GEOTECHNICAL INVESTIGATION

RIDGEGATE MESA TOPS PEDESTRIAN BRIDGE
SOUTHWEST OF RIDGEGATE PARKWAY AND INTERSTATE 25
LONE TREE, COLORADO

Prepared for:

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Project No. DN52,378.000-125-R1

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SCOPE

This report presents the results of our Geotechnical Investigation for the Ridgegate Mesa Tops Pedestrian Bridge that will span the proposed extension of Cabela Drive southwest of Ridgegate Parkway and Interstate 25 in Lone Tree, Colorado (Fig. 1). The purpose of our investigation was to evaluate the subsurface conditions and provide geotechnical design and construction criteria for the bridge. The scope was described in a Contract Modification dated February 13, 2025 (DN-24-0331-CM1).

This report is based on conditions found in exploratory borings, results of field and laboratory tests, previous investigation, engineering analysis of field and laboratory data, and our experience. The American Association of State Highway Transportation Official's (AASHTO) Load and Resistance Factor Design (LRFD) Bridge Design Specifications, 9th Edition, 2020 was considered during the preparation of this report. The report contains descriptions of the strata encountered in exploratory borings, discussion of foundation support of abutment and wingwalls, recommended design and construction criteria for foundations, and drainage. The recommendations are based on the construction as currently planned. Revisions to the planned construction could affect our recommendations. If the construction will differ from the descriptions herein, we should be contacted to review our recommendations and determine if revisions are needed. A summary of our conclusions and recommendations follows, with more detailed discussion and design criteria presented in the report.

SUMMARY OF CONCLUSIONS

1. Strata encountered in exploratory borings generally consisted of about 2 to 6 feet of native sand or clay underlain by sandstone, interbedded claystone/sandstone, and weathered and comparatively weathered claystone bedrock to the maximum depth explored of 60 feet. The clay and clayey bedrock are expansive and the sand and sandstone are judged to be non-expansive.

2. Groundwater was encountered in all three borings at approximate depths of about 22 to 42 feet below existing grades (or approximate elevations 6243 to 6245 feet). Groundwater may be encountered during installation of deep foundations. Groundwater can fluctuate seasonally and may rise in response to changes in land use, precipitation, landscape irrigation, and the planned residential development upon the mesa top.

3. The presence of expansive soils and bedrock and steep slopes constitutes a geologic hazard. The potential for erosion and scouring is also a concern. We calculated 1 inch to 2½ inches of potential movement at the proposed ground surface based on a 24-foot depth of wetting. It is not certain this movement will occur. There is risk that improvements will heave or settle and be damaged. We believe the recommendations presented in this report will help to control risk of damage; they will not eliminate that risk.

4. We judge that drilled piers bottomed in bedrock is the safest foundation system from a geotechnical and constructability perspective. Complications for pier installation may include groundwater and lenses of very hard bedrock. Design and construction criteria for foundations are presented in the report.

5. Surface drainage should be designed, constructed, and maintained to provide rapid removal of runoff away from structures and flatwork to reduce potential subsurface wetting. Water should not be allowed to pond adjacent to structures or flatwork.

6. The design and construction criteria for foundation alternatives in this report were compiled with the expectation that all other recommendations related to grading, surface and subsurface drainage, backfill compaction, etc. will be incorporated into the project and that the structures and surface drainage will be maintained. It is critical that all recommendations in this report are followed.

SITE CONDITIONS AND PROPOSED CONSTRUCTION

The proposed pedestrian bridge is planned approximately 1 mile southwest of the Ridgegate Parkway and Interstate 25 intersection in Lone Tree, Colorado (Fig. 1 and Photo 1). Relatively steep slopes are present along the edges of the drainage. The site is vegetated with shrubs and trees along the slopes and grasses within the bottom of the drainage. The drainage slopes and conveys surface water to the north from the surrounding mesa top. Existing residential developments are located about ¼-mile north and ½-mile west and south of the site. The mesa top is currently used as a livestock pasture but it planned for residential development.

The bridge will realign a portion of the existing East/West Regional Trail to span the proposed extension of Cabela Drive located along the existing drainage. Two design options are being considered; one where the bridge spans the entire drainage and another incorporating a column at the center of the bridge. The bridge will likely be about 10 feet wide and 120-130 feet long, sit at a minimum height of 17.5 feet above Cabela Drive, and include abutments and wing walls. We anticipate up to about 10 feet of fill at the eastern abutment and less than about 5 feet of fill to about 10 feet of cut for the western abutment. Foundation loads have not been provided at this time.

Photo 1 – Google Earth© aerial site photo, October 2023. Approximate bridge location in red.

PREVIOUS INVESTIGATIONS

Our firm performed a Preliminary Geotechnical Investigation within the Ridgegate Mesa Tops development under our Project No. DN52,036-115-R1 and presented our results in a report dated November 20, 2023. We drilled and sampled 19 widely-spaced exploratory borings. We encountered sandy clay and clayey sand over claystone and sandstone bedrock. Bedrock was very hard and practical drill rig refusal was encountered in nine borings at depths of 12 to 21 feet below existing grades. The primary geotechnical concerns identified were expansive clay and claystone and potentially unstable slopes.

We have recently performed a Preliminary Geotechnical Investigation for the proposed Cabela Road extension that the bridge is planned to overpass. We presented results in a report and letter under our Project No. DN52,378-115, dated March 12 and April 18, 2025, respectively. We drilled and sampled eleven exploratory borings along the road alignment. We encountered sandy clay and clean to clayey sand underlain by weathered and comparatively unweathered claystone bedrock. Data from our previous investigations was considered in preparation of this report.

INVESTIGATION

Subsurface conditions were investigated on May 16, 2025, by drilling and sampling three exploratory borings at the west and east abutment locations and near the center of the drainage (Fig. 1). Boring locations and elevations were staked and surveyed by others. The borings were advanced to depths of 30 to 60 feet using 4-inch diameter, continuous-flight, solid-stem auger powered by a truck-mounted drill rig. Samples were obtained at approximate 2- to 5-feet intervals using 2.5-inch diameter (O.D.) modified California barrel samplers driven by blows of an automatic 140-pound hammer falling 30 inches. Bulk samples of the auger cuttings from the upper 5 feet were collected at each boring. Our field representatives were present to observe drilling operations, log the strata encountered, and obtain samples. Graphical boring logs are presented in Appendix A.

Samples obtained during drilling were returned to our laboratory where they were examined and testing was assigned. Laboratory tests included moisture content, dry density, swell-consolidation, Atterberg limits, particle size analysis (gradation and percent passing the No. 200 sieve), unconfined compressive strength, and water-soluble sulfate concentration. Swell-consolidation tests were performed by wetting samples under approximate overburden pressures (i.e. the pressure exerted by the overlying soil). Load-back analysis was performed on the swell tests to help estimate swelling pressures. Results of laboratory tests are presented in Appendix B and summarized in Table B-I.

SUBSURFACE CONDITIONS

Strata encountered in exploratory borings generally consisted of about 2 to 6 feet of colluvial/alluvial deposits comprised of sand and clay underlain by sandstone, interbedded claystone/sandstone, and weathered and comparatively unweathered claystone bedrock to the maximum depth explored of 60 feet. Some pertinent engineering characteristics of the subsoils are described in the following paragraphs.

Native Soils

We encountered about 2 feet of native clayey sand (colluvium) on the slope of the drainage in TH-1 and TH-2. Within the drainage in TH-3, we encountered about 6 feet of native sandy clay (alluvium). The clay was very stiff based on results of resistance penetration testing. A bulk sample of the clay contained 58 percent silt- and clay-sized particles (passing the No. 200 sieve) and exhibited moderate plasticity with a liquid limit of 44 and plasticity index of 25. The bulk sample from TH-1, containing a mix of native sand and sandstone bedrock, contained 34 percent silt- and clay-sized particles and 7 percent gravel (retained on the No. 4 sieve) and exhibited moderate plasticity with a liquid limit of 33 and plasticity index of 15. Based on testing from previous investigations, the clay is expansive. We judge the sand is non-expansive to low swelling. The slopes along the drainage are likely covered with relatively thin layers of sand dominated colluvium while the overburden soil within the drainage is likely thicker and consists of clay dominated alluvium.

Bedrock

The bedrock directly below the overburden soils in our borings along the slopes (TH-1 and TH-2) consists of about 4 to 10 feet of sandstone bedrock overlaying about 20 to 26 feet of claystone bedrock. Below the soils in our boring within the drainage (TH-3) we encountered about 2 feet of weathered claystone overlaying about 11 feet of comparatively unweathered claystone bedrock. Underlying the claystone in each boring is primarily sandstone bedrock with zones of interbedded claystone/sandstone bedrock. The unweathered claystone was medium hard to very hard, the interbedded bedrock was hard, and the sandstone was hard to very hard. Layers of relatively harder sandstone were encountered at both shallow and deeper depths. One sandstone sample contained 21 percent silt- and clay- sized particles. Six claystone samples swelled 0.1 to 2.1 percent and one did not swell when wetted. Five claystone samples developed load-back swell pressures of approximately 3,300 to 19,500 psf. Five claystone samples had unconfined compressive strengths of approximately 8,960 to 16,200 psf. Three claystone samples contained 55 to 77 percent silt- and clay-sized particles and exhibited moderate plasticity with liquid limits of 40 to 54 and plasticity indices of 20 to 22.

Groundwater

Groundwater was encountered during drilling in TH-1 at a depth of about 54 feet. When the borings were checked after drilling on May 21, 2025, groundwater was measured in all three borings at depths of about 22 to 42 feet below grade (approximate elevations 6243 to 6245). Groundwater may be encountered during installation of deep foundations. Groundwater may fluctuate seasonally and develop and rise in response to changes in land use, precipitation, landscape irrigation, and the planned residential development upon the mesa top.

SITE GEOLOGY

The 1977 Geologic map of the Highlands Ranch quadrangle[1] indicates the site is primarily underlain by the Upper Part of the Dawson Arkose sandstone facies (map symbol Tda) and the Castle Rock Conglomerate (map symbol Tcr). Based on our borings, the Dawson formation contained much thicker layers of claystone than the mapping implied. The Castle Rock Conglomerate is a pebble, cobble, and boulder arkosic conglomerate and can be difficult to excavate. Overburden soils primarily consist of colluvial deposits with lesser amounts of alluvial deposits. The geologic map indicates landslides have occurred on this mesa to the north of the subject site. Based on our experience, there is a risk of landslides within the subject site. More details are contained below in **GEOLOGIC HAZARDS**.

[1] Maberry, J.O., and Lindvall, R.M. "Geologic map of the Highlands Ranch quadrangle, Arapahoe and Douglas Counties, Colorado," Geologic Quadrangle Map GQ-1413, 1977

Photo 2 – Excerpt from geologic map[1]. Approximate bridge location in red.

GEOLOGIC HAZARDS

Geologic hazards can affect development risks and costs. We believe potential hazards can be mitigated with proper engineering, design, and construction practices as discussed in this report. No geologic hazards were identified within the scope of this investigation which, in our opinion, will preclude development provided mitigation techniques as discussed in this report are implemented. The identified hazards include:

- Steep Slopes, Landslides, and Debris Flows
- Erosion and Scour
- Expansive Soil and Bedrock
- Seismicity

These hazards and conceptual mitigation methods are discussed in the following sections. The risks can be mitigated but not eliminated by careful design, construction and maintenance procedures.

Steep Slopes, Landslides, and Debris Flows

Moderate to steep slopes are present on this site. Existing slopes appear to be currently stable. As part of our evaluation, we reviewed the 2018 Landslide Inventory and Susceptibility Map of Douglas County, Colorado[2] and the 2018 Debris Flow Susceptibility Map of Douglas County, Colorado[3]. Excerpts of these maps are shown below in Photos 2 and 3, respectively. There are limitations of these maps, associated with map scaling and scope of the investigation.

The landslide map (Photo 2) indicates there are relatively small zones of "medium susceptibility" for landslides adjacent to the project site. This susceptibility rating was based on mapped geology and slope angle. We are aware of post-construction slope failures on nearby sites. Slope stability analysis is outside the scope of this investigation. Addition of water to the subsurface from precipitation or irrigation reduces relative stability and may initiate slope failures. Surface drainage should be designed, constructed, and maintained to provide rapid removal of surface runoff.

The debris flow map (Photo 3) indicates debris flow hazard areas are present downstream of the drainage the bridge will overpass. The hazard rating was based on evaluation of source areas and modeled runout zones. In large precipitation events, erosion and debris/mud flows can occur through these pathways. We judge there is currently low susceptibility of debris flows at the project site due to relatively minimal source areas of material. Risk may increase with development and land-use changes on the mesa top. The project may increase the debris flow susceptibility downstream. The Civil Engineer should evaluate drainage basins present on the site slopes and design surface drainage structures that will reduce the risk of debris flows.

[2] Lindsey, Kassandra, "Landslide Inventory and Susceptibility for Douglas County, Colorado," Colorado Geological Survey, (Open File Report, OF-18-07), 2018.

[3] McCoy, Kevin M., F. Scot Fitzgerald, Matthew L. Morgan, and Karen A. Berry. "Debris Flow Susceptibility Map of Douglas County, Colorado," Colorado Geological Survey (Open File Report, OF-18-08), 2018.

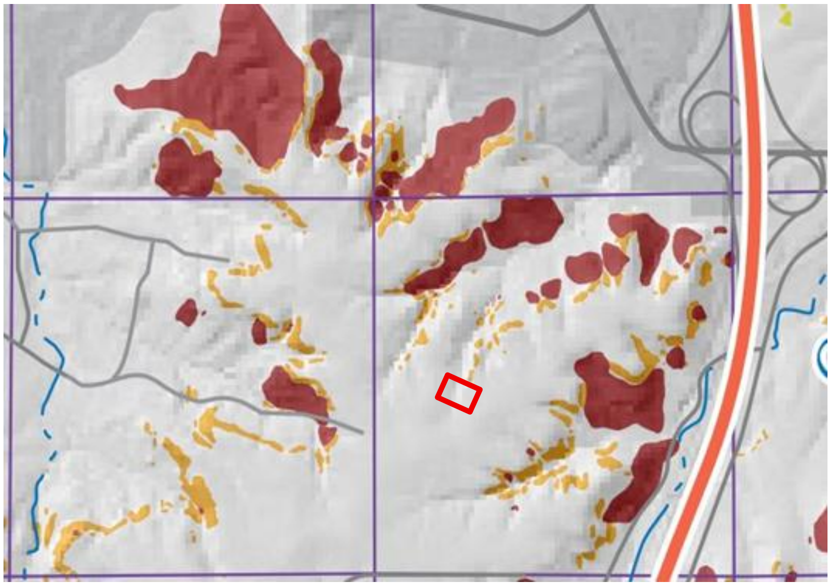


Photo 2: Excerpt from the Landslide Inventory and Susceptibility Map of Douglas County. Approximate site boundary outlined in red.

