



REPORT OF SUBSURFACE EXPLORATION
AND GEOTECHNICAL EVALUATION
606 CENTRAL AVE DEVELOPMENT
BENTONVILLE, ARKANSAS
BUILDING & EARTH PROJECT NO.: BV260045

PREPARED FOR:
Peel Compton Foundation

MAY 27, 2026



Geotechnical, Environmental, and Materials Engineers

May 27, 2026

Peel Compton Foundation
312 N. Main Street
Bentonville, AR 72712

Attention: Ms. Cheryl Humann – Sr. Project Manager

Subject: Report of Subsurface Exploration and Geotechnical Evaluation
606 Central Ave Development
Bentonville, Arkansas
Building & Earth Project No: BV260045

Dear Ms. Humann:

Building & Earth Sciences, LLC (Building & Earth) has completed the authorized subsurface exploration and geotechnical engineering evaluation for the development at 606 East Central Ave in Bentonville, Arkansas, in anticipation of renovations or additions.

The purpose of this exploration and evaluation was to determine general subsurface conditions at the site and to address applicable geotechnical aspects of the proposed construction and site development. The recommendations in this report are based on a physical reconnaissance of the site, conditions observed at the time of the exploration, and observation and classification of samples obtained from twelve (12) test borings conducted at the site. These recommendations are based on the conditions encountered at the time of the exploration. Confirmation of the anticipated subsurface conditions during construction is an essential part of geotechnical services.

We appreciate the opportunity to provide consultation services for the proposed project. If you have any questions regarding the information in this report or need any additional information, please call us.

Respectfully Submitted,

BUILDING & EARTH SCIENCES, LLC

AR Certificate of Authorization No. 569, Expiration Date 12/31/2027



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APPENDIX

1.0 PROJECT & SITE DESCRIPTION

The subject site is located at 606 East Central Avenue in Bentonville, Arkansas. Information relative to the proposed site and the proposed development is listed in Table 1 below. Photographs depicting the current site condition are presented on the following page.

Development Item	Detail	Description
General Site	Size (Ac.)	±8.25
	Existing Development	Vacant house and a few outbuildings, concrete drives, walkways and chain link fence were noted on the site. Underground utilities were marked along the site boundaries include communication, water, and sanitary sewer lines. Overhead electric lines were noted.
	Vegetation	Manicured grass, bushes and multiple trees across the site
	Slopes	This site slopes down to the west with elevation change of about 20 feet and slopes down to the south with an elevation change of about 6 feet.
	Retaining Walls	None noted
	Drainage	The site appears to have fair drainage, with natural surface runoff toward the existing creek along the west side of the site.
	Cuts & Fills	Existing fill was noted in the one test boring IB-01, the boring inside the house.
Proposed Additions	No. of Bldgs	Two (2) Additions (assumed)
	Square Ft.	Unknown
	Stories	Single Story and Basement (assumed)
	Construction	Wood Framing (assumed)
	Column Loads	Maximum of 50 kips (assumed)
	Wall Loads	Maximum of 3 kips per linear foot (klf)
	Preferred Foundation	Conventional Shallow Foundations (assumed)
	Preferred Slab	Slab-on-Grade (assumed)
Proposed pedestrian bridge	No. of Bridges	One (1) (provided)
	Dimensions	25 feet X 12 feet (provided)
	Load at each end of the Bridge	25 kips (provided)
Pavements	Traffic	Not Provided
	Standard Duty	Yes, Rigid and Flexible
	Heavy Duty	Yes, Rigid and Flexible

Table 1: Project and Site Description

References:

- Survey control sheet (C-002) prepared by Crafton Tull dated 05/23/2025
- Basement and Crawl Space New Plans - Sheets 1.02 and 3.03 prepared by Dirtworks undated
- Email from Mr. Kyle Bridges ENFRA Solutions on May 12, 2026

Notes:

1. ***If actual loading conditions exceed our assumed loads, Building & Earth should be allowed to review the proposed structural design and its effects on our recommendations for foundation design.***
2. ***When a grading plan is finalized, Building & Earth should be allowed to review the plan and its effects on our recommendations.***



Figure 1: Northwest side boundary of the site looking east (facing the proposed location of the pedestrian bridge)



Figure 2: Southeast of the existing house looking northwest



Figure 3: Proposed Elevator Location

2.0 SCOPE OF SERVICES

The authorized subsurface exploration was performed on May 5 and 6, 2026 in conformance with our proposal BV28079, dated April 24, 2026. Notice To Proceed was provided by signing and returning our contract on April 24, 2026.

The purpose of the geotechnical exploration was to determine general subsurface conditions at specific boring locations and to gather data on which to base a geotechnical evaluation with respect to the proposed construction. For this exploration, we performed twelve (12) test borings: eleven (11) SPT test borings and one (1) hand auger boring with dynamic cone penetrometer (DCP) inside an existing structure. The SPT borings were drilled using a Geoprobe 7822DT equipped with an automatic hammer for performing Standard Penetration Tests (SPT) to help evaluate the relative soil strength. Refer to the Appendix for a description of the drilling and sampling procedures.

The boring locations were determined in the field by a representative of our staff using a GPS-equipped mobile phone (iPhone 15 Pro). As such, the boring locations shown on the Boring Location Plan attached to this report should be considered approximate.

The soil/rock samples recovered during our site investigation were visually classified and specific samples were selected by the project engineer for laboratory analysis. The laboratory analysis consisted of:

Test	ASTM	No. of Tests
Natural Moisture Content	D2216	42
Atterberg Limits	D4318	5
Material Finer Than No. 200 Sieve by Washing	D1140	5

Table 2: Scope of Laboratory Tests

The results of the laboratory analysis are presented on the enclosed Boring Logs and in tabular form in the Appendix of this report. Descriptions of the laboratory tests that were performed are also included in the Appendix.

The results of the work are presented within this report that addresses:

- General site geology.
- Summary of existing surface conditions.
- A description of the subsurface conditions encountered at the boring locations.
- A description of the groundwater conditions observed in the boreholes during drilling. Long-term monitoring is not included in this proposal.

- Presentation of field and laboratory test results.
- Site preparation considerations including material types to be expected at the site, treatment of any encountered unsuitable soils, excavation considerations, and surface drainage.
- Recommendations to be used for foundation design, including appropriate foundation types, bearing pressures, and depths.
- Presentation of expected total and differential settlements, and recommendations to reduce the expected movements, if appropriate.
- Recommendations to be used for design of slabs-on-grade, including modulus of subgrade reaction.
- Seismic Site Classification per IBC based on SPT test boring information.
- Compaction requirements and recommended criteria to establish suitable material for structural backfill.
- Recommended typical minimum flexible and rigid pavement sections based on assumed traffic loading conditions.

3.0 GEOTECHNICAL SITE CHARACTERIZATION

The following discussion is intended to create a general understanding of the site from a geotechnical engineering perspective. It is not intended to be a discussion of every potential geotechnical issue that may arise, nor to provide every possible interpretation of the conditions identified. It is important to note that the following conditions and subsequent recommendations are based solely on the subsurface conditions observed at the boring locations, and the assumption that significant changes in subsurface conditions do not occur between boreholes. However, anomalous conditions can occur due to existing fill, the geologic conditions at the site, and it will be necessary to evaluate the assumed conditions during site grading and foundation installation. The seasonal weather conditions and rainfall amount prior to and during site grading can have a significant impact on the near surface site conditions.

3.1 GENERAL SITE GEOLOGY

According to the Geologic Map of Arkansas, published by the Arkansas Geological Commission and the United States Geological Survey, the site is underlain by the Boone Formation.

The Boone consists of gray, fine to coarse grained fossiliferous limestone interbedded with chert. Some sections may be predominantly limestone or chert. The cherts tend

to be dark in color in the lower part of the sequence and light in color in the upper part of the section. The quantity of chert varies considerably both vertically and horizontally.

The Boone is well known for dissolutional features such as sinkholes, caves, and enlarged fissures. The thickness of the Boone is 300 to 350 feet in most of northern Arkansas but as much as 390 feet has been reported.

3.2 EXISTING SURFACE CONDITIONS

At the time of our exploration, the site was mainly covered with manicured grass, few bushes, and several trees across the site. Underground utilities were marked along the site boundaries include communication, water, and sanitary sewer lines. Overhead electric lines were noted.

This site slopes to the west with elevation change of about 20 feet and slopes to the south with an elevation change of about 6 feet. The site appears to have fair drainage, with natural surface runoff toward the existing creek along the west side of the site.

Topsoil was encountered at the surface across the site with a thickness of about 3 to 6 inches. Topsoil thicknesses could vary in unexplored areas of the project site. For this report, topsoil is defined as the soil horizon which contains the root mat of the noted light vegetation (grass and weeds). No testing has been performed to verify that soils meet any project requirements of "topsoil".

3.3 SUBSURFACE CONDITIONS

A generalized stratification summary has been prepared using data from the test borings and is presented in the table below. The stratification depicts the general soil/rock conditions and strata encountered in the test borings.

Stratum No.	Description	Consistency/Relative Density
1	Existing Fill Material – Lean Clay (CL)	Very soft to medium stiff, isolated area of hard consistency
2	Silty - Clayey Residuum - Silty Clay (CL-ML), Lean Clay (CL) , Lean to Fat Clay (CL-CH),and Fat Clay (CH)	Soft to medium stiff in the upper 2.5 feet with isolated areas of medium stiff to stiff. Stiff to hard at greater depths with isolated area of medium stiff to stiff
3	Gravelly Residuum - Poorly Graded Gravel (GP), Clayey Gravel (GC)	Medium dense to very dense, isolated area of loose relative density

Table 3: Stratification Summary

Existing fill materials consisting of lower plasticity clays with various amounts of sand and gravel were countered in test boring IB-01 and extended to 1.8 feet below existing grades.

Beneath the surficial topsoil, lean clay residuum containing varying amounts of sand and gravel was encountered in nine of the twelve test borings. These materials were encountered at the following depths: up to 1 foot in B-11; up to 1.5 feet in B-01; up to 2.0 feet in B-02, and B-05; up to 3.0 feet in B-06; up to 4.0 feet in B-08; up to 8.5 feet in B-03; from 2.5 to 5.0 feet in B-04; and from 5.0 to 8.5 feet in B-07.

Sandy Silty clay (CL-ML) residual soils were encountered in test boring B-04 at 5 feet below existing grades and extended up to the termination depth. A layer was also encountered in test boring B-07 at a depth of 2 feet below existing grades and extended up to 5 feet.

Lean to fat clay (CL-CH) residual soils with various amounts of sand and gravel were encountered in in test boring B-07 and B-09 beneath the topsoil and extended up to 2 feet below existing grades.

Fat clay (CH) residual soils with various amounts of sand and gravel were encountered in test borings B-01 and B-02 at a depth of 8.5 feet and extended up to 13.5 feet below existing grades.

Poorly graded gravel with clay and sand (GP-GC) and clayey gravel (GC) residual soils were encountered in 11 of the 12 test borings. These materials were encountered as shallow as immediately beneath the topsoil in test boring B-10 and generally extended to the termination depth of the borings, except in test boring B-04, where they extended only to a depth of 2.5 feet below existing grades.

Subsurface profiles, presented in the Appendix, have also been prepared based on data obtained at specific boring locations. For specific details on the information obtained from individual test borings, refer to the Boring Logs included in the Appendix. The ground surface elevations at the boring locations indicated in this report were estimated based on Google Earth elevation information, and as such should be considered approximate.

3.3.1 DETAILED LAB TESTING RESULTS

The following section details specific lab testing results performed on our test borings at the site. For exact location of lab testing for the collected samples, please refer to the attached Boring Logs

- Lean Clay (CL)
 - Moisture content: 17% to 29%
 - Liquid Limit (LL): 29 to 30, Plasticity Index (PI): 10 to 16
 - Fine content: 64% to 83%
- Silty Lean Clay (CL-ML)

- Moisture content: 21% to 23%
 - LL: 28, PI: 6
 - Fine content: 65%
- Lean to Fat Clay (CL-CH)
 - Moisture content: 15% to 22%
- Fat Clay (CH)
 - Moisture content: 37% to 49%
- Poorly Graded Gravel with Clay and Sand (GP-GC)
 - Moisture content: 7%
 - LL: 24, PI: 1
 - Fine content: 11%
- Clayey Gravel (GC)
 - Moisture content: 9% to 39%

3.4 AUGER REFUSAL

Auger refusal is the drilling depth at which the borehole can no longer be advanced using soil drilling procedures. Auger refusal can occur on boulders, buried debris or rock units. Coring is required to sample the material below auger refusal, which was beyond the approved scope of work for this project. Auger refusal was encountered at the locations and depths below.

Boring No.	Depth (ft)	Boring No.	Depth (ft)
B-01	17.8	B-05	5
B-02	18	B-09	3
B-03	10.2	IB-01 ⁽¹⁾	1.5
B-04	7.3	---	---

Table 4: Approximate Auger Refusal Depth

Note:

1. *Hand auger refusal*

3.5 GROUNDWATER AT BORING LOCATIONS

Groundwater was not encountered at the time of drilling or prior to backfilling during our subsurface exploration. For purposes of this report, "groundwater" refers to water

encountered at the borehole locations. Designation between perched water and a true groundwater table was not included in our scope of work.

Water levels reported are accurate only for the time and date that the borings were drilled. Long term monitoring of the boreholes was not included as part of our subsurface exploration. The borings were backfilled the same day that they were drilled so the stabilized water level may be different than shown.

The prevalent weather conditions prior to construction will influence the amount and depth of water at the borehole locations. Identifying the static depth to the groundwater table verses the presence of perched water was not included in our scope of services.

3.6 SEISMIC SITE CLASSIFICATION

Basis of Evaluation	Recommended Site Classification
2021 International Building Code (IBC) and ASCE 7, Chapter 20	C
This recommended seismic site classification is based on the 2021 Edition of the International Building Code, the subsurface conditions encountered in the borings, and our knowledge of the geologic conditions of the site. Our subsurface exploration extended to a maximum depth of about 18 feet; hence the seismic site classification should be re-evaluated in the event subsurface information is made available to a depth of 100 feet.	

Table 5: Seismic Site Classification

4.0 SITE DEVELOPMENT CONSIDERATIONS

A site grading plan was not available for our review at the time of this report. For the purposes of this report, we assume limited cuts and fills of up to 2 feet will be required to achieve design grades for the proposed building and pavement areas. ***When a grading plan is finalized, Building & Earth should be allowed to review the plan and its effects on our recommendations.***

Our recommendations are based on the subsurface conditions observed during the exploration and do not account for seasonal weather variations that may occur prior to or during construction.

The primary geotechnical concerns for this project are:

- Existing development was noted across the site including, a main house, small buildings, pavements, sidewalks, utilities, and gate structures.
- Existing fill materials were encountered in test boring IB-01 in the upper 1.8 feet below existing grades.
- Low consistency soils were encountered in the upper 2 to 4 feet across portions of the project site. In addition, moisture sensitive lean clays were encountered

near the surface in portions of the project site. These soils are prone to losing strength and stability with increases in soil moisture contents.

- Lean to fat clays and fat clays were encountered across portions of the project site at depths as shallow as beneath the topsoil in test boring B-07 and at 2.5 feet below the existing grades at test boring B-08. These soils pose a high risk for volume change with fluctuations in soil moisture contents.
- Although groundwater was not encountered during our exploration, elevated moisture contents were noted in the soils based on the visual classification and the prevalence of clayey soils encountered across the site indicate a high risk for the development of perched water conditions during and following rainy periods.
- Shallow auger refusal was encountered in the area of boring B-04, B-05, B-09, and IB-01 during our exploration at as shallow as 1.5 feet below the surface depth.

Recommendations addressing the site conditions are presented in the following sections.

4.1 INITIAL SITE PREPARATION

All trees, roots, topsoil and any other deleterious materials should be removed from the proposed construction areas. Approximately 3 to 6 inches of topsoil were observed in the borings. The topsoil thickness is accurate only for the specific boring locations and likely will vary in unexplored areas of the site. Grubbing of trees should include removal of tree stumps and root systems. Desiccated clay soils may be present in the zone surrounding the trees. Desiccated clay soils should be undercut and replaced with structural fill. Removal of desiccated soils is particularly important when they exhibit higher plasticity characteristics, with a high swell potential as was encountered at this site.

The geotechnical engineer or their designated field representative should observe stripping and grubbing operations to evaluate that all unsuitable materials are removed from locations for proposed construction.

Because of current use of the site, buried structures could be encountered such as foundations, basement walls and slabs, utility lines, septic tanks, etc. If encountered, they should be removed and backfilled in accordance with requirements outlined in the Structural Fill section of this report.

Materials disturbed during clearing operations should be stabilized in place or, if necessary, undercut to undisturbed materials and backfilled with properly compacted, approved structural fill.

During site preparation activities, the contractor should identify borrow source materials that will be used as structural fill and provide samples to the testing laboratory so that

conformance to the Structural Fill requirements outlined below and appropriate moisture-density relationship curves can be determined.

4.2 SUBGRADE EVALUATION

We recommend that the project geotechnical engineer, or their representative, assess the subgrade after site preparation. Unstable or unsuitable areas may exist in unexplored portions of the site. All locations requiring fill or structural support should be proofrolled using a loaded tandem-axle dump truck (gross vehicle weight: 20–25 tons) or, in areas with limited access, evaluated through shallow test pits and probings at the appropriate stages:

- After an area has been stripped, and undercut if required, prior to the placement of any fill.
- After grading an area to the finished subgrade elevation in a building or pavement area.
- After areas have been exposed to any precipitation, and/or have been exposed for more than 48 hours.

Some instability may exist during construction, depending on climatic and other factors immediately preceding and during construction. If any soft or otherwise unsuitable soils are identified during the proofrolling process, they should be undercut or stabilized prior to fill placement, or construction of floor slab and pavement. All unsuitable material identified during construction should be removed and replaced in accordance with the *Structural Fill* section of this report.

4.3 MOISTURE SENSITIVE SOILS

Moisture sensitive lean clays (CL), lean to fat clays (CL-CH), silty clays (CL-ML) were encountered near the surface in portions of the project site.

These moisture sensitive soils will degrade if allowed to become saturated. Accordingly, construction during wet weather conditions should be avoided, as this could result in soft and unstable soil conditions that would require ground modification, such as in place stabilization or undercutting.

Due to the moisture sensitive soils at the site and the increased potential of perched water conditions forming on the clayey soils, site grading during historically wet periods should be avoided.

Refer to the *Wet Weather Construction* section of this report for additional considerations.

It is not Building & Earth's responsibility to evaluate expected weather patterns for the time of year construction is planned, as these conditions can significantly affect site

preparation and earthwork activities. This is particularly important where moisture-sensitive soils are present, since periods of rainfall or freezing conditions may adversely impact soil performance, excavation and compaction.

