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GEOTECHNICAL INVESTIGATION

INSPIRATION METROPOLITAN DISTRICT SERVICE BUILDING
23392 EAST GLIDDEN DRIVE
AURORA, COLORADO

Prepared for:

INSPIRATION METROPOLITAN DISTRICT
c/o PUBLIC ALLIANCE
13131 West Alameda Parkway, Suite 200
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Attention:
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Project No. DN51,790.000-125-R1

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SCOPE

This report presents the results of our Geotechnical Investigation to support design and construction of the Inspiration Metropolitan District Service Building planned at 23392 East Glidden Drive in Aurora, Colorado (Fig. 1). The purpose of our investigation was to evaluate the subsurface conditions and provide geotechnical design and construction criteria for the project. The scope was described in a Service Agreement (DN 22-0378) dated August 15, 2022. This service was provided under the Terms and Conditions described in a contract between CTL|T and the Inspiration Metropolitan District, executed by Board President, Aaron Curtiss, dated October 20, 2022. The service was provided to adhere to the requirements described in a Request for Proposal letter by Quintessence Design Group, dated August 1, 2022.

This report was prepared considering data developed during current and previous field exploration, laboratory testing, engineering analysis, and experience with similar conditions. The report contents are based on construction as currently planned, as conceptualized in Architectural Site Schematic Site Plans and Renderings by Quintessence Design Group, dated July 18, 2022. Other types of construction may require revision of this report and the recommended design criteria. If plans change, we should be contacted to re-evaluate the opinions and recommendations in this report and whether they remain valid.

This report includes descriptions of site conditions, proposed construction, subsurface conditions encountered during field exploration and laboratory tests, and the identified geologic hazards along with our opinions and recommendations for design criteria and construction details for foundations and floor systems, slabs-on-grade, pavements, and surface-subsurface drainage precautions. The report was prepared for exclusive use of the Inspiration Metropolitan District and their design and construction team for design and construction of the project. A brief summary of our conclusions and recommendations follows. The summary should not be solely relied upon in lieu of reading the entirety of this report. Detailed design criteria are presented within the report.

SUMMARY OF CONCLUSIONS

1. Strata found in our borings consisted of about nil to 4.5 feet of fill and up to 1.5 feet of sandy clay underlain by sandstone and claystone bedrock to the maximum explored depth of 35 feet. The bedrock generally became harder with depth. Testing indicates the overburden soils are predominantly low swelling or non-expansive while the claystone is expansive, when wetted.

2. Groundwater was encountered in each boring at depths of about 20 to 24 feet or approximate elevations 6150.5 to 6154.5 feet. The water appears to be confined in the bedrock. Groundwater is not expected to influence construction unless drilled piers are used. Water levels may fluctuate seasonally and rise in response to precipitation, landscape irrigation, and changes in land-use.

3. The presence of expansive and compressible soil and bedrock constitutes a geologic hazard and the existing fill constitutes a geotechnical concern. There is risk that foundations, slab-on-grade floors, and improvements will experience heave or settlement, and subsequent damage. We believe the recommendations in this report will help to reduce risk of damage; they will not eliminate the risk.

4. Hard bedrock will likely be encountered during excavations. The contractor should anticipate this and implement appropriate measures.

5. Spread footing foundations are considered appropriate for the structures included in this study after sub-excavation to a depth of 5 feet below bottom of foundations or 8 feet below existing grade, whichever is deeper. Alternatively, drilled pier foundations can be used. Discussions and design and construction criteria for footing and drilled pier foundations are presented in the report.

6. Slab-on-grade floors can be used after sub-excavation provided the owner can accept risk of floor movement. Structurally supported floors over a crawl space or void can be used if virtually no movement is desired.

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

8. The design and construction criteria for foundations and floor systems in this report were compiled with the expectation that all other recommendations presented related to surface drainage, landscaping irrigation, backfill compaction, etc. will be incorporated into the project and that owners/property managers will maintain the structures and positive surface drainage, and use prudent irrigation practices. It is critical that all recommendations in this report are followed.

SITE CONDITIONS

The approximate 0.5-acre site is located at 23392 East Glidden Drive in Aurora, Colorado (Photo 1 and Fig. 1). There is an existing clubhouse and pool northeast of the property that was constructed between 2015 and 2017. The site is bordered by single-family residence to the south and west, a parking lot to the east, and the pool and clubhouse to the north. The site slopes to the north-northwest with overall topographic relief of about 10 feet, and about 5 feet within the proposed building pad according contours shown in Grading Plans by Calibre dated February 24, 2015. Most of the surrounding development is relatively new single-family residences.

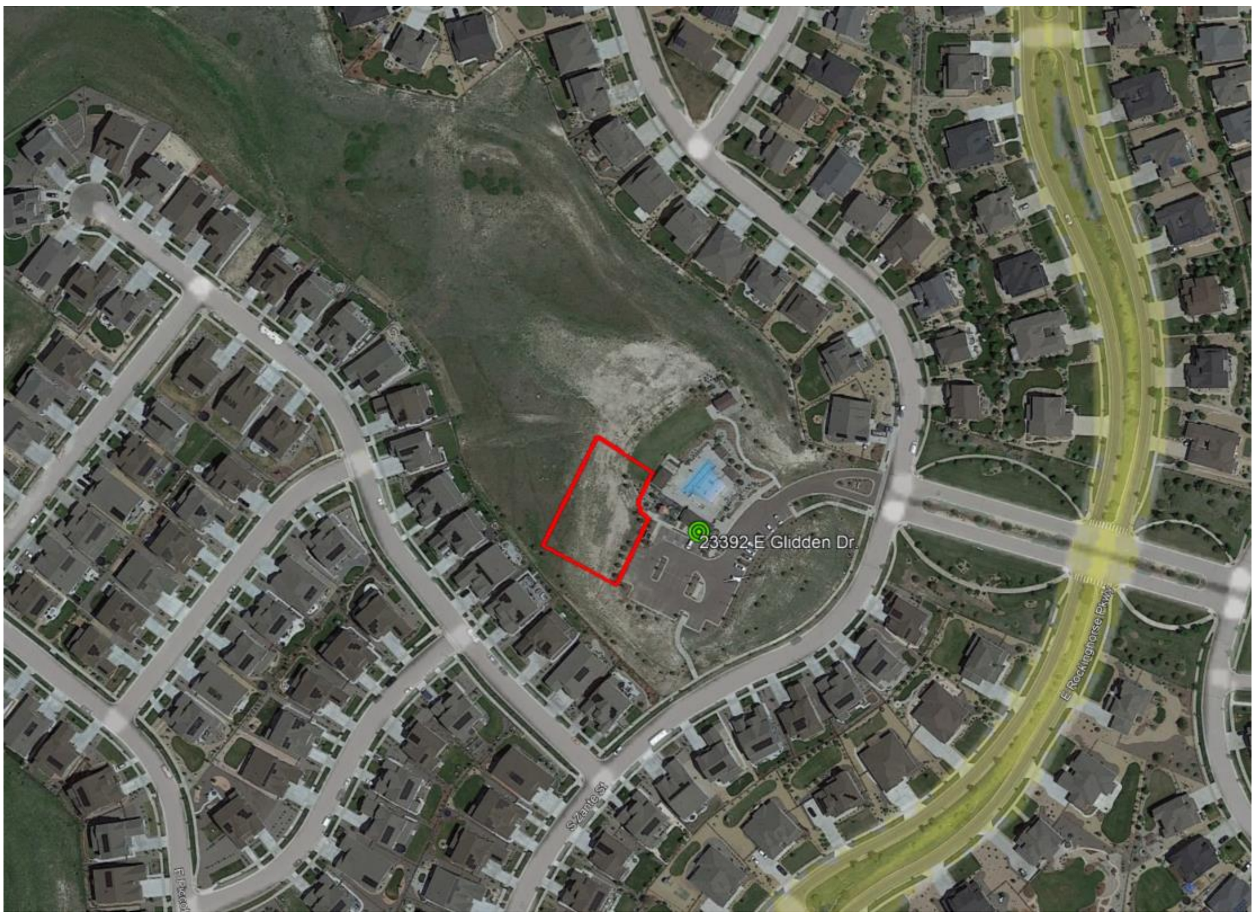

Photo 1 – Google Earth© Aerial Site Photo, June 2021

According to historical aerial photos dating back to 1937, a historical drainage terminated to the immediate northwest of the site. The drainage did not extend into the proposed area based on these historical aerial photos. Grading of the surrounding subdivision appears to begin in 2006. The site was also used as a staging area for the development for the clubhouse and pool. Much of the site has been unchanged since 2017. The ground surface generally consists of scattered bushes, grasses, and weeds. Fill from previous earthwork is more than likely present.

PROPOSED CONSTRUCTION

Architectural Site Plans and Renderings prepared by Quintessence Design Group (Project No 19001, dated July 25, 2022) show construction to include a single-story building containing a garage and meeting rooms. A community garden and hardscaped plaza are planned adjacent to the building. The building will be surrounded by pavements, flatwork, and landscaping. Below-grade areas are not planned. Grading plans have not been provided but we anticipate site grading cuts and fills will likely be limited to about 5 feet or less. We assume a finished floor elevation near EL 6177.

PREVIOUS INVESTIGATION

We prepared a Geotechnical Investigation report for Inspiration All Ages Amenity (Rocking Horse) under our Project No. DN46,748.000-125 (report dated January 2, 2015). We drilled five exploratory borings (Fig. 1) and encountered clayey sand underlain by claystone and sandstone bedrock at depths likely to influence performance of the development. The claystone was expansive while the sandstone and sand were judged to be low swelling or non-expansive. Groundwater was encountered in one boring at a depth of 34 feet. We reviewed pertinent information from the previous investigation in preparation of this report. Summary Logs of previous Exploratory Borings are included in Appendix B.

INVESTIGATION

We investigated subsurface conditions on December 7, 2022 by drilling and sampling 3 exploratory borings at the approximate locations shown on Fig. 1. We estimated the boring locations, GPS coordinates, and ground surface elevations using a Leica GS18 GPS unit referencing the NAD83. Prior to drilling, we contacted the Utility Notification Center of Colorado and local sewer and water districts to identify locations of buried utilities. The borings were drilled to depths of about 25 to 35 feet below existing grades using 4-inch diameter, continuous-flight solid-stem auger and a truck-mounted drill rig. Samples of the soil and bedrock were obtained at approximate 2 to 5 feet intervals using a 2.5-inch diameter (O.D.) modified California barrel sampler driven by blows from a 140-pound cat-head hammer falling approximately 30 inches. Our field representatives were present to observe drilling, log the strata encountered, and obtain samples. Summary Logs of the Exploratory Borings, including results of field penetration resistance tests and a portion of laboratory test results, are presented on Fig. 2.