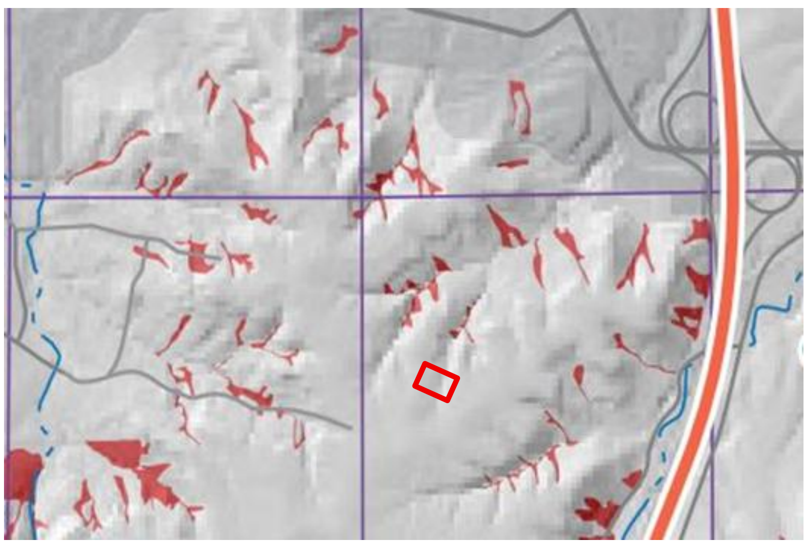


Photo 3: Excerpt from the Debris Flow Susceptibility Map of Douglas County. Approximate site boundary outlined in red.

Permanent, Unretained Slopes

We did not observe evidence of slope instability. Soil cut and fill slopes no steeper than 3H:1V (horizontal:vertical) should be stable. Slopes of 4H:1V are preferable. Engineered slope retention systems, such as wing walls, should be used if slopes steeper than 3H:1V are planned. There is potential for differential settlement and slope movements where new fill is placed on steep slopes. Sliding can result along sloping interfaces between new fill and existing ground. An effective way to deal with this issue is to bench slopes steeper than about 20 percent prior to fill placement as shown on Fig. 2. Addition of water to the subsurface from precipitation or landscape irrigation will likely reduce relative stability and may initiate slope failures. Surface drainage should be designed, constructed, and maintained to provide rapid removal of surface runoff. Global stability analysis results are presented in Appendix C and discussed further in the **Slope Stability Analysis** section.

Erosion and Scour

The topography of the site encourages water to flow from the higher elevations, through the drainage pathways, towards the lower lying areas. During peak precipitation events, some accumulation of surface sheet flow in the drainage is expected. Development will increase the number of impervious surfaces and may alter established drainage paths, which can lead to drainage problems and erosion and scour if surface water flow is not adequately designed. Scour of the soil by flowing water around bridge supports can cause bridge failure. Sandy soils are highly susceptible to scour. Slopes will require erosion control during and after construction. Scour protection, to some extent, should be considered. Revegetation or other erosion control measures should be employed.

Expansive Soil and Bedrock

Expansive soils and bedrock are present at this site. There is risk that ground heave or settlement will damage the proposed improvements. We performed estimates of potential heave by dividing the soil profile into layers and modeling the swell test result using a depth of wetting of 24 feet below the ground surface. Based on the subsurface profiles and swell-consolidation test results we have estimated the potential heave at the existing grade and proposed grade as shown in Table I below. Potential heave will be affected by site grading. Variation from our estimates should be anticipated.

TABLE I: ESTIMATED POTENTIAL HEAVE

Boring	Estimated Heave at Existing Grade (inches)	Estimated Heave at Proposed Grade (inches)
TH-1	1	1
TH-2	2	1½
TH-3	2½	2½

*Estimated heave is based on 24-feet depth of wetting from ground surface

Seismicity

According to the USGS, Colorado's Front Range and eastern plains are considered low seismic hazard zones. The earthquake hazard exhibits higher risk in western Colorado compared to other parts of the state. The Denver Metropolitan area has experienced earthquakes within the past 100 years, shown to be related to deep drilling, liquid injection, and oil/gas extraction. Naturally occurring earthquakes along faults due to tectonic shifts are rare in this area.

The soil and bedrock at this site are not expected to respond unusually to seismic activity. The current IBC and IRC defer the estimation of Seismic Site Classification to ASCE7-22, a structural engineering publication. Updates from the previous versions of ASCE7 include (1) incorporation of additional Site Classifications BC, CD, and DE, (2) removal of tabulated blow-count and shear-strength correlations to shear wave velocity, and (3) requires the engineer to reduce shear wave velocity values by a factor of 1.3 when empirically estimated or not directly measured. The table below summarizes ASCE7-22 Site Classification Criteria.

ASCE7-22 SITE CLASSIFICATION CRITERIA

Seismic Site Class	$\bar{v}_s$, Calculated Using Measured or Estimated Shear Wave Velocity Profile (ft/s)
A. Hard Rock	>5,000
B. Medium Hard Rock	>3,000 to 5,000
BC. Soft Rock	>2,100 to 3,000
C. Very Dense Sand or Hard Clay	>1,450 to 2,100
CD. Dense Sand or Very Stiff Clay	>1,000 to 1,450
D. Medium Dense Sand or Stiff Clay	>700 to 1,000
DE. Loose Sand or Medium Stiff Clay	>500 to 700
E. Very Loose Sand or Soft Clay	≥500
F. Soils requiring Site Response Analysis	See Section 20.2.1

Based on the results of our investigation, the reduced, empirically estimated average shear wave velocity values for the upper 100 feet range between 1,306 and 1,400 feet per second, with an average value of 1,355 feet per second. We judge the subsurface will likely classify as Seismic Site Classification CD. The field penetration test results along with the empirical estimates imply that shear-wave velocity seismic tests to directly measure $\bar{v}_s$ could likely result in a better Seismic Site Classification. The subsurface conditions indicate low susceptibility to liquefaction from a materials and groundwater perspective.

SITE DEVELOPMENT

Excavation

Excavations made at this site may be governed by local, state, or federal guidelines or regulations. Subcontractors should be familiar with these regulations and take whatever precautions they deem necessary to comply with the requirements and thereby protect the safety of their employees and the public.

We believe the soils encountered in our exploratory borings can be excavated with conventional, heavy-duty excavation equipment. Lenses of very hard, cemented sandstone were encountered during a previous investigation nearby. There may be layers of relatively harder sandstone bedrock that could require more effort to excavate or the use of rock excavation techniques such as heavy ripping or rock saws. We recommend the owner and the contractors become familiar with applicable local, state and federal safety regulations, including the current OSHA Excavation and Trench Safety Standards. We believe the clay and claystone will classify as Type B soil and the sand and sandstone as Type C, which require maximum slope inclinations of 1:1 and 1½:1 (horizontal:vertical), respectively, for temporary excavations in dry conditions. Flatter slopes will be required if seepage is present. The contractor's "competent person" is required to 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. Permanent unretained slopes should be designed no steeper than about 3H:1V and should be re-vegetated as soon as practical to control erosion; use of 4H:1V maximum slopes is preferable. A professional engineer should design excavations deeper than 20 feet.

Fill and Backfill

Onsite soils are suitable for reuse as new fill provided they are substantially free of vegetation, debris, oversized particles (larger than about 3 inches) and other deleterious materials. If import fill is necessary, it should ideally consist of similar material as currently onsite. Imported Colorado Department of Transportation (CDOT) Class 5 or 6 fill may also be used. A sample of import material should be submitted to our office for approval prior to importing to the site.

Prior to fill placement, the surface should be stripped of topsoil, scarified to a depth of at least 12 inches, moisture conditioned, and compacted to the criteria below. Subsequent fill should be placed in thin (8 inches or less) loose lifts, moisture conditioned to within 2 percent of optimum moisture content for sand and between 1 and 4 percent above optimum moisture content for clay. The fill should be compacted to at least 95 percent of standard Proctor maximum dry density (ASTM D 698). If Imported CDOT Class 5 or 6 fill is used, it should be compacted to at least 95 percent of the modified Proctor maximum dry density (ASTM D1557). Utility trench backfill, if any, should be moisture conditioned and compacted to Douglas County specifications.

Our experience indicates fill and backfill can settle under its own weight, even if properly compacted to the criteria provided above. We estimate potential settlement of about 1 to 2 percent of the fill thickness. Most of this settlement usually occurs during and soon after construction; for clayey fill, it may continue for longer. Heave or additional settlement may occur after development in response to wetting. There may be some areas where proposed grading will create non-uniform depths of fill below improvements. Where fill depths vary by more than 5 feet, sub-excavation or benching of existing slopes should be considered to create more uniform fill depths. **If fill will be placed on slopes of 20 percent or steeper the slopes should be benched prior to placing fill (Fig. 2**).

FOUNDATIONS

Our investigation indicates expansive clay and claystone are present at depths likely to influence the performance of foundations. Drilled piers bottomed in bedrock are the safest foundation type for the proposed bridge, abutments, and wing walls due to the presence of expansive materials and steep site topography.

Design and construction criteria for drilled shaft foundations are provided below. These criteria were developed from analysis of field and laboratory data and our experience. The foundation elements should be sized considering the unfactored side and end bearing resistance in the following table.

SUMMARY OF ULTIMATE GEOTECHNICAL FOUNDATION RESISTANCES

Subsurface Strata	Ultimate End Pressure (psf)	Ultimate Side Shear (psf)
Bedrock	35,000	3,500

Drilled shafts bottomed in bedrock are usually designed accounting for the nominal end bearing (tip) resistance plus the nominal side resistance along the shaft, neglecting the side resistance within the potential scour depth and the depth of shrinkage cracking. We recommend that the minimum shaft lengths be determined referring to Section 10.8 of the AASHTO LRFD Bridge Design Manual, 9th Edition. The static shaft lengths and minimum bedrock penetration should be determined by the structural engineer considering the factored loads and ultimate (nominal) end pressure and side shear values provided. The tabulated resistance factors should be applied by the structural engineer and have been used in this study to determine minimum lengths to resist uplift to due heave of expansive bedrock.

LRFD STATIC ANALYSIS RESISTANCE FACTORS DRILLED SHAFTS

Subsurface Material	Resistance Type	Static Resistance Factor (phi)
Bedrock	Side	0.60
	End/Tip	0.55

It is sometimes beneficial to verify the axial resistance of drilled shafts via static load testing. Side resistance and end resistance values obtained from load tests allows use of a higher resistance factor which in turn can result in shorter foundation lengths to resist axial loads than those predicted from static analysis methods. When the minimum foundation lengths are controlled by uplift of expansive soil and bedrock, in the case of these foundations, it is not usually beneficial to perform load testing as a value engineering measure. Load testing of drilled shafts may not provide cost-benefit value for this project but can be considered to check the geotechnical resistances used to design the foundations. The table below includes the resistance factors from load testing of drilled shafts (AASHTO LRFD Bridge Design Specifications, 9th Edition).

LRFD LOAD TEST RESISTANCE FACTORS DRILLED SHAFTS

Method of Capacity Determination (Load Tests)	Resistance Factor (phi)
Static Load Test, Compression	0.7
Static Load Test, Tension	0.6

Drilled Piers

1. The ultimate end and side shear pressures presented in the previous table should be appropriately factored by the structural engineer to be used for design. Side resistance in the soil should be neglected.

2. Potential scour depths will impact the design of foundations. The side resistance should be ignored along the foundation segment within the depth of potential scour.

3. Side resistance should be ignored in the overburden soils due to the potential for soil shrinkage away from the shaft diameter and the subsequent loss in geotechnical resistance.

4. Piers should have a minimum length of 30 feet and extend at least 6 feet into unweathered bedrock. Longer shafts may be necessary based on structural and scour considerations. The minimum diameter will depend on the length-to-diameter ratio (L/D). We recommend the piers be designed with a maximum L/D ratio not to exceed 30. We recommend drilling with a large, heavy-duty drill rig to facilitate the required bedrock penetration.

5. Formation of “mushrooms” or enlargements at the tops of piers should be avoided during pier drilling and subsequent construction operations.

6. A 4-inch or thicker continuous void should be constructed beneath all abutment walls, wing walls, and shaft caps to concentrate deadload onto the shafts and to prevent heave pressures on abutment walls between shafts. The void should be protected on the sides using a continuous high-strength plastic retainer that will prevent abutment backfill from migrating into the void and closing the void. The structural engineer should detail the void form system.

7. Shear rings should be installed in the penetration zone (lower 8 feet). Shear rings should have a height of at least 2 to 3 inches and extend 3 to 4 inches minimum beyond the pier shaft to increase the load transfer through skin friction. These shear rings should be spaced about 2 feet on-center for the design side shear penetration zone.

8. Pier drilling should produce shafts with relatively undisturbed bedrock exposed. Excessive remolding and caking of bedrock on pier walls should be removed. The bedrock surface should be rough or roughened. Pier drilling contractors should be required to have properly sized augers. Use of side cutters or teeth to increase the effective diameter should not be allowed.

9. Piers should be reinforced their full length and the reinforcement should extend an adequate distance into grade beams and pier caps. The reinforcement should be designed assuming an uplift force of (70 kips x pier diameter in feet minus the deadload) to resist tension in the event of swelling. Grade 60 (or better) steel is recommended. More reinforcement may be required for structural considerations. Reinforcement should be designed by the structural engineer.

10. Piers should have a center-to-center spacing of at least three pier diameters when designing for vertical loading conditions, or they should be designed as a group. Piers aligned in the direction of lateral forces should have a center-to-center spacing of at least 6 pier diameters.

11. Pier holes should be cleaned prior to placement of concrete. Use of temporary casing and concrete pumping may be required for proper dewatering and placement of concrete during drilled pier installation. Use of slurry to maintain holes for casing may be required. Concrete should be on-site and placed in the pier holes immediately after the holes are drilled, cleaned and inspected. Concrete should not be placed by free-fall in pier holes containing more than about 3 inches of water.

12. Concrete should be on-site and placed in the pier holes immediately after the holes are drilled, cleaned, and observed and reinforcing steel is set. Concrete placed in pier holes should have a slump of 6 inches (± 1 inch) to promote proper consolidation and reduce arching of concrete on the reinforcement and sides of the piers.

13. Some shaft-drilling contractors use casing with an O.D. equal to the specified shaft diameter. This practice results in a shaft diameter less than specified. The shaft design and specifications should consider the alternatives. If full-size casing is desired (I.D. of casing equal to specified pier diameter) it should be clearly specified. If the design considers the potential reduction in diameter, then the specifications should include a tolerance for a smaller diameter for cased piers.

14. Some movement of drilled pier foundations should be anticipated to mobilize the skin friction. We estimate this movement on the order of ¼ to ½-inch to mobilize skin friction. Differential movement may be equal to the total movement.

15. Installation of drilled piers should be observed by a representative of our firm to identify the bearing strata, confirm subsurface conditions are as anticipated and observe the contractor’s installation procedures.

Laterally Loaded Piles

Lateral load analysis of piles can be performed with the software analysis package LPILE by Ensoft, Inc. We believe this method of analysis is appropriate for piles with a pile length to width ratio of seven or greater. Lateral resistance factor of 1.0 is recommended by ASHTO for LRFD. Recommended soil input values for use with LPILE software is presented in the following table.

