



ECS Midwest, LLC

Geotechnical/Geophysical Engineering Report

Proposed McDonald's 24-1329 - Willard, MO

652 South Hunt Road
Willard, Greene County, MO 65781

ECS Project No. 16:16210

June 25, 2026





ECS MIDWEST, LLC

Geotechnical • Construction Materials • Environmental • Facilities

June 25, 2026

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ECS Project No. 16:16210

Reference: Geotechnical/Geophysical Engineering Report
Proposed McDonald's 24-1329 - Willard, MO
652 South Hunt Road
Willard, Greene County, MO 65781
Purchase Order No.: 2930577

Dear Ms. Switzer:

ECS Midwest, LLC (ECS) has completed the subsurface exploration, laboratory testing, and geotechnical/geophysical engineering analyses for the above-referenced project. Our services were performed in general accordance with our agreed-upon scope of work. This report presents our understanding of the geotechnical aspects of the project along with the results of the field exploration and laboratory testing conducted, and our design and construction recommendations.

It has been our pleasure to be of service to McDonald's USA LLC during the design phase of this project. We would appreciate the opportunity to remain involved during the continuation of the design phase, and we would like to provide our services during construction phase operations as well to verify the subsurface conditions anticipated based on this report. Should you have any questions concerning the information contained in this report, or if we can be of further assistance to you, please contact us.

Respectfully submitted,

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EXECUTIVE SUMMARY

ECS Midwest, LLC has completed the geotechnical/geophysical engineering evaluations for the proposed McDonald's restaurant development in Willard, Missouri. The project information that was made available to ECS by McDonald's at the time of this report and the results of our findings, conclusions, and recommendations are summarized below:

Topic	Overview Statement
Project Description	- Approximately 4,590 SF building structure with anticipated building FFE of EL. 1,224.5± feet MSL. - Based on a recent email communication with Rubix Consultants, LLC, dated July 11, 2025, below are the anticipated structural loads: ➤ Maximum Column Load: 20 kips ➤ Wall Loads: 2½ to 3½ kips per lineal foot ➤ Slab Loads: 100 to 120 psf Fill operations of 8 to 12 feet are anticipated within the building, southern and western parts of the site. - Based on the provided information 'Survey and Soil Borings' dated September 19, 2009 from McDonald's, the light and heavy-duty pavements will be designed for 1.5 and 4.0 - 18-kips ESAL's loads per day with 20 years of design life, respectively.
Geotechnical Characterization	- Below the surficial cover, lean and highly plastic 'fat' clays, were encountered at the test borings extending to depths of approximately ½ to 12½ feet. - The bottommost formation consisted of weathered limestone bedrock. ECS cored limestone bedrock at five (5) locations.
Shrink/Swell Potential	Highly plasticity 'fat clays' (CH) were encountered within the explored depths of the soil borings. These soils exhibit higher than typical shrink-swell potential and could cause potential rise/heave (PVR - potential vertical rise) in the range of 1½ to 2½ inches if encountered at lightly loaded proposed building foundations, floor slab, and pavements final subgrade/bearing elevations. Therefore, if observed, at the proposed foundations, slabs or pavement subgrade elevation, ECS recommends removal and replacement of higher plasticity 'fat clays' with low volume change material (LVC).
Earthwork	Existing higher plasticity 'fat clays' (CH) are not considered suitable for the grading operations due to high shrink-swell potential.
Geological Hazards	- The subject site is underlain by potentially karstic bedrock conditions, which are common below Willard, MO and the surrounding areas. The subsurface conditions encountered at this site are anticipated to be the same, or very similar, subsurface conditions present within the surrounding parcels. - Apparent active sink holes <u>were not observed</u> via the geophysical mapping and soil borings. No surficial indications of sink hole activities were observed at this site. However, the potential for future sinkhole development is present in karst bedrock formations.
Shallow Foundations	The proposed structure and light poles can be supported on shallow foundations at or below the frost depth of 24 inches below the final site grades bearing on low volume change material (LVC) for maximum net allowable soil bearing pressure of 2,500 psf. Some undercutting may be required at the foundation bearing elevation in soft/unsuitable soils.