Samples were returned to our laboratory where they were visually classified and testing was assigned. Laboratory tests included moisture content, dry density, percent silt- and clay-size particles (passing No. 200 sieve), Atterberg limits, swell-consolidation, 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). Laboratory test results are presented in Appendix A and summarized on Table A-I.

SUBSURFACE CONDITIONS

Strata generally consisted of nil to 4.5 feet of fill and up to 1.5 feet of sandy clay underlain by sandstone and claystone bedrock to the maximum explored depth of 35 feet. Hardness of bedrock generally increased with depth. Some of the pertinent engineering characteristics of the soil and bedrock are described in the following paragraphs.

Fill

Existing fill was encountered in two of the three borings at depths of 1 to 4.5 feet below existing grade. This fill is likely the result of surrounding site development. We are unsure if it was placed in a controlled manner. More or less fill could be present than our borings indicate. The fill was medium stiff based on the results of field penetration resistance testing. One sample swelled 1.3 percent when wetted, contained 51 percent silt- and clay-size particles, and exhibited high plasticity. The field penetration test indicates the fill was unlikely compacted as engineered fill.

Sandy Clay

Sandy clay was encountered at the ground surface of one boring. No samples of the clay were obtained. Based on visual observations during drilling, the soil was relatively moist.

Bedrock

Shallow bedrock is present. Sandstone and claystone bedrock were encountered below the overburden soils in each boring at depths of about 1 to 4.5 feet or approximate elevations 6169.5 to 6178 feet. Seven sandstone samples contained 11 to 32 percent silt- and clay-size particles. Sandstone was predominant and one claystone sample was very sandy. The bedrock is considered medium hard to very hard and generally became harder with depth. One claystone sample swelled 1.3 percent when wetted.

Groundwater

Groundwater was encountered during drilling in each boring at depths of about 22 to 24 feet below existing grades. When the holes were checked after drilling on December 9, 2022, water was measured in two borings depths of about 20 and 22.5 feet or approximate elevations 6154 to 6154.5 feet. Groundwater appeared to be generally confined within the bedrock. Groundwater may be encountered during drilled pier installation (if used), but is not otherwise expected to influence construction. Groundwater levels may fluctuate seasonally and rise in response to precipitation, landscape irrigation, and changes in land-use.

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 and southern 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 2021 International Building Code (Section 16.13.2.2) defers the estimation of Seismic Site Classification to ASCE7-22, a structural engineering publication. 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 1418 and 1565 feet per second with an average value of 1502 feet per second. We judge the subsurface classifies as Seismic Site Classification C. 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 no susceptibility to liquefaction from a materials and groundwater perspective.

GEOLOGIC HAZARDS

Colorado is a challenging location to practice geotechnical engineering. The climate is relatively dry and the near-surface soils are typically dry and comparatively stiff. These soils and related sedimentary bedrock formations tend to react to changes in moisture content. Some soils swell as they increase in moisture and are referred to as expansive soils. Other soils can compress significantly upon wetting and are identified as collapsible soils. Much of the land available for development east of the Front Range is underlain by expansive clay or claystone bedrock near the surface. The soils that exhibit collapsible behavior are more likely west of the Continental Divide; however, both types of soils occur throughout the state.

Covering the ground with buildings, pavements and flatwork, coupled with landscape irrigation and changing drainage patterns leads to an increase in subsurface moisture conditions. As a result, some soil movement is inevitable. It is critical that all recommendations in this report are followed to increase the chances that the slabs-on-grade, pavements, and other improvements will perform satisfactorily. After construction, property owners/managers and the District must assume responsibility for maintaining the structures and use appropriate practices regarding drainage and landscaping.

Expansive soils and bedrock were encountered in our borings, which constitutes a geologic hazard. Expansive soil and bedrock may swell upon wetting. There is risk that ground heave will damage slabs-on-grade, pavements and improvements. Non-engineered fill was encountered in our borings. The tests indicate the fill is susceptible to settlement after new loads are applied and/or when the fill is wetted. <u>All existing fill should be removed via sub-excavation below foundations and floor systems and pavements</u>. The risk of foundation and slab movements can be mitigated, but not eliminated by careful design, construction, and maintenance procedures. We believe the recommendations in this report will help control risk of damage; they will not eliminate that risk. The owner(s) should understand that surface improvements may be affected by movement of the subsoils. Proper design, construction and maintenance will be required to control risk.

Estimated Potential Heave

We estimated total potential ground heave considering wetting to a depth of 24 feet below existing grade. The analysis of estimated potential heave involves dividing the soil profile into layers and modeling the heave of each layer from representative swell/consolidation tests. We calculated heave using the Partial Wetting Adjustment Factor (PWAF) method. Research by Houston et al.[1] indicates the highest degree of wetting occurs near-surface with a gradually decreasing degree of wetting with depth. We estimate up to about 1 inch of heave is possible without sub-excavation. It is uncertain whether the estimated heave will occur. <u>Proposed grading will affect these estimates</u>.

[1] "Use of the Net Partial Wetting Factor (NPWF) Method of Computation of Remaining Heave: A Forensic Study" by Houston, Stauffer, West, Bradford, and Houston, 2017.

TABLE I - ESTIMATED POTENTIAL HEAVE AT EXISTING GROUND SURFACE
BASED ON 24 FEET DEPTH OF WETTING

Boring	Potential Heave (inches)
TH-1	<½
TH-2	1
TH-3	<½

SITE DEVELOPMENT

Existing Fill

Existing fill was encountered in two of three exploratory borings. This fill is likely associated with previous site development and grading. The fill is judged unsuitable to support shallow foundations and slab-on-grade floors. Existing fill below the building footprint plus a 5-foot margin on all sides should be removed and replaced, in its entirety, as described in <u>Fill and Backfill</u>. Sub-excavation of existing fill below flatwork and pavements are considered optional. Electing to sub-excavate fill below hardscapes and pavements is an informed decision that only the District can make. Should the District elect to leave the existing fill in-place, settlement and damage and poor performance of pavements and flatwork should be anticipated. Replacement and/or increased maintenance of pavements and flatwork is anticipated if the fill is not sub-excavated.

Excavation

We believe soils encountered in our exploratory borings can generally be excavated with conventional, heavy-duty excavation equipment. We recommend the owner and the contractor become familiar with applicable local, state and federal safety regulations, including the current OSHA Excavation and Trench Safety Standards. We believe the claystone and clay will classify as Type B soils, and the fill and sandstone as Type C soils, which require maximum slope inclination of 1H:1V and 1½H:1V (horizontal:vertical) for temporary excavations in dry conditions. Flatter slopes will be required where seepage is present (if any). 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 horizontal distance equal to one-half the excavation depth, from the crest of an excavation. A Registered Professional Engineer should design excavations greater than 20 feet deep.

Sub-Excavation

We encountered existing, undocumented fill and expansive claystone within the upper 7 feet of the existing ground surface and estimate up to about 1 inch of potential heave is possible at existing grades. Heave can deform and crack floor slabs and can damage interior partitions, especially where claystone is at shallow depths. Existing fill the previous development can lead to settlements. <u>To reduce potential movement and provide a uniform subgrade, we recommend sub-excavation to at 5 feet below lowest foundation level or 8 feet below existing grades, which-ever is deeper.</u> Sub-excavation should extend to a uniform elevation below the entire building. Alternatively, a deep foundation system, such as drilled piers bottomed in bedrock can be used in lieu of sub-excavation.

Sub-excavation has been used in the Denver area with satisfactory performance for most sites. Sub-excavation should reduce potential heave and enhance performance of slab-on-grade floors, pavements, and other surface improvements. We estimate potential movements and differential movements can be significantly reduced if sub-excavation is performed as out-lined in our specifications.

The extent and depth of sub-excavation should be surveyed and an “as-built” plan of the sub-excavated areas should be prepared. We have seen isolated instances where settlement of sub-excavation fill has led to damage. In most cases, the settlement was caused by wetting as-sociated with poor surface drainage and/or poorly compacted fill placed at the horizontal limits of the sub-excavation. The bottom of the sub-excavation should be extended to 5 feet beyond the largest building footprint. Special precautions should be taken for compaction of fill at cor-ners, access ramps and edges of the sub-excavation due to equipment access constraints. The contractor should have the appropriate equipment to reach and compact these areas. A concep-tual sub-excavation profile is presented on Fig. 3.

The excavation contractor should be chosen carefully to assure they have experience with fill placement at over-optimum moisture and have the necessary compaction equipment. The contractor should provide a construction disc to break down fill materials. Soils chunks should be broken down to 3 inches and less. Due to the relative hardness of the bedrock, additional effort may be needed to break down chunks of claystone to acceptable sizes. We anticipate use of excavator and loader sub-excavation operations due to the relatively small building footprint. We strongly suggest the use of a roller and sheepsfoot compactor.

The sub-excavation operation may be relatively slow. In order for the procedure to be performed properly, close contractor control of fill placement to specifications is required. Sub-excavation fill should be moisture-conditioned between 1 and 4 percent above optimum moisture content for clay and claystone or within 2 percent of optimum for sand and sandstone. Fill should be compacted at least 95 percent of standard Proctor maximum dry density. The borings indicate existing soil and bedrock can more than likely be reused as engineered fill. Our representative should observe and test compaction of fill during placement. The swell of the sub-excavation fill should be tested during fill placement to better evaluate whether the desired results have been achieved.

Fill and Backfill

The on-site soils are suitable for reuse as new fill, provided they are free of debris, vegetation/organics, and other deleterious materials. The borings indicate existing soil and bedrock can more than likely be reused as engineered fill. Soil particles larger than 3 inches in diameter should not be used for fill. Imported fill should ideally consist of soil having a maximum particle size of 3 inches, less than 50 percent passing the No. 200 sieve, a liquid limit less than 30, and a plasticity index less than 15. Potential fill materials should be submitted to our office for approval at least two weeks prior to importing to the site.