SOIL INPUT DATA FOR "LPILE"

	CDOT Class 5/6 Fill	Sand	Clay	Weathered Bedrock	Unweathered Bedrock
Soil Type Per L-Pile	Reese Sand	Reese Sand	Medium Stiff Clay w/o Free Water	Stiff Clay w/o Free Water	Hard Clay w/o Free Water
Effective Unit Weight (pcf)	125	122	120	123	125
Undrained Cohesion, c (psf)	-	-	500	2,000	6,000
Friction Angle, ø (degrees)	34	32	-	-	-
Soil Strain, ε_{50} (in/in)	-	-	0.01	0.005	0.004
p-y Modulus, k_s (pci)	60	60	100	1,000	2,000
p-y Modulus, k_c (pci)	-	-	-	400	800

The ε_{50} represents the strain corresponding to 50 percent of the maximum principal stress difference.

Closely-Spaced Pile Reduction Factors

For axial loading, no reduction is needed for a minimum spacing of three diameters (center-to-center). At one diameter (piles touching), the skin friction reduction factor for both piles would be 0.5. End pressure values would not be reduced provided the bases of the piles are at similar elevations. Interpolation can be used between one and three diameters.

For lateral loading, no reduction is needed for piles in-line with the direction of lateral loads with a minimum spacing of six widths (center-to-center) based upon the larger pile. If a closer spacing is required, the modulus of subgrade reaction for initial and trailing piles should be reduced. At a spacing of three widths, the effective modulus of subgrade reaction of the first pile can be estimated by multiplying the given modulus by 0.6; for trailing piles in a line at three-width spacing, the factor is 0.4. Linear interpolation can be used for spacing between three and six widths.

Reductions to the modulus of subgrade reaction can be accomplished in LPILE by inputting the appropriate modification factors for p-y curves. Reducing the modulus of subgrade reaction in trailing piles will result in greater computed deflections on these piles. In practice, a grade beam can force deflections of all piles to be equal. Load-deflection graphs can be generated for each pile by using the appropriate p-multiplier values. The sum of the piles lateral load resistance at selected deflections can be used to develop a total lateral load versus deflection graph for the system of piles. For lateral loads perpendicular to the line of piles, a minimum spacing of three widths can be used with no capacity reduction. At one width (piles touching) the piles should be analyzed as one unit. Interpolation can be used for intermediate conditions.

The above method has been used by our firm for years with success but sometimes results in overly conservative values. We believe the prediction equations proposed by Reese and Van Impe[4] result in more practical solutions for group efficiency. They were formulated by fitting curves to data representing group efficiency versus pile spacing. No differentiation was made between soil type, pile diameter, or penetration. The data indicates that for side-by-side piers, group efficiency becomes unity at spacing of about 4 pier diameters. For in-line piers, the lead piers were found to have efficiency of unity with spacing of about 4 diameters, and the trailing piers were unity efficiency with spacing of 7 diameters. The equations for solving group efficiency for side-by-side, leading and trailing piers are shown below, where the variable “s” is the pile spacing and “b” is the pile diameter.

Side-by-side piers:

$$e = 0.64\left(\frac{s}{b}\right)^{0.34} for\ 1 \leq \frac{s}{b} \leq 3.75, e = 1.0, \frac{s}{b} \geq 3.75 \qquad \textit{(Equation 5.39)}$$

Leading piers:

$$e = 0.7\left(\frac{s}{b}\right)^{0.26} for\ 1 \leq \frac{s}{b} \leq 4.0, e = 1.0, \frac{s}{b} \geq 4.0 \qquad \textit{(Equation 5.40)}$$

Trailing piers:

$$e = 0.48\left(\frac{s}{b}\right)^{0.38} for\ 1 \leq \frac{s}{b} \leq 7.0, e = 1.0, \frac{s}{b} \geq 7.0 \qquad \textit{(Equation 5.41)}$$

[4] “Single Piles and Pile Groups Under Lateral Loading,” Authored by Lymon C. Reese and William F. Van Impe, 2001; Section 5.7.5, Pages 158 and 159

For piers that are skewed at an angle (i.e. between in-line and side-by-side), the group efficiency is taken as a modification to shadow and edge effects. The efficiency can be estimated by:

$\mathrm{e} = ({e_\mathrm{i}}^2 \cos^2 \emptyset + {e_\mathrm{s}}^2 \sin^2 \emptyset)^2$; where e_i = efficiency of pile in-line,
e_s = efficiency of pier side-by-side, and
$\emptyset$ = angle between piers (Reese & Wang, 1996)

ABUTMENTS AND WING WALLS

We anticipate wing walls will be used on either side of each abutment. Wing walls should be constructed on the same foundation type as the abutments. The wing walls should be designed and constructed separate from the abutments and be sufficiently flexible so “active” earth pressure conditions can be used in design. The wing walls will likely be constructed in the embankment slopes. The backfill behind wing walls is anticipated to slope upwards.

Lateral earth pressures are dependent on the type of backfill, the slope of the backfill, and other surcharge loads. Fig. 3 presents methods for evaluating additional pressures due to additional loading conditions. The backfill behind walls should be compacted as described above in **Fill and Backfill**. Care should be taken when compacting backfill directly against the walls as to not damage the wall and increase lateral pressures. Waiting at least seven days after the walls are placed to allow the concrete to gain strength can reduce the risk of damage.

We recommend wing walls and abutments be provided with a wall drain system behind walls to reduce the risk of developing hydrostatic pressures. A subsurface drain detail is presented on Fig. 4. The drain should consist of a free-draining gravel layer and weep holes or slotted pipe to drain water from behind the walls. Miradrain or a similar manufactured drain is a suitable alternative.

Slope Stability Analysis

We performed slope stability analyses for the west and east bridge abutments. We analyzed the stability of the slopes to evaluate the global stability and risk of failure related to the proposed construction. To perform our analysis, we used proposed grading and retaining wall plans, estimated subsurface conditions and soil profile, and estimated strength properties of the soil and bedrock types within the subsurface profile. The following paragraphs summarize our analyses. Results are presented in Appendix C.

Two cross-sections were considered for each abutment, as shown on Fig. 5. We were provided with the “Pedestrian Bridge Over Hillcamp Drive” construction plans prepared by San Engineering, dated October 29, 2025, to create our cross sections. Our analyses and recommendations are based on construction as currently planned. Slope stability should be reevaluated if revisions are made to the planned construction.

Subsurface data was obtained from our exploratory borings during the current and previous investigations. We used current and previous laboratory test results and our experience to select strength parameters for the soil and bedrock. Strength parameters using Mohr-Coulomb criterion and are summarized in the table below. Strength parameters and slope geometry were input into a limit equilibrium slope stability modeling program, Slide2, by Rocscience. The Spencer method and the non-circular Cuckoo failure search.

SLIDE2 INPUT VALUES

Material	Unit Weight (pcf)	Angle of Internal Friction (degrees)	Cohesion (psf)
Fill, Sand Clay	120	26	25
Granular Fill	125	36	1
Clay CL/CH	123	26	75
Weathered Claystone	123	16	395
Claystone	125	18	1000
Sandstone	125	30	50
Rosetta Stone	134	38	2,671
Leveling Pad	140	40	1
Concrete	150	1	216,000
Reinforced Soil	140	34	1

In discussions that follow, the term “factor of safety” (FOS) is used. This term describes the ratio of strength available to resist sliding compared to the forces that cause sliding. A FOS of 2.0 means the forces resisting sliding exceed the driving force by 2.0 times. For long-term stability, a typical minimum industry accepted FOS is 1.3, while a FOS of 1.5 is preferred. A FOS of less than 1.0 implies failure conditions.

Results of our analysis of the west abutment, cross-sections A and B, indicate stabile conditions based on proposed construction with FOS of 2.2 and 2.6, respectively. Results for cross-sections C and D along the east abutment indicate FOS of 1.4 and 1.3, respectively, if fill placed along the slope consists of granular material with a minimum internal friction angle of 36 degrees. Import or borrow material used along the east abutment should be tested prior to placement to confirm this specification. A summary of the results is presented in the table below.

RESULTS OF SLOPE STABILITY ANALYSIS

Cross-Section	Factor of Safety
A	2.2
B	2.6
C	1.4*
D	1.3*

*Assumes granular fill material with internal friction angle of at least 36°

LATERAL EARTH PRESSURES

Abutment and wing walls should be designed to resist lateral earth pressures. The amount of pressure on a wall is a function of the type of backfill, drainage conditions, slope of the backfill surface, and the allowable rotation of the wall. An “active” earth pressure resistance can be used provided deflection of the wall is acceptable. If very little movement is desired, an “at-rest” resistance should be used. A “passive” earth pressure resistance can be used to resist sliding and overturning. Much larger wall movements are required for passive earth pressure to engage. This larger movement should be accounted for before relying on passive earth pressures. We have tabulated equivalent fluid pressure for use in lateral earth pressure restraint design below. The values given are for backfill material. All backfill should be well compacted, as discussed in the **Fill and Backfill** section. The pressures given do not include allowances for surcharge loads such as sloping backfill or vehicle traffic. Appropriate LRFD load factors should be applied to the lateral earth pressures.

A clean granular material (less than 10 percent passing a No. 200 sieve) such as CDOT Class 5 or 6 material can be used in backfill zones. Granular backfill can be used to reduce lateral pressures compared to clay, and can result in lower settlement, possibly as little as half of the settlement expected with clayey soils. If this material is used, we recommend capping the granular material with a geofabric and at least 2 feet of compacted on-site clayey soils to provide a less permeable surface layer to reduce surface water infiltration where this condition occurs.

LATERAL EQUIVALENT FLUID DENSITIES FOR BACKFILL

Backfill Type	Site Developed Fill	CDOT Class 5 or 6 Fill
Active Equivalent Fluid Density (pcf)	45	35
At Rest Equivalent Fluid Density (pcf)	65	50
Passive Equivalent Fluid Density (pcf)*	320*	400*

*Assumes compacted backfill will never be removed and there is adequate wall movement to engage passive pressure resistance.

CONCRETE

Concrete in contact with soil can be subject to sulfate attack. We measured water-soluble sulfate concentrations of 0.03 and 0.05 percent in two samples from this investigation. As indicated in our tests and ACI 318-19, the sulfate exposure class is *S0* or *Not Applicable*.

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 no 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 Restrictions</td></tr>
<tr><td colspan="2">S1</td><td>0.50</td><td>4000</td><td>II B</td><td>Type with (MS) Designation</td><td>MS</td><td>No Restrictions</td></tr>
<tr><td colspan="2">S2</td><td>0.45</td><td>4500</td><td>V B</td><td>Type with (HS) Designation</td><td>HS</td><td>Not Permitted</td></tr>
<tr><td>S3</td><td>Option 1</td><td>0.45</td><td>4500</td><td>V + Pozzolan or Slag Cement C</td><td>Type with (HS) Designation plus Pozzolan or Slag Cement C</td><td>HS + Pozzolan or Slag Cement C</td><td>Not Permitted</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) Designation</td><td>HS</td><td>Not Permitted</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

Surface drainage should be designed, constructed, and maintained to provide rapid removal of surface water runoff away from the proposed structures. We recommend the following precautions be observed during construction and maintained after construction is completed.

1. Wetting or drying of open foundation and earthwork excavations should be avoided.

2. Positive drainage should be provided away from structure(s). We recommend a minimum slope of 5 percent in the first 5 to 10 feet away from the foundations in landscaped areas.

3. Backfill around foundations should be moistened and compacted according to criteria presented in **Fill and Backfill**. Areas behind flatwork should be backfilled and compacted to reduce ponding of surface water.

4. Landscaping should be carefully designed to minimize irrigation. Plants used close to the bridge should be limited to those with low moisture requirements. Irrigation should be limited to the minimum amount sufficient to maintain vegetation.

5. 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.

6. Concentrated flow from drains or swales should be directed away from the bridge and discharge beyond backfill zones or into appropriate storm sewer or detention area.

CONSTRUCTION OBSERVATIONS

We recommend that CTL|Thompson, Inc. provide construction observation services to allow us the opportunity to verify whether the soil conditions encountered in the excavations 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. Owners/operators must assume responsibility for maintaining the structures and use appropriate practices regarding drainage and landscaping. Improvements 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 JR Engineering, LLC and the design team for the purpose of providing geotechnical design and construction criteria for the proposed project. The information, conclusions and recommendations presented herein are based upon consideration of many factors including, but not limited to, the type of structures proposed, and the subsurface conditions encountered. The conclusions and recommendations contained in the report are not valid for use by others. The recommendations provided are appropriate for about three years. If the proposed project is not constructed within about three years, we should be contacted to determine if we should update this report.

The recommendations presented in this report are based on the construction as currently planned. Revisions in the planned construction could affect our recommendations. We should be contacted if construction will differ from descriptions in this report to review and revise our recommendations, if necessary. The exploratory borings were drilled to obtain a reasonably accurate indication of foundation soil conditions. The borings are representative of conditions encountered only at the locations drilled. Subsurface variations not indicated by the borings are possible.

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

CTL|THOMPSON, INC.