Topic	Overview Statement
	• Due to variable limestone bedrock depths across the site, the northern building foundations could be at or near the bedrock formation. ECS recommends that the building foundations should not be simultaneously supported on soil and bedrock due to high risk of differential settlement. • The estimated total and differential settlement will be on the order of 1 inch and ½ inch.
Deep Foundations	• Deep foundations are not necessary to support the proposed building. The proposed light poles can be supported either on shallow foundations or drilled shafts.
Subgrade Modulus	• The proposed floor slab supported on 24-inches of new engineered fill (LVC - low volume change material) can be designed using a subgrade modulus of 150 psi/in. • The proposed pavements supported on 12-inches of new engineered fill (LVC - low volume change material) can be designed using a subgrade modulus of 125 psi/in.
California Bearing Ratio	• The proposed flexible pavement can be designed based on an anticipated California Bearing Ratio (CBR) of 3.5 considering upper 12 inches of low volume change material (LVC).
Below-Grade Structures	None anticipated
Seismic Site Class	Site Class C
Pavements	With subgrade prepared as noted in the Earthwork section of this report, the estimated pavement sections are as follows: Rigid Concrete Pavement: ➤ Light-Duty: 5 inches of Portland Cement Concrete (PCC) over 6 inches of crushed aggregate. ➤ Heavy-Duty: 6 inches of Portland Cement Concrete (PCC) over 6 inches of crushed aggregate. Flexible Asphalt Pavement: ➤ Light-Duty: 4 inches of Asphalt Concrete over 8 inches of crushed aggregate. ➤ Heavy-Duty: 5 inches of Asphalt Concrete over 10 inches of crushed aggregate.
<i>This Executive Summary is intended as a very brief overview of the primary geotechnical conditions that are expected to affect design and construction. Information gleaned from this Executive Summary should not be utilized in lieu of reading the entire geotechnical report. This summary should not be considered apart from the entire text of the report with the qualifications and considerations mentioned herein. Details of our conclusions and recommendations are discussed in the report text.</i>	

1.0 INTRODUCTION

The purpose of this study was to provide geotechnical information for the design of building foundations, high-rise sign, light poles and pavements for the proposed McDonald's development in Willard, Missouri. The recommendations developed for this report are based on project information supplied by the McDonald's USA, LLC team.

ECS provided these services in accordance with the Service Supplier Proposal and authorized by Ms. Amy Switzer through McDonald's Purchase Order No.: 2930577 Rev 1, dated June 9, 2026.

This report contains the procedures and results of our subsurface exploration and laboratory testing programs, review of existing site conditions, engineering analyses, and recommendations for the design and construction of the project. The report includes the following items:

- A brief review and description of our field and laboratory test procedures and the results of the testing conducted.
- A review of surface topographical features and site conditions.
- A review of subsurface soil and bedrock stratigraphy with pertinent physical properties.
- Final soil and bedrock exploration/test boring logs and subsurface cross section.
- Recommendations for site preparation and construction of compacted engineered fills, including an evaluation of on-site soils for use as compacted engineered fills and identification of potentially unsuitable soils and/or soils exhibiting excessive moisture at the time of sampling.
- Recommended foundation type(s), including allowable soil bearing pressures and anticipated foundation settlements for the proposed building, high-rise sign and light poles.
- Geophysical testing to map the karstic formation and sink holes.
- Seismic Site Classification in accordance with the International Building Code 2018 based on the weighted average SPT 'N' blow counts and provide applicable seismic site coefficients (no liquefactions analysis).
- Recommendations for the design and construction of pavements.
- Evaluation and recommendations related to groundwater control, including recommendations for pavement underdrains.

2.0 PROJECT INFORMATION

2.1 PROJECT LOCATION/CURRENT SITE USE/PAST SITE USE

The project site is located at 652 South Hunt Road in Willard, Missouri. The project site is bounded to the north by Melville Road, to the east by Hunt Road, and to the west and south by low-lying vegetation and is utilized for cattle grazing. Based on the recent site visit by ECS personnel and ECS completed Phase-I ESA report (ECS Report No.: 53-6130 dated April 10, 2026), the subject site is currently undeveloped with an overall area of approximately 2.02 acres. The site location is shown below, and in wider scope on the **Site Location Diagram** in Appendix A:



Site Location Map (outlined in RED)

Based on the publicly available historical images dating back to 1938 and to date, the subject property remained undeveloped. The eastern adjoining properties, Melville Road and Hunt Road were developed during 1979 through early 2000. Based on the client provided topographical Map, the existing site grades range from about elevation EL. 1,209 feet to EL. 1,227 feet MSL, and slope from north to south and east to west. Site grades determined without a professional survey are approximate and should not be used in the final design of structures and site amenities.