Prior to fill placement, the excavation surface should be scarified to a depth of at least 8 inches, moisture-conditioned, and compacted to the criteria below. Subsequent fill should be placed in thin (8 inches or less) loose lifts, moisture conditioned and compacted to within 2 percent of optimum moisture content for sand or between 1 and 4 percent above optimum moisture content for clay and compacted to at least 95 percent of standard Proctor maximum dry density (ASTM D 698).

Utility Trench Backfill

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 and compacted as outlined above. Our experience indicates use of self-propelled compactors results in more reliable performance compared to fill 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. The placement and compaction of utility trench backfill should be observed and tested by a representative of our firm during construction.

Our experience indicates fill and backfill can settle, even if properly compacted to criteria provide above. Factors that influence the amount of settlement are depth of fill, material type, degree of compaction, amount of wetting and time. The degree of compression of fill under its own weight will likely be about 1 percent.

Permanent Slopes

We recommend permanent cut and fill slopes be designed with a maximum grade of 3:1 (horizontal to vertical). Use of flatter slopes (4:1) is preferable to control erosion. If site constraints (property boundaries and streets) do not permit construction with recommended slopes, we should be contacted to evaluate the subsurface soils and steeper slopes. Surface drainage should not be allowed to sheet flow across slopes or pond near the crest of slopes. All cut and fill slopes should be designed and re-vegetated as soon as possible after grading to reduce potential for erosion problems. Excavation contractors should evaluate ground conditions and control slopes in accordance with OSHA criteria.

FOUNDATIONS

Our investigation revealed existing fill, sandy clay, and sandstone with erratic claystone beds at depths likely to influence shallow foundation performance. We believe spread footings are suitable for the proposed structures provided sub-excavation is performed. Sub-excavation will not eliminate the potential for movement but will likely reduce differential movements and provide more uniform foundation and floor subgrade. Alternatively, straight-shaft drilled piers bottomed in bedrock can also be used if sub-excavation is forgone. The bedrock generally became harder with depth. This should be anticipated during drilling of the drilled piers. Design and construction criteria for drilled piers are presented below.

Spread Footings (after sub-excavation)

1. Footings should be constructed on new, moisture-conditioned, compacted fill at least 5 feet inside the sub-excavation limits. Where soft soils are present or are the result of the excavation and footing forming process, the soils should be removed and re-compacted prior to placing concrete.

2. Footings can be designed for a maximum allowable soil pressure of 2,000 psf. The maximum pressure can be calculated using deadload plus ½-live load.

3. Minimum deadload pressure on foundations and void form below grade beams are not required.

4. Lateral earth pressures can be calculated based on equivalent fluid densities presented in **LATERAL EARTH PRESSURES**. The coefficient of friction for a footing sliding on the site soils is 0.3. These values have not been factored. The structural engineer should apply appropriate factors of safety in design.

5. Continuous wall footings should have a minimum width of at least 18 inches. Foundations for isolated columns should have minimum dimensions of 20 inches square. Footing widths may be greater depending upon foundation loads.

6. Exterior footings must be protected from frost action. Normally, 3 feet of frost cover is assumed in this area.

7. Foundation walls and grade beams 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.

8. The completed foundation excavations should be observed by a representative of our firm to confirm subsurface conditions are as anticipated from our borings, and to evaluate the presence of any poorly compacted fill. Our representative should observe and test moisture and compaction of the fill and backfill placed below foundations.

9. 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 building is critical to control wetting.

Drilled Piers Bottomed in Bedrock

1. Piers should be designed for a maximum allowable end pressure of 30,000 psf and skin friction value of 3,000 psf for the portion of pier in comparatively un-weathered bedrock. The skin friction should be neglected within the overburden soils, within 3 feet of foundation walls, grade beams and pier caps, and where casing is used within the bedrock (if any).

2. Piers should penetrate a minimum of 5 feet into bedrock and have a minimum total length of 20 feet. Longer piers may be necessary for structural considerations. A minimum diameter of 12 inches is recommended.

3. A 4-inch or thicker continuous void should be constructed beneath grade beams, foundation walls and pier caps.

4. We recommend piers be drilled with a large, heavy-duty drill rig to facilitate the required bedrock penetration. Formation of "mushrooms" or enlargements at the tops of piers should be avoided during pier drilling and subsequent construction operations. Use of sonotube may be one alternative to reduce risk of "mushroomed" pier tops.

5. Shear rings in the lower portion of piers are not required.

6. 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.

7. Piers should be designed to resist an ultimate uplift force calculated as (35 kips x pier diameter in feet minus the deadload) to resist tension in the event of swelling. Piers should be reinforced their full length and the reinforcement should extend an adequate distance into grade beams, foundation walls, and pier caps. Grade 60 (or better) steel is recommended. More reinforcement may be required for structural considerations.

8. 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. Reduction factors for closely spaced piers are provided in a following section.

9. Pier holes should be cleaned prior to placement of concrete. Concrete should be on-site and placed in the pier holes immediately after the holes are drilled, cleaned and inspected using drill and pour construction procedure. Groundwater may be encountered during pier drilling and installation, although unlikely. Use of temporary casing and tremie pipe or pumping may be required for proper dewatering and placement of concrete during pier installation. Concrete should not be placed by free-fall in pier holes containing more than 3 inches of water.

10. 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, casing and sides of the piers.

11. Some pier-drilling contractors use casing with an outer diameter (O.D.) equal to the specified pier diameter. This practice results in a nominal pier diameter less than specified. The pier 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.

12. Some movement of drilled pier foundations should be anticipated to mobilize the skin friction. We estimate the movement will be on the order of ¼ to ½-inch. Differential movement between adjacent piers may equal total movement.

13. Installation of drilled piers should be observed by a geotechnical firm to identify the bearing strata, confirm subsurface conditions are as anticipated from our borings and observe the contractor's installation procedures.

Laterally Loaded Piers

Lateral load analysis of piers can be performed with the software analysis package LPILE by Ensoft, Inc. We believe this method of analysis is appropriate for piers with a pier length to diameter ratio of seven or greater. Suggested criteria for LPILE analysis are presented in the following table.

SOIL INPUT DATA FOR "LPILE"

Soil Type	Sandy Clay	Bedrock
Soil Model Type	Stiff Clay w/o Free Water	Stiff Clay w/o Free Water
Effective Unit Weight (psf)	125	130
Cohesive Strength, c (psf)	1500	7920 (LPile maximum)
Soil Strain, ε_{50} (in/in)	Default LPILE Values	Default LPILE Values
p-y Modulus, k_s (pci)	Default LPILE Values	Default LPILE Values
p-y Modulus, k_c (pci)	Default LPILE Values	Default LPILE Values

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

Closely Spaced Pier Reduction Factors

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

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

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 piers will result in greater computed deflections on these piers. In practice, a grade beam can force deflections of all piers to be equal. Load-deflection graphs can be generated for each pier by using the appropriate p-multiplier values. The sum of the piers lateral load resistance at selected deflections can be used to develop a total lateral load versus deflection graph for the system of piers.

For lateral loads perpendicular to the line of piers, a minimum spacing of three diameters can be used with no capacity reduction. At one diameter (piers touching) the piers 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[2] result in more practical solutions for group efficiency. They were formulated by fitting curves to data representing group efficiency versus pier spacing. No differentiation was made between soil type, pier 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 pier spacing and "b" is the pier diameter.

Side-by-side piers:

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

Leading piers:

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

Trailing piers:

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

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 $e_{\mathrm{i}} =$ efficiency of pile in-line,
$e_{\mathrm{s}} =$ efficiency of pier side-by-side, and
$\emptyset =$ angle between piers (Reese & Wang, 1996)

FLOOR SYSTEMS

Risk of floor slab distress due to expansive soils is considered comparatively low after sub-excavation. Potential heave or settlement of about 1 inch or less is considered probable after sub-excavation. This movement could cause damage to brittle floor finish materials and interior partitions. A structurally supported floor (over a crawl space) should be used if you wish to positively control floor movement or if sub-excavation is not performed.

[2] "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

Slab-On-Grade

If the owner understands and accepts the risk of slab-on-grade movement and selects a slab-on-grade floor system, we recommend the following design and construction criteria. These recommendations will not prevent movement. Rather, they tend to reduce damage if movement occurs.

1. Slabs should be poured directly on properly moisture-conditioned, well-compacted fill.

2. The International Building Code (IBC) requires a vapor retarder between base course or subgrade soils and the concrete slab-on-grade floor. The merits of installation of a vapor retarder below floor slabs depend on the sensitivity of floor coverings and building use to moisture. A properly installed vapor retarder (6 mil minimum, 10 mil for better durability) 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 derations and recommendations for the installation of vapor retarders below concrete slabs are outlined in Section 5.2.3.2 of the 2015 report of American Concrete Institute (ACI) Committee 302, “Guide to Concrete Floor and Slab Construction (ACI 302.1R-15).

3. Slabs should be separated from exterior foundation walls, grade beams, and interior bearing members with a slip joint that allows free vertical movement of the slabs. This detail can reduce cracking if movement occurs.

4. Slab-bearing partition walls should be designed and constructed to allow at least 1 inch of slab movement. This can be accomplished by providing a 2-inch void space above or below interior partition walls. In our experience, voiding the below the partitions can provide better performance.

5. Load bearing walls should be supported by the foundation system. Interior, non-load bearing masonry walls can be supported on a thickened slab provided they are detached from walls that are supported by the foundations and voided at the top.

6. Plumbing and utilities that pass through the slab should be isolated from the slab and constructed with flexible couplings. Utilities, as well as electrical and mechanical equipment should be constructed with sufficient flexibility to allow for movement. Details should be prepared by the mechanical engineer.

7. Mechanical systems supported by the slab (if any) should be provided with flexible connections capable of withstanding at least 2 inches of movement. Details should be prepared by the mechanical engineer.

8. The American Concrete Institute (ACI) recommends frequent control joints in slabs to reduce problems associated with shrinkage cracking and curling. To reduce curling, the concrete mix should have a high aggregate content and a low slump. If desired, a shrinkage compensating admixture could be added to the concrete to reduce the risk of shrinkage cracking. We can perform a mix design or assist the design team in selecting a pre-existing mix.

Structurally Supported Floors

To our knowledge, there are no soil treatments combined with slab-on-grade floors that will result in the same reduction in risk of floor movement (relative to the risk inherent for a floor slab placed directly on the natural soils), as would be provided by a structural floor. If floor movement cannot be tolerated, then a structurally supported floor should be used.