4.4 EVALUATING OF EXISTING FILL MATERIALS

Based on the subsurface conditions encountered and following initial site preparation, very soft to medium stiff fill materials comprising lean clays with gravel were encountered in borings IB-01 (the test boring for the elevator foundation recommendations) to depths up to 1.8 feet. Due to the low consistency of the soils encountered, we recommended the uncontrolled, low consistency fill materials be removed as discussed below.

4.5 UNDERCUTTING OF LOW CONSISTENCY SOILS

Low consistency soils were encountered in the upper 2 to 4 feet across most of the project site during our exploration. Low consistency soils should be undercut to a stable, suitable subgrade. The undercutting should extend laterally 5 feet outside the building footprint.

In the planned pavement areas, the undercutting should extend laterally 3 feet outside of the edge of pavement. It may be possible to stabilize the soft soils in the pavement areas in place. Typical stabilization methods vary widely and include modification of the soft soils with the addition of shot rock or No. 2 stone, as well as utilization of geogrids and graded aggregates. The design of a specific stabilization method is beyond the scope of this investigation but can be provided by Building & Earth as an additional service if desired during construction depending on the conditions encountered at that time. Any undercutting or stabilization performed in pavement areas should be conducted under the observation of the geotechnical engineer or their designated representative.

Some unsuitable or unstable areas may be present in unexplored areas of the site. Once the known undercut is complete, the areas planned for construction should be proofrolled to identify any additional soft soils requiring further removal.

Undercut soils should be replaced with structural fill. Clean, non-organic, non-saturated soils taken from the undercut area can be re-used as structural fill. The placement procedure, compaction and composition of the structural fill should meet the requirements of the *Structural Fill* section of this report.

The undercutting should be conducted under the observation of the geotechnical engineer or their designated representative. *Weather conditions at the time of construction will affect the undercutting depths and quantities.* Some instability may exist during construction, depending on climatic and other factors immediately preceding and during construction.

4.6 UNDERCUTTING OF HIGHLY PLASTIC SOILS

Based on visual classification, lean to fat clays and fat clays (CL-CH & CH) soils are present across portions of the site. These materials were encountered at depths as shallow as the bottom of topsoil. Based on the provided information, the retaining wall will have a height of approximately 8 feet. Fat clay soils (CH) may be encountered at the base of the wall, as Borings B-01 and B-02 indicated fat clay at a depth of 8.5 feet, extending to 13.5 feet below existing grade. We recommend that these soils are maintained at least 3 feet below finished subgrade for the building area and 1 foot below finished subgrade (bottom of base layer) for the pavement areas. A minimum of 1 foot of low plasticity soil should be provided between the bottom of the retaining wall foundation and top of fat clay (CH) layer. The undercutting should extend at least 5 feet horizontally outside the building footprint and 3 feet outside the edge of pavements.

The undercutting should be conducted under the observation of the geotechnical engineer or his representative. *Weather conditions at the time of construction will affect the undercutting depths and quantities.* Some instability may exist during construction, depending on climatic and other factors immediately preceding and during construction.

4.7 STRUCTURAL FILL

Structural fill recommendations for this project are as follows:

Soil Type	USCS Classification	Property Recommendations	Placement Location
Imported Lean Clay, Clayey Sand, Clayey Gravel	CL, SC, GC	LL<40, PI<20, P200>15%, Maximum 3" particle size in any dimension	Low Plasticity Structural Fill to be used for construction of building pads and for support of pavements
Onsite Lean Clay, Clayey Gravel	CL, GC	Same as above	Likely suitable for placement as low plasticity structural fill in building pads and pavement areas ⁵
Onsite Lean to Fat Clay, Fat Clay, Silty Clay	CL-CH, CH, CL-ML	Not Applicable	Not suitable for placement as structural fill due to high plasticity characteristics, reduced gravel content, and high silt content

Table 6: Structural Fill Recommendations

Notes:

1. LL indicates the soil Liquid Limit; PI indicates the soil Plasticity Index; P200 indicates percentage of fine particles passing through a #200 sieve.

2. **Laboratory testing of the soils proposed for fill should be performed to verify their conformance with the above recommendations.**
3. **Any fill to be placed at the site should be reviewed by the geotechnical engineer.**
4. **If onsite residual soils are to be used for structural fill, bulk samples of the material should be provided prior to construction for laboratory analysis to determine suitability**
5. **Material native to the region not meeting the above structural fill criteria may be used if it contains more than 70 percent chert sand and gravel retained on a No. 200 sieve (with maximum particle size of 3 inches in the upper 24 inches of the building pad, and 6 inches elsewhere) and is approved by the geotechnical engineer. Bulk sampling of such material should be provided for, but not necessarily limited to, particle size analysis, Atterberg limits, and moisture-density relationship testing.**

Recommendations for structural fill placement are as follows:

Specification	Recommendation
Lift Thickness	Maximum loose lift thickness of 8 to 12 inches, depending on type of compaction equipment used.
Density	98% of the standard Proctor (ASTM D698) maximum dry density.
Moisture	2% below to 2% above the optimum moisture content as determined by ASTM D698
Density Testing Frequency	Building areas: One test per 2,500 square feet (SF) per lift with a minimum of three tests performed per lift. Pavement areas: One test per 5,000 SF per lift with a minimum of three tests per lift. Utility trenches: One test per 150 linear feet per lift with a minimum of two tests performed per lift. The testing frequency can be increased or decreased by the Geotechnical Engineer of Record in the field based on uniformity of material being placed and compactive effort used.

Table 7: Structural Fill Placement Recommendations

Notes:

1. **All utility trenches should be backfilled and compacted in the manner specified above for structural fill. It may be necessary to reduce the lift thickness to 4 to 6 inches to achieve compaction using hand-operated equipment.**

4.8 EXCAVATION CONSIDERATIONS

All excavations performed at the site should follow OSHA guidelines for temporary excavations. Excavated soils should be stockpiled according to OSHA regulations to limit the potential cave-in of soils.

4.8.1 DIFFICULT EXCAVATION

Shallow auger refusal was encountered in the area of boring B-04, B-05, B-09, and IB-01 during our exploration, as shallow as 1.5 feet below the surface depth. Due to this refusal during drilling and the elevated SPT N-values, there is a high likelihood that chert cobbles, boulders, and seams could be encountered during excavations extended into the clayey gravel residuum at the site. The contractor should anticipate the need for a large track hoe equipped with rock teeth to excavate into the clayey gravel residuum during construction.

4.8.2 TEMPORARY EXCAVATION CONSIDERATIONS

Care should be taken during undercutting operations to not undermine the existing building foundations. The bearing materials of the foundation elements supporting the existing building should be protected during excavations.

The contractor may need to develop a shoring or underpinning plan to protect the bearing materials and shallow footings adjacent to excavations.

4.8.3 GROUNDWATER EFFECT ON GRADING

Groundwater was not encountered at any boring location at the time of our exploration or prior to backfilling of the boring. It should be noted that fluctuations in the water level could occur due to seasonal variations in rainfall, and prevalent weather conditions during site grading. The contractor should be prepared to remove water seepage from excavations if encountered during construction. Excavations extending below water levels will require dewatering systems (such as well points, sump pumps or trench drains). The contractor should evaluate the most economical and practical dewatering method based on the conditions encountered during construction, as the design of permanent or temporary dewatering systems was not included in our scope of services.

4.9 UTILITY TRENCH BACKFILL

Following completion of building pad and pavement subgrade preparation, trench excavations typically will occur for the installation of under-slab and under-pavement utility lines. The performance of a slab-on-grade and pavement is dependent on the subgrade conditions; therefore, it is important that backfill of utility trench excavations be monitored by a representative of Building & Earth. The field monitoring of backfill placement should include verification of material type, lift thickness, compaction, and moisture content for each lift, and follow the recommendations outlined in the *Structural Fill* section of this report.

At the perimeter wall crossings, we recommend that clay soils or a flowable fill be used to backfill the utility trench. The clay or flowable fill will act as a relatively impermeable plug

reducing the risk of water migration from the outside into the interior of the building. The plug should be at least 36 inches wide and should extend below the perimeter walls to provide for a proper seal.

4.10 LANDSCAPING AND DRAINAGE CONSIDERATION

The potential for soil moisture fluctuations within building areas and pavement subgrades should be reduced to lessen the potential of subgrade movement. Site grading should include positive drainage away from buildings and pavements. Excessive irrigation of landscaping poses a risk of saturating and softening soils below shallow footings and pavements, which could result in settlement of footings and premature failure of pavements

4.11 WET WEATHER CONSTRUCTION

Excessive movement of construction equipment across the site during wet weather may result in ruts, which will collect rainwater, prolonging the time required to dry the subgrade soils. Therefore, the grading contractor should maintain positive drainage to avoid surface water ponding and provide temporary dewatering methods as required to help avoid degradation and softening of the soils.

During rainy periods, additional effort will be required to properly prepare the site and establish/maintain an acceptable subgrade. The difficulty will increase in areas where moisture sensitive soils are exposed at the subgrade elevation.

Likewise, rainwater may become perched on the clayey soils encountered across the site, which could require additional dewatering efforts not needed during dry conditions.

A perched water condition occurs when water seeping downward is slowed by a low permeability soil layer, such as the onsite clayey soils and rock units. The perched water level can be any number of feet above the true groundwater level. Due to the prevalence of clayey soils and a rock unit encountered across the project site, the successful contractor should expect to encounter perched water during wet weather construction.

Grading contractors typically postpone grading operations during wet weather to wait for conditions that are more favorable. Contractors can typically disk or aerate the upper soils to promote drying during intermittent periods of favorable weather. When deadlines restrict postponement of grading operations, additional measures such as undercutting and replacing saturated soils or stabilization can be utilized to facilitate placement of additional fill material.

5.0 FOUNDATION RECOMMENDATIONS

5.1 SHALLOW FOUNDATIONS – PROPOSED ADDITIONS

Specific structural loading conditions were not known at the time of this report; however, based on our experience with similar projects, we anticipate that the individual column loads will be less than 50 kips and wall loads will be less than 3 kips per linear foot. ***If these assumptions concerning structural loading are incorrect, our office should be contacted, such that our recommendations can be reviewed.***

Based on the conditions encountered during our field investigation and after our site preparation and grading recommendations are implemented, the proposed structures/building additions can be supported on conventional shallow foundations that bear in properly evaluated and approved existing residuum and/or newly placed and compacted structural fill.

Column footings should be at least 24 inches wide and strip footings should be at least 18 inches wide. These dimensions facilitate hand cleaning of footing subgrades disturbed by the excavation process and the placement of reinforcing steel. They also reduce the potential for localized punching shear failure. All exterior footings should bear at least 24 (varies by freeze depth) inches below the adjacent exterior grade.

Shallow foundations designed and constructed as recommended above can be designed using a maximum net allowable bearing capacity of 2,500 psf.

Total settlement of footings designed and constructed as recommended above is estimated at 1 inch or less. Differential settlement between any two points along a continuous footing or between isolated spread footings spaced 40 feet is estimated at ½-inch.

5.2 SHALLOW FOUNDATION – PROPOSED PEDESTRIAN BRIDGE

Based on the provided information, vertical loads on the order of 25 kips are anticipated at each abutment. ***If the structural loading is incorrect, our office should be contacted, such that our recommendations can be reviewed.***

The borings performed in the vicinity of the bridge abutments, B-04 and B-05, encountered auger refusal at depths of 7.3 feet and 5.2 feet, respectively. As an alternative to deep foundations, the proposed bridge can be supported on conventional shallow foundations designed using an allowable soil bearing capacity of 2,500 psf. Some undercut and replacement of low consistency soils may be required to obtain the recommended bearing capacity. All bridge foundations should bear at least 60 inches below the adjacent exterior grade. Additionally, the face of the bridge foundations should

be at least 10 feet away laterally from the face of existing slopes. Total settlement of footings designed and constructed as recommended above should be 1 inch or less.

It should be noted that scour analysis and scour countermeasure design are beyond the scope of this report. The effects of scour on the bridge foundations should be considered, and scour countermeasures may be appropriate particularly in the bends of the creek bank.

The analysis of lateral/uplift loading was not included in our scope of services. The structural engineer should design the foundations to resist shear and uplift.

5.3 MAT FOUNDATIONS- INTERIOR ELEVATOR

We understand that an interior elevator will be installed within the existing house. Loading information was not available at the time of preparing this report. We assumed that the elevator pad foundation will have loads of up to 20 kips and wall loads will be on the order of 2 kips per linear foot. ***If this assumption is incorrect, our office should be contacted, such that our recommendations can be reviewed.***

Following excavations to foundation bearing elevation, mat or slab-on-grade foundation design based on a recommended modulus of subgrade reaction (k) of 150 may be used. If wall or column foundations are required, a net allowable bearing capacity of 2,500 psf may be used.

5.4 GENERAL FOUNDATION CONSIDERATIONS

The following items should be considered during the preparation of construction documents and foundation installation:

- The geotechnical engineer of record should observe the exposed foundation bearing surfaces prior to concrete placement to verify that the conditions anticipated during the subsurface exploration are encountered.
- Wherever possible, the foundation concrete should be placed "neat", using the sides of the excavations as forms. Where this is not possible, the excavations created by forming the foundations must be backfilled with suitable structural fill and properly compacted.
- Concrete should be placed the same day as the excavations are completed and the bearing materials verified by the engineer. If the excavations are left open for an extended period, or if the bearing surfaces are disturbed after the initial observation, then the bearing surfaces should be re-evaluated prior to concrete placement.

- Water should not be allowed to pond in foundation excavations prior to concrete placement or above the concrete after the foundation is completed.
- Grades around the structure should be sloped to drain away from the foundations.

5.5 DEEP FOUNDATIONS

If loads necessitate the use of deep foundations for support of the abutments, options are presented below. The relative shallow depth to auger refusal could inhibit penetration and capacities, especially uplift.

5.5.1 MICRO PILES

Based on the subsurface conditions noted at the bridge abutments, the encountered lower consistency soils, one option for support of the abutments would be micro-piles. Micro-piles are recommended for ease of construction and to reduce the risk of settlement and foundation erosion by the existing creek system.

Micro-piles are a proprietary ground improvement system developed as a practical alternative to deep foundations and traditional over-excavation and replacement method of strengthening subsoils for settlement control and bearing capacity improvement.

The ground improvement system should be designed and constructed by a licensed company. The installer should provide details design calculations sealed by a professional engineer licensed in the State of Arkansas. Estimates from the design calculations should demonstrate that soil reinforcement from the Micro-pile system will control long-term settlements to less than 1-inch total and ½ inch differential. The design parameters should be verified by a full-scale “load” test performed in the field. Building & Earth should be retained to monitor micro-pile installations.

5.5.2 HELICAL PILES

Helical piles could also prove to be a feasible alternative. This intermediate foundation system utilizes high-capacity helical anchors advanced into the underlying strata by applying torque to the central shaft. As the helical sections extend into the soil, they provide support through end-bearing on the soil plates. The helical piles should be extended to refusal material or the dense clayey chert layer. The design and performance criteria for this system are typically provided by a specialty installation contractor.

5.5.3 DRILLED PIERS

Drilled piers socketed into the rock unit encountered below the residuum could also be considered. Piers need to extend through the overburden soils and any weathered rock into the underlying competent rock unit.

A design decision to be addressed when using rock sockets is whether to neglect one or the other component of resistance (side or base), for the purpose of evaluating the drilled pier capacity. For strong rock, the Federal Highway Administration (FHWA) recommends to use either side friction or end bearing resistance only, but not both. It is our opinion that the use of drilled piers sized for only end bearing is appropriate for the site for the following reasons:

- The drilled pier base will be bearing in rock which is either massive or tightly jointed. The unconfined compressive strength of the dolomite is very high and the rock is mostly continuous. The presence of cavities, voids or soft soil seams can be verified with test probes.
- The conditions are suitable for the preparation of a clean base. This can be verified by visual observation prior to concrete placement.
- The amount of rock removal is a major factor in the drilled pier foundation costs. The use of a high allowable end bearing capacity will reduce the amount of rock removal required for the shafts.

Most of the upper rock surface is varying in a range of 5.2 to 7.4 feet below the surface in test borings B-04 and B-05. Hard rock excavation will be required in order to prepare a level bearing surface for the drilled piers. The amount of rock excavated will vary at any particular location. The amount of rock excavation required to achieve a suitable bearing condition will have a major impact on the foundation construction costs. Based on the information obtained at the boring locations, we anticipate that competent bedrock will be encountered on the order of about 1.5 feet below the upper rock surface (auger refusal). For preliminary budgeting purposes, we recommend that rock excavation quantities averaging twice the pier diameter be used in areas of competent rock. A concrete overage factor of 1.4 times the neat shaft volume should also be used. The table below summarizes the approximate depths where rock surface and competent rock were encountered at each boring location.

Boring Number	Approximate Depth to Rock Surface (ft)	Approximate Depth to Competent Rock (ft)
B-04	7.4	8.9
B-05	5.2	6.7

Table 8: Summary of Subsurface Conditions for Drilled Piers for the Proposed Pedestrian Bridge

Notes:

- 1. The depths provided in this table are measured from the existing ground surface.**
- 2. The depths and elevations of competent rock are based on evaluation of rock cores extracted at the boring locations and may not depict all conditions within the subject site.**

3. The approximate depth to competent rock was assumed to be equal to the pile diameter.

A factor of safety of 3.0 was used for the drilled pier analysis. The following table summarizes the design parameters for drilled piers bearing in continuous dolomite:

Parameter	Value
Minimum Diameter	18 in.
Allowable End Bearing Pressure	10 ksf
Allowable Skin Friction for Uplift Resistance	5 ksf (concrete on rock socket)
Minimum Spacing	3 x Pier Diameter
Group Reduction Factor	None*

Table 9: Design Parameters for Drilled Piers

Note:

1. We understand that pier groups will not be used.

Total settlement of drilled piers designed and constructed as recommended above is anticipated to be ½ inch or less.

A Building & Earth engineer or qualified representative should observe the installation of production piers to check that the installation criteria are implemented. **Test holes must be performed in order to verify rock continuity. A minimum of one 2-inch diameter test hole should be drilled in each pier excavation. The test hole should be drilled in the center of the pier excavation, and should extend a minimum of one pier diameter, or 3 feet, whichever is greater.** We recommend that drilled piers have a minimum diameter of 18 inches to allow for cleaning and reviewing the exposed bearing surface. Loose material and fractured rock should be removed from the bearing surface. Bearing elevations will be adjusted based on actual conditions encountered. Alternative recommendations can be provided if anomalous or unforeseen conditions occur.

