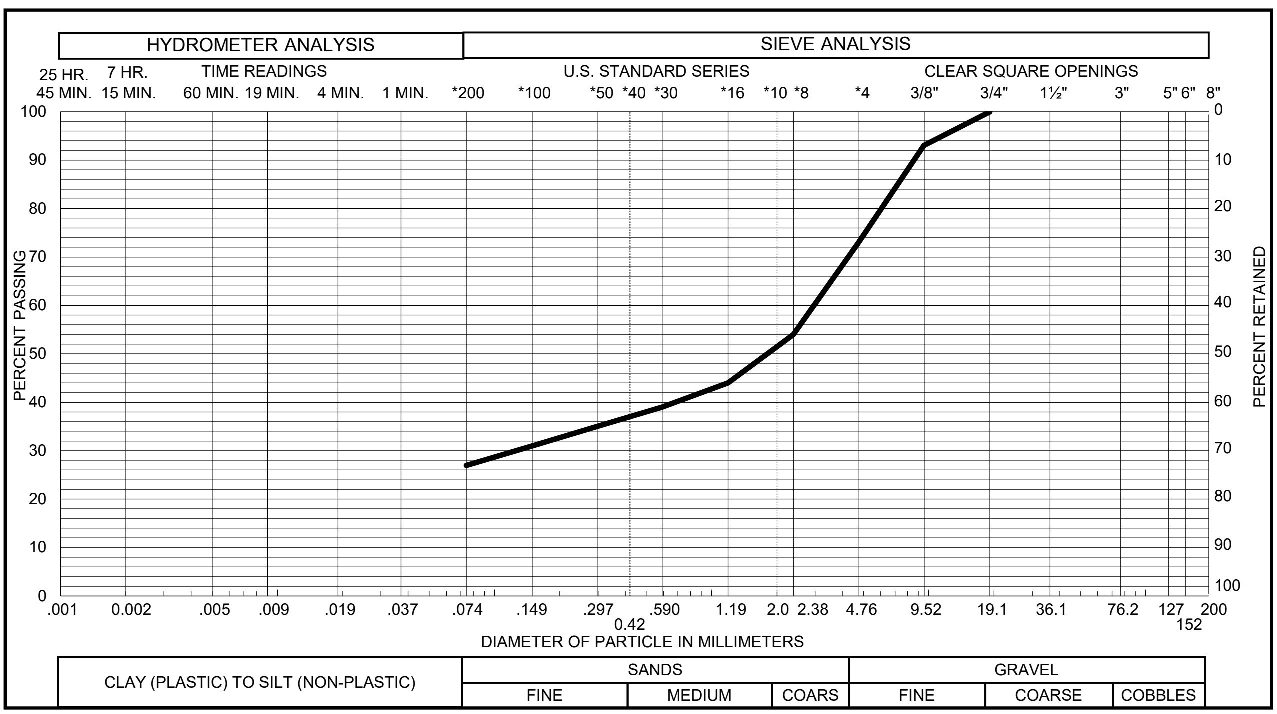
WEIDNER APARTMENTS
ARROWSWEST APARTMENTS
CTL|T PROJECT NO. CS19280-120

Swell Consolidation Test Results

FIG. B-11



Sample of	FILL, CLAY, VERY SANDY (CL)	GRAVEL	11 %	SAND	33 %
From	TH - 4 AT 4 FEET	SILT & CLAY	56 %	LIQUID LIMIT	
		PLASTICITY INDEX			