A structural floor is supported by the foundation system. Design and construction issues associated with structural floors include ventilation and lateral loads. Where structurally supported floors are installed over a crawl space, the required air space depends on the materials used to construct the floor and the potential expansion of the underlying soils.

Where structurally supported floors are installed, we recommend a minimum void of 4 inches between the ground surface and the underside of the floor system. If untreated wood floor components are used, we recommend an air gap of at least 18 inches between the wood and soil. The minimum void (4 inches) should be constructed below beams and utilities that penetrate the floor. Concrete floor systems may also be cast over void forming material. Void form should be chosen that will break down quickly after the slab is placed. We do not recommend wax or plastic-coated void boxes unless measures are taken to allow moisture to penetrate the void.

Where structurally supported floors are used, utility connections, including water, gas, air duct, and exhaust stack connections to floor supported appliances, should be capable of absorbing some deflection of the floor. Plumbing that passes through the floor should ideally be hung from the underside of the structural floor and not lain on the bottom of the excavation. This configuration may not be achievable for some parts of the installation. It is prudent to maintain the minimum clear space below all plumbing lines. If trenching below the lines is necessary, we recommend sloping these trenches so they discharge to the foundation drain.

Control of humidity in crawl spaces is important for indoor air quality and performance of wood floor systems. We believe the best current practices to control humidity involve the use of a vapor retarder or vapor barrier (6 mil minimum) placed on the soils below accessible subfloor areas. The vapor retarder/barrier should be sealed at joints and attached to concrete foundation elements. If desired, we can provide designs for ventilation systems that can be installed in association with a vapor retarder/barrier to improve control of humidity in crawl space areas.

Exterior Flatwork

We recommend exterior flatwork and sidewalks be isolated from foundations to reduce the risk of transferring heave, settlement or freeze-thaw movement to the structure. One alternative would be to construct the inner edges of the flatwork on haunches or steel angles bolted to the foundation walls and detailing the connections such that movement will cause less distress to the building, rather than tying the slabs directly into the building foundation. Construction on haunches or steel angles and reinforcing the sidewalks and other exterior flatwork will reduce the potential for differential settlement and better allow them to span across wall backfill. Frequent control joints should be provided to reduce problems associated with shrinkage. Panels that are approximately square perform better than rectangles.

LATERAL EARTH PRESSURES

Below-grade walls should be designed to resist lateral earth pressures. The pressure is a function of the wall height, type of backfill, drainage conditions, slope of the backfill surface, and the allowable rotation of the wall. If the walls will be essentially rigid and unable to rotate to mobilize the strength of the backfill soils, they should be designed for an "at rest" earth pressure condition. For walls that are free to rotate (e.g., retaining walls not attached to the buildings), an “active” earth pressure resistance can be used. A “passive” earth pressure resistance and sliding friction can be used to resist sliding and overturning. Passive resistance requires movement to generate the resistance. Passive resistance should only be used when movement is tolerable and the soil is well compacted and will be not be removed. We have tabulated equivalent fluid density values for use in design below.

LATERAL EQUIVALENT FLUID DENSITY FOR BACKFILL

Backfill Type	On-Site Soil
Active Equivalent Fluid Density (pcf)	40
At Rest Equivalent Fluid Density (pcf)	55
Passive Equivalent Fluid Density (pcf)*	325*

*Assumes compacted backfill will never be removed.

All backfill should be well compacted, as discussed in the **Fill and Backfill** section. The pressures given above do not include allowances for surcharge loads such as sloping backfill, vehicle traffic or hydrostatic pressure, and assume foundation drains are installed. We can provide criteria for surcharge loads if desired.

SUBSURFACE DRAINAGE

The site slopes to the northwest with about 10 feet of topographic relief and about 5 feet within the proposed building footprint. Due to this difference in elevation, below-grade walls could be constructed. Water from landscape irrigation frequently flows through relatively permeable backfill placed adjacent to a building and collects on the surface of less permeable soils occurring at the bottom of foundation or crawl space excavations. This process can cause wet or moist conditions in any below-grade areas such as crawl spaces, if present. To reduce the likelihood water pressure will develop outside foundation walls and the risk of accumulation of water at the lower levels (and subsequent wetting), you may consider provision of a foundation drain around the entire perimeter of any below-grade areas. A foundation drain is recommended around the perimeter of crawl spaces, if they are used. Subsurface drains should consist of a 4-inch diameter, perforated pipe encased in free-draining gravel. The foundation drain should be connected to a gravity outfall or a sump pit where water can be removed with a pump. Conceptual foundation wall drain details for crawl space construction are presented on Figs. 4 through 7. High moisture conditions in the native soils or localized zones of free water could result in moisture transmission through the below-grade walls. If this cannot be tolerated, then some form of moisture retarder or waterproof barrier should be considered. The level of protection desired will depend on the level of performance desired. Building code considerations may control the required level of protection. The provision of a drain will not eliminate slab movement or prevent moist conditions in below-grade areas.

CONCRETE

Concrete in contact with soil can be subject to sulfate attack. Samples were predominantly sandstone at shallow depths. We measured low water-soluble sulfate concentrations in the surrounding Inspiration filings (less than 0.1 percent). As indicated in our tests and ACI 318-19, the sulfate exposure class is *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 cement type requirements for sulfate resistance as indicated in the table below.

CONCRETE DESIGN REQUIREMENTS FOR SULFATE EXPOSURE PER ACI 318-19

Exposure Class		Maximum Water/ Cement Ratio	Minimum Compressive Strength (psi)	Cementitious Material Types [A]			Calcium Chloride Admixtures
				ASTM C150/ C150M	ASTM C595/ C595M	ASTM C1157/ C1157M	
S0		N/A	2500	No Type Restrictions	No Type Restrictions	No Type Restrictions	No Restrictions
S1		0.50	4000	II [B]	Type with (MS) Designation	MS	No Restrictions
S2		0.45	4500	V [B]	Type with (HS) Designation	HS	Not Permitted
S3	Option 1	0.45	4500	V + Pozzolan or Slag Cement [C]	Type with (HS) Designation plus Pozzolan or Slag Cement [C]	HS + Pozzolan or Slag Cement [C]	Not Permitted
S3	Option 2	0.4	5000	V [D]	Type with (HS) Designation	HS	Not Permitted

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, even though sulfate levels are relatively low. 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

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 structures 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. Excessive wetting before, during and/or after construction may cause movement of foundation, slabs-on-grade, and pavements. 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. Paved surfaces should be positively sloped to drain away from the buildings. 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. Backfill around foundations should be moistened and compacted according to criteria presented in Fill and Backfill. 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 used 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 buildings and improvements. Use of area drains can assist draining areas that cannot be provided with adequate slope.

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. Roof drains should be directed away from the structures and discharge beyond backfill zones or into 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 outlets, not foundation drains. Roof drains should not be directed below slab-on-grade floors. Roof drain outlets should be maintained.

GEOTECHNICAL RISK

The concept of risk is an important aspect with any geotechnical evaluation, primarily because the methods used to develop geotechnical recommendations do not comprise an exact science. We never have complete knowledge of subsurface conditions. Our analysis must be tempered with engineering judgment and experience. Therefore, the recommendations presented in any geotechnical evaluation should not be considered risk-free. Our recommendations represent our judgment of those measures that are necessary to increase the chances that the structure and improvements will perform satisfactorily. It is critical that all recommendations in this report are followed during construction. Owners or property managers 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 the Inspiration Metropolitan District for the purpose of providing geotechnical design and construction criteria for the Inspiration Metropolitan District Service Building planned at 23392 East Glidden Drive in Aurora, Colorado. The information, conclusions, and recommendations presented herein are based upon consideration of many factors including, but not limited to, the type of structures 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 development 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 plans change to review and revise our recommendations, if necessary.

Our borings were spaced to obtain a reasonably accurate picture of subsurface conditions below the proposed structure. The borings are representative of conditions encountered only at the location 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 structures or any other aspect of the proposed construction, please call.

CTL | THOMPSON, INC.

Hayden Ehrle
Staff Engineer

Reviewed by:

Matthew D. Monteith, P.E., D.GE
Senior Geotechnical Engineer, Associate

COLORADO LICENSED
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PROFESSIONAL ENGINEER

01/11/23 11:12 AM

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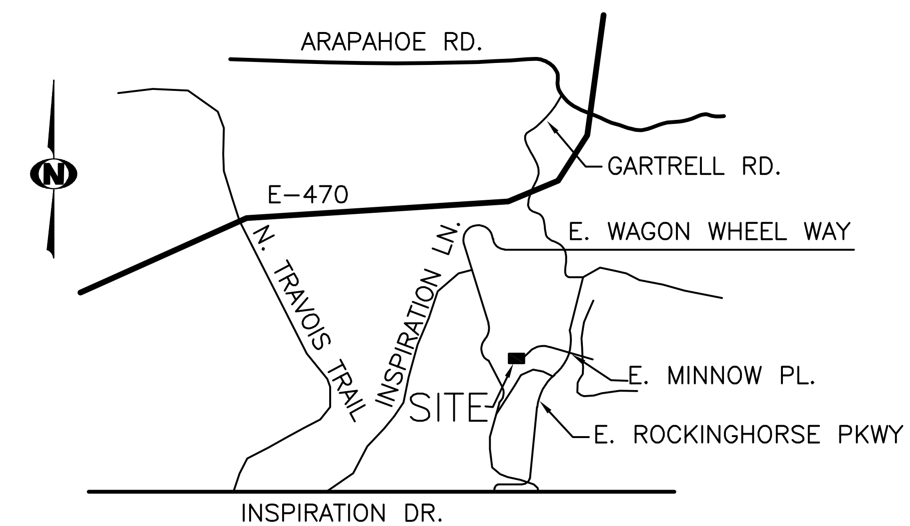
geol@publicalliancellc.com
jennifer@quintessence-dg.com



VICINITY MAP
NOT TO SCALE

LEGEND:

TH–1 APPROXIMATE LOCATION OF EXPLORATORY BORING, 2022

TH–1 APPROXIMATE LOCATION OF EXPLORATORY BORING, 2015

0 30 60
SCALE: 1" = 60'