Proper installation of drilled piers is essential to the success of the foundation system. In addition to structural considerations, certain safety hazards to personnel exist during construction and observation. The following guidelines should be observed during the installation of drilled piers:

- Prior to shaft entry or inspection:
 - Install metal casing in excavations over 4 feet or where the risk of sidewall collapse is present.
 - Monitor subsurface atmosphere with appropriate gas detection equipment.
 - Outfit personnel with OSHA (or MSHA) approved safety equipment including harness and separate safety line.

- Concrete Placement:
 - Concrete slump of 5 to 8 inches.
 - The concrete may be placed by “free-fall” if the concrete is not allowed to strike the reinforcement or the sides of the temporary casing. Otherwise, tremies or centering chutes should be used.
 - Clean bearing and socket surface of loose debris.
 - Place concrete the same day excavation is completed.
 - Maintain positive head of concrete during casing removal to prevent contamination.
- Should water enter the excavation:
 - Maintain a maximum water level of less than 2 inches deep.
 - Continue dewatering until concrete is placed.
 - Should water infiltration exceed pump limits:
 - Allow water to attain static level.
 - Pump or tremie concrete to bottom of excavation.
 - Maintain a minimum 5 feet hose tremie embedment into the concrete during placement.

The analysis of uplift effects was not included in our scope of services. The length and diameter required to resist uplift loads should be determined by the structural engineer of record, applying the provided skin friction uplift resistance. We understand that the lateral evaluation of the drilled pier will be performed by the structural engineer. Soil parameters to be used for the evaluation of individual pier behavior under lateral loading conditions are presented in the Appendix. The soil parameters were estimated based on visual inspection of the soil and rock samples, laboratory test results, SPT N-values and engineering judgment.

6.0 BELOW GRADE WALL RECOMMENDATIONS

We understand that a basement wall with approximate maximum height of 8-feet is planned at the project site. We recommend that all below grade walls be backfilled with free-draining granular fill such as No. 57 stone (ASTM D 448). The lowest earth pressures are developed when stone fill is placed in the zone defined by projecting a 1(H):1(V) line from the base of the wall to the finished subgrade elevation / ground surface as shown in the schematics presented in this section. If soil is used for backfill the pressures will be

greater and, as a minimum, a drainage blanket (at least 2 feet wide) should be used so that hydrostatic pressure does not develop on the wall as shown in the below. For either case, a layer of suitable filtration fabric, such as Amoco/ProPex 4545 or Mirafi N140 or equivalent, should be placed between the soil and stone backfill to reduce migration of soil fines into the drainage zone behind the wall.

Because the stone fill behind the walls may support slabs and other hardscapes, the fill must be placed and compacted in a systematic manner. The stone fill should be placed in lifts not exceeding six (6) inches and compacted using a vibratory plate compactor. Because density tests cannot be performed in open graded aggregate, the fill placement and compaction should be visually monitored.

The following earth pressures are recommended for design of stem walls and below-grade walls. At rest parameters should be used to design walls that are not allowed to rotate or translate. For retaining walls allowed to move, the active condition can be used. Appropriate surcharge loads must be included in the design.

Backfill Material	Soil Parameters		Equivalent Fluid Weights for Active and At-Rest Pressure (pcf) ¹	
	Wet Unit Weight (pcf)	Effective Angle of Internal Friction (°)	At-Rest Condition	Active Condition
ASTM No. 57 Stone	110	36°	45	29
New Low Plasticity Structural Fill or Existing Residuum	125	28°	66	45

Table 10: Soil Parameters and Earth Pressure Values

Notes:

- 1. Assumes level backfill and no surcharge loading**
- 2. A triaxial test was not performed for determination of soil parameters. Soil parameters were instead estimated based on N-values, our experience with similar soils, and engineering judgment only.**

Lateral earth pressures are greater for walls with free-draining material placed in a zone steeper than the 1(H):1(V) projection recommended above. In that case, we recommend that walls be designed using the soil parameter values for structural fill or residuals soils as shown in the table above.

The recommended lateral earth pressure values are based on a fully drained condition. If hydrostatic pressure is allowed to build up behind walls, additional pressures will develop. The No. 57 aggregate backfill will function as a drainage blanket. The drainage blanket should have a minimum width of 2 feet and should be wrapped in filter fabric to minimize intrusion of fines. A perforated drain line should be installed at the base of wall footings and should extend to a sump where water can be collected and removed, or drains should

discharge by gravity flow to a suitable outfall. Weep holes can be installed in the face of the wall to allow the water to drain, if feasible.

Generalized below grade wall schematics are presented below.

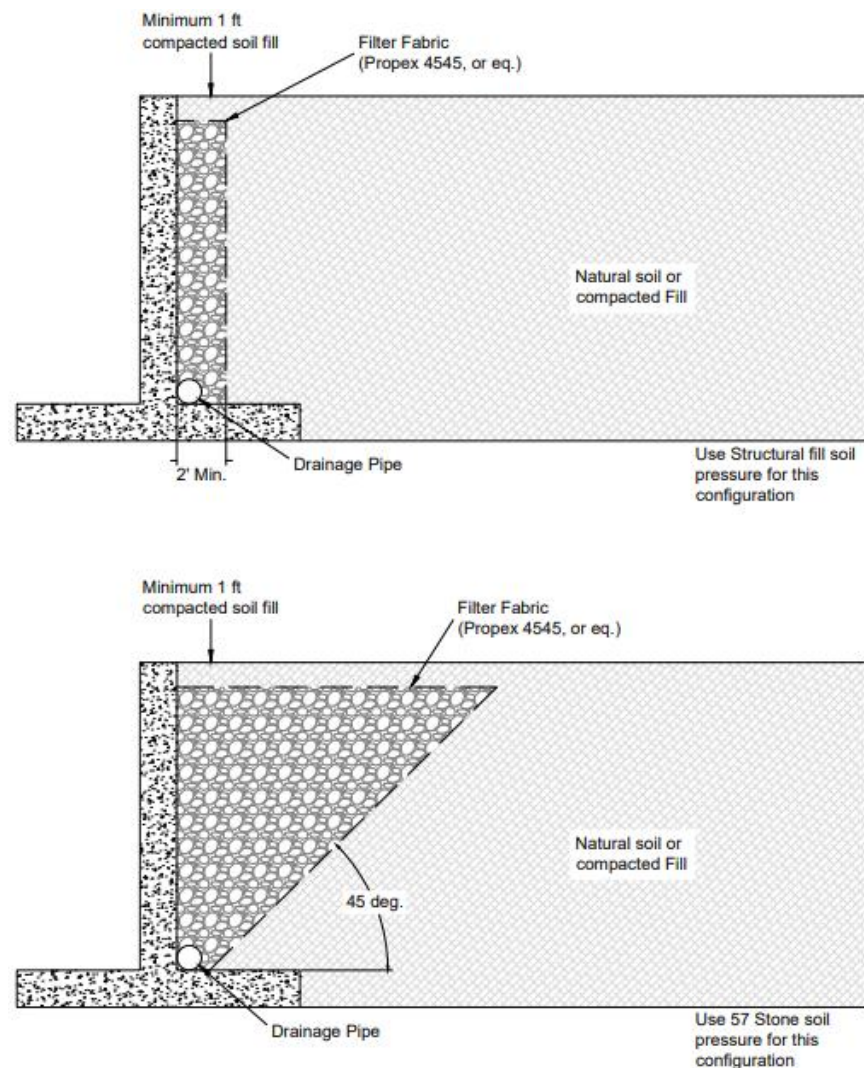


Figure 4: Wall Backfill Details

Lateral pressures arising from surcharge loading, earthquake loading, and groundwater, should be added to the above earth pressures to determine the total lateral pressures. In addition, transient loads imposed on the retaining walls by construction equipment during backfilling should be taken into consideration. Excessively heavy grading equipment (that could impose temporary excessive lateral pressures) should not be allowed within 5 feet (horizontally) of the walls.

Passive earth pressures of materials adjacent to the retaining wall footings as well as bearing material friction at the footing bases may be used to resist shear. The following

table presents recommended allowable friction coefficient values and passive earth pressure values for the anticipated materials.

Material Description	Friction Coefficient	Passive Earth Pressure (pcf)
Residual Soils or Structural Fill/Existing Fill	0.53	346

Table 11: Soil Parameter Values for Resisting Shear

The use of passive resistance requires that the material adjacent to the footing base is undisturbed and should not be removed (for utility installation, excavation, etc.).

7.0 FLOOR SLABS

Site development recommendations presented in this report should be followed to provide for subgrade conditions suitable for support of grade supported slabs. Floor slabs will be supported on approved existing residuum and/or newly placed and compacted structural fill.

We recommend floor slabs for the proposed structure be supported on a minimum four-inch layer of clean, densely-graded granular material commonly referred to as "crusher-run" materials. Alternatively, DOT approved road base with 100% passing the 1-1/2 in sieve, 15% to 55% passing the No. 4 sieve and less than 12% passing the No 200 sieve. The material passing the #200 sieve should be clean, granular fill with less than 3% clay or friable particles. The purpose of this layer is to help provide a uniform loading condition and act as a capillary break for moisture migration through the subgrade soil. This gravel material should be consolidated in-place with vibratory equipment. A modulus of subgrade reaction of 150 pci can be used in the design of a grade-supported building floor slab.

We recommend a minimum 10-mil thick vapor retarder meeting ASTM E 1745, Class C requirements be placed directly below the slab-on-grade floors. A higher quality vapor retarder (Class A or B) may be used if desired to further inhibit the migration of moisture through the slab-on-grade and should be evaluated based on the floor covering and use. The vapor retarder should extend to the edge of the slab-on-grade floors and should be sealed at all seams and penetrations. The slab should be appropriately reinforced (if required) to support the proposed loads.

Where applicable, we recommend that the floor slab be isolated from the foundation footings so differential settlement of the structure will not induce shear stresses on the floor slab. Temperature and shrinkage reinforcements in slabs on grade maybe considered and incorporated accordingly in the slab design. ACI 360R-10 provides guidance on the proper quantity of such reinforcement. The slab should also be appropriately reinforced to support the proposed loads as required. If welded-wire mesh

reinforcement is utilized, the mesh reinforcement should be placed 2 inches below the slab surface or upper one-third of the slab thickness, whichever is closer to the surface. Adequate construction joints, contraction joints and isolation joints should also be provided in the slab to reduce the impacts of cracking and shrinkage, in general accordance with ACI standards and guidelines (ACI 360R-10).

8.0 PAVEMENT CONSIDERATIONS

Specific traffic information was not provided to us at the time of preparing this report. We assume standard duty pavements will be subjected to passenger cars, pick-up trucks, and occasional light delivery box trucks with 18-kip Equivalent Single Axle Loads (ESALs) of 75,000. Heavy duty pavements will be subjected to light delivery box truck and occasional garbage truck traffic with ESALs of 300,000. Based on the materials encountered at the boring locations and after our recommendations for site preparation are implemented, pavements at the subject site may be designed based on a California Bearing Ratio (CBR) of four (4). Note that no CBR or plate load testing was completed to develop these recommendations.

It has been our experience that parking lots experience a certain level of wear and stress greater than roadways designed for similar traffic volumes. Therefore, parking lots are typically designed using the AASHTO method and adjusted based on experience. If the owner would like Building & Earth to assess other likely traffic volumes, we will gladly review other options.

In addition, we have assumed the following design parameters:

Design Criteria	Value
Design life (Years)	20
Initial Serviceability	4.2
Terminal Serviceability	2.0
Reliability	85%
Standard Deviation (Flexible)	0.45
Standard Deviation (Rigid)	0.35

Table 12: Assumed Design Parameters

Material	Specification Section
Portland Cement Concrete Pavement	501
Bituminous Asphalt Wearing Layer	407
Bituminous Asphalt Binder Layer	406
Mineral Aggregate Base Materials	303

Table 13: ArDOT Specification Sections

8.1 FLEXIBLE PAVEMENT

The asphalt pavement sections described herein were designed using the "AASHTO Guide for Design of Pavement Structures, 1993". Alternative pavement sections were designed by establishing the structural numbers used for the AASHTO design system and substituting materials based upon structural equivalency as follows:

Material	Structural Number
Asphalt Concrete	0.44
Crushed Stone Base	0.14

Table 14: Structural Equivalent Coefficient

The following flexible pavement sections are based on the design parameters presented above:

Minimum Recommended Thickness (in)		Material
Standard Duty	Heavy Duty	
3.0	2.0	Surface Course
---	2.5	Binder Course
8.0	8.0	Crushed Stone Base

Table 15: Asphalt Pavement Recommendations

8.2 RIGID PAVEMENT

The following rigid pavement sections are based on the design parameters presented above. We assume an effective modulus of subgrade reaction (k) of 150 pci. We have assumed concrete elastic modulus (E_c) of 3.6×10^6 psi, and a concrete modulus of rupture (S'_c) of 650 psi.

Minimum Recommended Thickness (in)		Material
Standard Duty	Heavy Duty	
5.0	6.0	Portland Cement Concrete, $f'_c=4000$ psi
4.0	4.0	Crushed Stone Base

Table 16: Rigid Pavement Recommendations

The concrete should be protected against moisture loss, rapid temperature fluctuations, and construction traffic for several days after placement. All pavements should be sloped for positive drainage. We recommended that the pavements be reinforced to hold any cracks that might develop tightly together and restrain their growth.

All pavement components and construction operations should be in accordance with the applicable sections of the *Arkansas Department of Transportation (ArDOT) Standard Specifications for Highway Construction, 2014 edition unless otherwise noted in this report.*

8.3 TRAIL PAVING CONSIDERATIONS

We understand that trail pavement will be included in the new development. We recommend that the trail consists of Portland cement concrete with a minimum thickness of 4 inches. The concrete is to be underlain by at least 4 inches of aggregate base material (Class 7 aggregate) compacted to 95% of the material's modified proctor maximum density, and subgrade compacted to at least 95% of the material's standard Proctor maximum density.

All subgrade, base and pavement construction operations should meet minimum requirements of the *Arkansas Department of Transportation (ArDOT) Standard Specifications for Highway Construction, 2014 edition unless otherwise noted in this report.*

9.0 SUBGRADE REHABILITATION

The subgrade soils often become disturbed during the period between initial site grading and construction of surface improvements. The amount and depth of disturbance will vary with soil type, weather conditions, construction traffic, and drainage.

The engineer should evaluate the subgrade soil during final grading to verify that the subgrade is suitable to receive pavement and/or concrete slab base materials. The final evaluation may include proofrolling or density tests.

Subgrade rehabilitation can become a point of controversy when different contractors are responsible for site grading and building construction. The construction documents should specifically state which contractor will be responsible for maintaining and rehabilitating the subgrade. Rehabilitation may include moisture conditioning and re-compacting soils. When deadlines or weather restrict grading operations, additional measures such as undercutting and replacing saturated soils or chemical stabilization can often be utilized.

10.0 CONSTRUCTION MONITORING

Field verification of site conditions is an essential part of the services provided by the geotechnical consultant. In order to confirm our recommendations, it will be necessary

for Building & Earth personnel to make periodic visits to the site during site grading. Typical construction monitoring services are listed below.

- Periodic observations and consultations by a member of our engineering staff during site development.
- Continuous monitoring during structural fill placement.
- Field density test during structural fill placement.
- Observations and verification of the bearing surface exposed after foundation excavation.
- Reinforcing steel inspections.
- Molding and testing of concrete cylinders.
- Continuous monitoring during pavement installation.

11.0 CLOSING AND LIMITATIONS

This report was prepared for Peel Compton Foundation, for specific application to the 606 East Central Development located in Bentonville, Arkansas. The information in this report is not transferable. This report should not be used for a different development on the same property without first being evaluated by the engineer.

The recommendations in this report were based on the information obtained from our field exploration and laboratory analysis. The data collected and presented in this report is representative of the subsurface conditions encountered at the specific boring locations at the time of the exploration. Variations in soil and groundwater conditions may occur between boring locations and throughout other areas of the site. Engineering judgment was applied in regard to conditions between borings. In addition, seasonal or short-term changes in weather patterns, such as rainfall or drought, can further influence subsurface conditions. Evaluation of these potential variations and changes falls outside the scope of this report and should be addressed by the client and contractor during construction.

This report has been prepared in accordance with generally accepted standards of geotechnical engineering practice. No other warranty is expressed or implied. In the event that changes are made, or anticipated to be made, to the nature, design, or location of the project as outlined in this report, Building & Earth must be informed of the changes and given the opportunity to either verify or modify the conclusions of this report in writing, or the recommendations of this report will no longer be valid.

The scope of services for this project did not include any environmental assessment of the site or identification of pollutants or hazardous materials or conditions. If the owner

is concerned about environmental issues Building & Earth would be happy to provide an additional scope of services to address those concerns.

This report is intended for use during design and preparation of specifications and may not address all conditions at the site during construction. Contractors reviewing this information should acknowledge that this document is for design information only and is not a substitute for means, methods, or construction practices, which remain the sole responsibility of the contractor.

An article published by the Geoprofessional Business Association (GBA), titled *Important Information About Your Geotechnical Report*, has been included in the Appendix. We encourage all individuals to become familiar with the article to help manage risk.

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GEOTECHNICAL INVESTIGATION METHODOLOGIES

The subsurface exploration, which is the basis of the recommendations of this report, has been performed in accordance with industry standards. Detailed methodologies employed in the investigation are presented in the following sections.

DRILLING PROCEDURES – STANDARD PENETRATION TEST (ASTM D1586)

At each boring location, soil samples were obtained at standard sampling intervals with a split-spoon sampler. The borehole was first advanced to the sample depth by augering and the sampling tools were placed in the open hole. The sampler was then driven 18 inches into the ground with a 140-pound automatic hammer free-falling 30 inches. The number of blows required to drive the sampler each 6-inch increment was recorded. The initial increment is considered the “seating” blows, where the sampler penetrates loose or disturbed soil in the bottom of the borehole.

The blows required to penetrate the final two (2) increments are added together and are referred to as the Standard Penetration Test (SPT) N-value. The N-value, when properly evaluated, gives an indication of the soil’s strength and ability to support structural loads. Many factors can affect the SPT N-value, so this result cannot be used exclusively to evaluate soil conditions.

The SPT testing was performed using a drill rig equipped with an automatic hammer. Automatic hammers mechanically control the height of the hammer drop, and doing so, deliver higher energy efficiency (90 to 99 % efficiency) than manual hammers (60 % efficiency) which are dropped using a manually operated rope and cathead system. Because historic data correlations were developed based on use of a manual hammer, it is necessary to adjust the N-values obtained using an automatic hammer to make these correlations valid. Therefore, an energy correction factor of 1.3 was applied to the recorded field N-values from the automatic hammer for the purpose of our evaluation. The N-values discussed or mentioned in this report and shown on the boring logs are recorded field values.