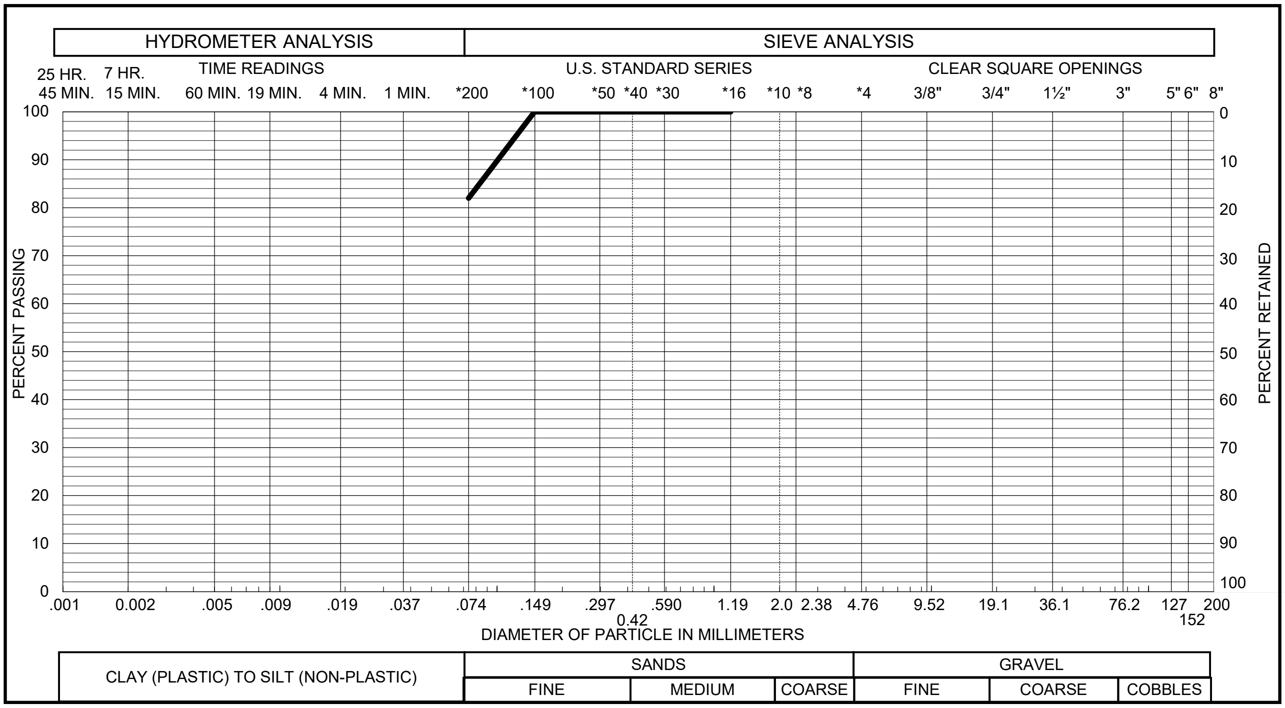


Sample of	SAND, CLAYEY, GRAVELLY (SC)	GRAVEL	27 %	SAND	46 %
From	TH - 17 AT 4 FEET	SILT & CLAY	27 %	LIQUID LIMIT	
		PLASTICITY INDEX			

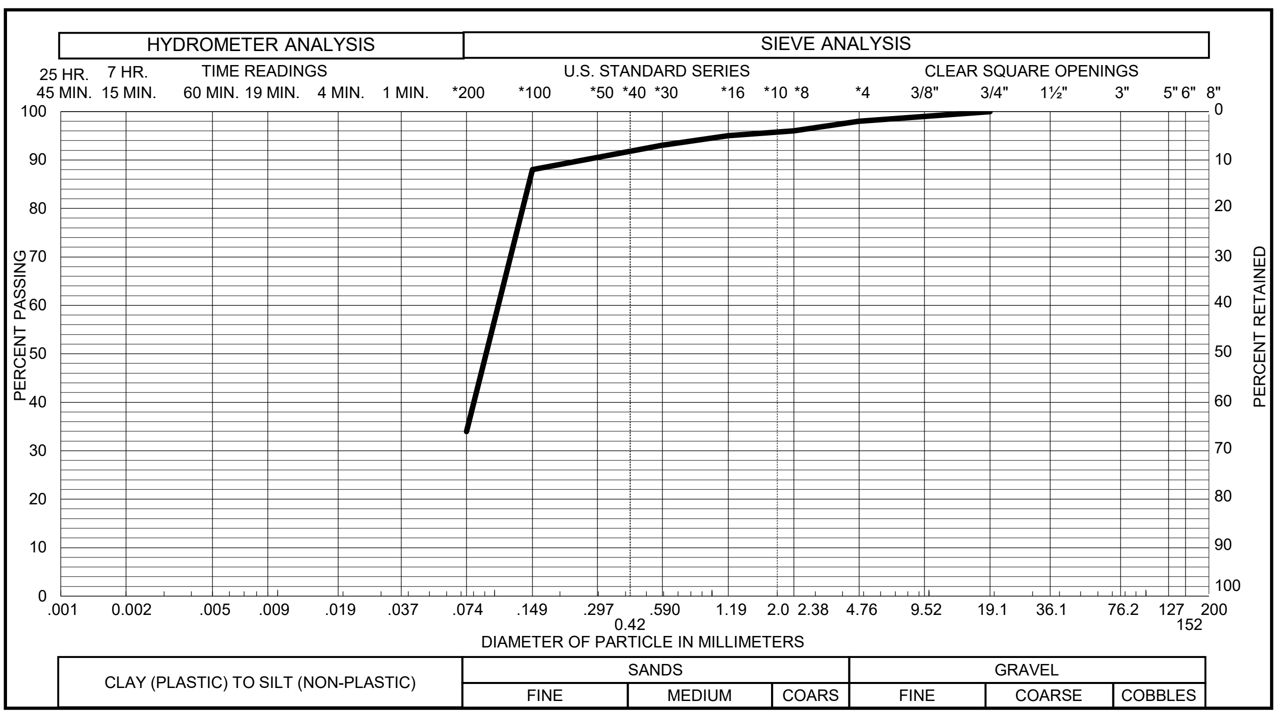
Gradation Test Results

FIG. B-12

WEIDNER APARTMENTS
ARROWSWEST APARTMENTS
CTL|T PROJECT NO. CS19280-120



Sample of	FILL, CLAY, SANDY	GRAVEL	0 %	SAND	18 %
From	TH - 19 AT 4 FEET	SILT & CLAY	82 %	LIQUID LIMIT	
		PLASTICITY INDEX			