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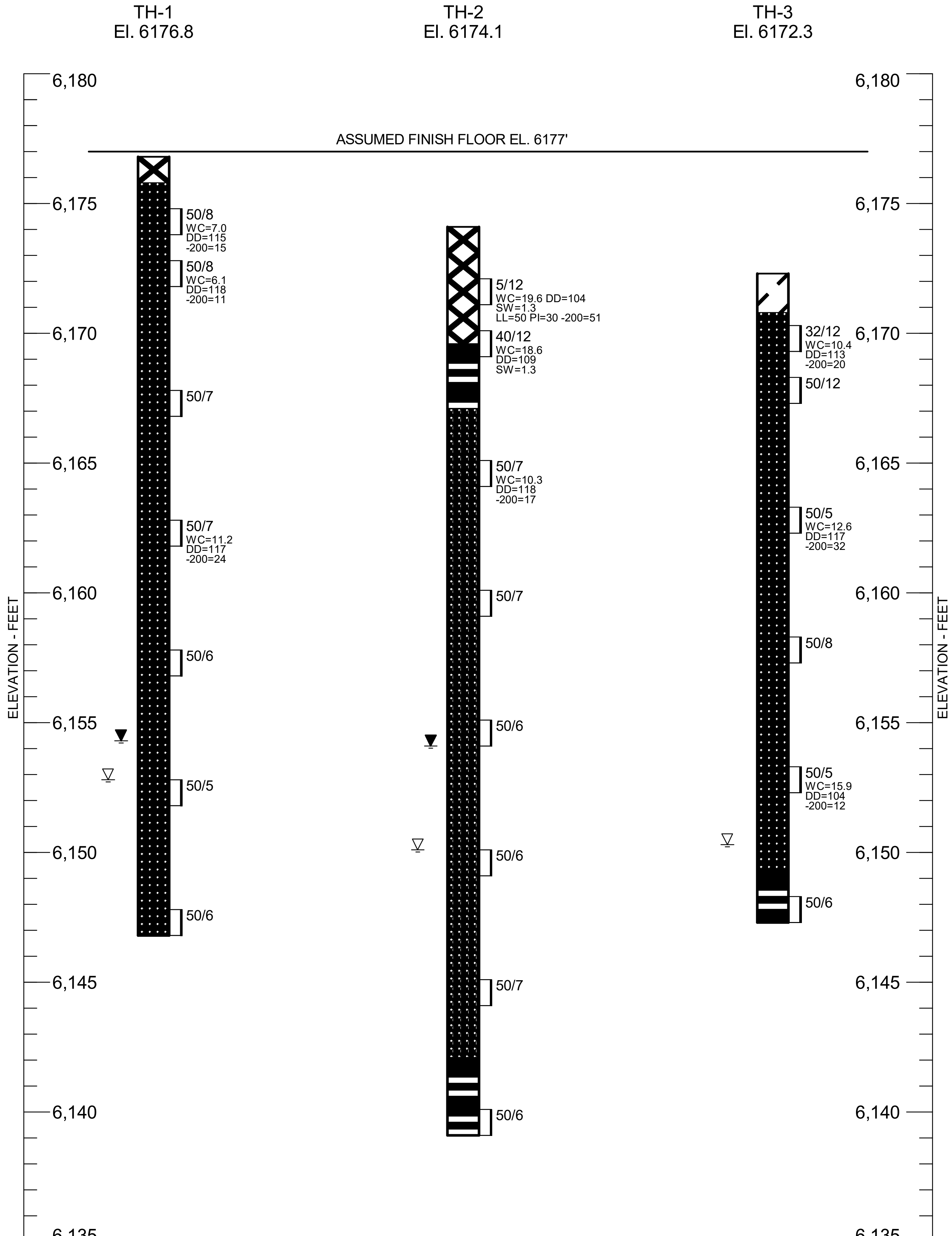
Locations of Exploratory Borings

Fig. 1



SUMMARY LOGS OF EXPLORATORY BORINGS

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LEGEND:

FILL, CLAY, VERY SANDY, MEDIUM STIFF, SLIGHTLY MOIST, BROWN, DARK BROWN, DARK GRAY (CL/CH).

CLAY, SANDY, MOIST, BROWN (CL/CH).

BEDROCK, CLAYSTONE, MEDIUM HARD TO HARD, SLIGHTLY MOIST, BROWN, GRAY (CL/CH).

BEDROCK, SANDSTONE, SLIGHTLY SILTY-CLAYEY TO SILTY-CLAYEY, HARD TO VERY HARD, SLIGHTLY MOIST, BROWN, GRAY, RUST (SM, SC, SP-SM, SP-SC).

DRIVE SAMPLE. THE SYMBOL 50/8 INDICATES 50 BLOWS OF A 140-POUND HAMMER FALLING 30 INCHES WERE REQUIRED TO DRIVE A 2.5-INCH O.D. SAMPLER 8 INCHES.

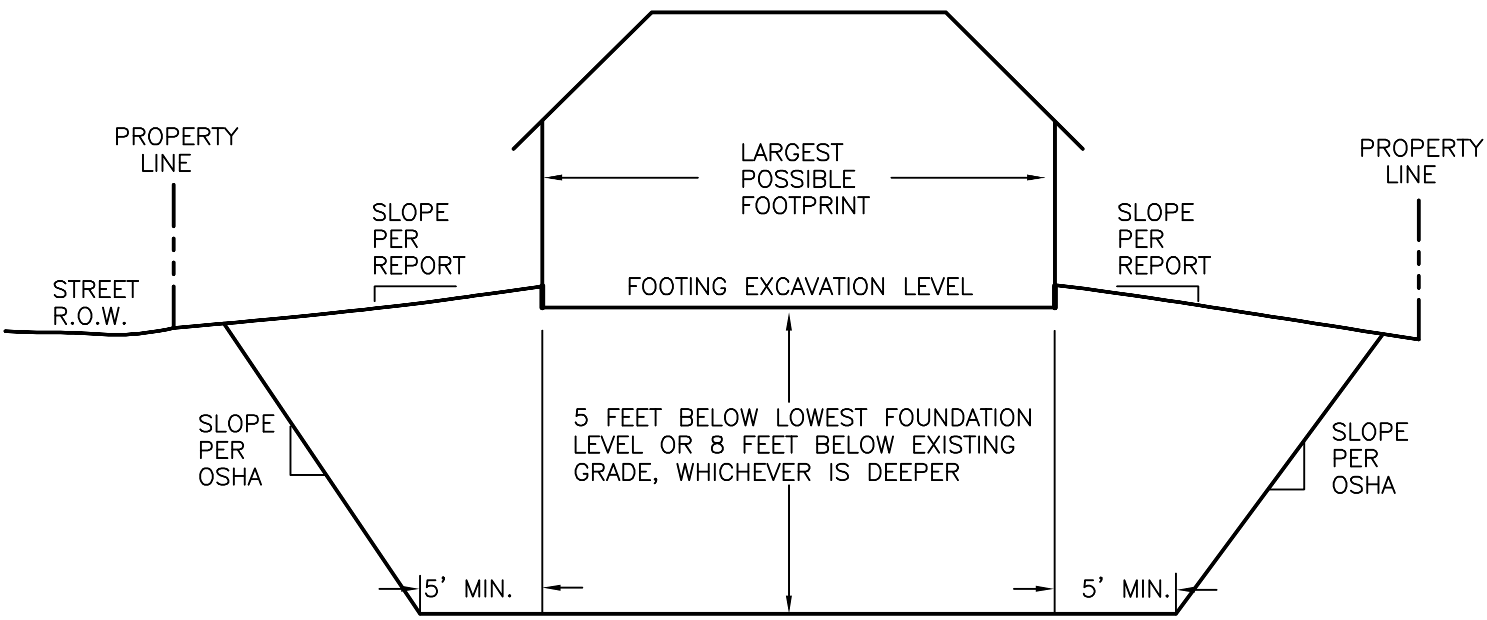
WATER LEVEL MEASURED AT TIME OF DRILLING.

WATER LEVEL MEASURED AFTER DRILLING ON DECEMBER 9, 2022.

NOTES:

1. THE BORINGS WERE DRILLED ON DECEMBER 7, 2022 USING 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 (NAD 83).
3. WC - INDICATES MOISTURE CONTENT (%).
   DD - INDICATES DRY DENSITY (PCF).
   SW - INDICATES SWELL WHEN WETTED UNDER APPROXIMATE OVERBURDEN PRESSURE (%).
   LL - INDICATES LIQUID LIMIT.
   PI - INDICATES PLASTICITY INDEX.
   -200 - INDICATES PASSING NO. 200 SIEVE (%).
   SS - INDICATES WATER-SOLUBLE SULFATE CONTENT (%).
4. THESE LOGS ARE SUBJECT TO THE EXPLANATIONS, LIMITATIONS AND CONCLUSIONS CONTAINED IN THIS REPORT.