Samples retrieved from the boring locations were labeled and stored in plastic bags at the jobsite before being transported to our laboratory for analysis. The project engineer prepared Boring Logs summarizing the subsurface conditions at the boring locations.

HAND AUGER BORINGS AND DYNAMIC CONE PENETRATION TESTING

Hand auger borings were drilled with a 3-inch diameter auger to advance the hole below the existing grade. A Building & Earth representative collected samples of the subsurface soils at regular depth intervals and at depths where a change in lithology occurred.

Dynamic Cone Penetration (DCP) testing was performed in the hand auger borings to evaluate the consistency of the subgrade soils. The DCP apparatus consists of a steel, cylindrical shaft with a conical tip at the end. The conical tip measures 1.5-inches in diameter, with a 45° tip angle. A 15-pound sliding ring weight is mounted to the shaft. When dropped from a height of 20 inches, the ring weight strikes a steel anvil, driving the point into the soil. After seating the point into the soil 2 inches, the weight is dropped

until the shaft travels an interval of 1.75 inches. The number of blows necessary to drive the tip each 1.75-inch increment is recorded. Given the material type and certain soil properties, this number can then be correlated to the Standard Penetration Test (ASTM D1586) N-values. The DCP test results are shown under the "Remarks" column on the boring logs.

BORING LOG DESCRIPTION

Building & Earth Sciences used the gINT software program to prepare the attached boring logs. The gINT program provides the flexibility to custom design the boring logs to include the pertinent information from the subsurface exploration and results of our laboratory analysis. The soil and laboratory information included on our logs is summarized below:

DEPTH AND ELEVATION

The depth below the ground surface and the corresponding elevation are shown in the first two columns.

SAMPLE TYPE

The method used to collect the sample is shown. The typical sampling methods include Split Spoon Sampling, Shelby Tube Sampling, Grab Samples, and Rock Core. A key is provided at the bottom of the log showing the graphic symbol for each sample type.

SAMPLE NUMBER

Each sample collected is numbered sequentially.

BLOWS PER INCREMENT, REC%, RQD%

When Standard Split Spoon sampling is used, the blows required to drive the sampler each 6-inch increment are recorded and shown in column 5. When rock core is obtained the recovery ratio (REC%) and Rock Quality Designation (RQD%) is recorded.

SOIL DATA

Column 6 is a graphic representation of four different soil parameters. Each of the parameters use the same graph, however, the values of the graph subdivisions vary with each parameter. Each parameter presented on column 6 is summarized below:

- **N-value** - The Standard Penetration Test N-value, obtained by adding the number of blows required to drive the sampler the final 12 inches, is recorded. The graph labels range from 0 to 50.
- **Qu** – Unconfined Compressive Strength estimate from the Pocket Penetrometer test in tons per square foot (tsf). The graph labels range from 0 to 5 tsf.
- **Atterberg Limits** – The Atterberg Limits are plotted with the plastic limit to the left, and liquid limit to the right, connected by a horizontal line. The difference in the plastic and liquid limits is referred to as the Plasticity Index. The Atterberg Limits test results are also included in the Remarks column on the far right of the boring log. The Atterberg Limits graph labels range from 0 to 100%.
- **Moisture** – The Natural Moisture Content of the soil sample as determined in our laboratory.

SOIL DESCRIPTION

The soil description were prepared in accordance with ASTM D2488, Visual Description of Soil Samples. Strata changes are indicated by a solid line, with the depth of the change indicated on the left side of the line and the elevation of the change indicated on the right side of the line. If subtle changes within a soil type occur, a broken line is used. The Boring Termination or Auger Refusal depth is shown as a solid line at the bottom of the boring.

GRAPHIC

The graphic representation of the soil type is shown. The graphic used for each soil type is related to the Unified Soil Classification chart. A chart showing the graphic associated with each soil classification is included.

REMARKS

Remarks regarding borehole observations, and additional information regarding the laboratory results and groundwater observations.

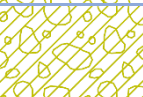
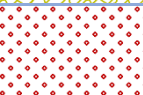
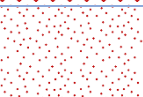
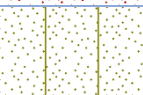
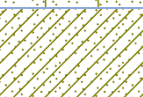
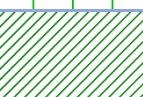
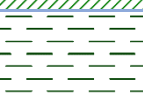
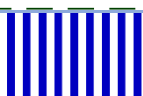
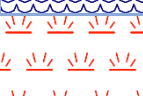
Major Divisions			Symbols		Group Name & Typical Description
			Lithology	Group	
Coarse Grained Soils More than 50% of material is larger than No. 200 sieve size	Gravel and Gravelly Soils More than 50% of coarse fraction is larger than No. 4 sieve	Clean Gravels (Less than 5% fines)		GW	Well-graded gravels, gravel – sand mixtures, little or no fines
				GP	Poorly-graded gravels, gravel – sand mixtures, little or no fines
		Gravels with Fines (More than 12% fines)		GM	Silty gravels, gravel – sand – silt mixtures
				GC	Clayey gravels, gravel – sand – clay mixtures
	Sand and Sandy Soils More than 50% of coarse fraction is smaller than No. 4 sieve	Clean Sands (Less than 5% fines)		SW	Well-graded sands, gravelly sands, little or no fines
				SP	Poorly-graded sands, gravelly sands, little or no fines
		Sands with Fines (More than 12% fines)		SM	Silty sands, sand – silt mixtures
				SC	Clayey sands, sand – clay mixtures
Fine Grained Soils More than 50% of material is smaller than No. 200 sieve size	Silts and Clays Liquid Limit less than 50	Inorganic		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silt with slight plasticity
				CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
		Organic		OL	Organic silts and organic silty clays of low plasticity
	Silts and Clays Liquid Limit greater than 50	Inorganic		MH	Inorganic silts, micaceous or diatomaceous fine sand, or silty soils
				CH	Inorganic clays of high plasticity
		Organic		OH	Organic clays of medium to high plasticity, organic silts
		Highly Organic Soils			

Table 1: Soil Classification Chart (based on ASTM D2487)

Building & Earth Sciences classifies soil in general accordance with the Unified Soil Classification System (USCS) presented in ASTM D2487. Table 1 and Figure 1 exemplify the general guidance of the USCS. Soil consistencies and relative densities are presented in general accordance with Terzaghi, Peck, & Mesri’s (1996) method, as shown on Table 2, when quantitative field and/or laboratory data is available. Table 2 includes Consistency and Relative Density correlations with N-values obtained using either a manual hammer (60 percent efficiency) or automatic hammer (90 percent efficiency). The *Blows Per Increment* and *SPT N-values* displayed on the boring logs are the unaltered values measured in the field. When field and/or laboratory data is not available, we may classify soil in general accordance with the Visual Manual Procedure presented in ASTM D2488.

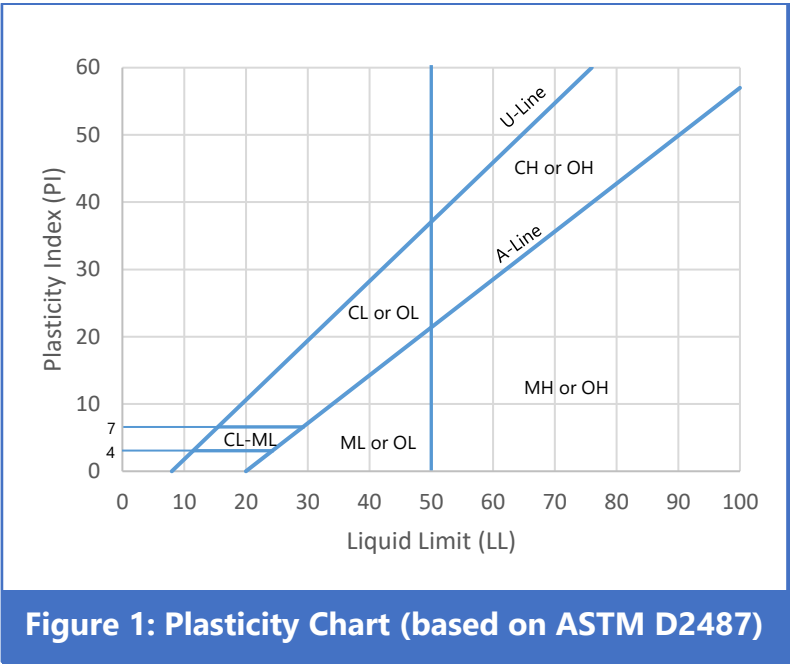


Figure 1: Plasticity Chart (based on ASTM D2487)

Non-cohesive: Coarse-Grained Soil			Cohesive: Fine-Grained Soil			
SPT Penetration (blows/foot)		Relative Density	SPT Penetration (blows/foot)		Consistency	Estimated Range of Unconfined Compressive Strength (tsf)
			Automatic Hammer*	Manual Hammer		
Automatic Hammer*	Manual Hammer		< 2	< 2	Very Soft	< 0.25
0 - 3	0 - 4	Very Loose	2 - 3	2 - 4	Soft	0.25 – 0.50
3 - 7	4 - 10	Loose	3 - 5	4 - 8	Medium Stiff	0.50 – 1.00
7 - 20	10 - 30	Medium Dense	5 - 10	8 - 15	Stiff	1.00 – 2.00
20 - 33	30 - 50	Dense	10 - 20	15 - 30	Very Stiff	2.00 – 4.00
> 33	> 50	Very Dense	> 20	> 30	Hard	> 4.00

Table 2: Soil Consistency and Relative Density (based on Terzaghi, Peck & Mesri, 1996)

* - Modified based on 90% hammer efficiency

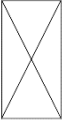
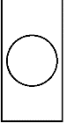
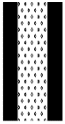
	Standard Penetration Test ASTM D1586 or AASHTO T-206		Dynamic Cone Penetrometer (Sower DCP) ASTM STP-399
	Shelby Tube Sampler ASTM D1587		No Sample Recovery
	Rock Core Sample ASTM D2113		Groundwater at Time of Drilling
	Auger Cuttings		Groundwater as Indicated

Table 3: Symbol Legend

Soil	Particle Size	U.S. Standard
Boulders	Larger than 300 mm	N.A.
Cobbles	300 mm to 75 mm	N.A.
Gravel	75 mm to 4.75 mm	3-inch to #4 sieve
Coarse	75 mm to 19 mm	3-inch to ¾-inch sieve
Fine	19 mm to 4.75 mm	¾-inch to #4 sieve
Sand	4.75 mm to 0.075 mm	#4 to #200 Sieve
Coarse	4.75 mm to 2 mm	#4 to #10 Sieve
Medium	2 mm to 0.425 mm	#10 to #40 Sieve
Fine	0.425 mm to 0.075 mm	#40 to #200 Sieve
Fines	Less than 0.075 mm	Passing #200 Sieve
Silt	0.075 mm to 2 µm	N.A.
Clay	Less than 2 µm	N.A.

Table 4: Standard Sieve Sizes

N-Value 	Standard Penetration Test Resistance calculated using ASTM D1586 or AASHTO T-206. Calculated as sum of original, field recorded values.	Atterberg Limits 	A measure of a soil's plasticity characteristics in general accordance with ASTM D4318. The soil Plasticity Index (PI) is representative of this characteristic and is bracketed by the Liquid Limit (LL) and the Plastic Limit (PL).
Qu 	Unconfined compressive strength, typically estimated from a pocket penetrometer. Results are presented in tons per square foot (tsf).	% Moisture 	Percent natural moisture content in general accordance with ASTM D2216.

Table 5: Soil Data

Hollow Stem Auger	Flights on the outside of the shaft advance soil cuttings to the surface. The hollow stem allows sampling through the middle of the auger flights.
Mud Rotary / Wash Bore	A cutting head advances the boring and discharges a drilling fluid to support the borehole and circulate cuttings to the surface.
Solid Flight Auger	Flights on the outside bring soil cuttings to the surface. Solid stem requires removal from borehole during sampling.
Hand Auger	Cylindrical bucket (typically 3-inch diameter and 8 inches long) attached to a metal rod and turned by human force.

Table 6: Soil Drilling Methods

Descriptor	Meaning
Trace	Likely less than 5%
Few	5 to 10%
Little	15 to 25%
Some	30 to 45%
Mostly	50 to 100%

Table 7: Descriptors

Manual Hammer	The operator tightens and loosens the rope around a rotating drum assembly to lift and drop a sliding, 140-pound hammer falling 30 inches.
Automatic Trip Hammer	An automatic mechanism is used to lift and drop a sliding, 140-pound hammer falling 30 inches.
Dynamic Cone Penetrometer (Sower DCP) ASTM STP-399	Uses a 15-pound steel mass falling 20 inches to strike an anvil and cause penetration of a 1.5-inch diameter cone seated in the bottom of a hand augered borehole. The blows required to drive the embedded cone a depth of 1-3/4 inches have been correlated by others to N-values derived from the Standard Penetration Test (SPT).

Table 8: Sampling Methods

Non-plastic	A 1/8-inch thread cannot be rolled at any water content.
Low	The thread can barely be rolled and the lump cannot be formed when drier than the plastic limit.
Medium	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be re-rolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit.
High	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be re-rolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit.

Table 9: Plasticity

Dry	Absence of moisture, dusty, dry to the touch.
Moist	Damp but no visible water.
Wet	Visible free water, usually soil is below water table.

Table 10: Moisture Condition

Stratified	Alternating layers of varying material or color with layers at least 1/2 inch thick.
Laminated	Alternating layers of varying material or color with layers less than 1/4 inch thick.
Fissured	Breaks along definite planes of fracture with little resistance to fracturing.
Slickensides	Fracture planes appear polished or glossy, sometimes striated.
Blocky	Cohesive soil that can be broken down into small angular lumps which resist further breakdown.
Lensed	Inclusion of small pockets of different soils, such as small lenses of sand scattered through a mass of clay.
Homogeneous	Same color and appearance throughout.

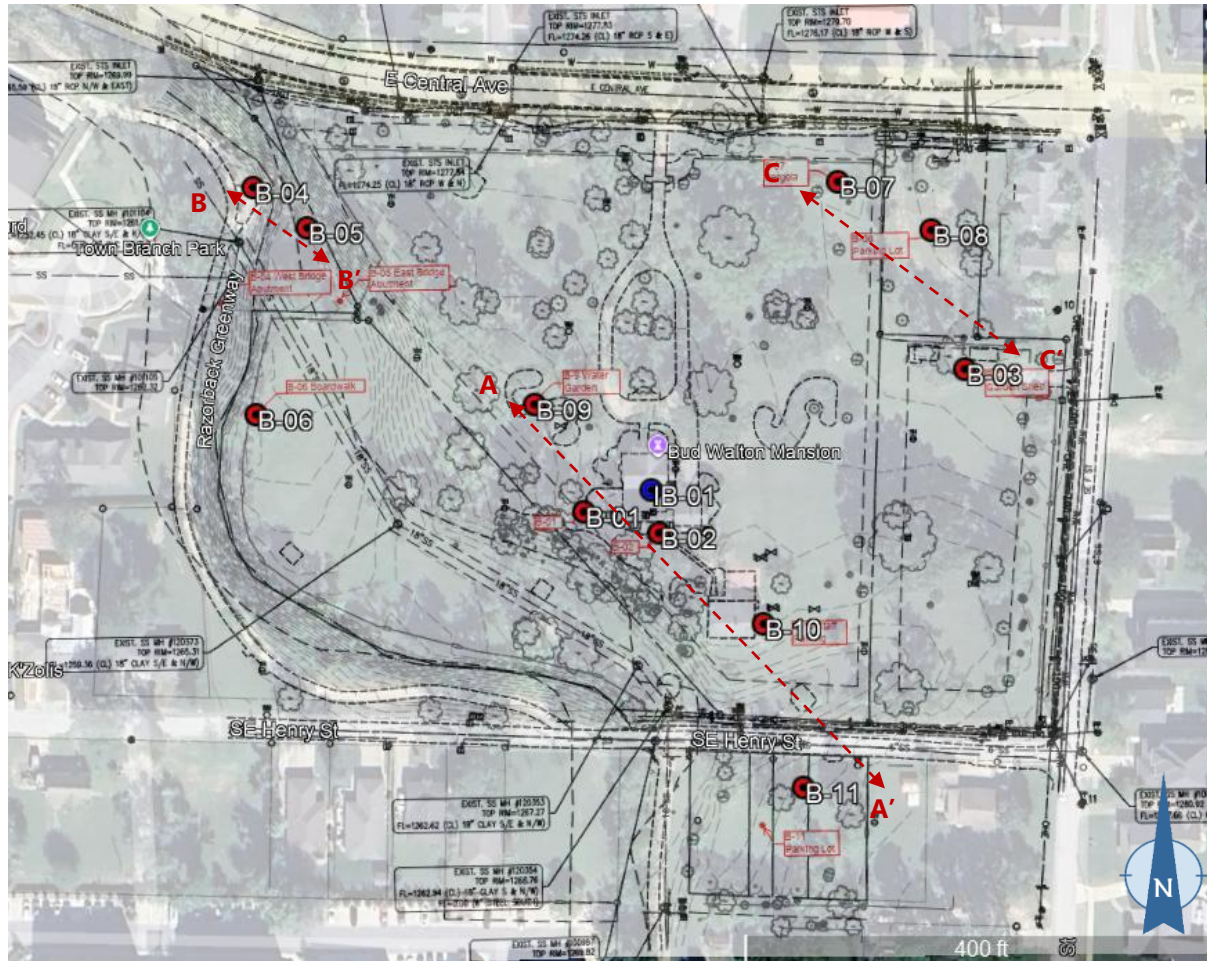
Table 11: Structure

KEY TO HATCHES

Hatch	Description	Hatch	Description	Hatch	Description
	GW - Well-graded gravels, gravel – sand mixtures, little or no fines		Asphalt		Clay with Gravel
	GP - Poorly-graded gravels, gravel – sand mixtures, little or no fines		Aggregate Base		Sand with Gravel
	GM - Silty gravels, gravel – sand – silt mixtures		Topsoil		Silt with Gravel
	GC - Clayey gravels, gravel – sand – clay mixtures		Concrete		Gravel with Sand
	SW - Well-graded sands, gravelly sands, little or no fines		Coal		Gravel with Clay
	SP - Poorly-graded sands, gravelly sands, little or no fines		CL-ML - Silty Clay		Gravel with Silt
	SM - Silty sands, sand – silt mixtures		Sandy Clay		Limestone
	SC - Clayey sands, sand – clay mixtures		Clayey Chert		Chalk
	ML - Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silt with slight plasticity		Low and High Plasticity Clay		Siltstone
	CL - Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays		Low Plasticity Silt and Clay		Till
	OL - Organic silts and organic silty clays of low plasticity		High Plasticity Silt and Clay		Sandy Clay with Cobbles and Boulders
	MH - Inorganic silts, micaceous or diatomaceous fine sand, or silty soils		Fill		Sandstone with Shale
	CH - Inorganic clays of high plasticity		Weathered Rock		Coral
	OH - Organic clays of medium to high plasticity, organic silts		Sandstone		Boulders and Cobbles
	PT - Peat, humus, swamp soils with high organic contents		Shale		Soil and Weathered Rock

Table 12: Key to Hatches Used for Boring Logs and Soil Profiles

BORING LOCATION PLAN



**REFERENCE USED
TO PRODUCE THIS
DRAWING:**

Google Earth Satellite
Imagery dated
10/04/2025 with overlay
of provided survey control
sheet dated 05/23/2025

BORING LOCATION PLAN

PROJECT NO.