Sample of	CLAY, SANDY (CH)	GRAVEL	2 %	SAND	64 %
From	S - 4 AT 0-4 FEET	SILT & CLAY	34 %	LIQUID LIMIT	
		PLASTICITY INDEX			

Gradation Test Results

FIG. B-13

WEIDNER APARTMENTS
ARROWSWEST APARTMENTS
CTL|T PROJECT NO. CS19280-120

TABLE B - I

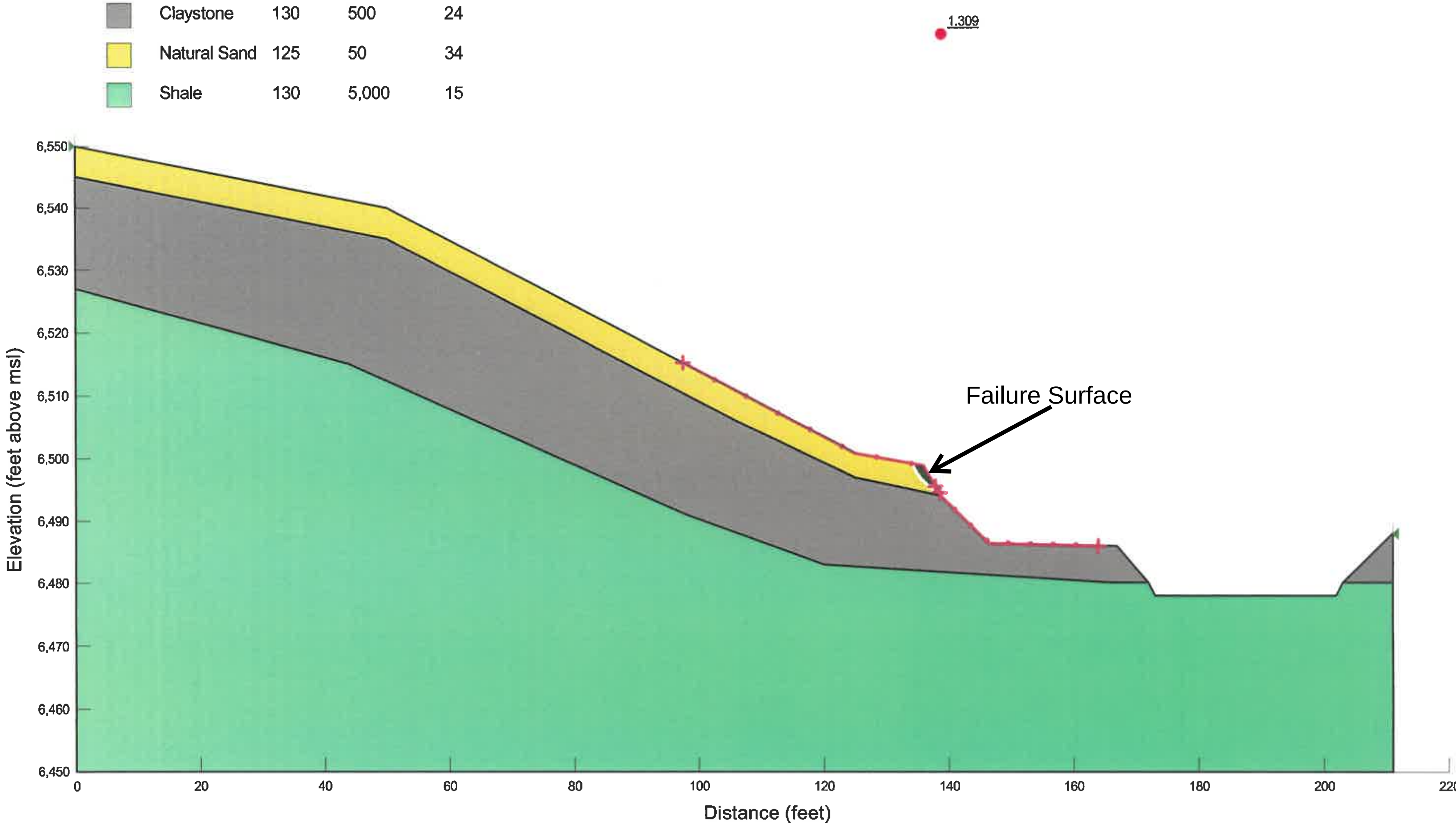
SUMMARY OF LABORATORY TEST RESULTS

BORING	DEPTH (ft)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	SWELL TEST DATA SWELL (%)	APPLIED PRESSURE (psf)	ATTERBERG LIMITS LIQUID LIMIT	PLASTICITY INDEX	SOLUBLE SULFATE CONTENT (%)	PASSING NO. 200 SIEVE (%)	SOIL TYPE
TH-2	4	11.5	105			40	21	0.245	62	FILL, CLAY, SANDY, SLIGHLTY GRAVELLY
TH-3	9	25.7	97			49	28		87	FILL, CLAY, SANDY, SLIGHLTY GRAVELLY
TH-4	4	8.7	111						56	CLAY, VERY SANDY (CL)
TH-6	4	20.9	109	5.3	500				91	CLAY, SLIGHTLY SANDY (CL)
TH-6	9	19.9	110	5.5	1,100	65	40		100	CLAYSTONE
TH-7	4	16.1	115	5.7	500				99	CLAYSTONE
TH-8	9	10.9	129	3.8	1,100				99	CLAYSTONE
TH-9	14	11.5	128	1.0	1,800				99	CLAYSTONE
TH-11	4	11.4	127	5.6	500				99	CLAYSTONE
TH-12	9	17.4	112	2.8	1,100				85	CLAY, SANDY (CL)
TH-14	4	14.2	108	7.0	500				89	CLAY, SLIGHTLY SANDY (CL)
TH-16	9	14.0	121	3.1	1,100				99	CLAYSTONE
TH-16	14	14.1	122	1.4	1,800				98	CLAYSTONE
TH-17	4	5.9	129						27	SAND, CLAYEY, GRAVELLY
TH-18	4	12.1	116	9.7	500	54	34	0.03	85	CLAY, SANDY (CL)
TH-19	4	12.1	123						82	FILL, CLAY, SANDY
S-2	1	13.3	124	10.8	200				98	FILL, CLAY, SANDY
S-4	0-4	12.1				53	34		83	CLAY, SANDY (CH)