FIG. 2



NOT TO SCALE

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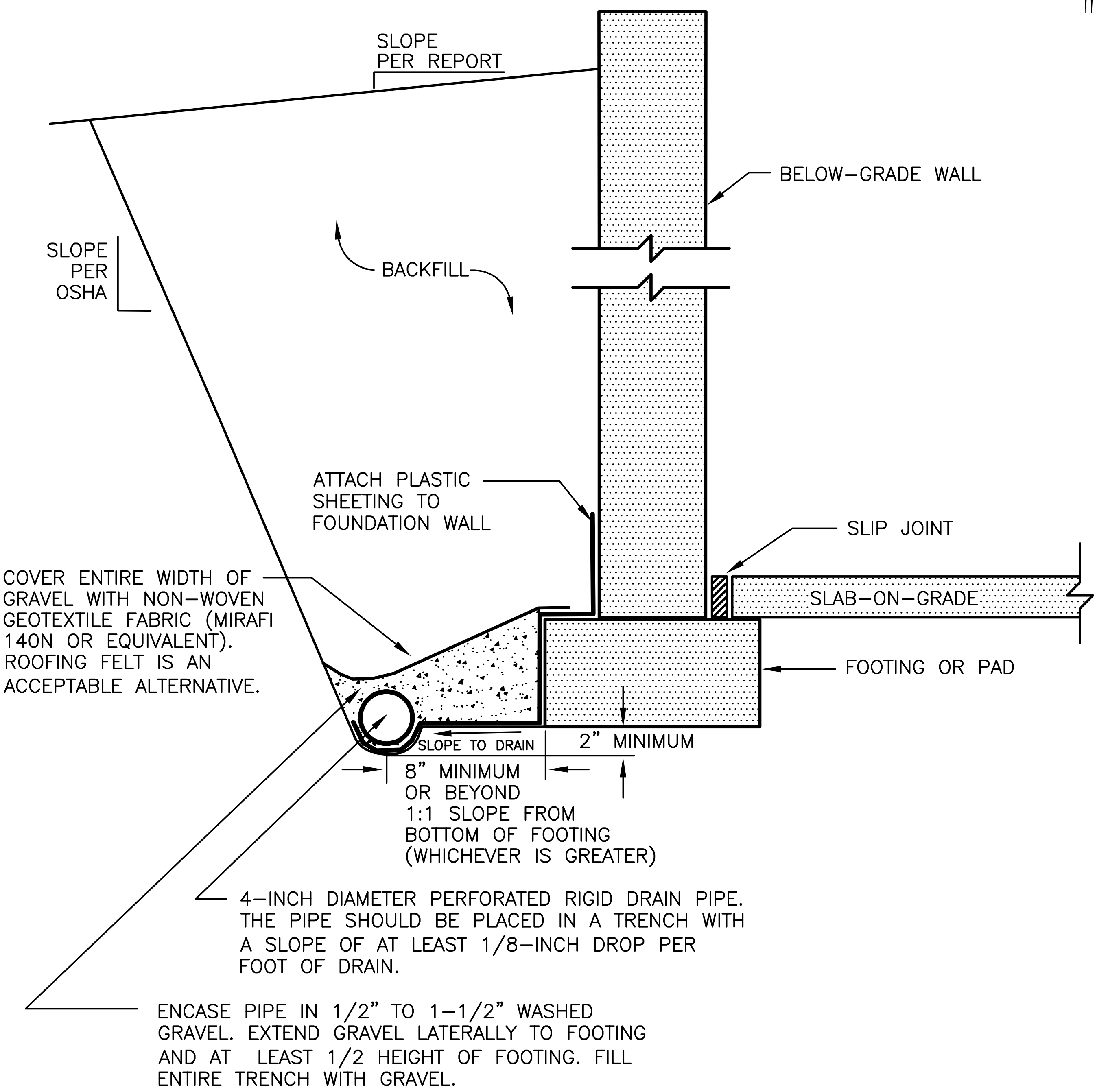
Conceptual Sub-Excavation Profile

Fig. 3

95-SUB-EXC_01



01-FTG-XTR-STF-02

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.

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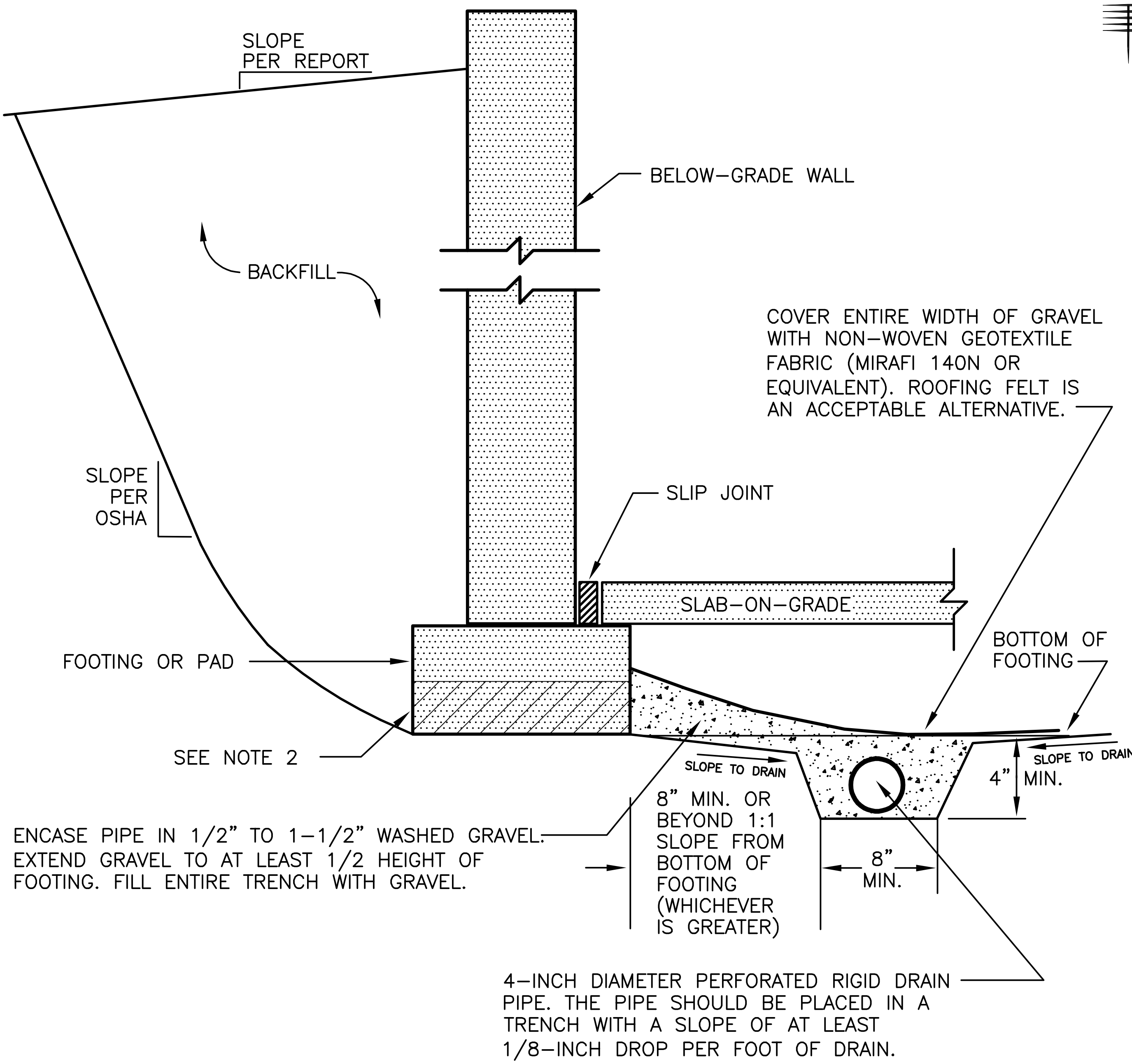
Exterior Foundation Wall Drain

Fig. 4



01-FTG-INT-STF-01

NOTES:

1) THE BOTTOM OF THE DRAIN SHOULD BE AT LEAST 4 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.

2) FOR FOOTINGS IN BASEMENT AREAS, WE RECOMMEND PLACING A 8-INCH THICK, 8-INCH WIDE SECTION OF VOID FORM PERPENDICULAR TO THE FOOTING ABOUT EVERY 10 TO 15 FEET AND AT WINDOW WELLS, TO ALLOW WATER IN WALL BACKFILL TO PASS BENEATH THE FOOTING INTO THE INTERIOR DRAIN. THIS CAN ALSO BE ACCOMPLISHED BY "TUNNELING" UNDER FOOTINGS AT THE TIME OF DRAIN INSTALLATION. THIS DETAIL SHOULD BE REVIEWED BY THE STRUCTURAL ENGINEER DURING FOUNDATION DESIGN AND INCORPORATED INTO THE FOUNDATION PLAN. ALTERNATIVELY, AN EXTERIOR FOUNDATION DRAIN CAN BE USED.

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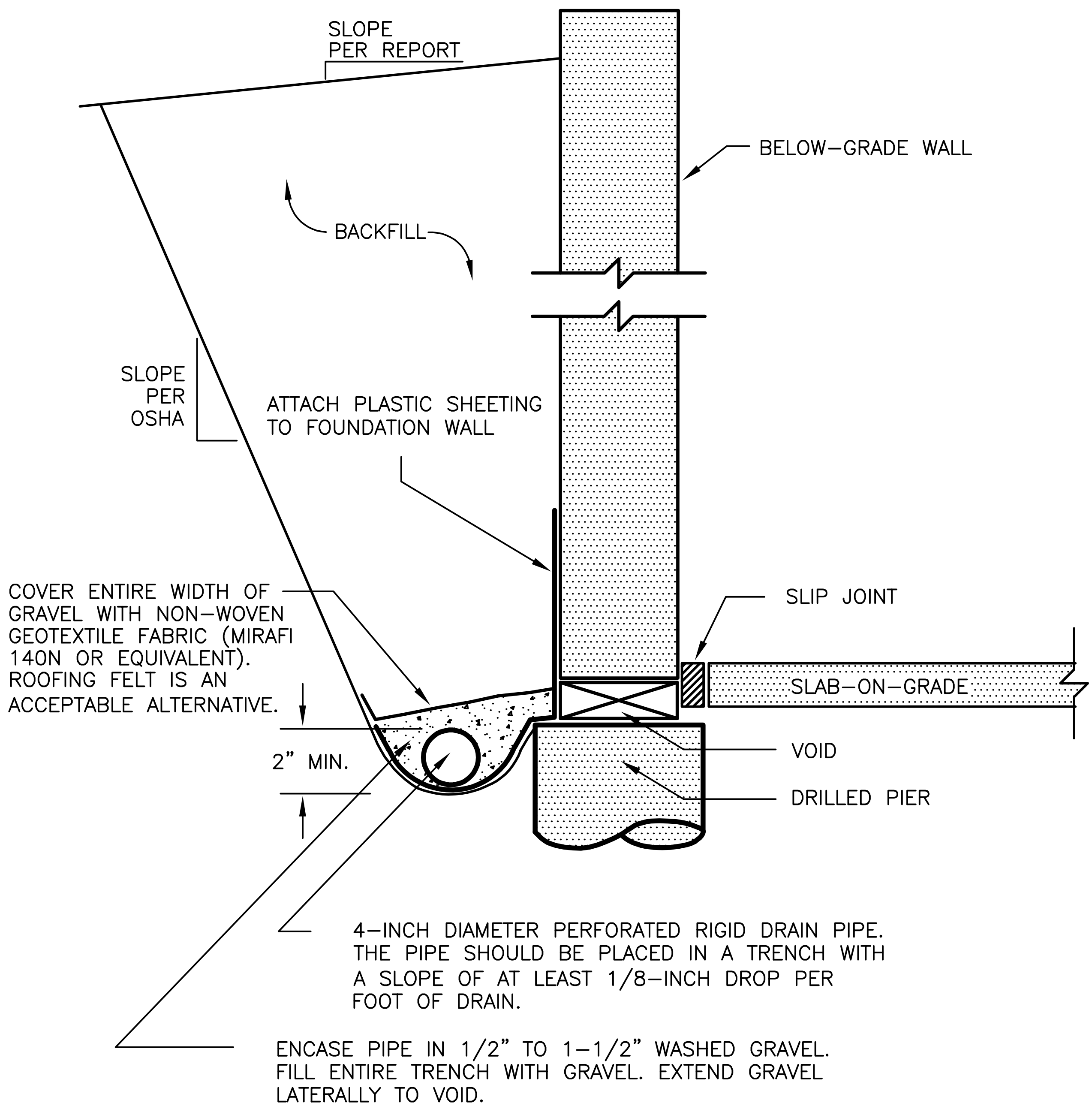
Interior Foundation Wall Drain