BV260045

PROJECT NAME / LOCATION:

606 Central Avenue
Bentonville, Arkansas

DATE: 05/06/2026

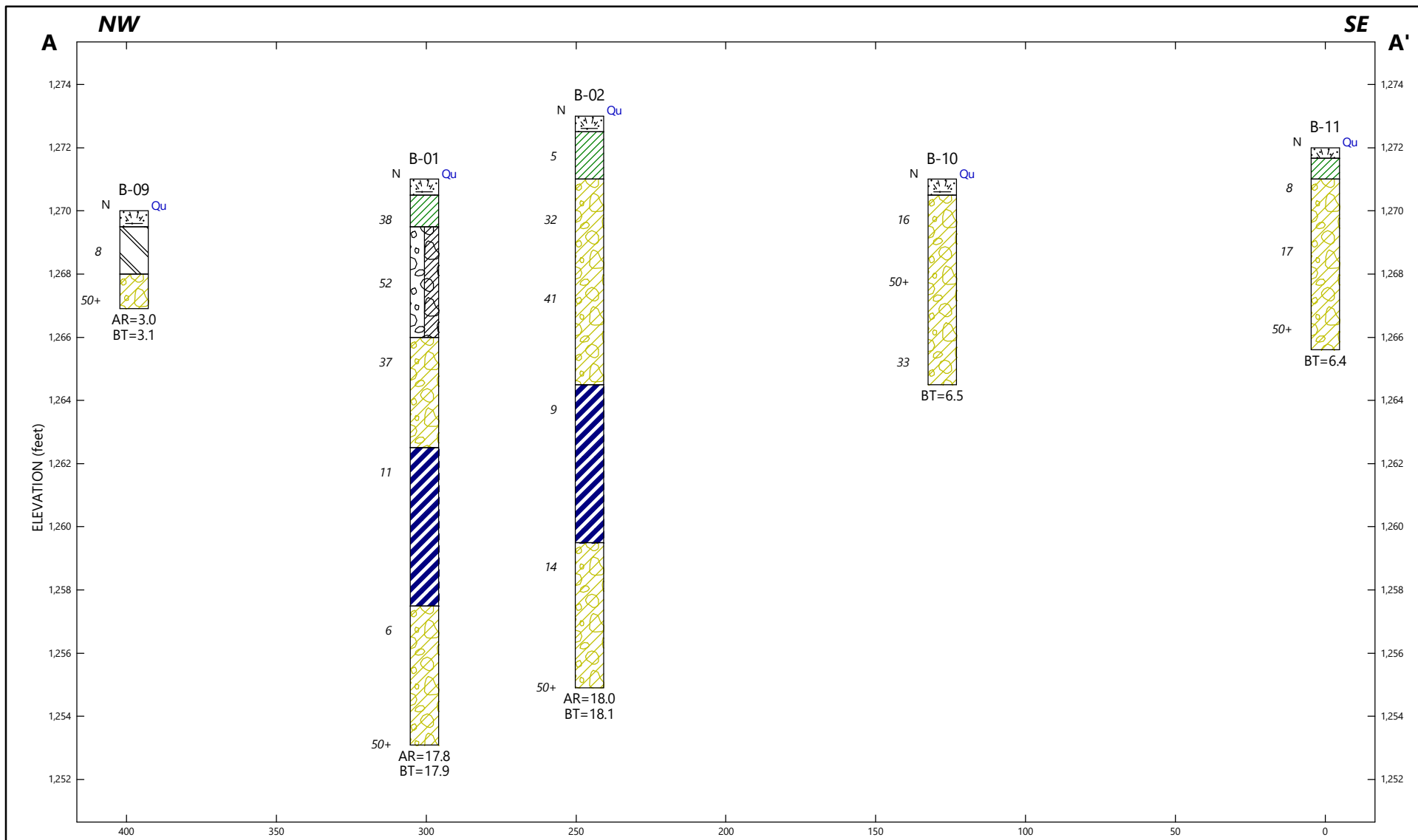
SCALE:

As Shown

BUILDING & EARTH

Geotechnical, Environmental, and Materials Engineers

SUBSURFACE SOIL PROFILES



Site Map Scale 1 inch equals 290 feet

Key to Hatches

Topsoil	USCS Low Plasticity Clay	USCS Poorly-graded Gravel with Clay
USCS Clayey Gravel	USCS High Plasticity Clay	USCS Low to High Plasticity Clay

Legend

BT=Boring Termination, TPT=Test Pit Terminated
 AR=Auger Refusal, ER=Excavation Refusal
 N=Standard Penetration Test N-Value
 Qu=Unconfined compressive strength estimate from pocket penetrometer test (tsf)

Water Level Reading at time of drilling.
 Water Level Reading after drilling.

Horizontal Scale (feet)

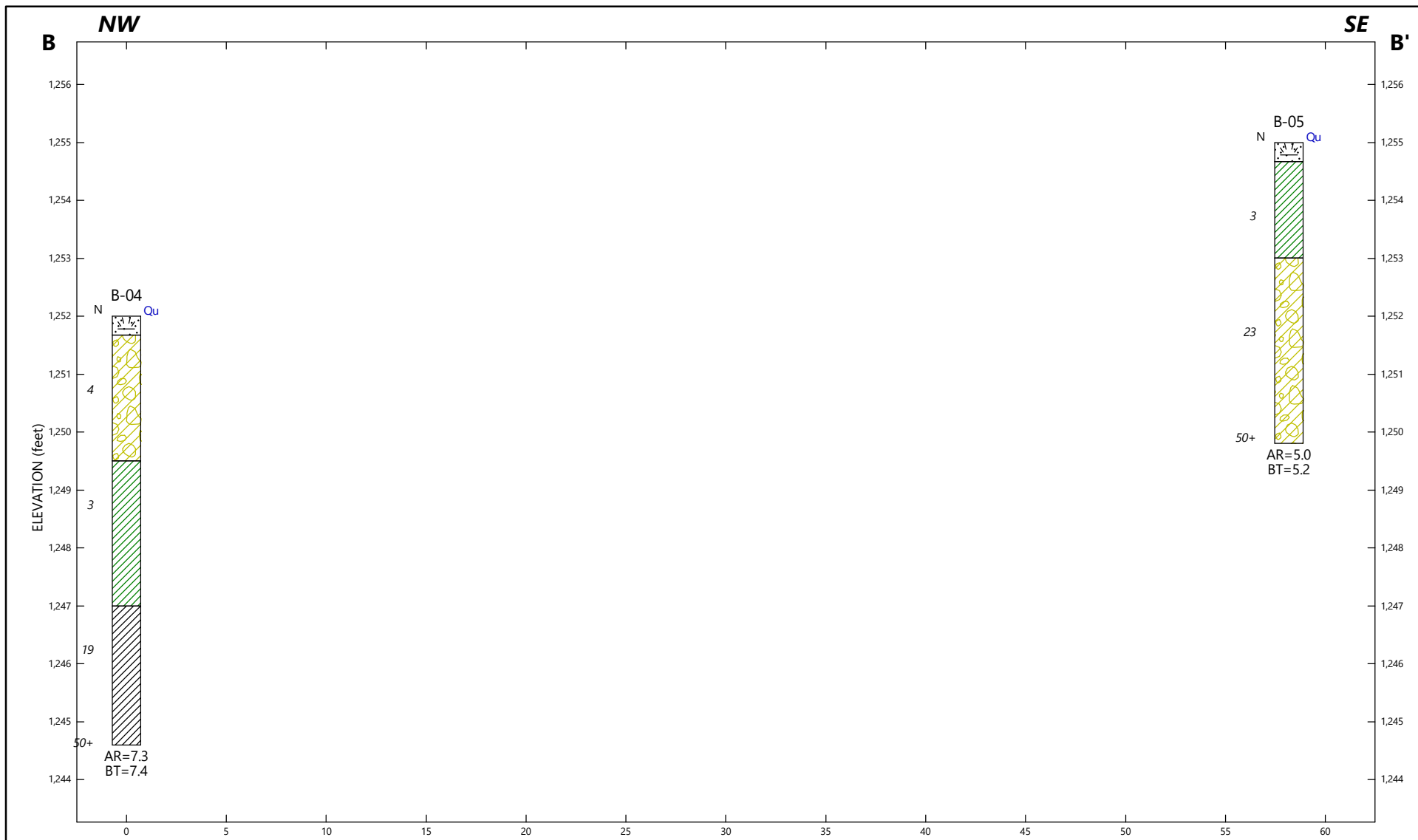
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Building & Earth Sciences, LLC
 3165 Pine Woods Road, Springdale, AR
 606 Central Ave Project
 Bentonville, AR

A-A': Subsurface Profile

PROJECT NO: BV260045	PLATE NO: A'	DATE: 5/27/26
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 Geotechnical, Environmental, and Materials Engineers



Site Map Scale 1 inch equals 45 feet

Key to Hatches

	Topsoil		USCS Clayey Gravel		USCS Low Plasticity Clay
	USCS Low Plasticity Silty Clay				

Legend

BT=Boring Termination, TPT=Test Pit Terminated
AR=Auger Refusal, ER=Excavation Refusal
N=Standard Penetration Test N-Value
Qu=Unconfined compressive strength estimate from pocket penetrometer test (tsf)

Water Level Reading at time of drilling.
 Water Level Reading after drilling.

Horizontal Scale (feet)
Vertical Exaggeration: 3x

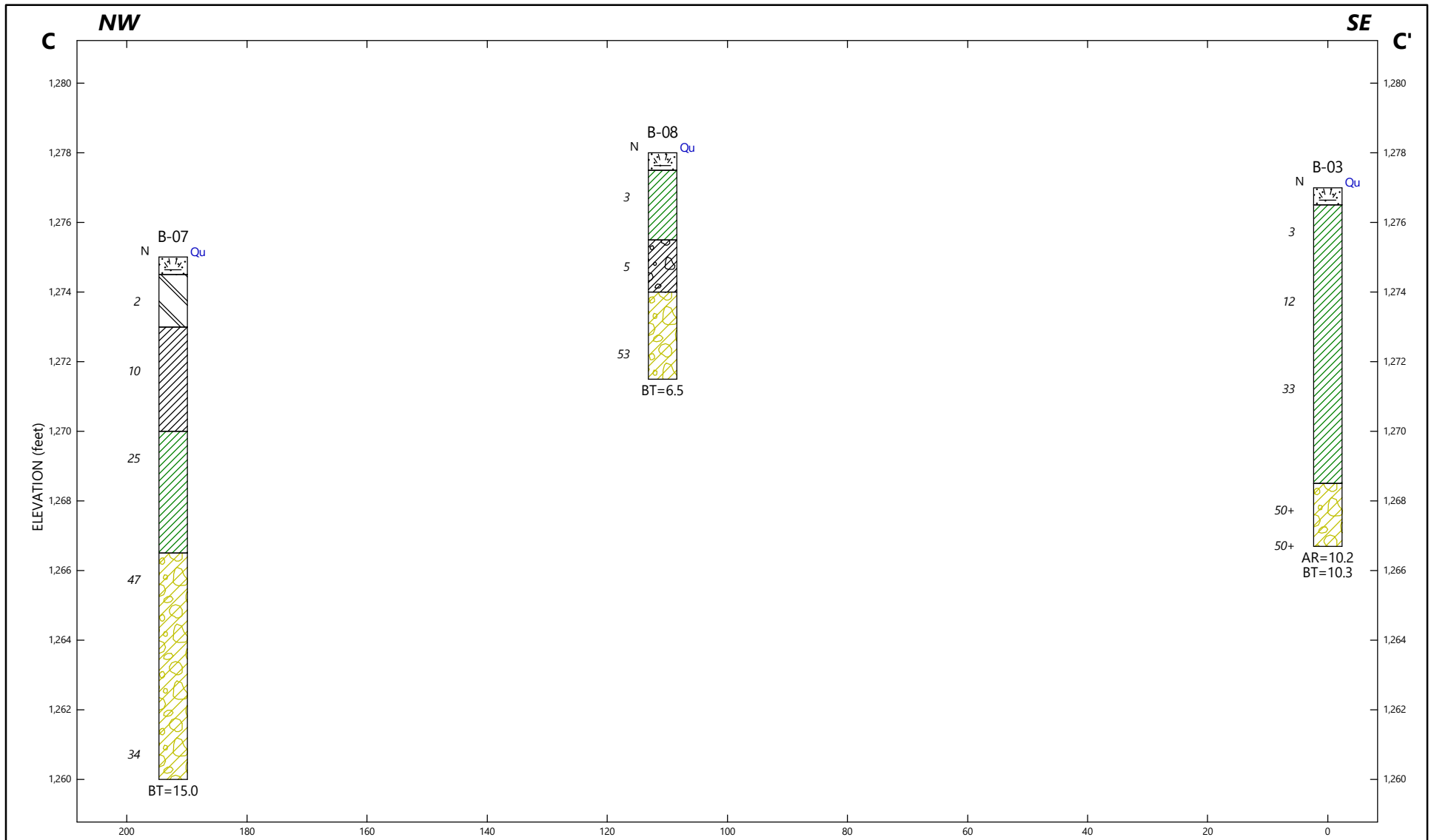
Building & Earth Sciences, LLC
3165 Pine Woods Road, Springdale, AR

606 Central Ave Project
Bentonville, AR

B-B': Subsurface Profile

PROJECT NO: BV260045	PLATE NO: B'	DATE: 5/27/26
----------------------	--------------	---------------

BUILDING & EARTH
Geotechnical, Environmental, and Materials Engineers



Site Map Scale 1 inch equals 145 feet

Key to Hatches

Topsoil	USCS Low Plasticity Clay	USCS Clayey Gravel
USCS Low to High Plasticity Clay	USCS Low Plasticity Silty Clay	USCS Low Plasticity Gravelly Clay

Legend

BT=Boring Termination, TPT=Test Pit Terminated
 AR=Auger Refusal, ER=Excavation Refusal
 N=Standard Penetration Test N-Value
 Qu=Unconfined compressive strength estimate from pocket penetrometer test (tsf)

Water Level Reading at time of drilling.
 Water Level Reading after drilling.

Horizontal Scale (feet)
Vertical Exaggeration: 6x

Building & Earth Sciences, LLC

3165 Pine Woods Road, Springdale, AR

606 Central Ave Project
Bentonville, AR

C-C': Subsurface Profile

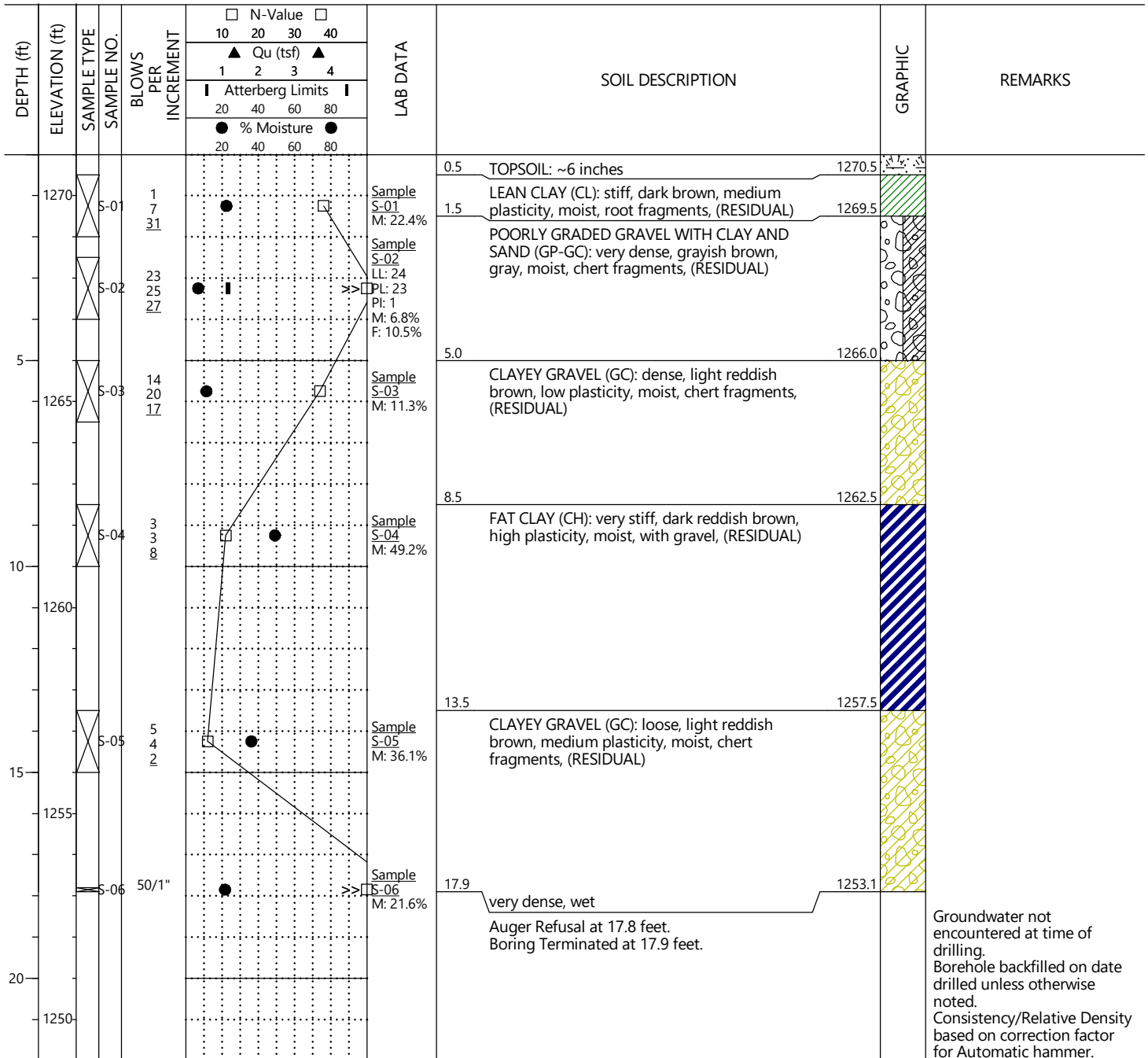
PROJECT NO: BV260045	PLATE NO: C'	DATE: 5/27/26
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BUILDING & EARTH
Geotechnical, Environmental, and Materials Engineers