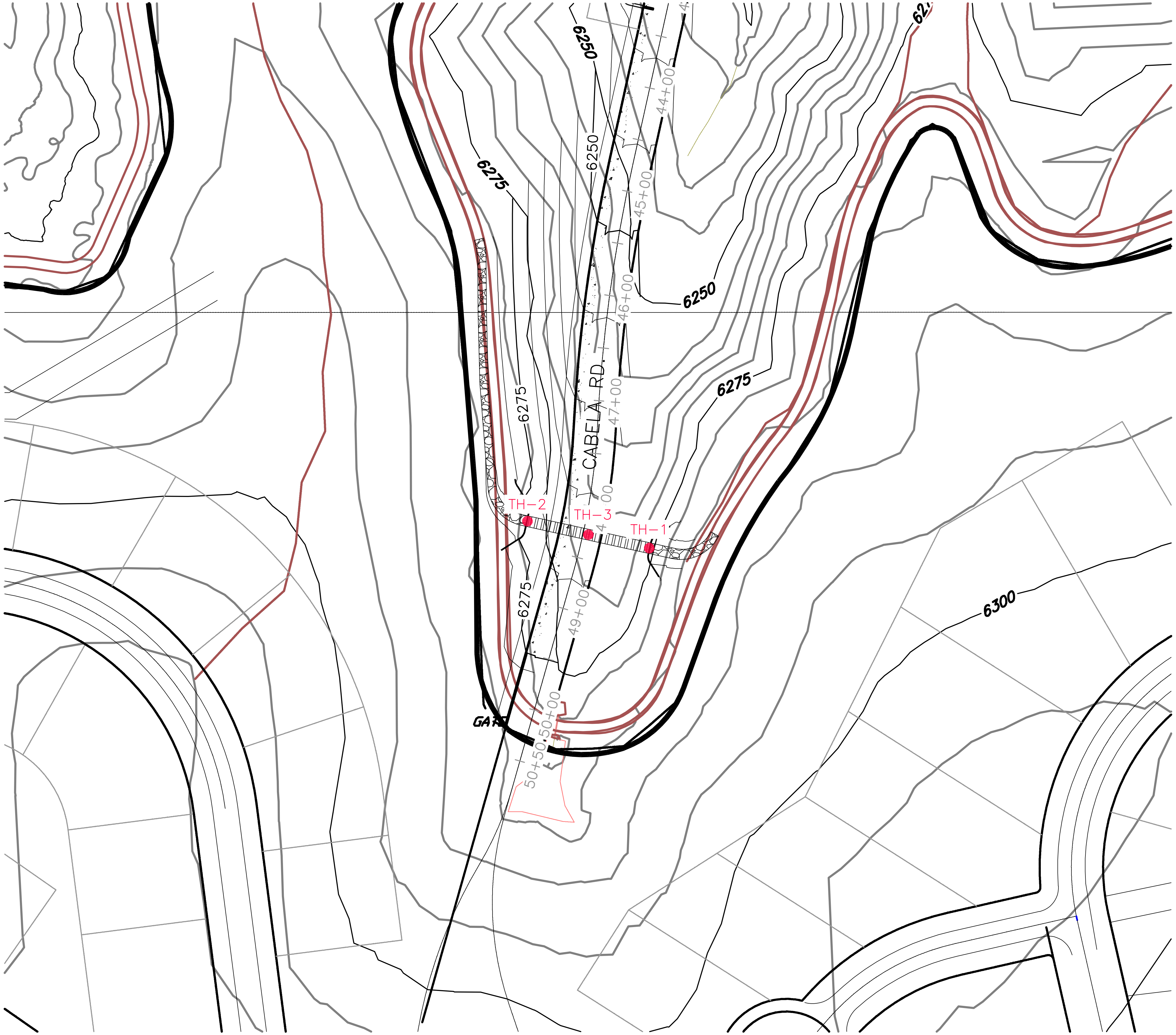
Robert J. Brown
Staff Geologist

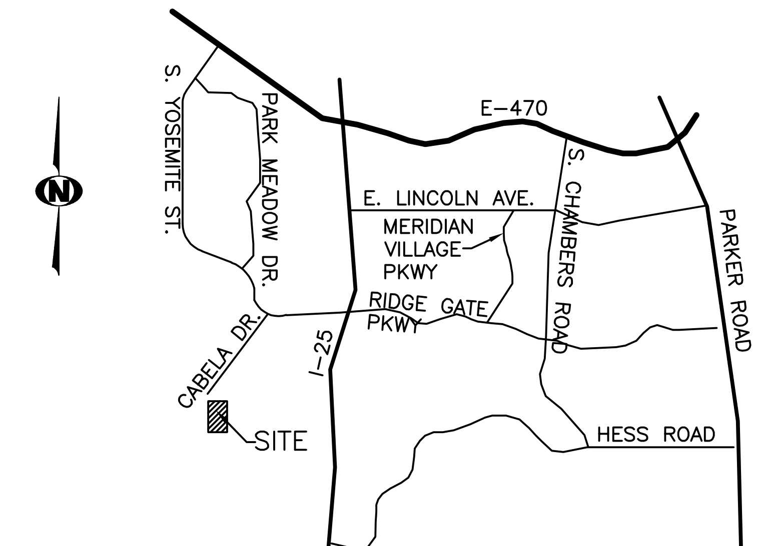
Reviewed by:

Erin Bouchet, P.E., P.G.
Associate Engineer | Denver Engineering Manager

11/20/2025

Via e-mail: krohrbough@jrengineering.com





VICINITY MAP
NOT TO SCALE

LEGEND:

TH-1 ● APPROXIMATE LOCATION OF EXPLORATORY BORING

0 50' 100'
SCALE: 1" = 100'

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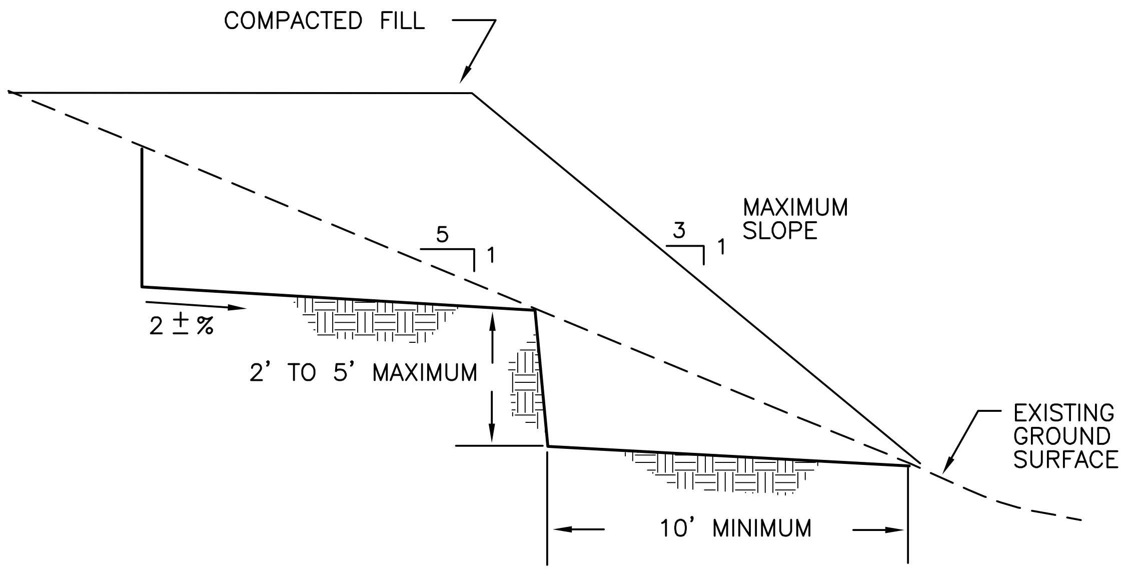
Locations of Exploratory Borings

Fig. 1

60-BENCHED-FILL_01



NOTES:

1) NATURAL SLOPES OF 20 PERCENT OR STEEPER ARE TO BE BENCHED PRIOR TO FILL PLACEMENT.

2) SLOPE BENCHES TO OUTSLOPE AT 2± PERCENT.

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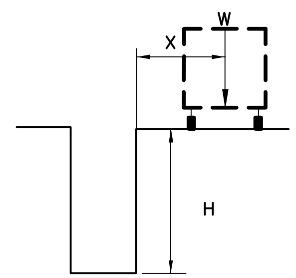
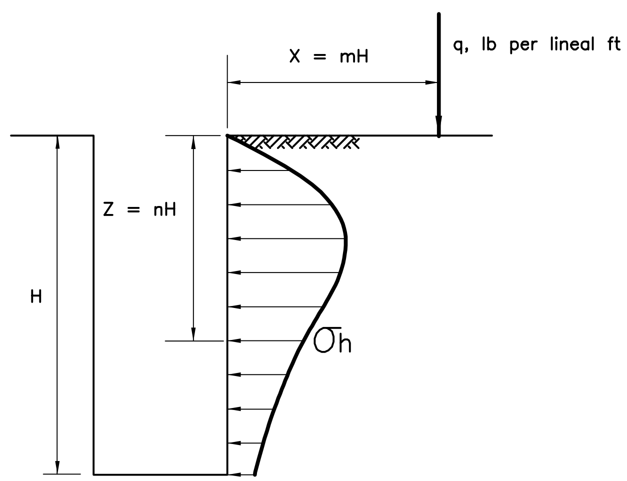
Benched Fill Detail

Fig. 2



W = TOTAL WEIGHT OF OBJECT
L or L_1 = LENGTH OF LOAD PARALLEL TO WALL/EXCAVATION
L_2 = LENGTH OF LOAD PERPENDICULAR TO WALL/EXCAVATION
X = DISTANCE FROM EDGE OF WALL/EXCAVATION MEASURED TO CENTER OF LOAD
H = TOTAL DEPTH OF WALL/EXCAVATION
Z = DEPTH OF WALL/EXCAVATION BELOW GROUND SURFACE



LATERAL PRESSURE AGAINST RIGID WALL DUE TO A LINE LOAD

q = LINE LOAD (plf)

$q = \frac{W}{L}$

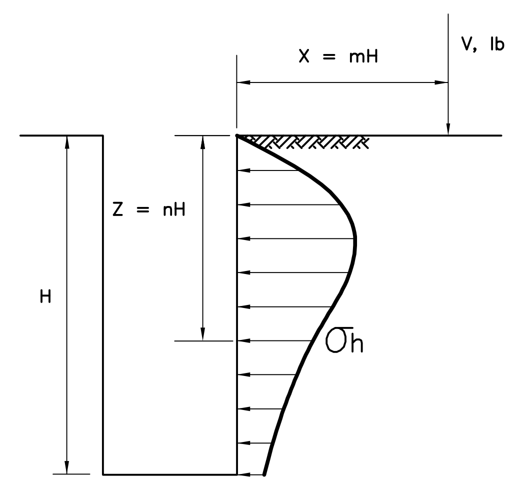
σ_h = LATERAL EARTH PRESSURE AT DEPTH Z DUE TO LINE LOAD q

FOR $m \leq 0.4$:

$$\sigma_h = \frac{q}{H} \times \frac{0.203n}{(0.16+n^2)^2}$$

FOR $m > 0.4$:

$$\sigma_h = \frac{4q}{\pi H} \times \frac{m^2 n}{(m^2+n^2)^2}$$



LATERAL PRESSURE AGAINST RIGID WALL DUE TO A POINT LOAD

V = POINT LOAD (lb)

$V = W$

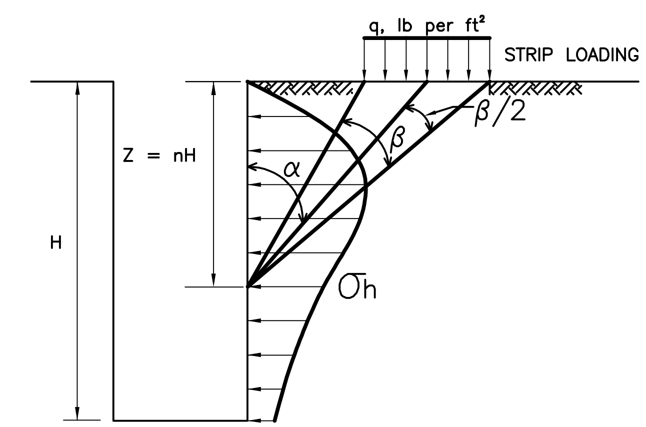
σ_h = LATERAL EARTH PRESSURE AT DEPTH Z DUE TO POINT LOAD V

FOR $m \leq 0.4$:

$$\sigma_h = \frac{0.28V}{H^2} \times \frac{n^2}{(0.16+n^2)^3}$$

FOR $m > 0.4$:

$$\sigma_h = \frac{1.77V}{H^2} \times \frac{m^2 n^2}{(m^2+n^2)^3}$$



LATERAL PRESSURE AGAINST RIGID WALL DUE TO A STRIP LOAD

q = STRIP LOAD (psf)

$q = \frac{W}{L_1 L_2}$

σ_h = LATERAL EARTH PRESSURE AT DEPTH Z DUE TO STRIP LOAD q

$$\sigma_h = \frac{2q}{\pi}(\beta - \sin\beta\cos 2\alpha)$$

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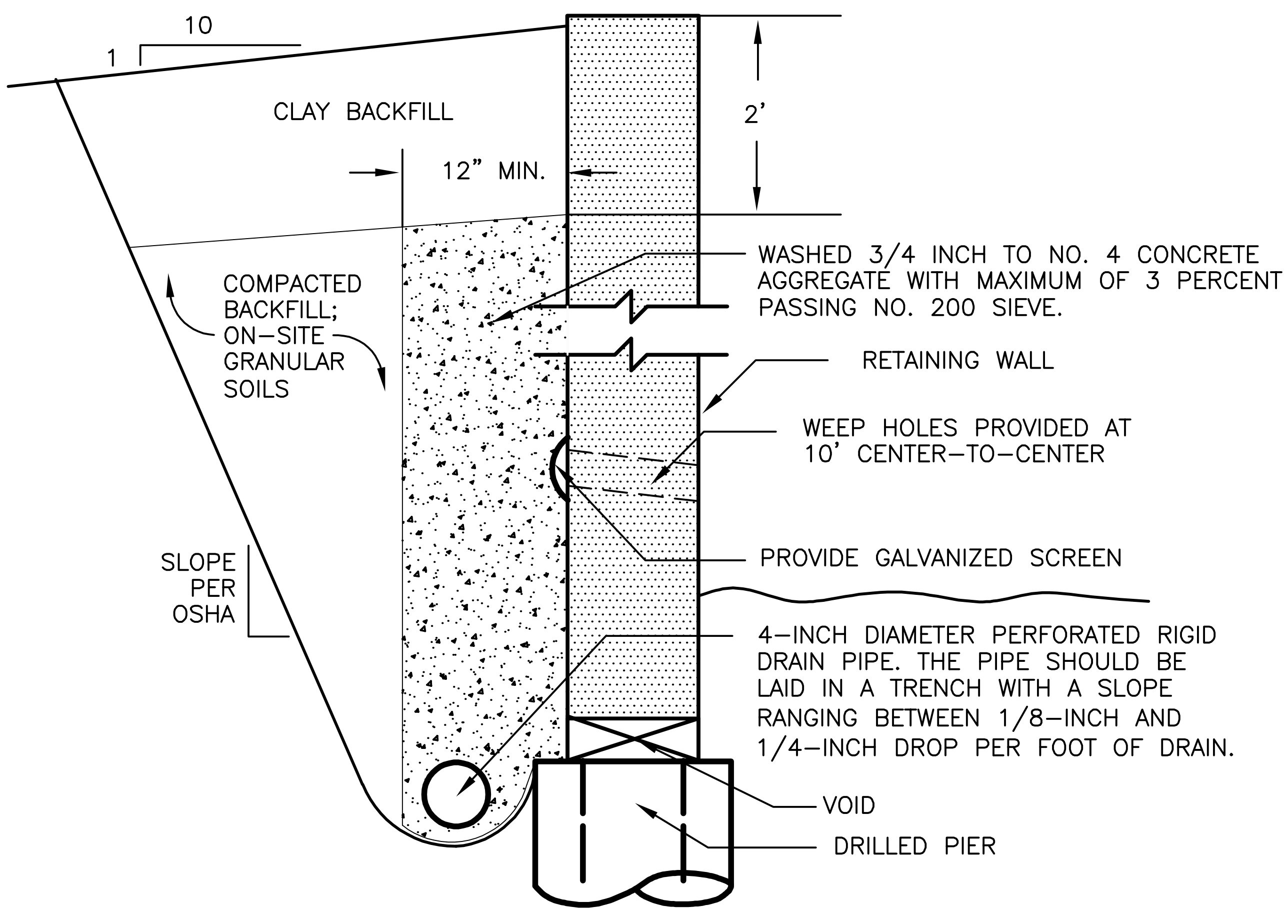
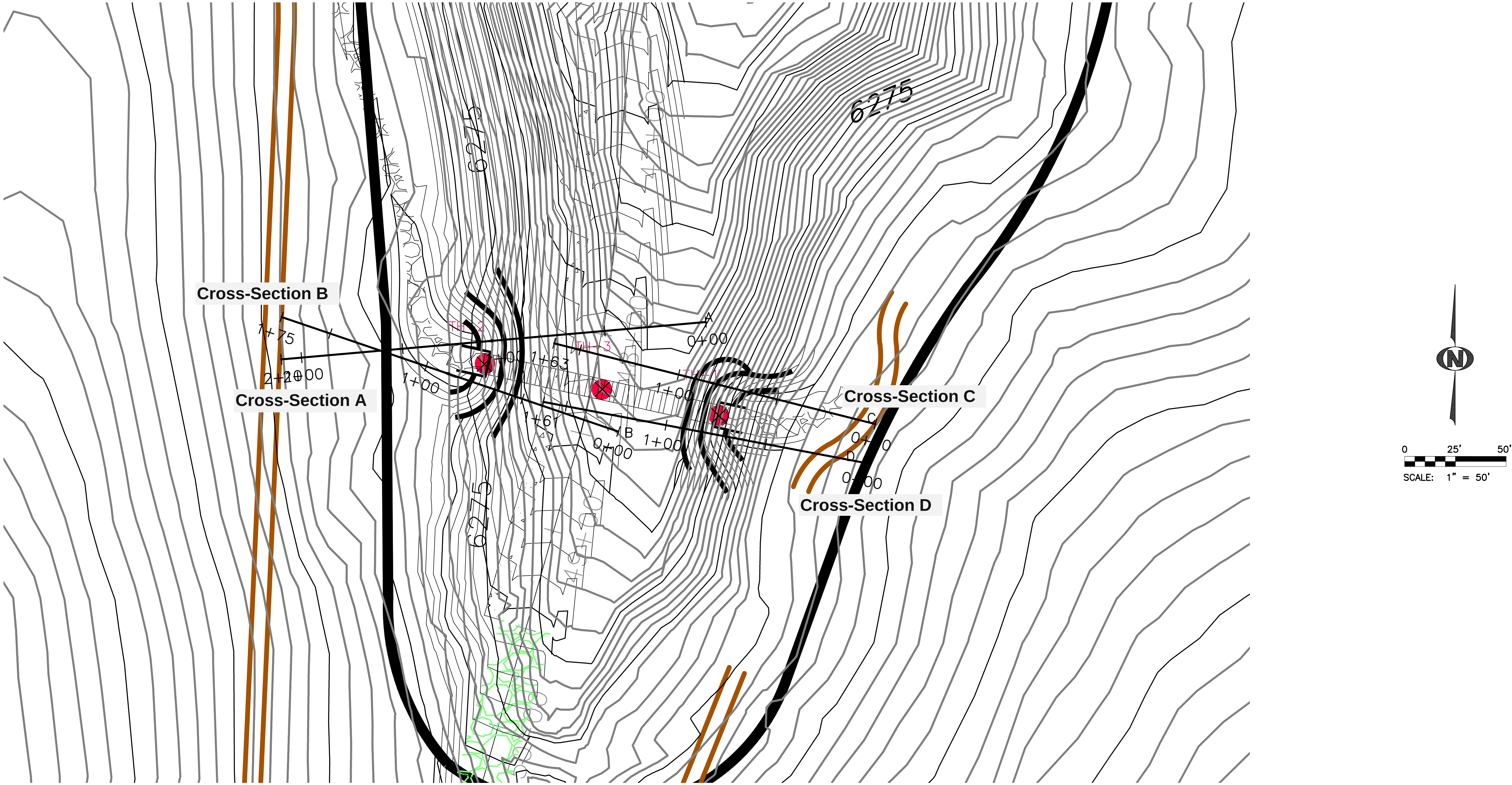
Surcharge Loads