APPENDIX C

SLOPE STABILITY ANALYSIS

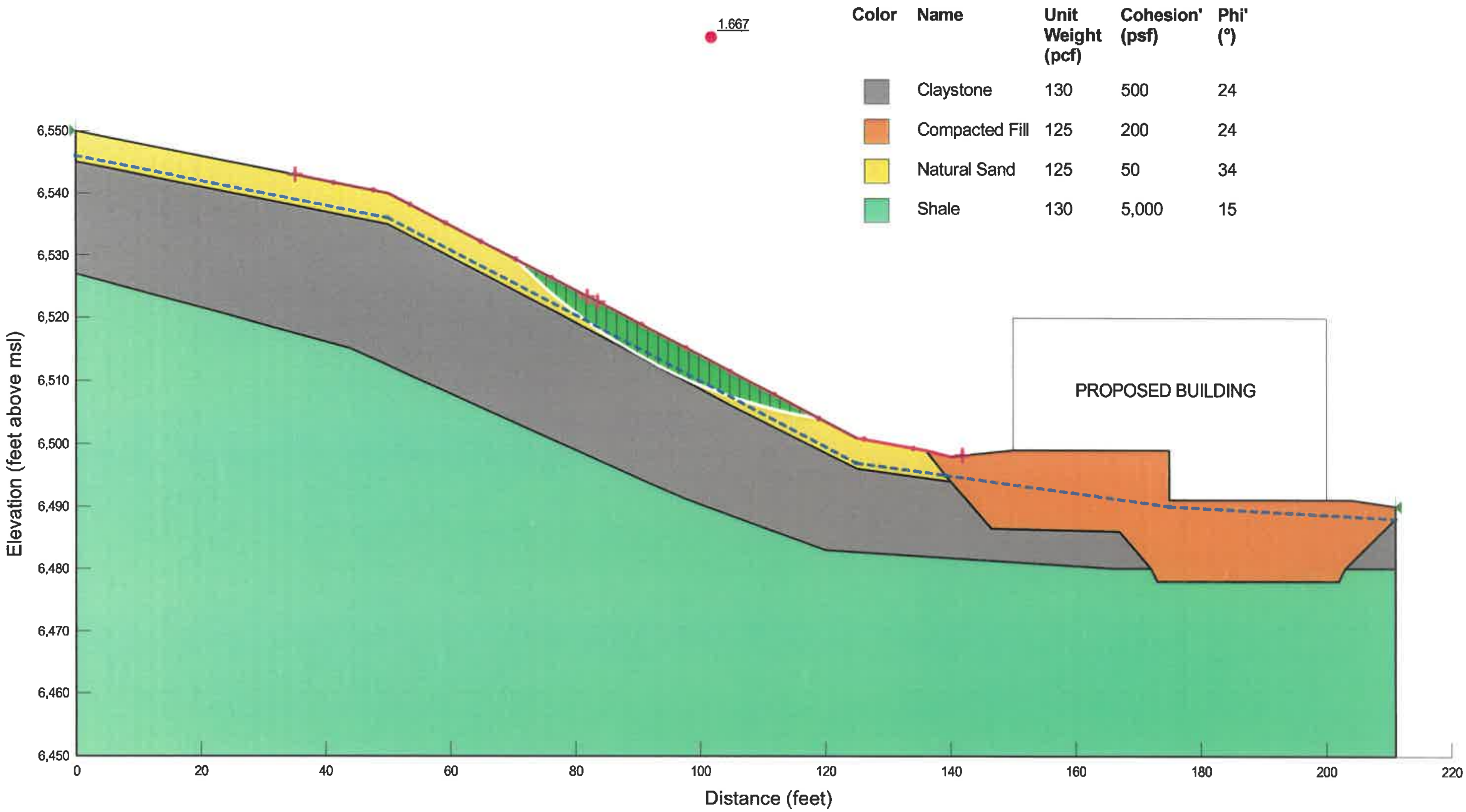
Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Claystone	130	500	24
	Natural Sand	125	50	34
	Shale	130	5,000	15



WEIDNER APARTMENT HOMES
ARROWSWEST APARTMENTS
CTL|T PROJECT NO. CS19280-120

DURING CONSTRUCTION - 10 FT DEEP SUB-EXCAVATION

FIG. C-1



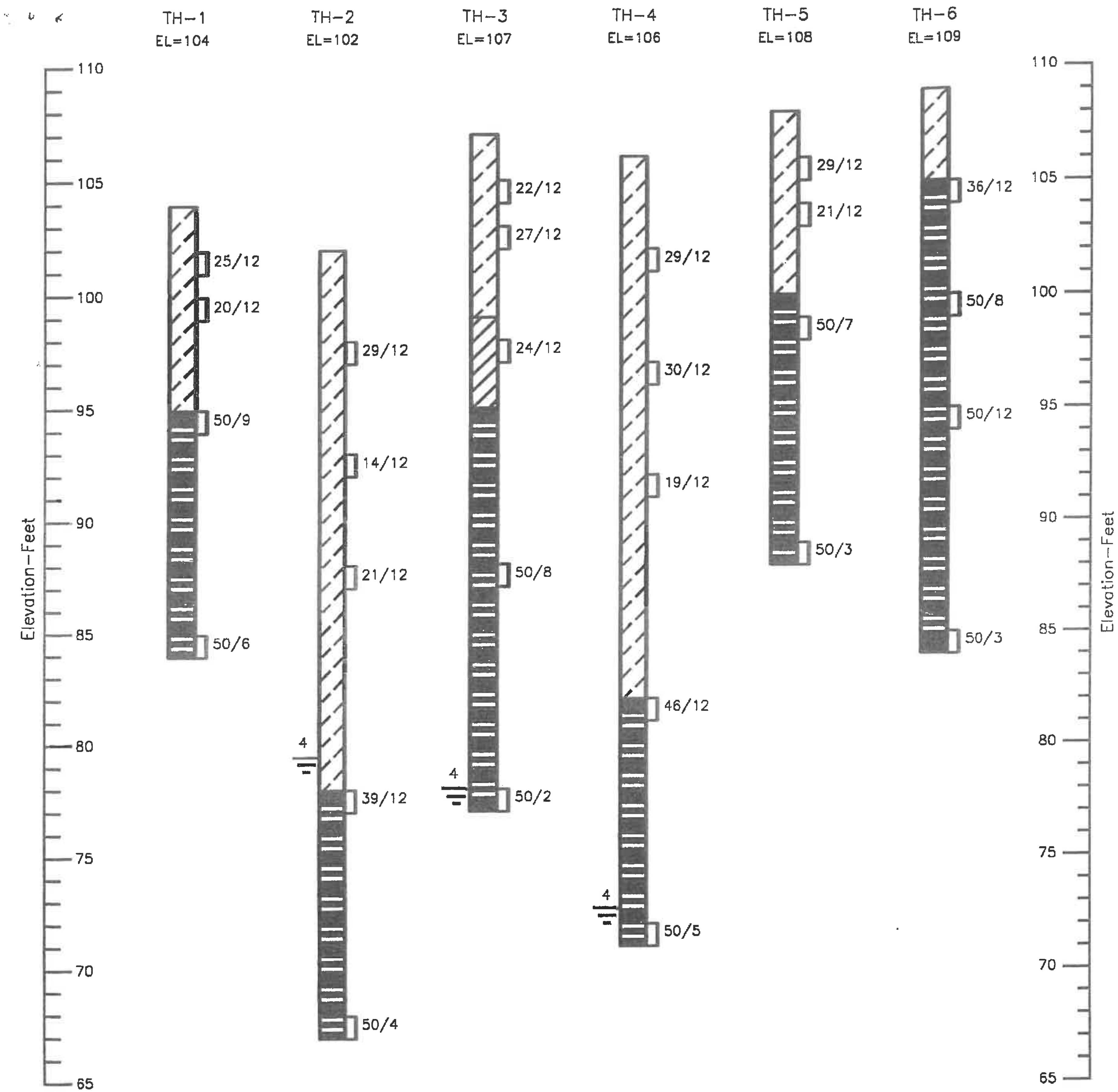
WEIDNER APARTMENT HOMES
ARROWSWEST APARTMENTS
CTL|T Project No. CS19280-120