BORING LOGS

PROJECT NAME: 606 Central Ave Project
PROJECT NUMBER: BV260045
DRILLING METHOD: Hollow Stem Auger
EQUIPMENT USED: Geoprobe 7822DT
HAMMER TYPE: Automatic
BORING LOCATION: West of Proposed Building

LOCATION: Bentonville, AR
DATE DRILLED: 5/5/26
WEATHER: Cloudy
ELEVATION: 1271
DRILL CREW: Building & Earth
LOGGED BY: D.Rios



SAMPLE TYPE ☒ Split Spoon

N-VALUE STANDARD PENETRATION RESISTANCE (AASHTO T-206)

% MOISTURE PERCENT NATURAL MOISTURE CONTENT

☒ GROUNDWATER LEVEL IN THE BOREHOLE AT TIME OF DRILLING

☒ STABILIZED GROUNDWATER LEVEL

REC RECOVERY

RQD ROCK QUALITY DESIGNATION

UD UNDISTURBED

Qu POCKET PENETROMETER UNCONFINED COMPRESSIVE STRENGTH

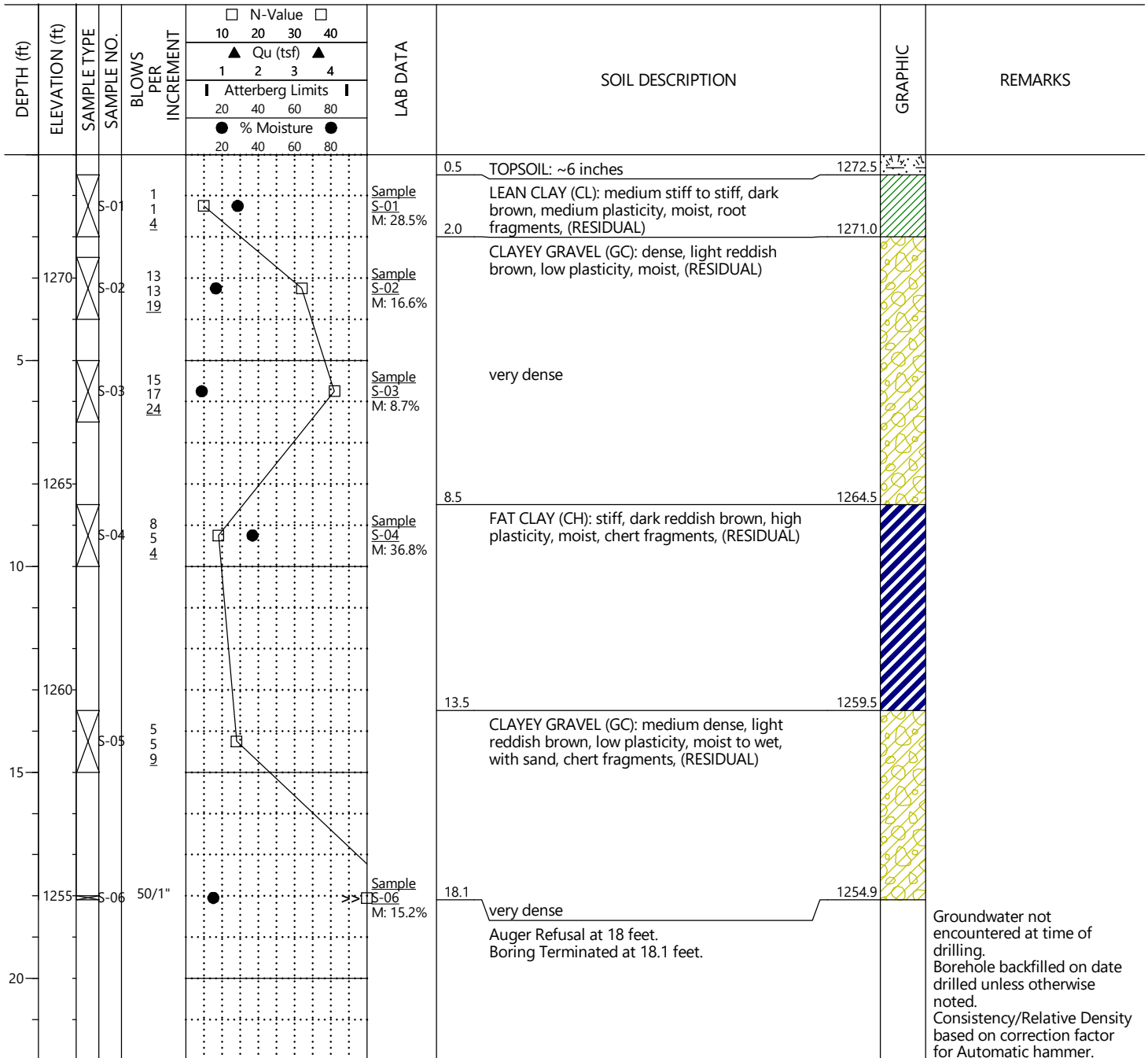
LL: LIQUID LIMIT **M:** NATURAL MOISTURE CONTENT

PL: PLASTIC LIMIT **F:** PERCENT PASSING NO. 200 SIEVE

PI: PLASTICITY INDEX

PROJECT NAME: 606 Central Ave Project
PROJECT NUMBER: BV260045
DRILLING METHOD: Hollow Stem Auger
EQUIPMENT USED: Geoprobe 7822DT
HAMMER TYPE: Automatic
BORING LOCATION: East of Proposed Building

LOCATION: Bentonville, AR
DATE DRILLED: 5/6/26
WEATHER: Cloudy
ELEVATION: 1273
DRILL CREW: Building & Earth
LOGGED BY: D.Rios



SAMPLE TYPE Split Spoon

N-VALUE STANDARD PENETRATION RESISTANCE (AASHTO T-206)

% MOISTURE PERCENT NATURAL MOISTURE CONTENT

GROUNDWATER LEVEL IN THE BOREHOLE AT TIME OF DRILLING

STABILIZED GROUNDWATER LEVEL

REC RECOVERY

RQD ROCK QUALITY DESIGNATION

UD UNDISTURBED

Qu POCKET PENETROMETER UNCONFINED COMPRESSIVE STRENGTH

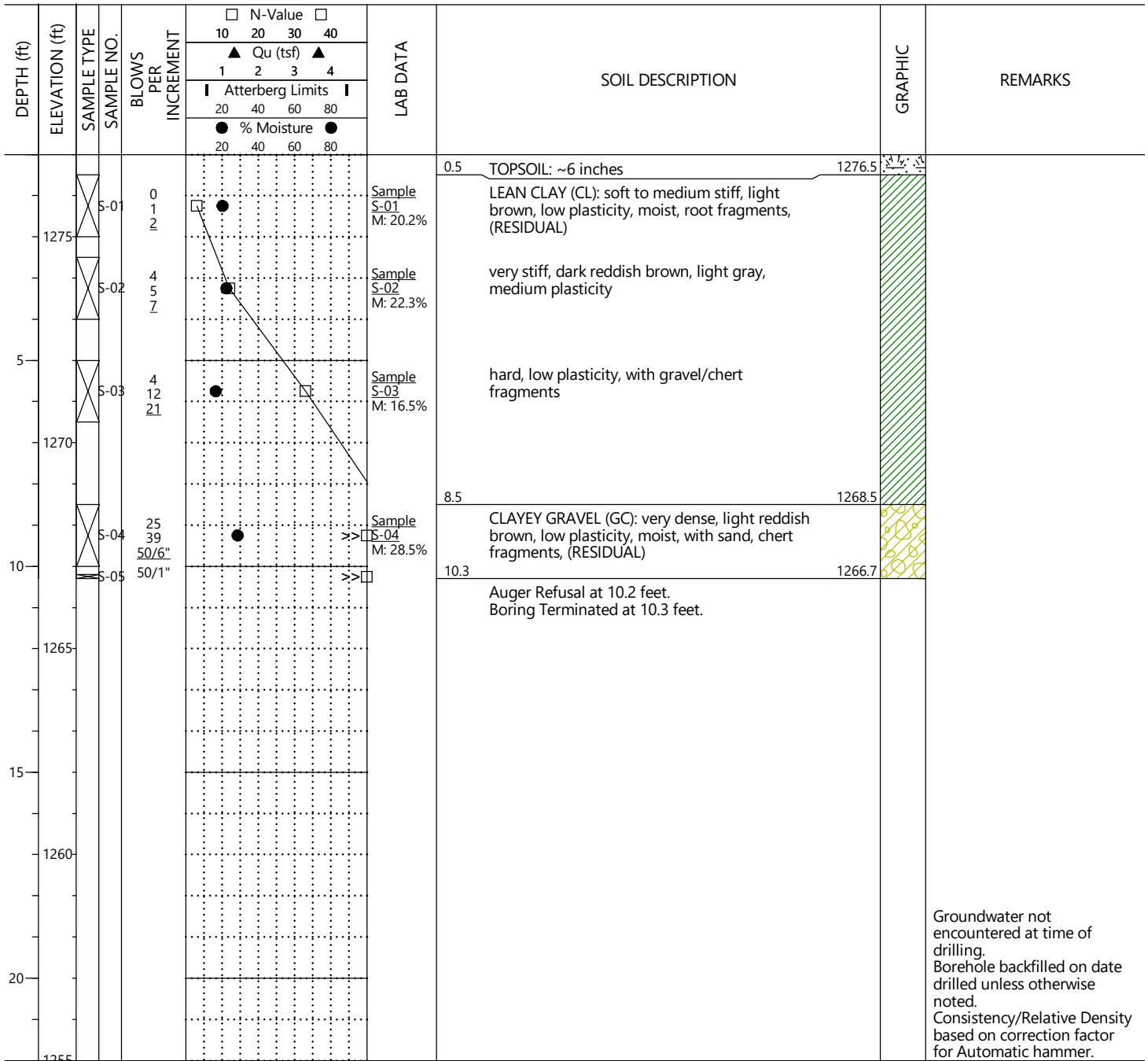
LL: LIQUID LIMIT **M:** NATURAL MOISTURE CONTENT

PL: PLASTIC LIMIT **F:** PERCENT PASSING NO. 200 SIEVE

PI: PLASTICITY INDEX

PROJECT NAME: 606 Central Ave Project
PROJECT NUMBER: BV260045
DRILLING METHOD: Hollow Stem Auger
EQUIPMENT USED: Geoprobe 7822DT
HAMMER TYPE: Automatic
BORING LOCATION: Garden Shed

LOCATION: Bentonville, AR
DATE DRILLED: 5/6/26
WEATHER: Cloudy
ELEVATION: 1277
DRILL CREW: Building & Earth
LOGGED BY: D.Rios



SAMPLE TYPE ☒ Split Spoon

N-VALUE STANDARD PENETRATION RESISTANCE (AASHTO T-206)

% MOISTURE PERCENT NATURAL MOISTURE CONTENT

☒ GROUNDWATER LEVEL IN THE BOREHOLE AT TIME OF DRILLING

☒ STABILIZED GROUNDWATER LEVEL

REC RECOVERY

RQD ROCK QUALITY DESIGNATION

UD UNDISTURBED

Qu POCKET PENETROMETER UNCONFINED COMPRESSIVE STRENGTH

LL: LIQUID LIMIT **M:** NATURAL MOISTURE CONTENT

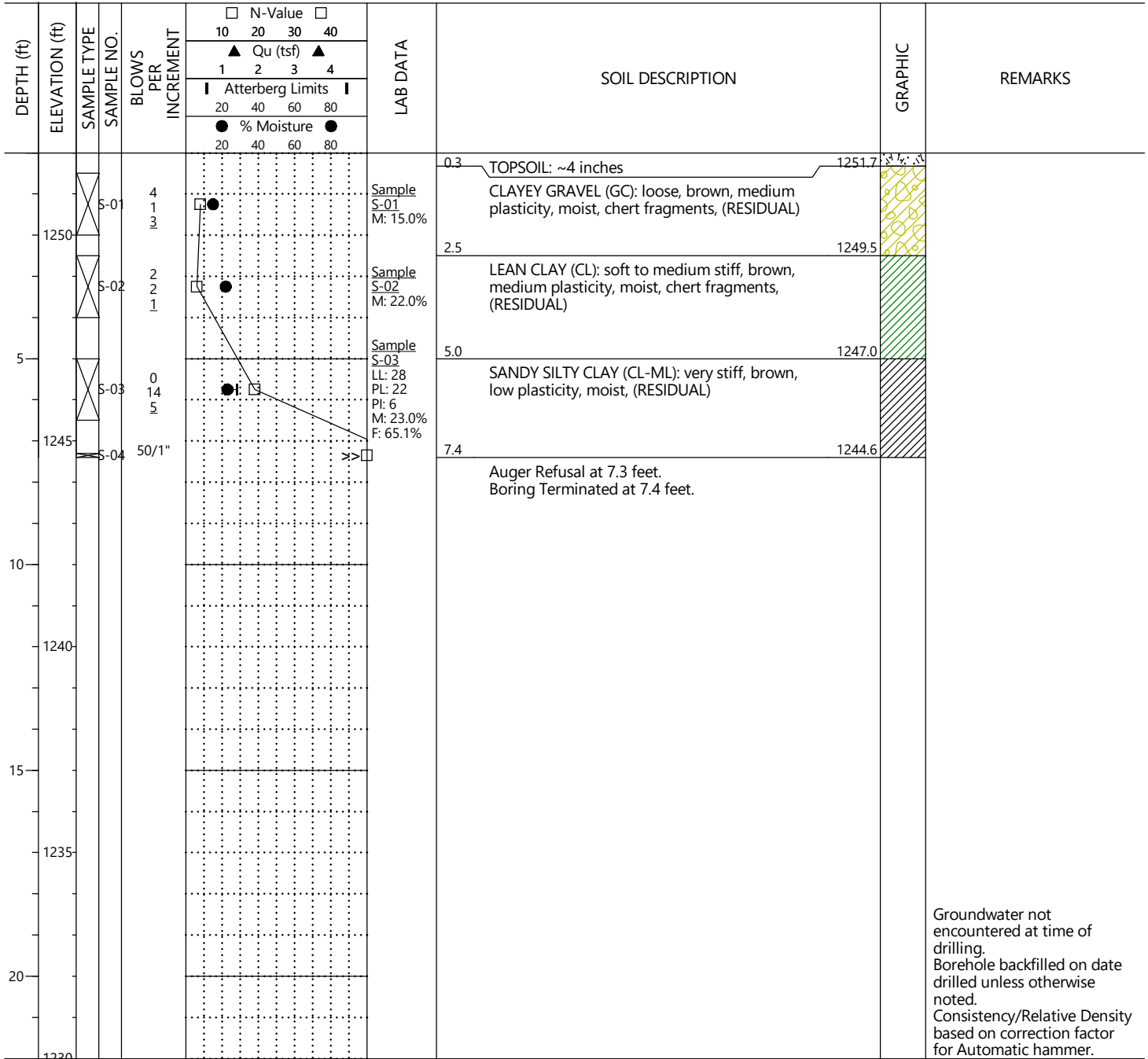
PL: PLASTIC LIMIT **F:** PERCENT PASSING NO. 200 SIEVE

PI: PLASTICITY INDEX

Groundwater not encountered at time of drilling. Borehole backfilled on date drilled unless otherwise noted. Consistency/Relative Density based on correction factor for Automatic hammer.

PROJECT NAME: 606 Central Ave Project
PROJECT NUMBER: BV260045
DRILLING METHOD: Hollow Stem Auger
EQUIPMENT USED: Geoprobe 7822DT
HAMMER TYPE: Automatic
BORING LOCATION: West of Proposed Bridge Abutment

LOCATION: Bentonville, AR
DATE DRILLED: 5/5/26
WEATHER: Cloudy
ELEVATION: 1252
DRILL CREW: Building & Earth
LOGGED BY: D.Rios



SAMPLE TYPE ☒ Split Spoon

N-VALUE STANDARD PENETRATION RESISTANCE (AASHTO T-206)

% MOISTURE PERCENT NATURAL MOISTURE CONTENT

☒ GROUNDWATER LEVEL IN THE BOREHOLE AT TIME OF DRILLING

☒ STABILIZED GROUNDWATER LEVEL

REC RECOVERY

RQD ROCK QUALITY DESIGNATION

UD UNDISTURBED

Qu POCKET PENETROMETER UNCONFINED COMPRESSIVE STRENGTH

LL: LIQUID LIMIT **M:** NATURAL MOISTURE CONTENT

PL: PLASTIC LIMIT **F:** PERCENT PASSING NO. 200 SIEVE

PI: PLASTICITY INDEX

LOG OF BORING

Designation: B-05

Sheet 1 of 1

3165 Pine Woods Road
Springdale, AR
Office: (479) 756-1555
www.BuildingAndEarth.com

PROJECT NAME: 606 Central Ave Project
PROJECT NUMBER: BV260045
DRILLING METHOD: Hollow Stem Auger
EQUIPMENT USED: Geoprobe 7822DT
HAMMER TYPE: Automatic
BORING LOCATION: East of Proposed Bridge Abutment

LOCATION: Bentonville, AR
DATE DRILLED: 5/5/26
WEATHER: Cloudy
ELEVATION: 1255
DRILL CREW: Building & Earth
LOGGED BY: D.Rios

DEPTH (ft)	ELEVATION (ft)	SAMPLE TYPE	SAMPLE NO.	BLOWS PER INCREMENT	N-Value	LAB DATA	SOIL DESCRIPTION	GRAPHIC	REMARKS
					□ N-Value □ 10 20 30 40				
					▲ Qu (tsf) ▲ 1 2 3 4				
					Atterberg Limits 20 40 60 80				
					● % Moisture ● 20 40 60 80				
1255		X 	S-01	2 1 =	Sample S-01 M: 27.4%	-0.3 TOPSOIL: ~3 inches LEAN CLAY (CL): soft to medium stiff, dark brown, low plasticity, moist, root fragments, (RESIDUAL) 2.0	1254.7 1253.0	[Green hatched pattern]	
		X 	S-02	5 11 12	Sample S-02 M: 9.3%	CLAYEY GRAVEL (GC): dense, dark brown, low plasticity, moist, chert fragments, (RESIDUAL)		[Yellow circle pattern]	
5	1250	X 	S-03	50/2"	Sample S-03 M: 9.6%	very dense Auger Refusal at 5 feet. Boring Terminated at 5.2 feet.	5.2 1249.8		
10	1245								
15	1240								
20	1235								Groundwater not encountered at time of drilling. Borehole backfilled on date drilled unless otherwise noted. Consistency/Relative Density based on correction factor for Automatic hammer.