Fig. 3



Typical Earth Retaining Wall Drain

Fig. 4



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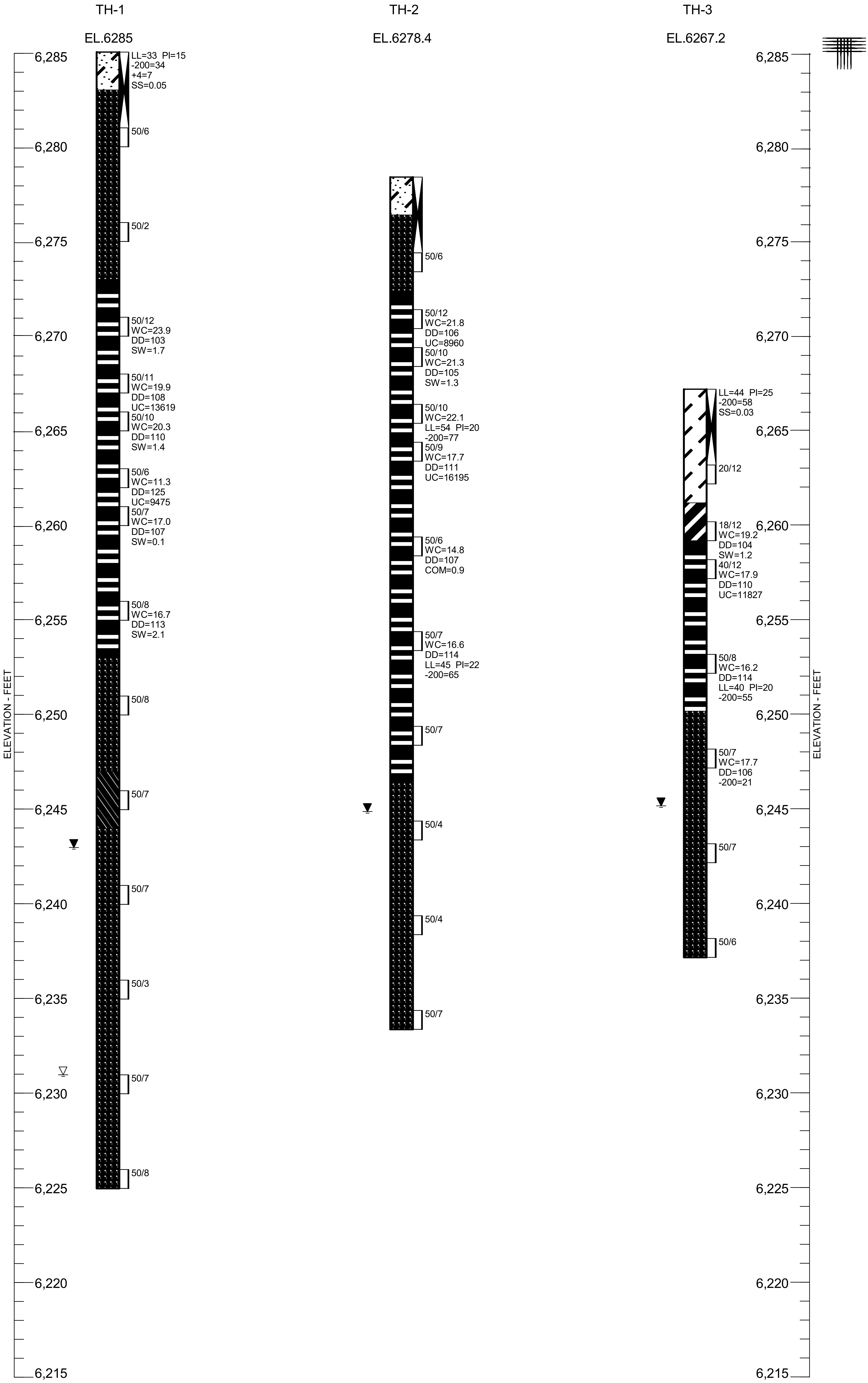
Cross Section Locations

Fig. 5

APPENDIX A

SUMMARY LOGS OF EXPLORATORY BORINGS



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SUMMARY LOGS OF EXPLORATORY BORINGS

FIG. A-1

LEGEND:

CLAY, SANDY, VERY STIFF, MOIST, DARK BROWN (CL).

SAND CLAYEY, SLIGHTLY MOIST, TAN, BROWN (SC).

WEATHERED BEDROCK CLAYSTONE, MOIST, GRAY.

BEDROCK, CLAYSTONE, SILTY AT TIMES, MEDIUM HARD TO VERY HARD, MOIST, GRAY, RUST, TAN, WHITE.

BEDROCK SANDSTONE, MAY CONTAIN THIN LENSES OF INTERBEDDED CLAYSTONE, HARD TO VERY HARD, SLIGHTLY MOIST TO VERY MOIST, TAN, RUST, WHITE.

BEDROCK, INTERBEDDED CLAYSTONE/SANDSTONE, HARD, MOIST, GRAY.

DRIVE SAMPLE. THE SYMBOL 50/6 INDICATES 50 BLOWS OF A 140-POUND HAMMER FALLING 30 INCHES WERE REQUIRED TO DRIVE A 2.5-INCH O.D. SAMPLER 6 INCHES.

INDICATES BULK SAMPLE OBTAINED FROM AUGER CUTTINGS.

WATER LEVEL MEASURED AT TIME OF DRILLING.

WATER LEVEL MEASURED AFTER DRILLING ON MAY 21, 2025.

NOTES:

1. THE BORINGS WERE DRILLED MAY 16, 2025 USING A 4-INCH DIAMETER, CONTINUOUS-FLIGHT SOLID-STEM AUGER AND TRUCK-MOUNTED CME-45 DRILL RIG.

2. BORING LOCATIONS AND ELEVATIONS ARE APPROXIMATE AND WERE DETERMINED BY A REPRESENTATIVE OF OUR FIRM USING A LEICA GS18 GPS UNIT REFERENCING THE NORTH AMERICAN DATUM OF 1983 (NAD83).

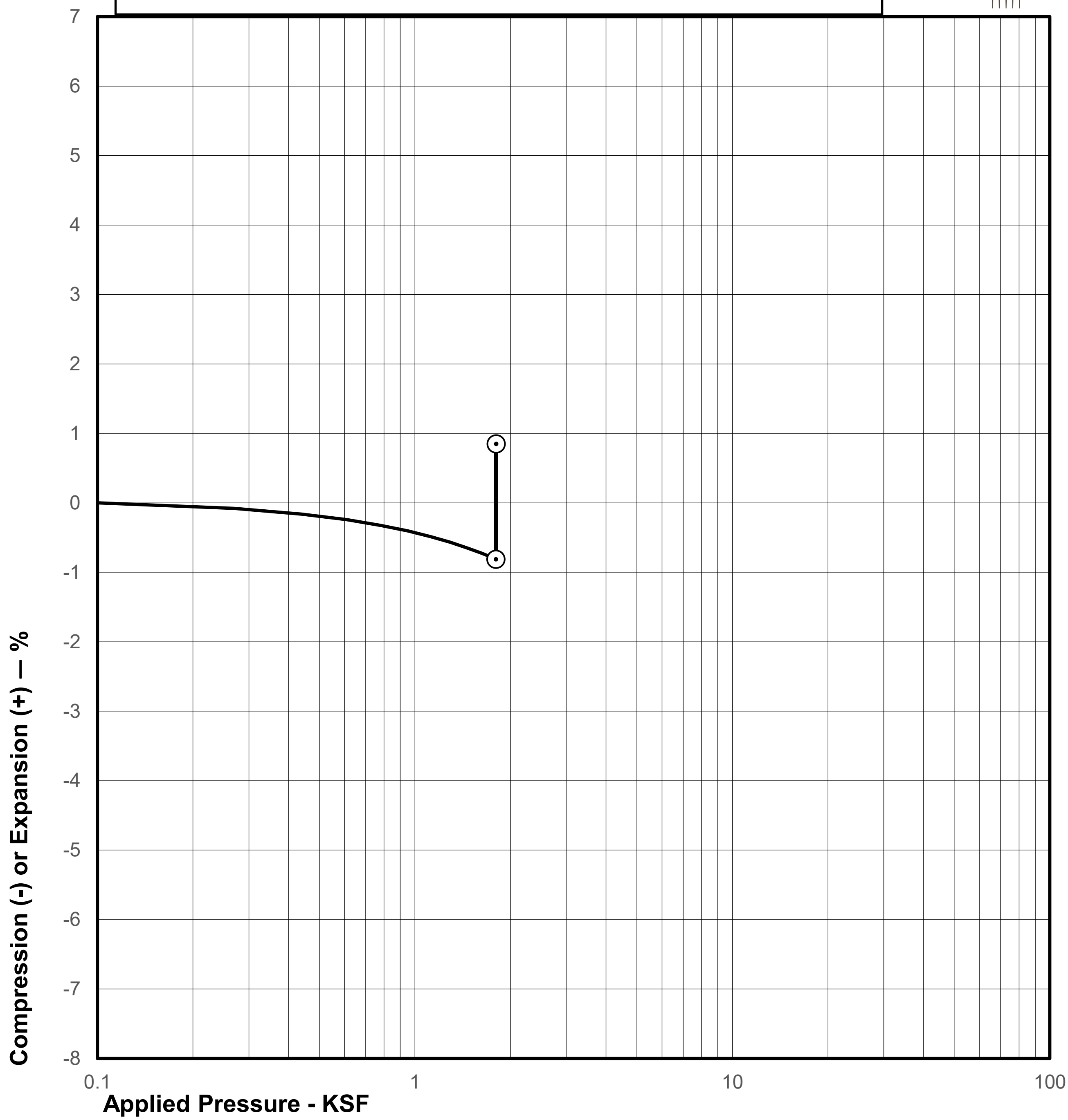
3. WC - INDICATES MOISTURE CONTENT (%).
   DD - INDICATES DRY DENSITY (PCF).
   SW - INDICATES SWELL WHEN WETTED UNDER APPROXIMATE OVERBURDEN PRESSURE (%).
   COM - INDICATES COMPRESSION WHEN WETTED UNDER APPROXIMATE OVERBURDEN PRESSURE (%).
   LL - INDICATES LIQUID LIMIT.
   PI - INDICATES PLASTICITY INDEX.
   -200 - INDICATES PASSING NO. 200 SIEVE (%).
   +4 - INDICATES RETAINED ON NO. 4 SIEVE (%).
   SS - INDICATES WATER-SOLUBLE SULFATE CONTENT (%).
   UC - INDICATES UNCONFINED COMPRESSIVE STRENGTH (PSF).

4. THESE LOGS ARE SUBJECT TO THE EXPLANATIONS, LIMITATIONS, AND CONCLUSIONS AS CONTAINED IN THIS REPORT.

APPENDIX B

LABORATORY TEST RESULTS
TABLES B-I – SUMMARY OF LABORATORY TEST RESULTS

Sample exhibited expansion of 1.7 percent when wetted under an applied pressure of 1800 psf.