PERMANANT CONDITION WITH PROPOSED BUILDING

FIG. C-2

APPENDIX D

SUMMARY LOGS OF PREVIOUS EXPLORATORY BORINGS



LEGEND:

Clay, sandy, very stiff, moist, brown (CL).

Severely weathered claystone, sandy, medium hard, moist, brown.

Claystone, sandy, medium hard to very hard, moist, brown, olive, dark gray.

Drive Sample. The symbol 25/12 indicates that 25 blows of a 140-pound hammer falling 30 inches were required to drive a 2.5-inch O.D. sampler 12 inches.

4 Indicates level of groundwater and the number of days after drilling the measurement was taken.

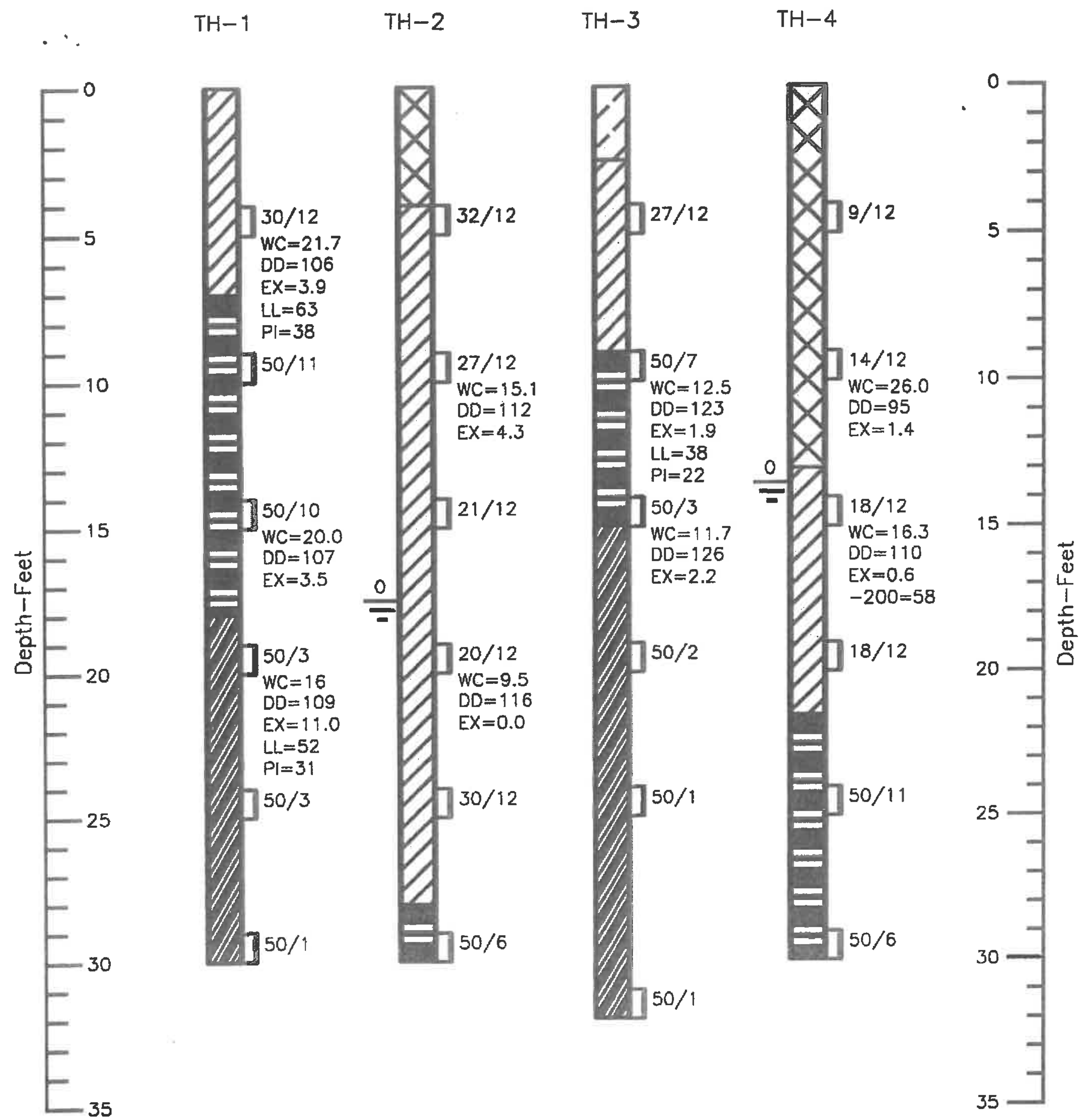
NOTES:

1. The borings were drilled November 16, 1998 using a 4-inch diameter, continuous-flight, truck-mounted, power auger.
2. The borings are subject to the explanations, limitations, and conclusions as contained in the report.
3. The elevations shown are approximate and were determined using a hand level and the benchmark shown on Fig. 1.

CONNELL CONSTRUCTION COMPANY
LOT 11, ARROWSWEST INDUSTRIAL PARK
Job No. CS-9300

Logs of Exploratory Borings

Fig. 3



LEGEND:

Fill, sandy clay and clayey sand, stiff, slightly moist to moist, olive to reddish brown.

Clay, sandy, slightly moist to moist, olive brown.

Bedrock. Weathered claystone, sandy, medium hard, steeply dipping bedding, slightly moist, gray to olive brown.

Bedrock. Claystone, sandy, hard to very hard, steeply dipping bedding, slightly moist, gray.

Bedrock. Shale, very hard, steeply dipping bedding, slightly moist, dark gray.

Drive Sample. The symbol 30/12 indicates that 30 blows of a 140-pound hammer falling 30 inches were required to drive a 2.5-inch O.D. sampler 12 inches.

0 Indicates the level of groundwater and the number of days after drilling the measurement was taken.

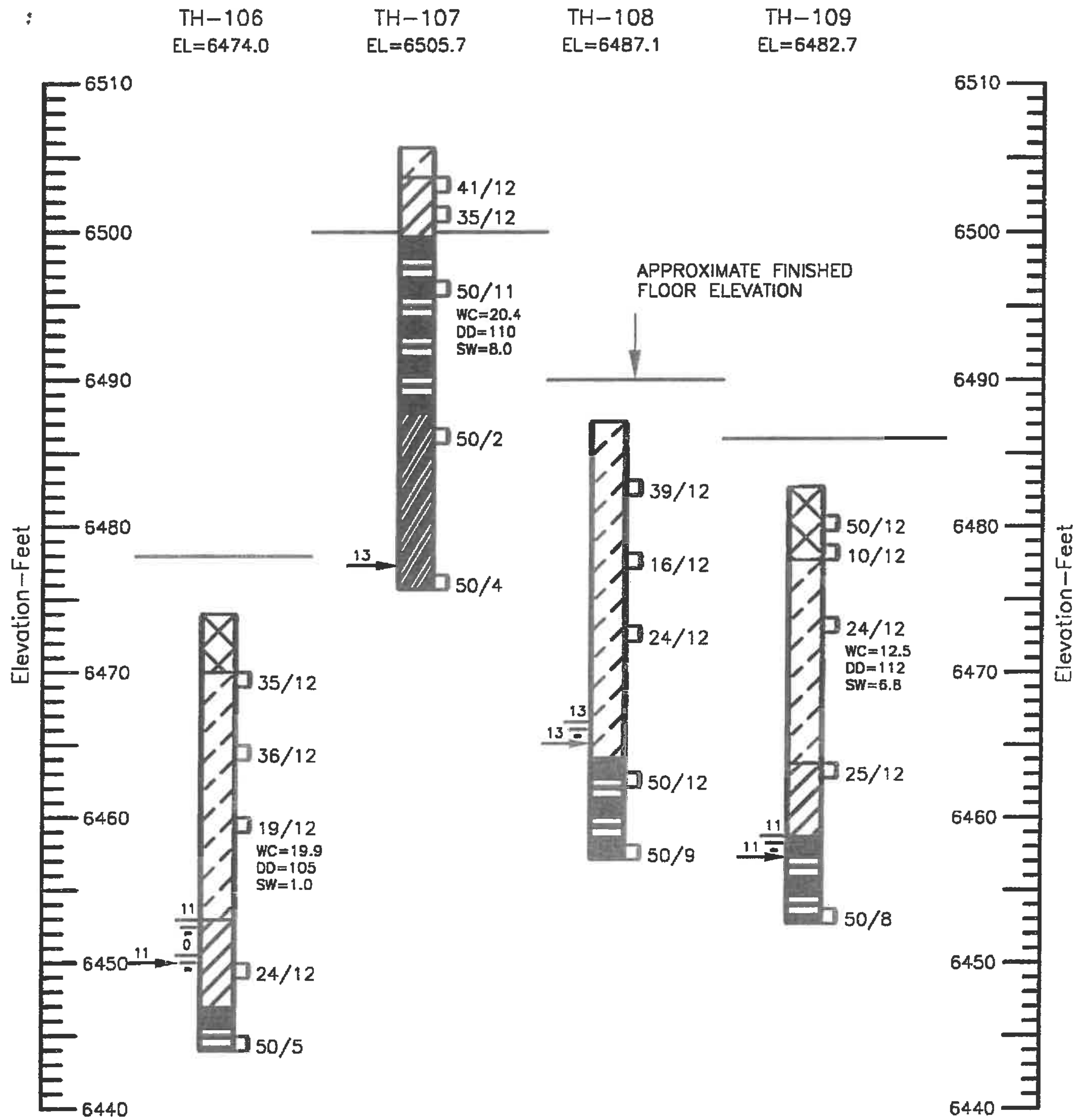
NOTES:

1. The borings were drilled March 31 and April 5, 2000 using a 4-inch diameter, continuous-flight, truck-mounted, power auger.
2. The borings are subject to the explanations, limitations, and conclusions as contained in the report.
3. WC – Indicates natural moisture content (%)
   DD – Indicates dry density (pcf)
   EX – Indicates percent expansion when sample wetted under a surcharge pressure of 1000 psf
   -200 – Indicates the percent passing the No. 200 sieve.
   LL – Indicates liquid limit (%).
   PI – Indicates plasticity index (%).