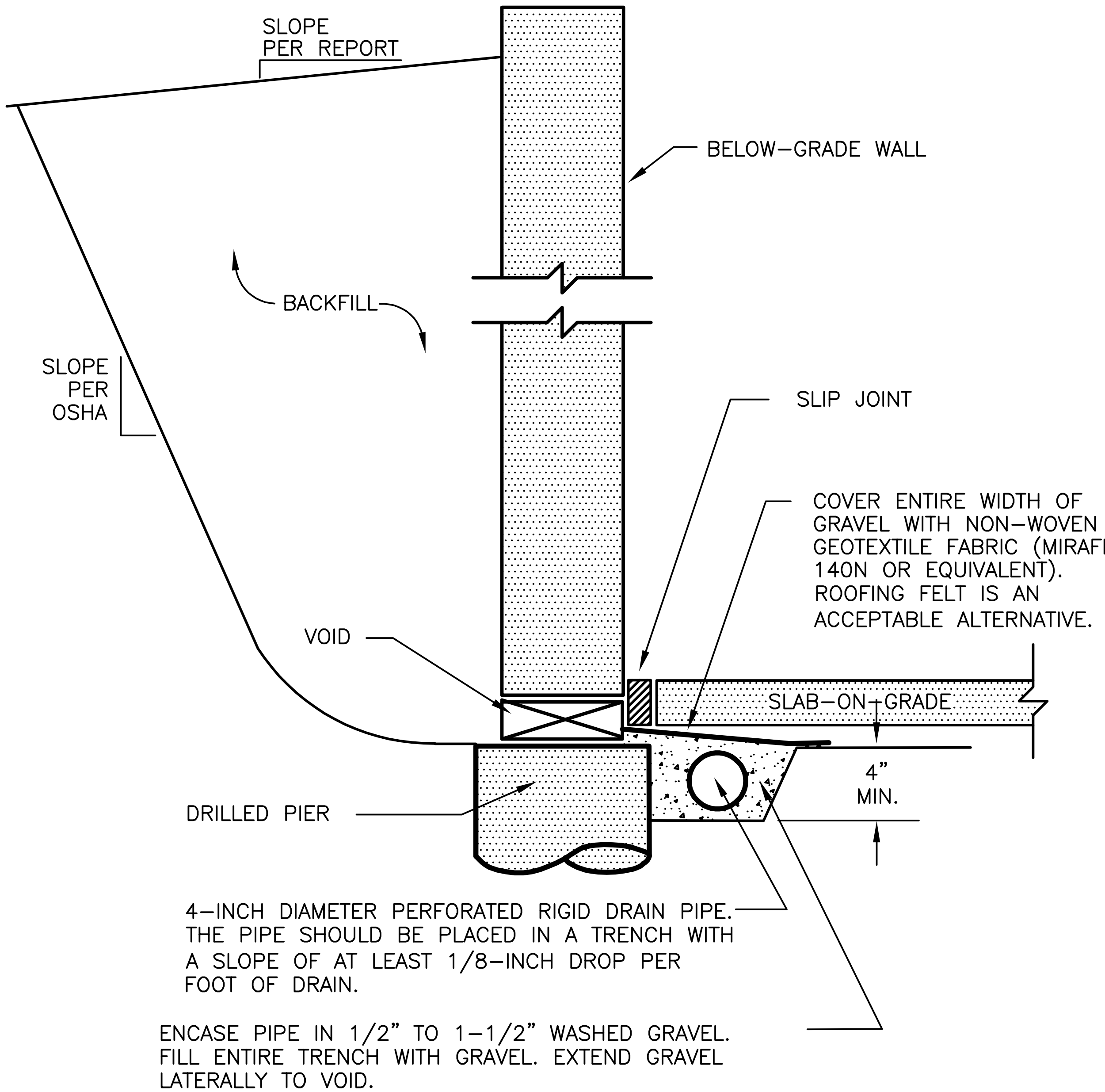
Fig. 5



NOTE:
THE BOTTOM OF THE DRAIN SHOULD BE AT LEAST 2 INCHES BELOW BOTTOM OF VOID AT THE HIGHEST POINT AND SLOPE DOWNWARD TO A POSITIVE GRAVITY OUTLET OR TO A SUMP WHERE WATER CAN BE REMOVED BY PUMPING.

Exterior Foundation Wall Drain

Fig. 6



NOTE:
THE BOTTOM OF THE DRAIN SHOULD BE AT LEAST 4 INCHES BELOW BOTTOM OF VOID AT THE HIGHEST POINT AND SLOPE DOWNWARD TO A POSITIVE GRAVITY OUTLET OR TO A SUMP WHERE WATER CAN BE REMOVED BY PUMPING.

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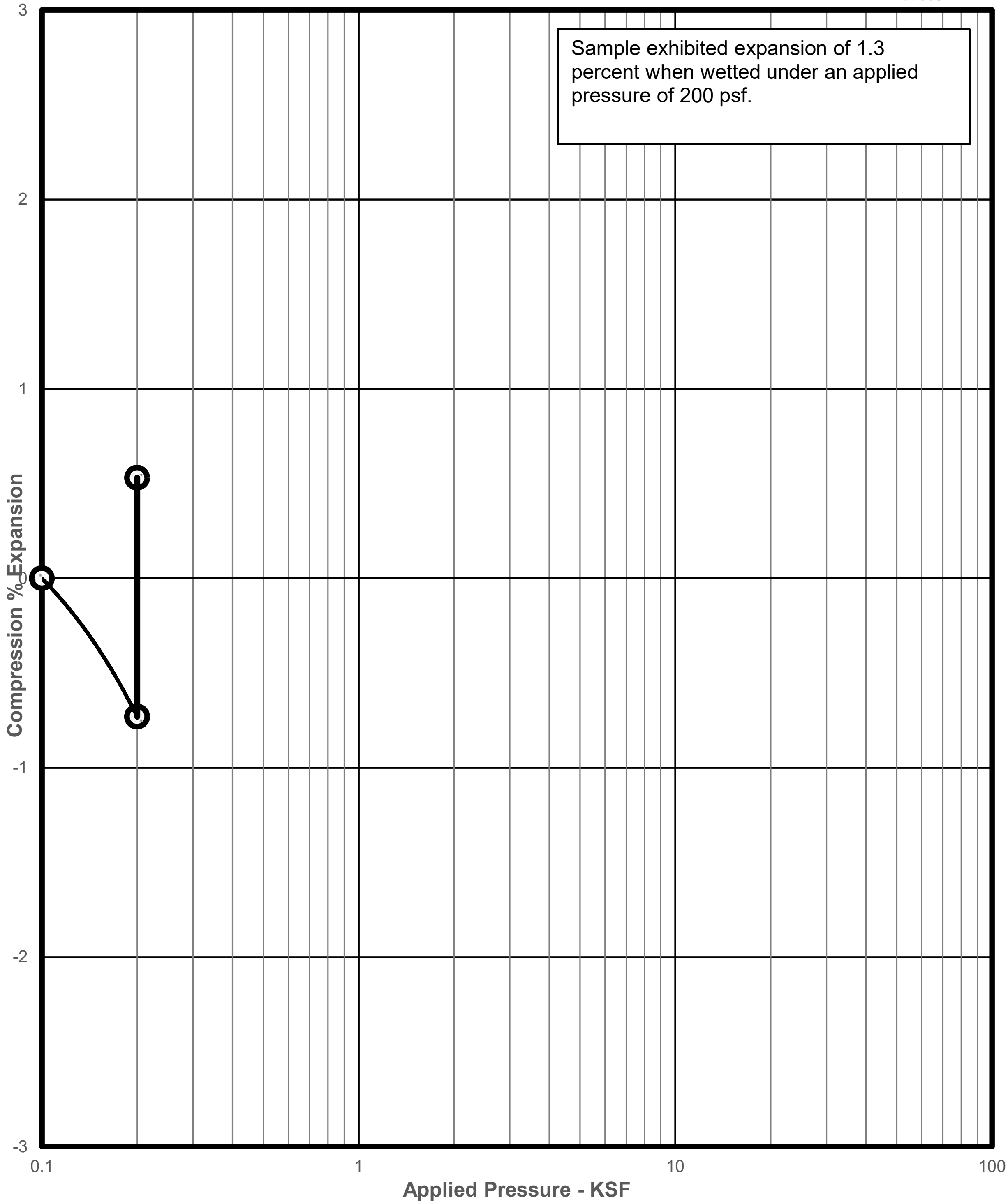
Interior Foundation Wall Drain