SAMPLE TYPE ☒ Split Spoon

N-VALUE STANDARD PENETRATION RESISTANCE (AASHTO T-206)

% MOISTURE PERCENT NATURAL MOISTURE CONTENT

▽ GROUNDWATER LEVEL IN THE BOREHOLE AT TIME OF DRILLING

STABILIZED GROUNDWATER LEVEL

REC RECOVERY

RQD ROCK QUALITY DESIGNATION

UD UNDISTURBED

Qu POCKET PENETROMETER UNCONFINED COMPRESSIVE STRENGTH

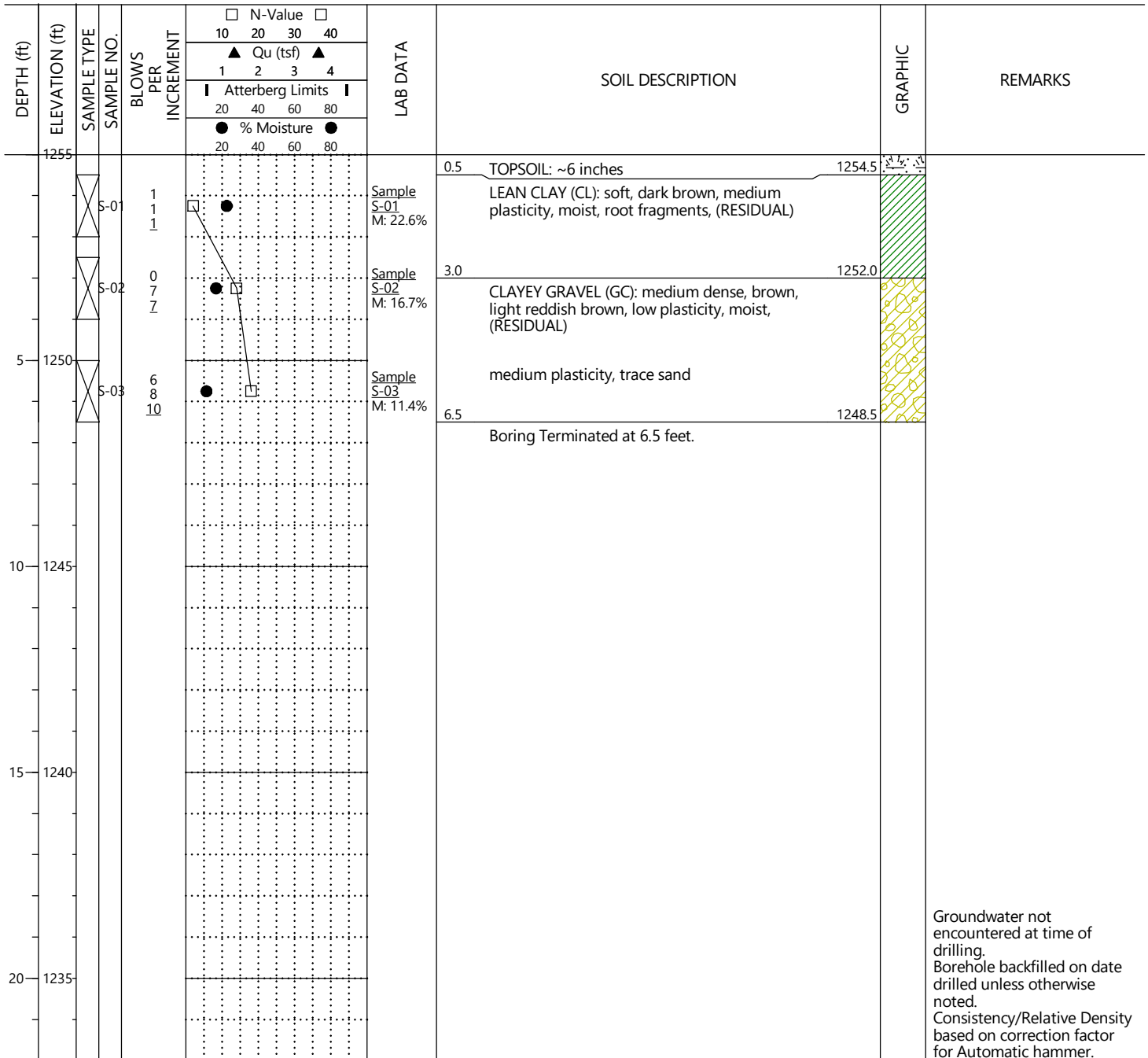
LL: LIQUID LIMIT **M:** NATURAL MOISTURE CONTENT

PL: PLASTIC LIMIT **F:** PERCENT PASSING NO. 200 SIEVE

PI: PLASTICITY INDEX

PROJECT NAME: 606 Central Ave Project
PROJECT NUMBER: BV260045
DRILLING METHOD: Hollow Stem Auger
EQUIPMENT USED: Geoprobe 7822DT
HAMMER TYPE: Automatic
BORING LOCATION: Northwest Boardwalk Area

LOCATION: Bentonville, AR
DATE DRILLED: 5/5/26
WEATHER: Cloudy
ELEVATION: 1255
DRILL CREW: Building & Earth
LOGGED BY: D.Rios



SAMPLE TYPE ☒ Split Spoon

N-VALUE STANDARD PENETRATION RESISTANCE (AASHTO T-206)

% MOISTURE PERCENT NATURAL MOISTURE CONTENT

☒ GROUNDWATER LEVEL IN THE BOREHOLE AT TIME OF DRILLING

☐ STABILIZED GROUNDWATER LEVEL

REC RECOVERY

RQD ROCK QUALITY DESIGNATION

UD UNDISTURBED

Qu POCKET PENETROMETER UNCONFINED COMPRESSIVE STRENGTH

LL: LIQUID LIMIT **M:** NATURAL MOISTURE CONTENT

PL: PLASTIC LIMIT **F:** PERCENT PASSING NO. 200 SIEVE

PI: PLASTICITY INDEX

Groundwater not encountered at time of drilling.
Borehole backfilled on date drilled unless otherwise noted.
Consistency/Relative Density based on correction factor for Automatic hammer.

LOG OF BORING

Designation: B-07

Sheet 1 of 1

3165 Pine Woods Road

Springdale, AR

Office: (479) 756-1555

www.BuildingAndEarth.com

PROJECT NAME: 606 Central Ave Project

PROJECT NUMBER: BV260045

DRILLING METHOD: Hollow Stem Auger

EQUIPMENT USED: Geoprobe 7822DT

HAMMER TYPE: Automatic

BORING LOCATION: Proposed Pergola

LOCATION: Bentonville, AR

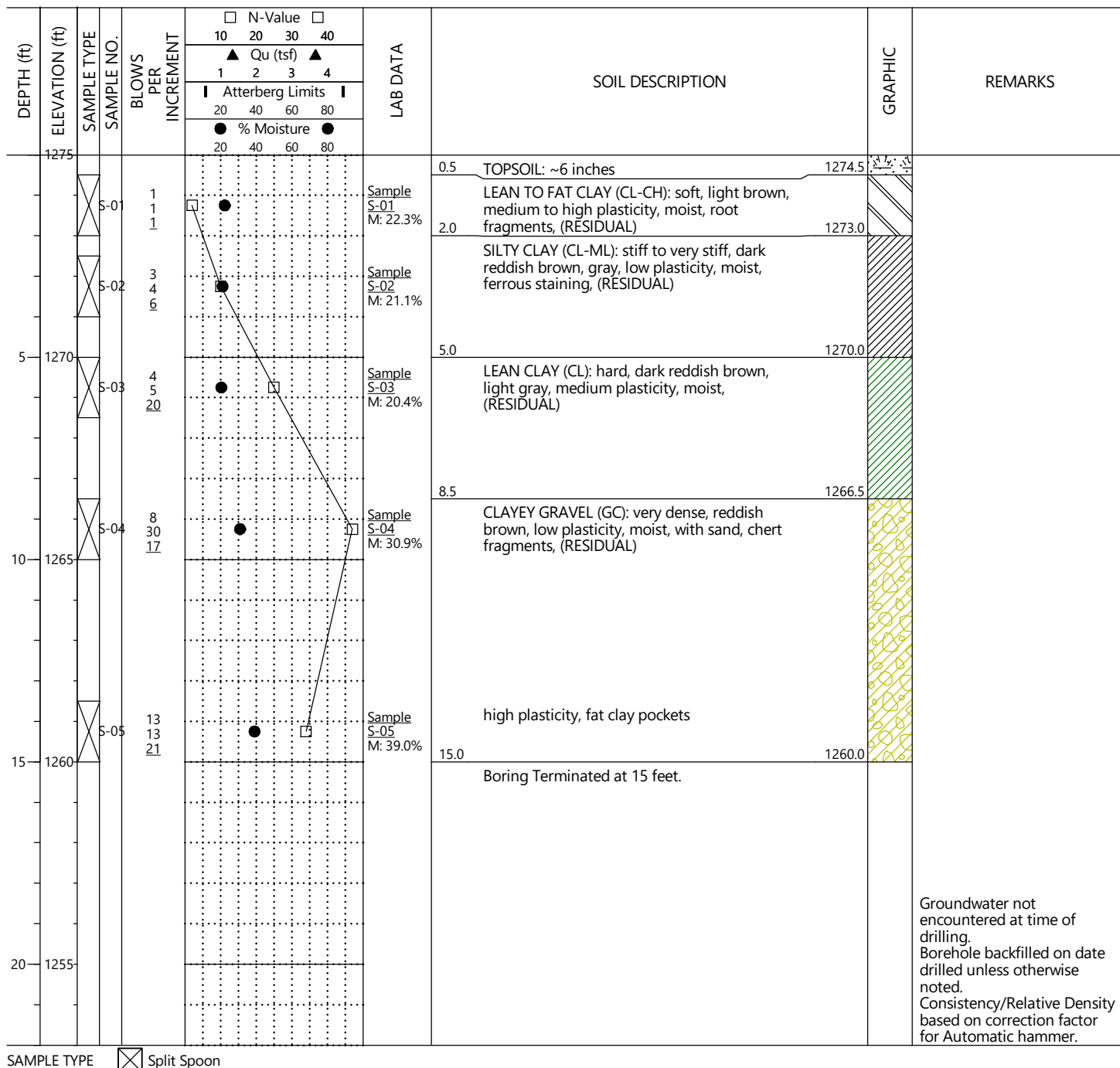
DATE DRILLED: 5/6/26

WEATHER: Cloudy

ELEVATION: 1275

DRILL CREW: Building & Earth

LOGGED BY: D.Rios



SAMPLE TYPE ☒ Split Spoon

N-VALUE STANDARD PENETRATION RESISTANCE (AASHTO T-206)

% MOISTURE PERCENT NATURAL MOISTURE CONTENT

▽ GROUNDWATER LEVEL IN THE BOREHOLE AT TIME OF DRILLING

 STABILIZED GROUNDWATER LEVEL**REC** RECOVERY

RQD ROCK QUALITY DESIGNATION

UD UNDISTURBED

Qu POCKET PENETROMETER UNCONFINED COMPRESSIVE STRENGTH

LL: LIQUID LIMIT **M:** NATURAL MOISTURE CONTENT

PL: PLASTIC LIMIT **F:** PERCENT PASSING NO. 200 SIEVE

PI: PLASTICITY INDEX

LOG OF BORING

Designation: B-08

Sheet 1 of 1

3165 Pine Woods Road
Springdale, AR
Office: (479) 756-1555
www.BuildingAndEarth.com

PROJECT NAME: 606 Central Ave Project
PROJECT NUMBER: BV260045
DRILLING METHOD: Hollow Stem Auger
EQUIPMENT USED: Geoprobe 7822DT
HAMMER TYPE: Automatic
BORING LOCATION: Northeast Parking Lot

LOCATION: Bentonville, AR
DATE DRILLED: 5/6/26
WEATHER: Cloudy
ELEVATION: 1278
DRILL CREW: Building & Earth
LOGGED BY: D.Rios

DEPTH (ft)	ELEVATION (ft)	SAMPLE TYPE	SAMPLE NO.	BLOWS PER INCREMENT	N-Value				LAB DATA	SOIL DESCRIPTION	GRAPHIC	REMARKS		
					10	20	30	40						
					▲ Qu (tsf) ▲									
					1	2	3	4						
					Atterberg Limits									
20 40 60 80				● % Moisture ●										
20 40 60 80														
1275	1275	S-01	S-01	3					Sample S-01	0.5	TOPSOIL: ~6 inches	1277.5		
				1					M: 21.6%	LEAN CLAY (CL): soft to medium stiff, dark brown, medium plasticity, moist, (RESIDUAL)	1275.5			
				2					Sample S-02	2.5	LEAN CLAY WITH GRAVEL (CL): medium stiff to stiff, dark brown, medium plasticity, moist, (RESIDUAL)	1274.0		
				2					LL: 30	4.0	CLAYEY GRAVEL (GC): very dense, light reddish brown, dark reddish brown, low plasticity, moist, chert fragments, (RESIDUAL)	1271.5		
				3					PL: 14	6.5	Boring Terminated at 6.5 feet.			
5	1275	S-02	S-02	2					M: 20.6%					
				2					F: 83.2%					
				3										
				12					Sample S-03					
				23					M: 19.9%					
1270	1270	S-03	S-03	30										
10	1265													
1260	1260													
20	1260													

SAMPLE TYPE ☒ Split Spoon

N-VALUE STANDARD PENETRATION RESISTANCE (AASHTO T-206)

% MOISTURE PERCENT NATURAL MOISTURE CONTENT

▽ GROUNDWATER LEVEL IN THE BOREHOLE AT TIME OF DRILLING

 STABILIZED GROUNDWATER LEVEL**REC** RECOVERY

RQD ROCK QUALITY DESIGNATION

UD UNDISTURBED

Qu POCKET PENETROMETER UNCONFINED COMPRESSIVE STRENGTH

LL: LIQUID LIMIT **M:** NATURAL MOISTURE CONTENT

PL: PLASTIC LIMIT **F:** PERCENT PASSING NO. 200 SIEVE

PI: PLASTICITY INDEX

PROJECT NAME: 606 Central Ave Project
PROJECT NUMBER: BV260045
DRILLING METHOD: Hollow Stem Auger
EQUIPMENT USED: Geoprobe 7822DT
HAMMER TYPE: Automatic
BORING LOCATION: Water Garden

LOCATION: Bentonville, AR
DATE DRILLED: 5/5/26
WEATHER: Cloudy
ELEVATION: 1270
DRILL CREW: Building & Earth
LOGGED BY: D.Rios

DEPTH (ft)	ELEVATION (ft)	SAMPLE TYPE	SAMPLE NO.	BLOWS PER INCREMENT	LAB DATA				SOIL DESCRIPTION	GRAPHIC	REMARKS	
					N-Value							
					Qu (tsf)							
					Atterberg Limits							
					% Moisture							
1270												
		X	S-01	1					Sample S-01 M: 14.6%	0.5 TOPSOIL: ~6 inches	1269.5	
		X	S-02	20					Sample S-02 M: 12.5%	LEAN TO FAT CLAY (CL-CH): stiff, dark brown, medium plasticity, moist, root fragments, (RESIDUAL)	1268.0	
				50/1"						CLAYEY GRAVEL (GC): medium dense, brown, low plasticity, moist, (RESIDUAL)	1266.9	
										Auger Refusal at 3 feet. Boring Terminated at 3.1 feet.		
5	1265											
10	1260											

SAMPLE TYPE ☒ Split Spoon

N-VALUE STANDARD PENETRATION RESISTANCE (AASHTO T-206)

% MOISTURE PERCENT NATURAL MOISTURE CONTENT

☒ GROUNDWATER LEVEL IN THE BOREHOLE AT TIME OF DRILLING

☐ STABILIZED GROUNDWATER LEVEL

REC RECOVERY

RQD ROCK QUALITY DESIGNATION

UD UNDISTURBED

Qu POCKET PENETROMETER UNCONFINED COMPRESSIVE STRENGTH

LL: LIQUID LIMIT **M:** NATURAL MOISTURE CONTENT

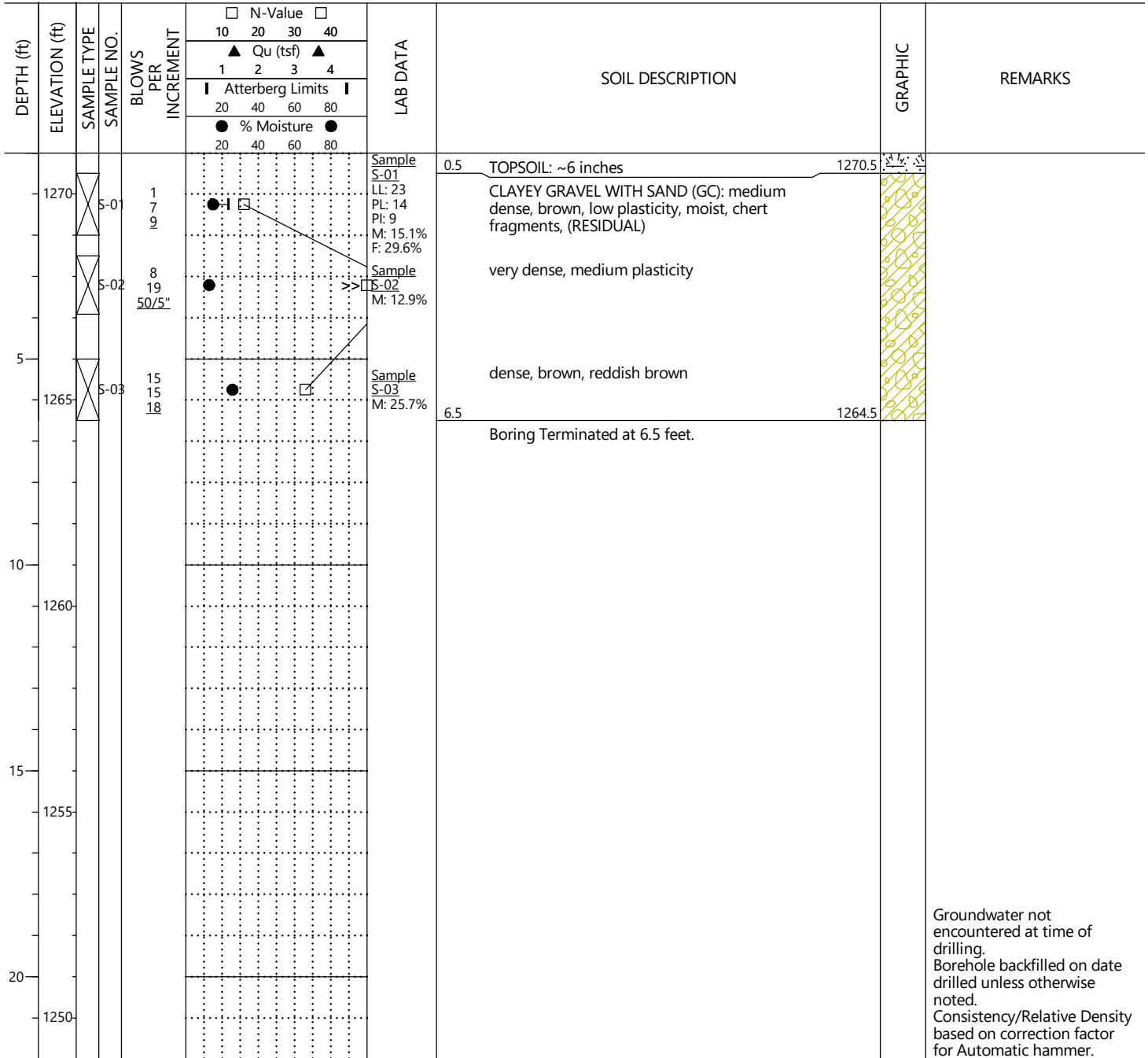
PL: PLASTIC LIMIT **F:** PERCENT PASSING NO. 200 SIEVE

PI: PLASTICITY INDEX

Groundwater not encountered at time of drilling. Borehole backfilled on date drilled unless otherwise noted. Consistency/Relative Density based on correction factor for Automatic hammer.