DELAY AND ASSOCIATES
ARROWSWEST, PARCELS 11 AND 12
Job No. CS-10.,738

Logs of Exploratory Borings

Fig. 2



LEGEND:

Fill, sand, clayey and clay, sandy, slightly gravelly to gravelly, medium dense to dense (sand) and very stiff (clay), slightly moist to moist, brown and olive.

Clay, slightly sandy to sandy, very stiff, slightly moist to moist, brown and tan. (CL, CH)

Bedrock Weathered claystone, near vertical bedding, with trace gypsum and iron staining, medium hard, slightly moist to moist, brown, gray and olive.

Bedrock. Claystone, near vertical bedding, with trace gypsum and iron staining, hard to very hard, slightly moist to moist, gray and olive.

Bedrock. Shale, near vertical bedding, very hard, slightly moist, gray.

Drive Sample. The symbol 28/12 indicates that 28 blows of a 140–pound hammer falling 30 inches were required to drive a 2.5–inch O.D. sampler 12 inches.

0 Indicates level of groundwater and the number of days after drilling the measurement was taken.

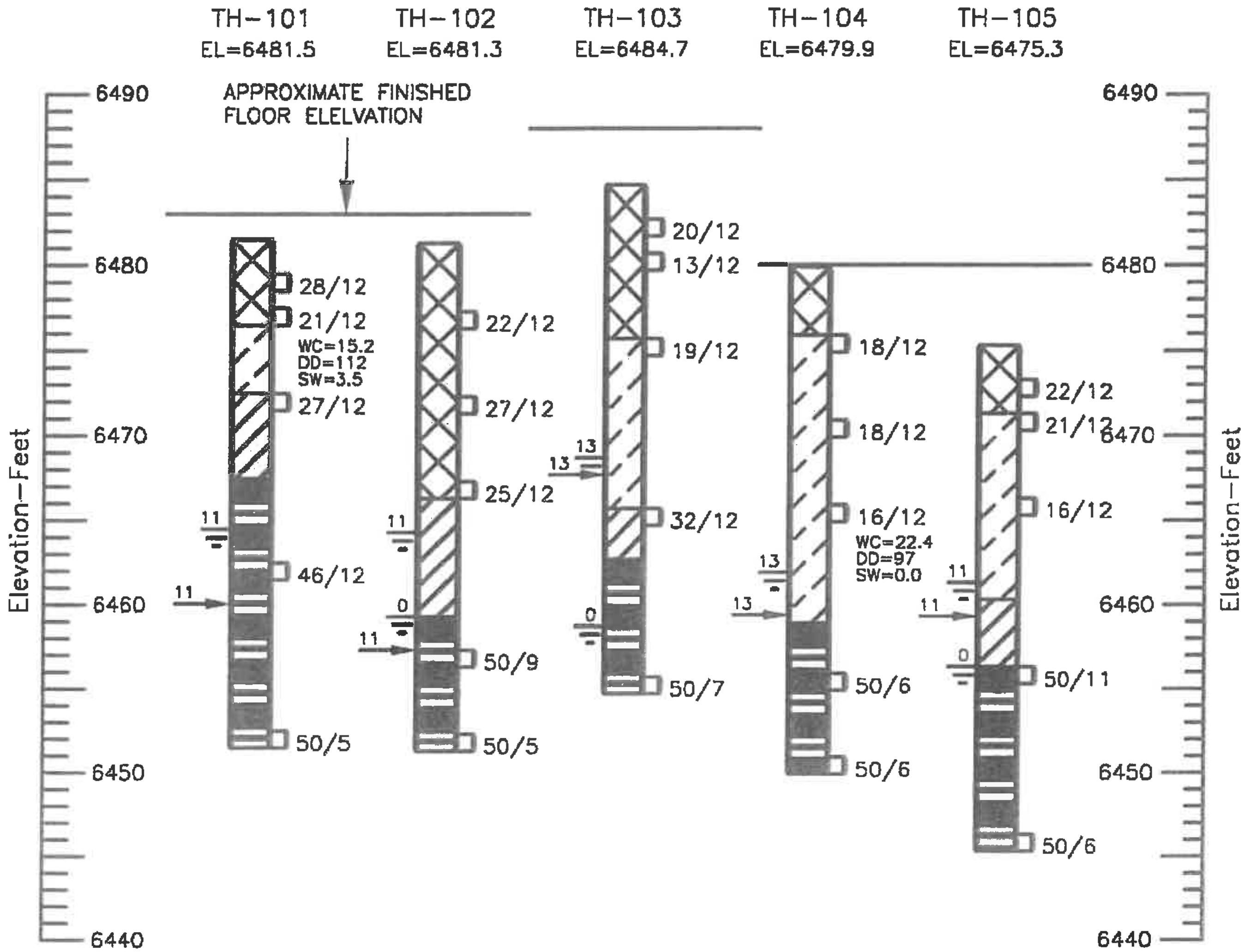
11 Indicates depth test hole caved and the number of days after drilling the measurement was taken.