Fig. 7

APPENDIX A

LABORATORY TEST RESULTS
TABLE A-I – SUMMARY OF LABORATORY TESTING



SAMPLE OF:	FILL, CLAY, SANDY (CL/CH)	DRY UNIT WEIGHT:	104	pcf
FROM:	TH-2 AT 2 FEET	MOISTURE CONTENT:	19.6	%

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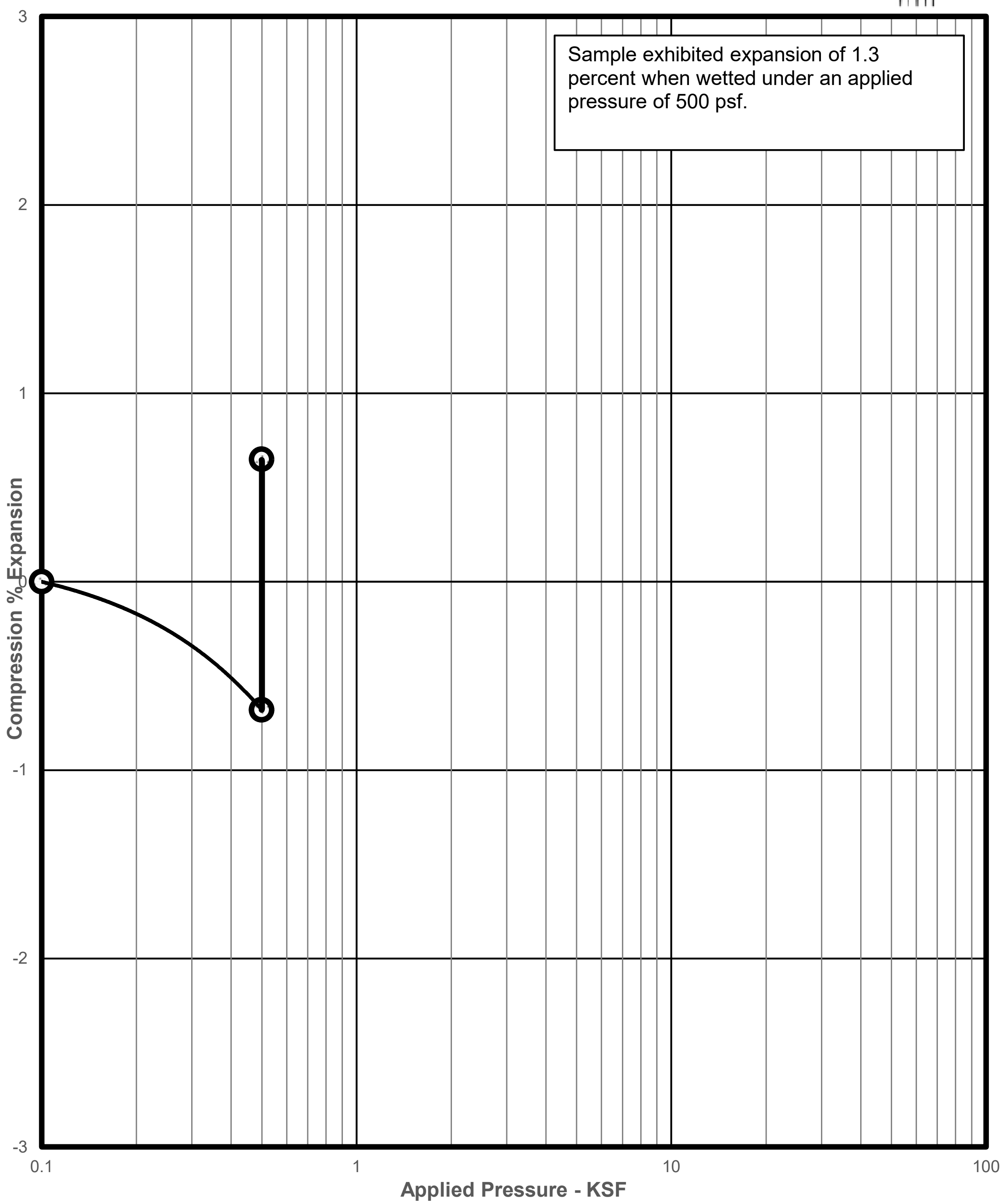
Swell Consolidation Test Results FIG. A- 1



SAMPLE OF:	CLAYSTONE (CL/CH)	DRY UNIT WEIGHT:	109	pcf
FROM:	TH-2 AT 4 FEET	MOISTURE CONTENT:	18.6	%

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Swell Consolidation Test Results FIG. A- 2

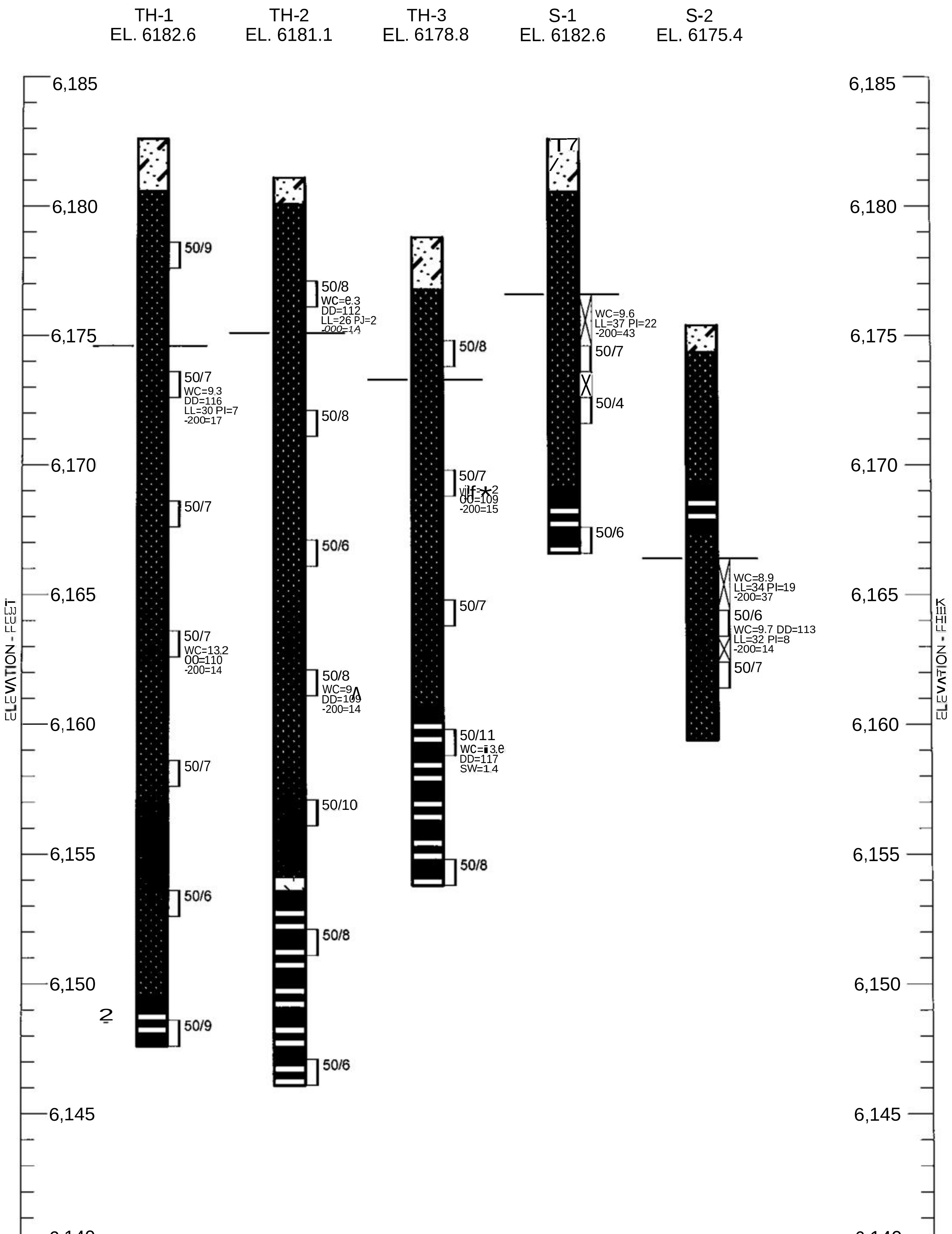
TABLE A - I

SUMMARY OF LABORATORY TEST RESULTS

BORING	DEPTH (ft)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	SWELL TEST DATA SWELL (%)	SWELL TEST DATA APPLIED PRESSURE (psf)	ATTERBERG LIMITS LIQUID LIMIT	ATTERBERG LIMITS PLASTICITY INDEX	PASSING NO. 200 SIEVE (%)	SOIL TYPE
TH-1	2	7.0	115					15	SANDSTONE (SM, SC, SP-SM, SP-SC)
TH-1	4	6.1	118					11	SANDSTONE (SM, SC, SP-SM, SP-SC)
TH-1	14	11.2	117					24	SANDSTONE (SM, SC, SP-SM, SP-SC)
TH-2	2	19.6	104	1.3	200	50	30	51	FILL, CLAY, SANDY (CL/CH)
TH-2	4	18.6	109	1.3	500				CLAYSTONE (CL/CH)
TH-2	9	10.3	118					17	SANDSTONE (SM, SC, SP-SM, SP-SC)
TH-3	2	10.4	113					20	SANDSTONE (SM, SC, SP-SM, SP-SC)
TH-3	9	12.6	117					32	SANDSTONE (SM, SC, SP-SM, SP-SC)
TH-3	19	15.9	104					12	SANDSTONE (SM, SC, SP-SM, SP-SC)

APPENDIX B

2015 SUMMARY LOGS OF EXPLORATORY BORINGS



SUMMARY LOGS OF EXPLORATORY BORINGS

LEGEND:

SAND, CLAYEY, MEDIUM DENSE, MOIST, BROWN, GRAY (SC).

BEDROCK, SANDSTONE, HARD, MOIST, BROWN, TAN.

BEDROCK, CLAYSTONE, HARD, MOIST, BROWN, GRAY.

CEMENTED SANDSTONE, HARD, MOIST, BROWN.

DRIVE SAMPLE. THE SYMBOL 50/9 INDICATES 50 BLOWS OF A 140-POUND HAMMER FALLING 30 INCHES WERE REQUIRED TO DRIVE A 2.5-INCH O.D. SAMPLER 9 INCHES.

BULK SAMPLE.

WATER LEVEL MEASURED AT TIME OF DRILLING.

PROPOSED FINISHED GRADE.

NOTES:

1. THE BORINGS WERE DRILLED ON NOVEMBER 25, 2014 USING 4-INCH DIAMETER, CONTINUOUS-FLIGHT AUGER AND A TRUCK-MOUNTED DRILL RIG.
2. BORING LOCATIONS AND ELEVATIONS WERE DETERMINED BY A REPRESENTATIVE OF OUR FIRM REFERENCING THE TEMPORARY BENCHMARK SHOWN ON FIG. 1.
3. WC - INDICATES MOISTURE CONTENT (%).
   DD - INDICATES DRY DENSITY (PCF).
   SW - INDICATES SWELL 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 (%).
4. THESE LOGS ARE SUBJECT TO THE EXPLANATIONS, LIMITATIONS AND CONCLUSIONS CONTAINED IN THIS REPORT.