SAMPLE OF: CLAYSTONE

FROM: TH-1 AT 14 FEET

MOISTURE CONTENT: 23.9 % LIQUID LIMIT: ____ SILT AND CLAY: ____ %

DRY UNIT WEIGHT: 103 pcf PLASTICITY INDEX: ____ SOIL SUCTION: ____ pF

Swell Consolidation Test Results

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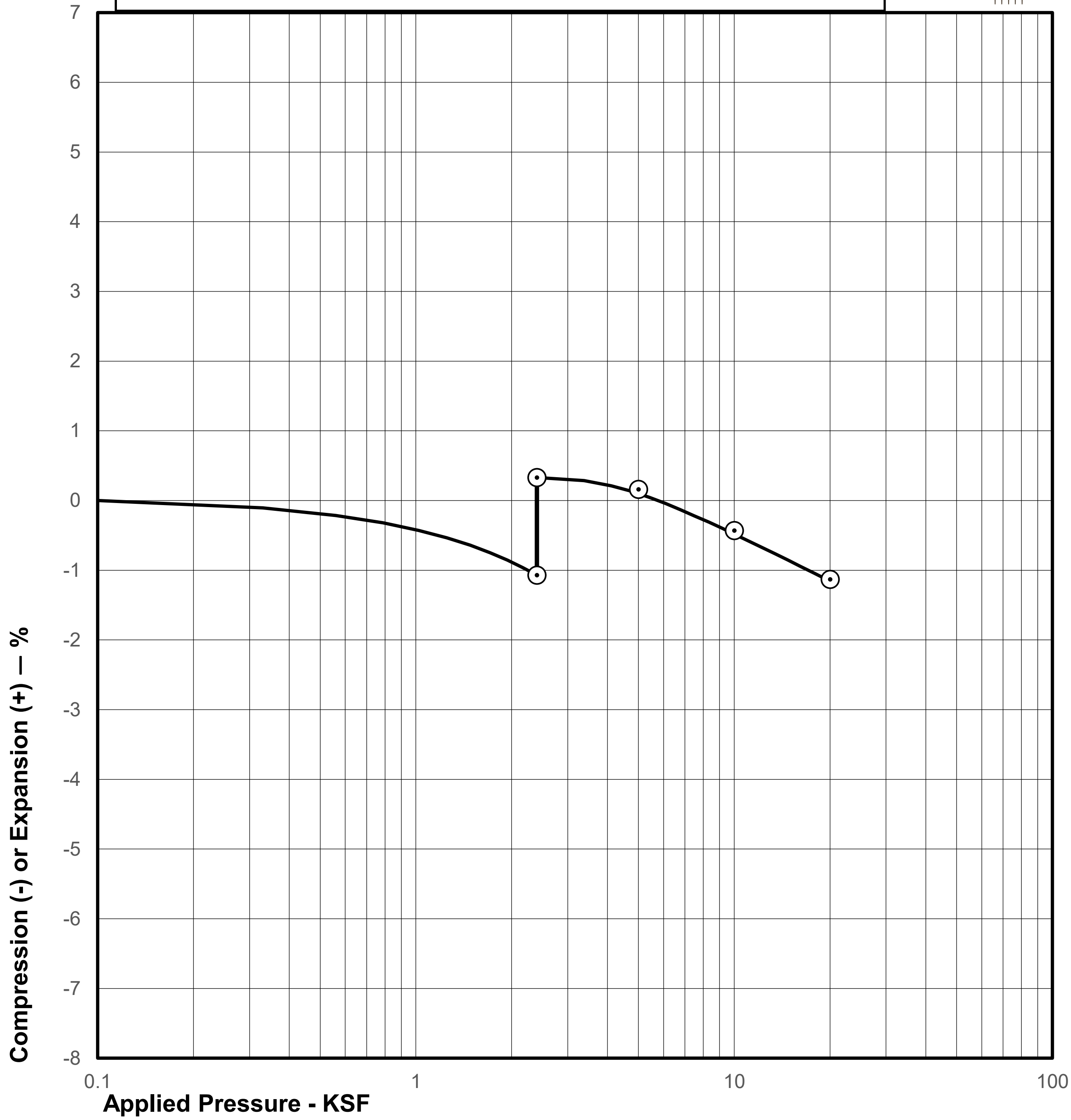
FIG. B-1

Sample exhibited expansion of 1.4 percent when wetted under an applied pressure of 2400 psf, with a calculated swell pressure of 19500 psf.



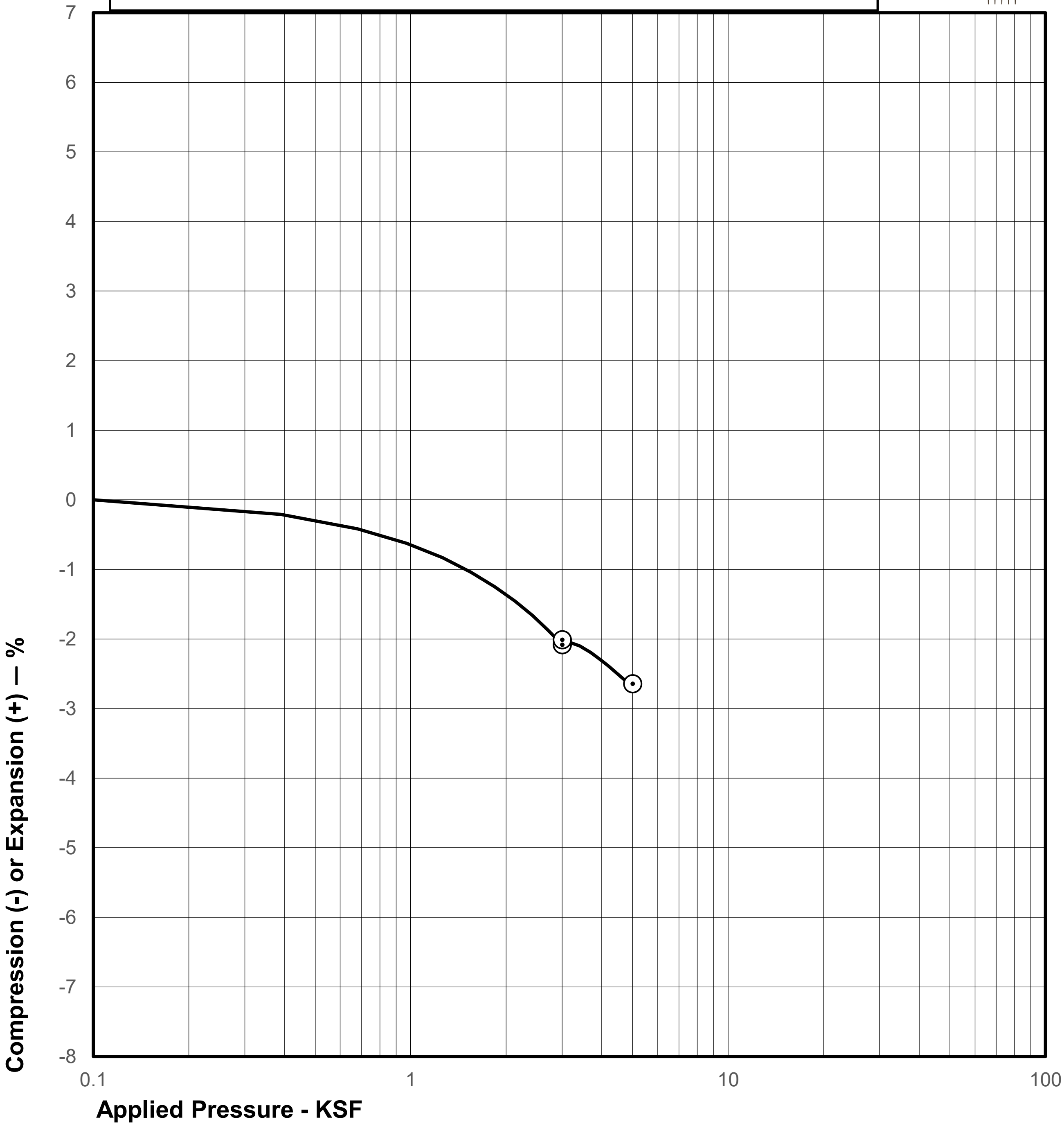
SAMPLE OF: CLAYSTONE

FROM: TH-1 AT 19 FEET

MOISTURE CONTENT:	20.3 %	LIQUID LIMIT:		SILT AND CLAY:	%
DRY UNIT WEIGHT:	110 pcf	PLASTICITY INDEX:		SOIL SUCTION:	pF

Swell Consolidation Test Results

Sample exhibited expansion of 0.1 percent when wetted under an applied pressure of 3000.0 psf, with a calculated swell pressure of 3400 psf.



SAMPLE OF: CLAYSTONE

FROM: TH-1 AT 24 FEET

MOISTURE CONTENT: 17.0 % LIQUID LIMIT: ____ SILT AND CLAY: ____ %

DRY UNIT WEIGHT: 107 pcf PLASTICITY INDEX: ____ SOIL SUCTION: ____ pF

Swell Consolidation Test Results

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RIDGEGATE MESA TOPS PEDESTRIAN BRIDGE
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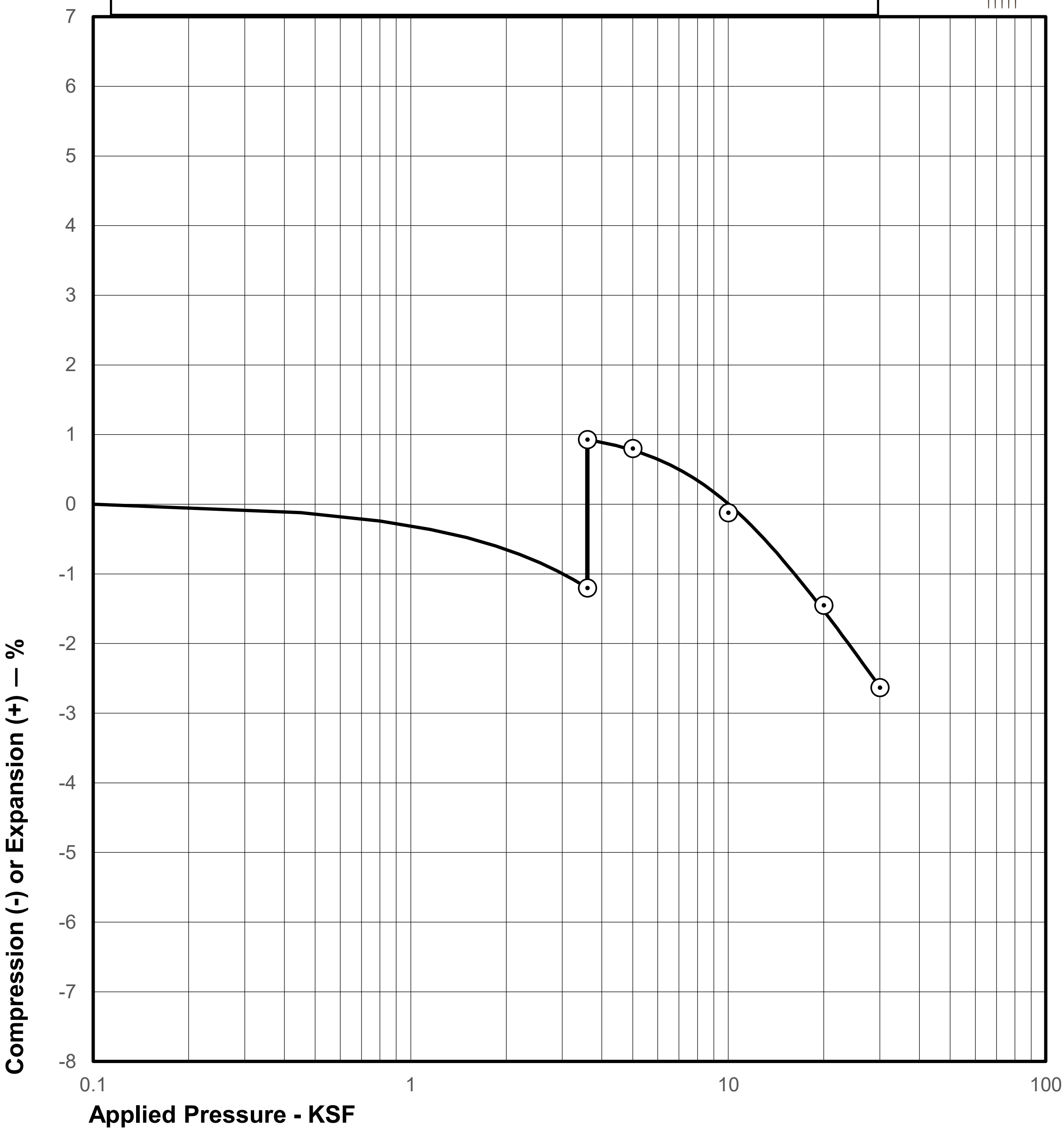
FIG. B-3

Sample exhibited expansion of 2.1 percent when wetted under an applied pressure of 3600 psf, with a calculated swell pressure of 17500 psf.