PROJECT NAME: 606 Central Ave Project
PROJECT NUMBER: BV260045
DRILLING METHOD: Hollow Stem Auger
EQUIPMENT USED: Geoprobe 7822DT
HAMMER TYPE: Automatic
BORING LOCATION: Proposed Staff Parking

LOCATION: Bentonville, AR
DATE DRILLED: 5/5/26
WEATHER: Cloudy
ELEVATION: 1271
DRILL CREW: Building & Earth
LOGGED BY: D.Rios



SAMPLE TYPE ☒ Split Spoon

N-VALUE STANDARD PENETRATION RESISTANCE (AASHTO T-206)

% MOISTURE PERCENT NATURAL MOISTURE CONTENT

☒ GROUNDWATER LEVEL IN THE BOREHOLE AT TIME OF DRILLING

☒ STABILIZED GROUNDWATER LEVEL

REC RECOVERY

RQD ROCK QUALITY DESIGNATION

UD UNDISTURBED

Qu POCKET PENETROMETER UNCONFINED COMPRESSIVE STRENGTH

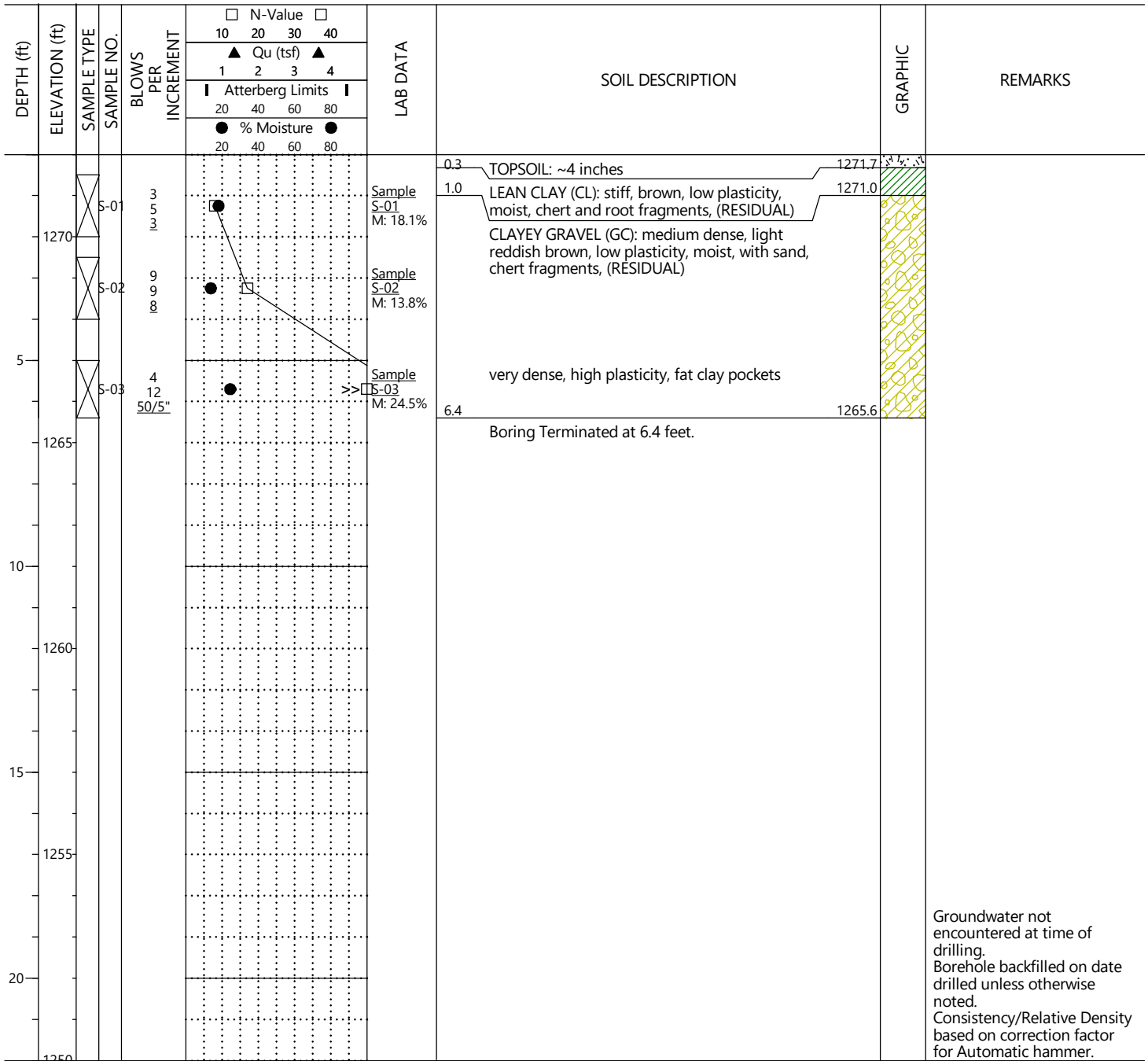
LL: LIQUID LIMIT **M:** NATURAL MOISTURE CONTENT

PL: PLASTIC LIMIT **F:** PERCENT PASSING NO. 200 SIEVE

PI: PLASTICITY INDEX

PROJECT NAME: 606 Central Ave Project
PROJECT NUMBER: BV260045
DRILLING METHOD: Hollow Stem Auger
EQUIPMENT USED: Geoprobe 7822DT
HAMMER TYPE: Automatic
BORING LOCATION: South Proposed Parking Lot Area

LOCATION: Bentonville, AR
DATE DRILLED: 5/5/26
WEATHER: Cloudy
ELEVATION: 1272
DRILL CREW: Building & Earth
LOGGED BY: D.Rios



SAMPLE TYPE ☒ Split Spoon

N-VALUE STANDARD PENETRATION RESISTANCE (AASHTO T-206)

% MOISTURE PERCENT NATURAL MOISTURE CONTENT

☒ GROUNDWATER LEVEL IN THE BOREHOLE AT TIME OF DRILLING

☒ STABILIZED GROUNDWATER LEVEL

REC RECOVERY

RQD ROCK QUALITY DESIGNATION

UD UNDISTURBED

Qu POCKET PENETROMETER UNCONFINED COMPRESSIVE STRENGTH

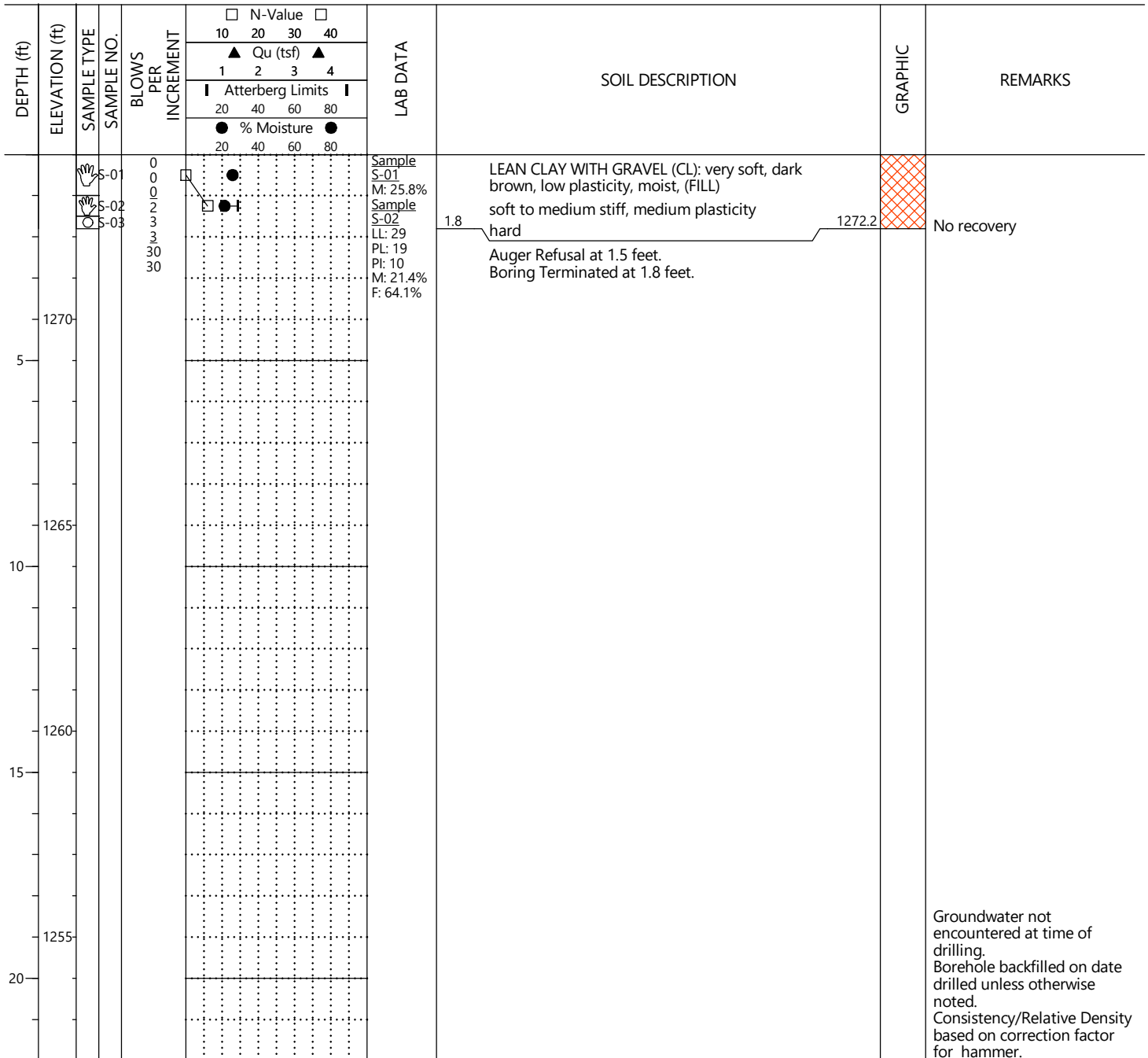
LL: LIQUID LIMIT **M:** NATURAL MOISTURE CONTENT

PL: PLASTIC LIMIT **F:** PERCENT PASSING NO. 200 SIEVE

PI: PLASTICITY INDEX

PROJECT NAME: 606 Central Ave Project
PROJECT NUMBER: BV260045
DRILLING METHOD: Hand Auger
EQUIPMENT USED: DCP
HAMMER TYPE:
BORING LOCATION: Proposed Interior Elevator

LOCATION: Bentonville, AR
DATE DRILLED: 5/5/26
WEATHER: Cloudy
ELEVATION: 1274
DRILL CREW: Building & Earth
LOGGED BY: A. Al-Saadi



SAMPLE TYPE Grab Sample No Recovery

N-VALUE STANDARD PENETRATION RESISTANCE (AASHTO T-206) **REC** RECOVERY **LL**: LIQUID LIMIT **M**: NATURAL MOISTURE CONTENT
% MOISTURE PERCENT NATURAL MOISTURE CONTENT **RQD** ROCK QUALITY DESIGNATION **PL**: PLASTIC LIMIT **F**: PERCENT PASSING NO. 200 SIEVE
 GROUNDWATER LEVEL IN THE BOREHOLE AT TIME OF DRILLING **UD** UNDISTURBED **PI**: PLASTICITY INDEX
 STABILIZED GROUNDWATER LEVEL **Qu** POCKET PENETROMETER UNCONFINED COMPRESSIVE STRENGTH

LABORATORY TEST PROCEDURES

A brief description of the laboratory tests performed is provided in the following sections.

DESCRIPTION OF SOILS (VISUAL-MANUAL PROCEDURE) (ASTM D2488)

The soil samples were visually examined by our engineer and soil descriptions were provided. Representative samples were then selected and tested in accordance with the aforementioned laboratory-testing program to determine soil classifications and engineering properties. This data was used to correlate our visual descriptions with the Unified Soil Classification System (USCS).

NATURAL MOISTURE CONTENT (ASTM D2216)

Natural moisture contents (M%) were determined on selected samples. The natural moisture content is the ratio, expressed as a percentage, of the weight of water in a given amount of soil to the weight of solid particles.

ATTERBERG LIMITS (ASTM D4318)

The Atterberg Limits test was performed to evaluate the soil's plasticity characteristics. The soil Plasticity Index (PI) is representative of this characteristic and is bracketed by the Liquid Limit (LL) and the Plastic Limit (PL). The Liquid Limit is the moisture content at which the soil will flow as a heavy viscous fluid. The Plastic Limit is the moisture content at which the soil is between "plastic" and the semi-solid stage. The Plasticity Index ($PI = LL - PL$) is a frequently used indicator for a soil's potential for volume change. Typically, a soil's potential for volume change increases with higher plasticity indices.

MATERIAL FINER THAN NO. 200 SIEVE BY WASHING (ASTM D1140)

Grain-size tests were performed to determine the partial soil particle size distribution. The amount of material finer than the openings on the No. 200 sieve (0.075 mm) was determined by washing soil over the No. 200 sieve. The results of wash #200 tests are presented on the boring logs included in this report and in the table of laboratory test results.

LABORATORY TEST RESULTS

LABORATORY TEST RESULTS

The results of the laboratory testing are presented in the following tables.

BORING NO.	DEPTH	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	% PASSING #200 SIEVE	CLASSIFICATION
B-01	0.5 - 2.0	22.4					
B-01	2.5 - 4.0	6.8	24	23	1	11	
B-01	5.0 - 6.5	11.3					
B-01	8.5 - 10.0	49.2					
B-01	13.5 - 15.0	36.1					
B-01	17.8 - 17.9	21.6					
B-02	0.5 - 2.0	28.5					
B-02	2.5 - 4.0	16.6					
B-02	5.0 - 6.5	8.7					
B-02	8.5 - 10.0	36.8					
B-02	18.0 - 18.1	15.2					
B-03	0.5 - 2.0	20.2					
B-03	2.5 - 4.0	22.3					
B-03	5.0 - 6.5	16.5					
B-03	8.5 - 10.0	28.5					
B-04	0.5 - 2.0	15.0					
B-04	2.5 - 4.0	22.0					
B-04	5.0 - 6.5	23.0	28	22	6	65	CL-ML
B-05	0.5 - 2.0	27.4					
B-05	2.5 - 4.0	9.3					
B-05	5.0 - 5.2	9.6					
B-06	0.5 - 2.0	22.6					
B-06	2.5 - 4.0	16.7					
B-06	5.0 - 6.5	11.4					
B-07	0.5 - 2.0	22.3					
B-07	2.5 - 4.0	21.1					
B-07	5.0 - 6.5	20.4					
B-07	8.5 - 10.0	30.9					
B-07	13.5 - 15.0	39.0					
B-08	0.5 - 2.0	21.6					
B-08	2.5 - 4.0	20.6	30	14	16	83	CL

TABLE L-1: General Soil Classification Test Results

Soils with a Liquid Limit (LL) greater than 50 and Plasticity Index (PI) greater than 25 usually exhibit significant volume change with varying moisture content and are considered to be highly plastic

⁽¹⁾ Indicates visual classification. WR indicates weathered rock.

LABORATORY TEST RESULTS

The results of the laboratory testing are presented in the following tables.

[illegible]

TABLE L-1: General Soil Classification Test Results

Soils with a Liquid Limit (LL) greater than 50 and Plasticity Index (PI) greater than 25 usually exhibit significant volume change with varying moisture content and are considered to be highly plastic

⁽¹⁾ Indicates visual classification. WR indicates weathered rock.

Important Information about This Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a civil engineer may not fulfill the needs of a constructor — a construction contractor — or even another civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. No one except you should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you — should apply this report for any purpose or project except the one originally contemplated.*

Read the Full Report

Serious problems have occurred because those relying on a geotechnical-engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

Geotechnical Engineers Base Each Report on a Unique Set of Project-Specific Factors

Geotechnical engineers consider many unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk-management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical-engineering report that was:

- not prepared for you;
- not prepared for your project;
- not prepared for the specific site explored; or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical-engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an

assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical-engineering report is based on conditions that existed at the time the geotechnical engineer performed the study. *Do not rely on a geotechnical-engineering report whose adequacy may have been affected by:* the passage of time; man-made events, such as construction on or adjacent to the site; or natural events, such as floods, droughts, earthquakes, or groundwater fluctuations. *Contact the geotechnical engineer before applying this report to determine if it is still reliable.* A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ — sometimes significantly — from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide geotechnical-construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are Not Final

Do not overrely on the confirmation-dependent recommendations included in your report. *Confirmation-dependent recommendations are not final*, because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations *only* by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's confirmation-dependent recommendations if that engineer does not perform the geotechnical-construction observation required to confirm the recommendations' applicability.*

A Geotechnical-Engineering Report Is Subject to Misinterpretation

Other design-team members' misinterpretation of geotechnical-engineering reports has resulted in costly

problems. Confront that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Constructors can also misinterpret a geotechnical-engineering report. Confront that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing geotechnical construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical-engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make constructors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give constructors the complete geotechnical-engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise constructors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure constructors have sufficient time to perform additional study.* Only then might you be in a position to give constructors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and constructors fail to recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help

others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Environmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform an *environmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold-prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, many mold- prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical- engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; *none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.*

Rely, on Your GBC-Member Geotechnical Engineer for Additional Assistance

Membership in the Geotechnical Business Council of the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your GBC-Member geotechnical engineer for more information.



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