NOTES:

1. The borings were drilled February 19 and 21, 2002 using a 4–inch diameter, continuous–flight, truck–mounted, power auger.
2. The borings are subject to the explanations, limitations, and conclusions as contained in the report.
3. The elevations shown are approximate and were provided by Classic Consulting Engineers and Surveyors, LLC.
4. WC – Indicates natural moisture content (%).
   DD – Indicates dry density (pcf).
   SW – Indicates percent swell when sample wetted under an applied pressure of 1000 psf.

Logs of Exploratory Borings

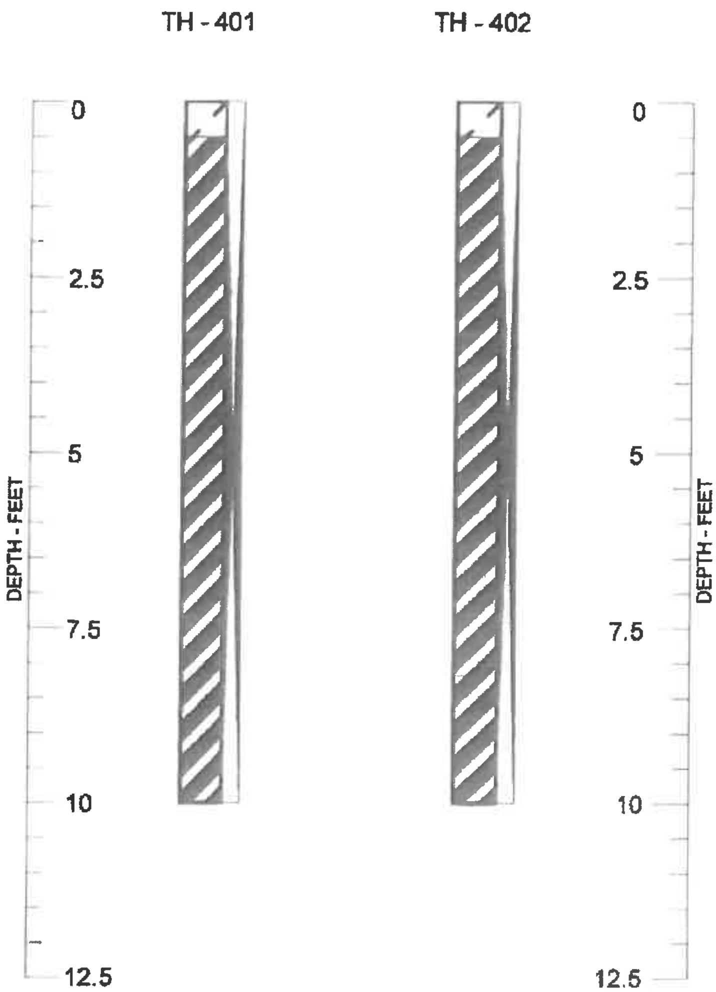
WESTERRA DEVELOPMENT COMPANY
ARROWS WEST LOTS 11 & 12
Job No. CS–12,228



WESTERRA DEVELOPMENT COMPANY
ARROWSWEST LOTS 11 & 12
Job No. CS-12,228

Logs of Exploratory Borings

Fig. 3



LEGEND:

CLAY, SANDY, MOIST, MEDIUM BROWN. (CL)

WEATHERED CLAYSTONE, MEDIUM HARD TO HARD, MOIST, MEDIUM BROWN.

INDICATES BULK SAMPLE OBTAINED FROM AUGER CUTTINGS.

NOTES:

1. THE BORINGS WERE DRILLED APRIL 22, 2008 USING A 4-INCH DIAMETER, CONTINUOUS-FLIGHT AUGER AND A CME-55, TRUCK-MOUNTED DRILL RIG.
2. THESE LOGS ARE SUBJECT TO THE EXPLANATIONS, LIMITATIONS, AND CONCLUSIONS AS CONTAINED IN THIS REPORT.
3. GROUND WATER WAS NOT ENCOUNTERED IN THE EXPLORATORY BORINGS DURING THIS INVESTIGATION.

Summary Logs of Exploratory Borings

FIG. 3

COLORADO SPRINGS
OLYMPIC CITY USA

Geologic Hazard Study Report

Applicant:	Weidner Apartment Homes	Telephone:	425-250-7312
Address:	9757 NE Juanita Drive, Suite 300	Email:	davida@weidner.com
City/State:	Kirkland, Washington	Fax:	
Zip Code:	98034		

The following documents have been included and considered as part of this report (checked off by individual(s) preparing the geologic report):

- [x] Development Plan
- [ ] Landscape Plan (if applicable)
- [ ] Grading Plan
- [ ] Drainage Report (necessary if debris and/or mud flow hazard is present)

ENGINEER'S STATEMENT

I hereby attest that I am qualified to prepare a Geologic Hazard Study in accordance with the provisions of Section 504 of the Geologic Hazards Ordinance of Colorado Springs. I am qualified as:

- [ ] A Professional Geologist as defined by CRS 34-1-201(3); or,
- [x] A Professional Engineer as defined by Board Policy Statement 50.2 - "Engineers in Natural Hazard Areas" of the Colorado State Board of Registration for Professional Engineers and Professional Land Surveyors. Board authority as defined by CRS 12-25-107(1).

Submitted by: CTL|Thompson, Inc. Jeff Jones, PE Date: September 14, 2020

This Geologic Hazard Study is filed in accordance with the Zoning Code of Colorado Springs, 2001, as amended.

City Engineer **Date** **Planning & Development Manager** **Date**