SAMPLE OF: CLAYSTONE

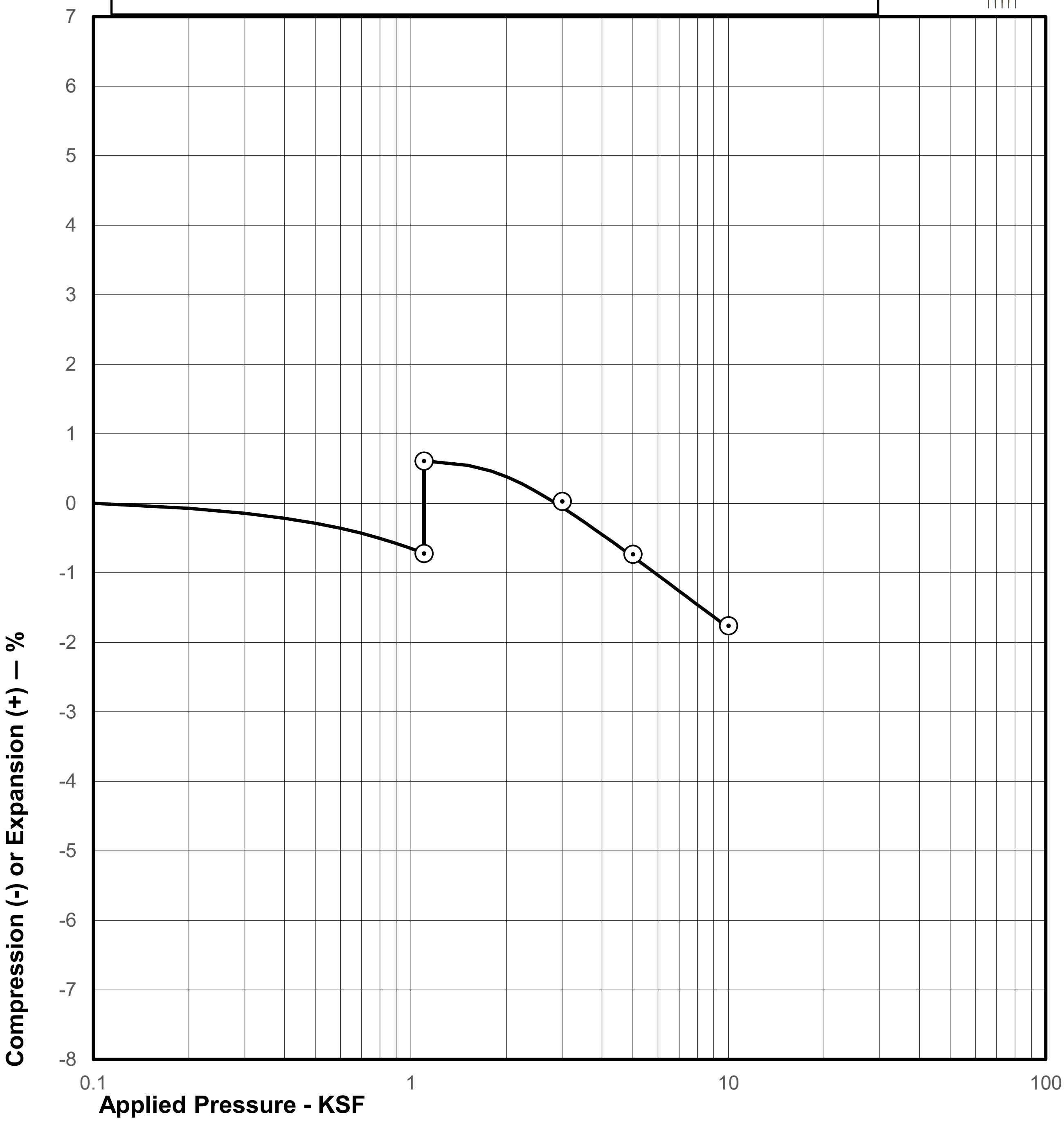
FROM: TH-1 AT 29 FEET

MOISTURE CONTENT: 16.7 % LIQUID LIMIT: ____ SILT AND CLAY: ____ %

DRY UNIT WEIGHT: 113 pcf PLASTICITY INDEX: ____ SOIL SUCTION: ____ pF

Swell Consolidation Test Results

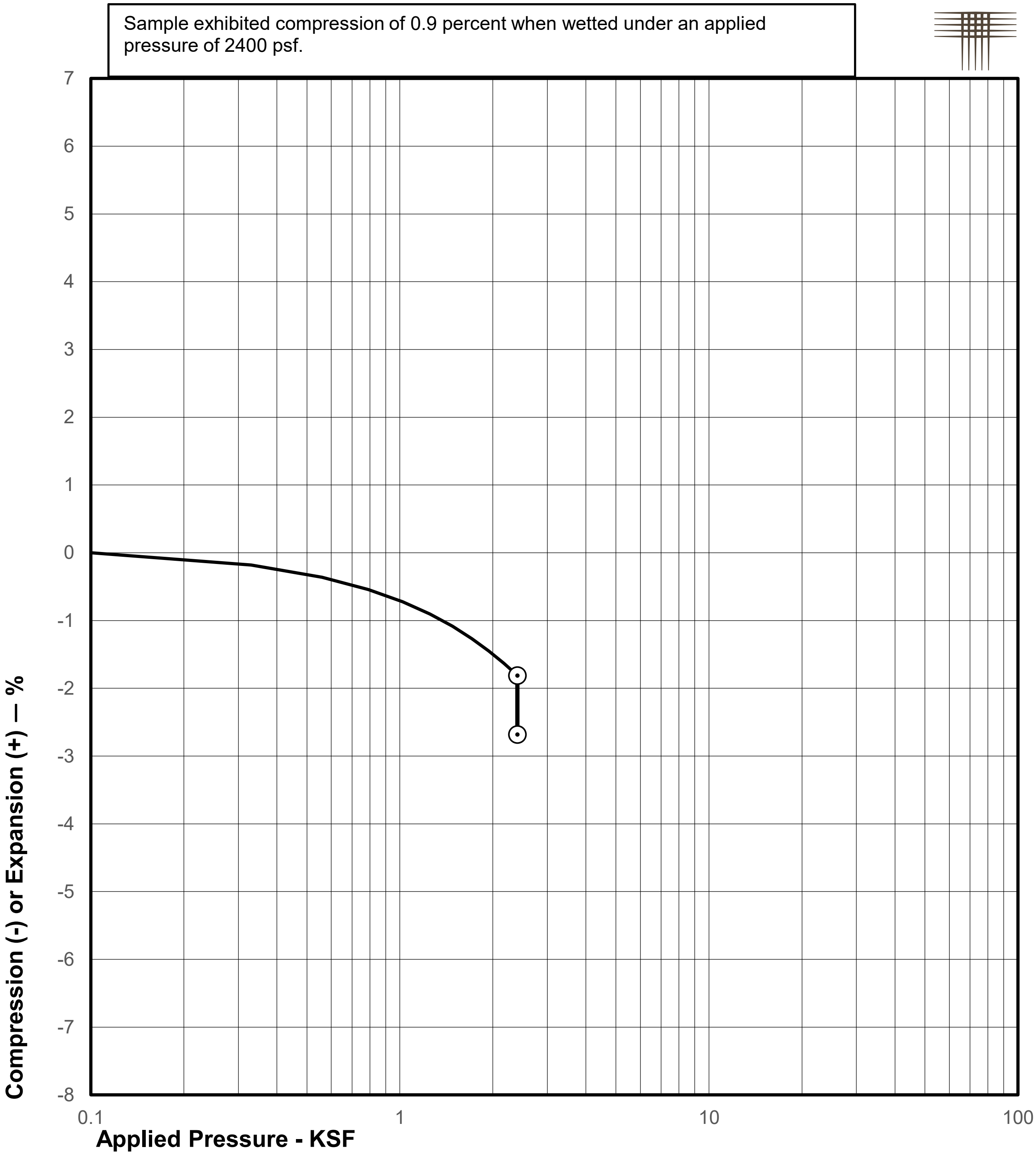
Sample exhibited expansion of 1.3 percent when wetted under an applied pressure of 1100 psf, with a calculated swell pressure of 4700 psf.



SAMPLE OF:	CLAYSTONE
FROM:	TH-2 AT 9 FEET

MOISTURE CONTENT:	21.3 %	LIQUID LIMIT:		SILT AND CLAY:	%
DRY UNIT WEIGHT:	105 pcf	PLASTICITY INDEX:		SOIL SUCTION:	pF

Swell Consolidation Test Results



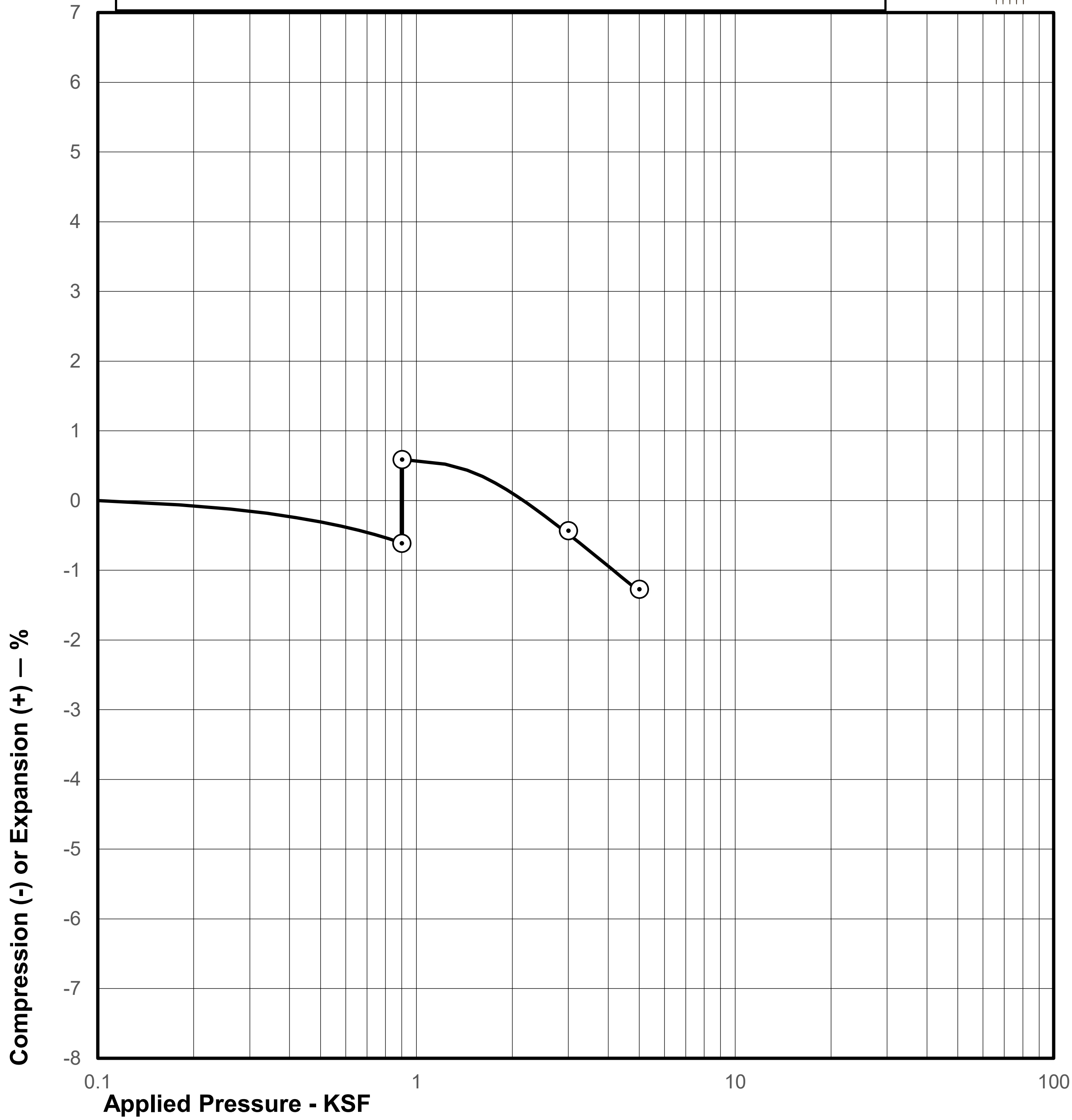
SAMPLE OF: CLAYSTONE

FROM: TH-2 AT 19 FEET

MOISTURE CONTENT:	14.8 %	LIQUID LIMIT:		SILT AND CLAY:	%
DRY UNIT WEIGHT:	107 pcf	PLASTICITY INDEX:		SOIL SUCTION:	pF

Swell Consolidation Test Results

Sample exhibited expansion of 1.2 percent when wetted under an applied pressure of 900 psf, with a calculated swell pressure of 3300 psf.



SAMPLE OF: WEATHERED CLAYSTONE

FROM: TH-3 AT 7 FEET

MOISTURE CONTENT:	19.2 %	LIQUID LIMIT:		SILT AND CLAY:	%
DRY UNIT WEIGHT:	104 pcf	PLASTICITY INDEX:		SOIL SUCTION:	pF

Swell Consolidation Test Results

JR ENGINEERING, LLC
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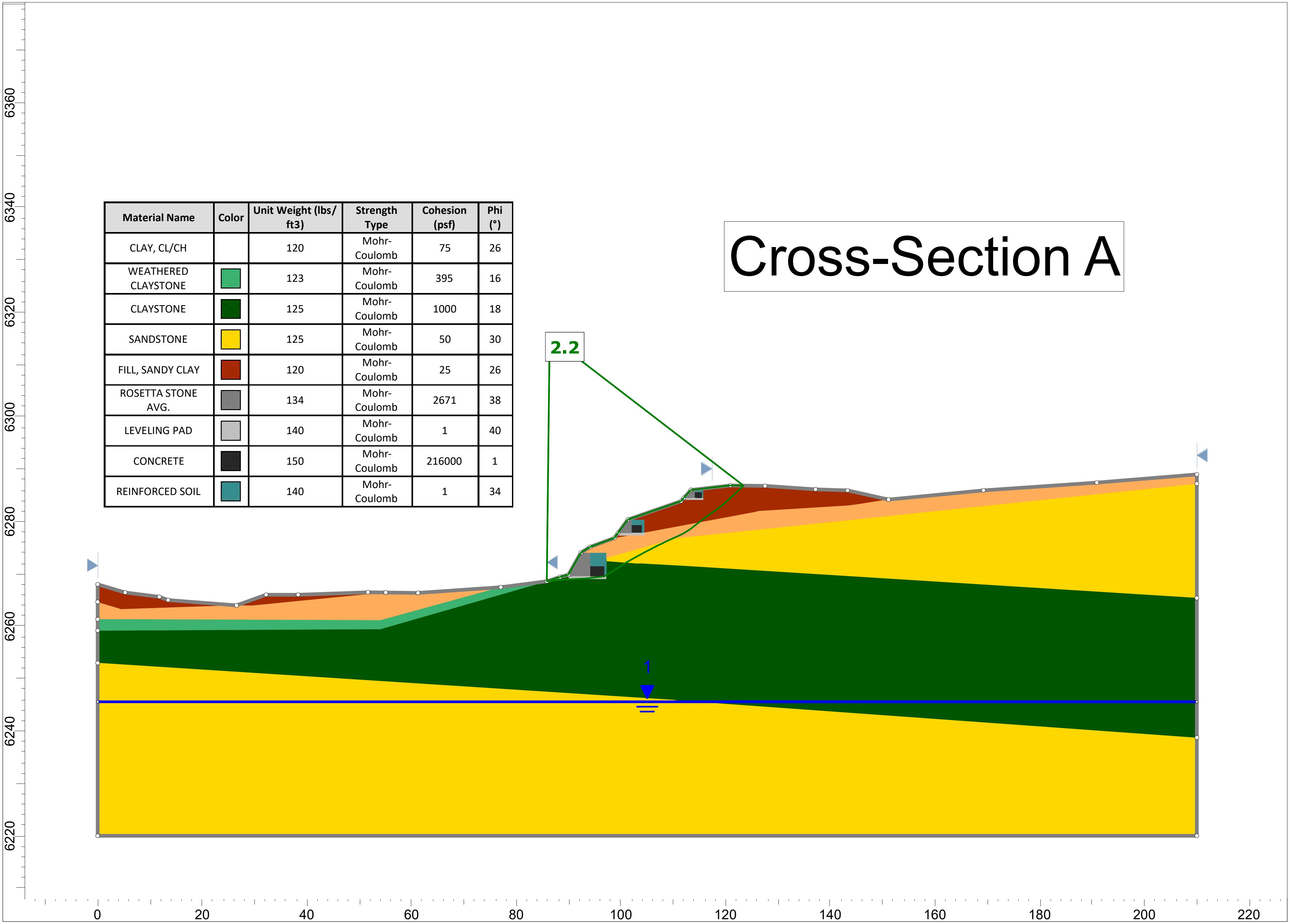
TABLE B - I

SUMMARY OF LABORATORY TEST RESULTS

BORING	DEPTH (ft)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	SWELL TEST DATA				ATTERBERG LIMITS		UNCONFINED COMPRESSIVE STRENGTH (psf)	SOLUBLE SULFATE CONTENT (%)	RETAINED NO. 4 SIEVE (%)	PASSING NO. 200 SIEVE (%)	SOIL TYPE
				SWELL (%)	COMPRESSION (%)	APPLIED PRESSURE (psf)	SWELL PRESSURE (psf)	LIQUID LIMIT	PLASTICITY INDEX					
TH-1	0-5							33	15		0.05	7	34	SAND, CLAYEY (SC)
TH-1	14	23.9	103	1.7		1800								CLAYSTONE
TH-1	17	19.9	108							13620				CLAYSTONE
TH-1	19	20.3	110	1.4		2400	19500							CLAYSTONE
TH-1	22	11.3	125							9480				CLAYSTONE
TH-1	24	17.0	107	0.1		3000	3400							CLAYSTONE
TH-1	29	16.7	113	2.1		3600	17500							CLAYSTONE
TH-2	7	21.8	106							8960				CLAYSTONE
TH-2	9	21.3	105	1.3		1100	4700							CLAYSTONE
TH-2	12	22.1						54	20				77	CLAYSTONE
TH-2	14	17.7	111							16200				CLAYSTONE
TH-2	19	14.8	107		0.9	2400								CLAYSTONE
TH-2	24	16.6	114					45	22				65	CLAYSTONE
TH-3	0-5							44	25		0.03		58	CLAY, SANDY (CL)
TH-3	7	19.2	104	1.2		900	3300							WEATHERED CLAYSTONE
TH-3	9	17.9	110							11830				CLAYSTONE
TH-3	14	16.2	114					40	20				55	CLAYSTONE
TH-3	19	17.7	106										21	SANDSTONE

APPENDIX C
SLOPE STABILITY RESULTS



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
CLAY, CL/CH		120	Mohr-Coulomb	75	26
WEATHERED CLAYSTONE		123	Mohr-Coulomb	395	16
CLAYSTONE		125	Mohr-Coulomb	1000	18
SANDSTONE		125	Mohr-Coulomb	50	30
FILL, SANDY CLAY		120	Mohr-Coulomb	25	26
ROSETTA STONE AVG.		134	Mohr-Coulomb	2671	38
LEVELING PAD		140	Mohr-Coulomb	1	40
CONCRETE		150	Mohr-Coulomb	216000	1
REINFORCED SOIL		140	Mohr-Coulomb	1	34

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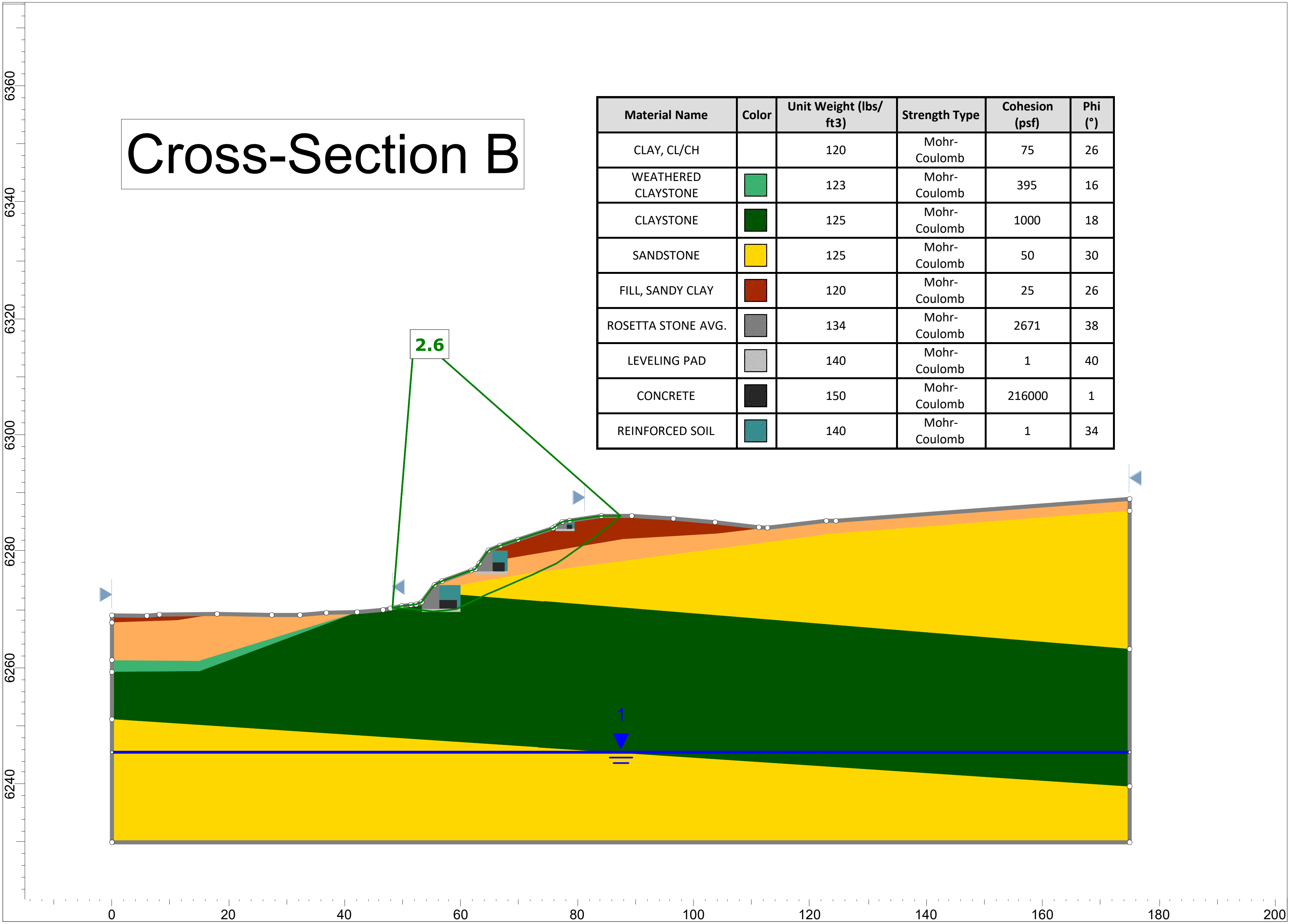
FIG. C-1

Cross-Section B



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
CLAY, CL/CH		120	Mohr-Coulomb	75	26
WEATHERED CLAYSTONE		123	Mohr-Coulomb	395	16
CLAYSTONE		125	Mohr-Coulomb	1000	18
SANDSTONE		125	Mohr-Coulomb	50	30
FILL, SANDY CLAY		120	Mohr-Coulomb	25	26
ROSETTA STONE AVG.		134	Mohr-Coulomb	2671	38
LEVELING PAD		140	Mohr-Coulomb	1	40
CONCRETE		150	Mohr-Coulomb	216000	1
REINFORCED SOIL		140	Mohr-Coulomb	1	34

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FIG. C-2

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
CLAY, CL/CH		120	Mohr-Coulomb	75	26
WEATHERED CLAYSTONE		123	Mohr-Coulomb	395	16
CLAYSTONE		125	Mohr-Coulomb	1000	18
SANDSTONE		125	Mohr-Coulomb	50	30
FILL, SANDY CLAY		120	Mohr-Coulomb	25	26
ROSETTA STONE AVG.		134	Mohr-Coulomb	2671	38
LEVELING PAD		140	Mohr-Coulomb	1	40
CONCRETE		150	Mohr-Coulomb	216000	1
REINFORCED SOIL		140	Mohr-Coulomb	1	34
GRANULAR FILL		125	Mohr-Coulomb	1	36

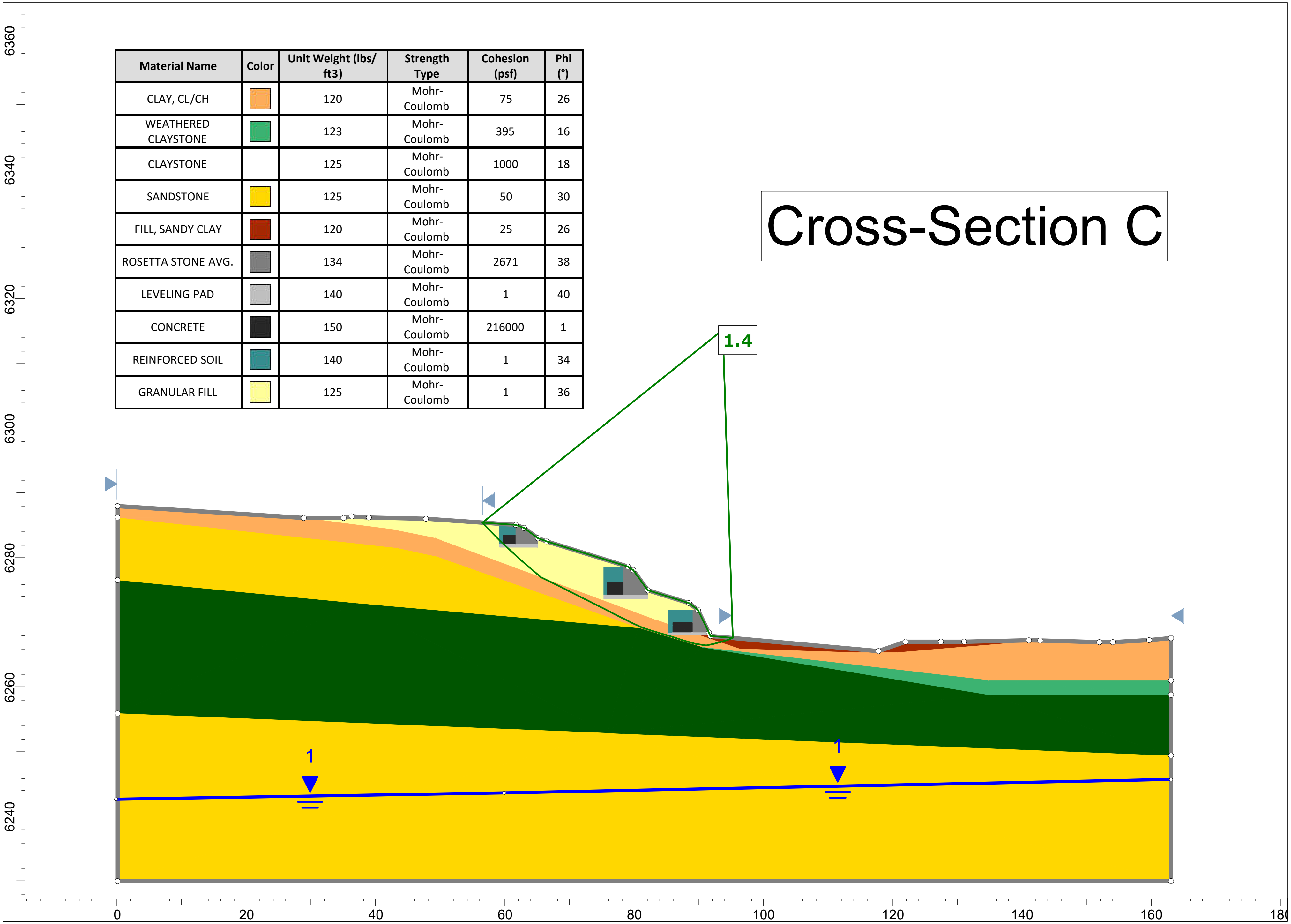

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FIG. C-3

Cross-Section D

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
CLAY, CL/CH		120	Mohr-Coulomb	75	26
WEATHERED CLAYSTONE		123	Mohr-Coulomb	395	16
CLAYSTONE		125	Mohr-Coulomb	1000	18
SANDSTONE		125	Mohr-Coulomb	50	30
FILL, SANDY CLAY		120	Mohr-Coulomb	25	26
ROSETTA STONE AVG.		134	Mohr-Coulomb	2671	38
LEVELING PAD		140	Mohr-Coulomb	1	40
CONCRETE		150	Mohr-Coulomb	216000	1
REINFORCED SOIL		140	Mohr-Coulomb	1	34
GRANULAR FILL		125	Mohr-Coulomb	1	36

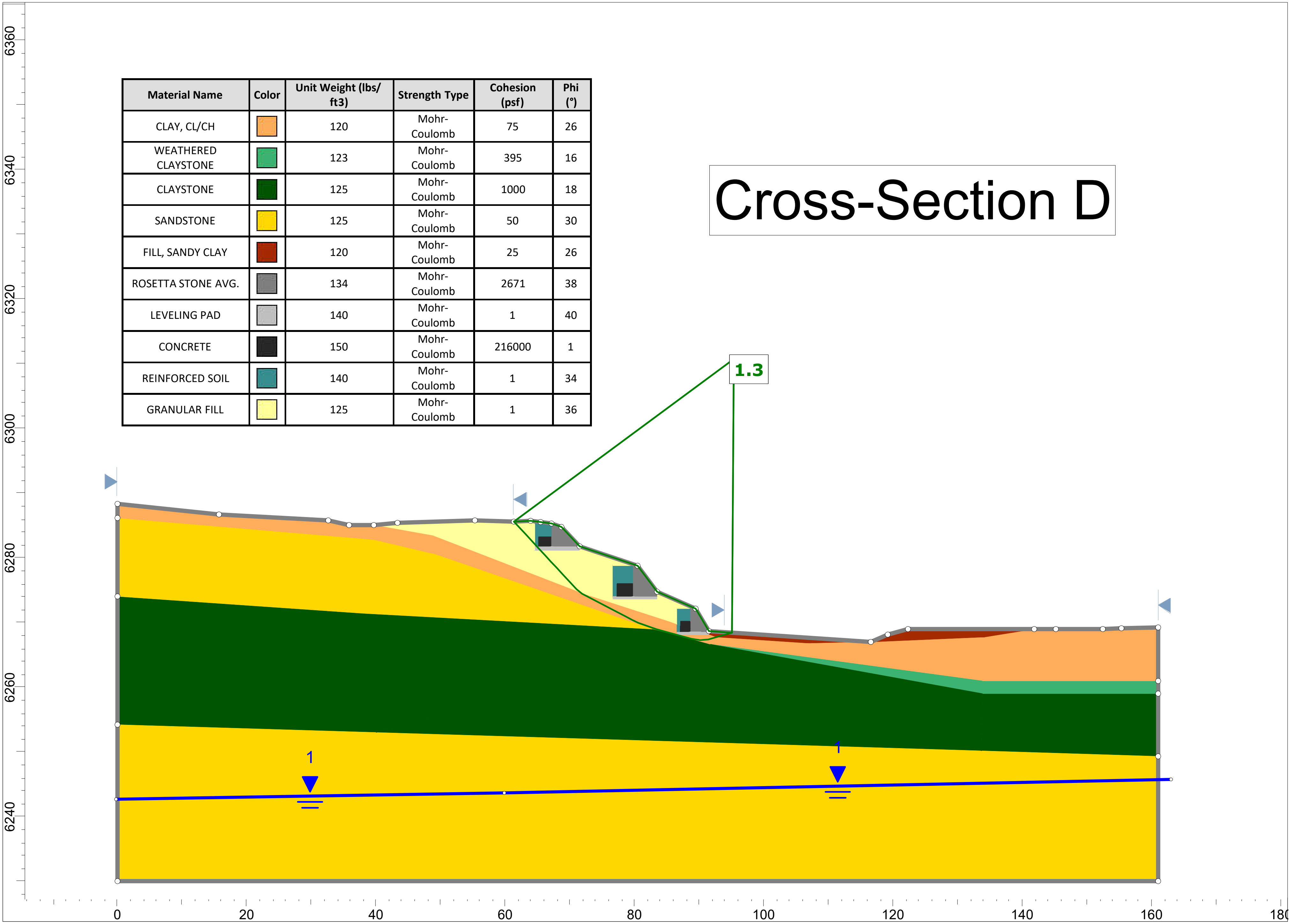

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FIG. C-4