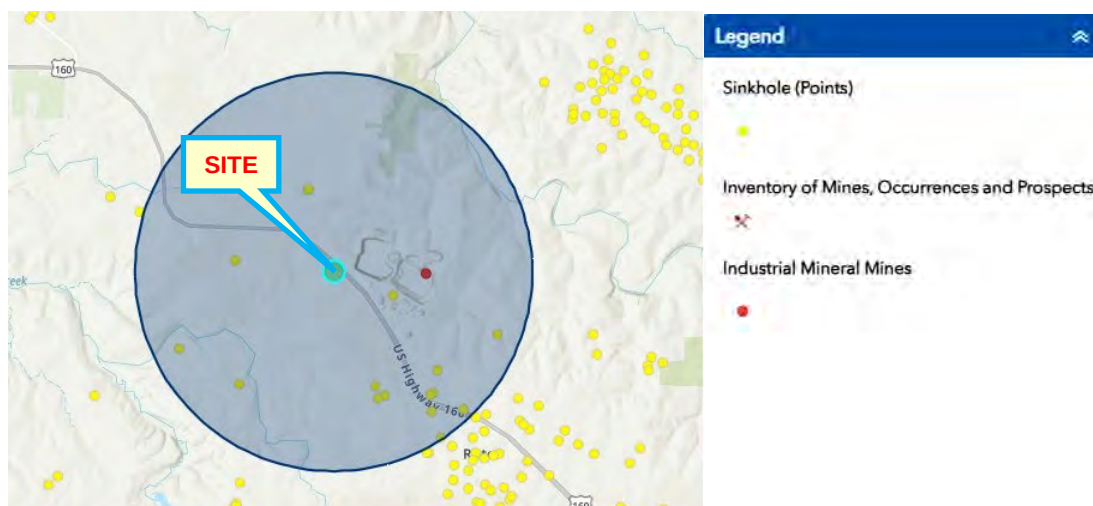
Viewing East/southeast across the site from northwestern property corner



Viewing east/northeast across the property from southwestern property corner

2.1.1 Mineral Mines and Sinkholes

Based on our review of the information available online (i.e., Missouri's Department of Natural Resources Geosciences Technical Resource Assessment Tool, or *GeoSTRAT*), the State of Missouri is known to be home to a wide variety of geological structures and features (including but not limited to fault lines, sinkholes, caves, etc., as well as areas with Karst activity). In particular, the Figure below depicts the geological features and structures within a radius of about 2 miles around the proposed project site:



Geological Features and Structures within 2 miles of the project site

Based on the information included in *GeoSTRAT*, the area in the vicinity of the project site hosts the following geologic structures and features within a small radius of 2 miles:

GEOLOGICAL FEATURES NEAR THE PROJECT SITE	
Type of Feature	Number of Occurrences
Reported Karstic Sinkholes	16
Old Industrial Mineral Mines	1
TOTAL	17

2.2 PROPOSED CONSTRUCTION

Based on the information provided and the schematic *Concept Plan (Drawings CP-12)*, ECS understands that the proposed development will include construction of a single-story, slab on grade (i.e., with no basement level) building structure, associated parking lot and drive lanes, and a stormwater detention pond. The Table below shows the provided and ECS estimated design details for the proposed development:

GENERAL DESIGN INFORMATION		
Subject	Design Characteristic	
Proposed Building	One-story, slab-on-grade, building structure encompassing approximately 4,590 square feet in plan area	R
Maximum Column/Wall Loads	20 kips/3½ kips per foot	R
Settlement Tolerance	1-inch total and 1/2-inch differential	R
Grade Supported Slab Loads	100 to 120 psf	R
Finished Floor Elevation	Estimated EL. 1,224.5± feet MSL	R
Grading Operations	• Building: Fill operations of up to 8 feet within the southern/southwestern part of the building. • Pavements and SWM Pond: Cut operations of up to 3 feet within the northern part of the site, and Fill operations of up to 12 feet within the western/ southern parts of the site and stormwater management pond.	R
Light Poles	Structure specific details were not available at this time. However, ECS understands that the proposed light poles can be supported on shallow foundations or drilled shafts.	B
Pavement	Flexible and Rigid concrete pavement based on the client provided 18-kips ESAL's data. Based on the client provided traffic data, ECS estimated 11,000 and 30,000 18-kips ESAL's for the light and heavy-duty pavements, respectively, for design life of 20 years	R
SWM Pond	A stormwater management pond is planned to be constructed within the southern part of the site with pond bottom elevation of EL. 1,213 feet and crest elevation of EL. 1,218 to EL. 1,222 feet MSL	

R: Reported by client and/or Design Team

B: Based on ECS estimate in absence of information from Client and/or Design Team

If ECS' understanding of the project is not correct, especially if the structural loads or elevations are different, please contact ECS so that we may review these changes and revise our recommendations as appropriate.

3.0 FIELD EXPLORATION AND LABORATORY TESTING

Our exploration procedures are explained in greater detail in Appendix B including the insert entitled 'Exploration Procedures'. Our initial scope of work included drilling thirteen (13) SPT test borings within the proposed development areas. However, during an email communication on June 3, 2026, ECS was notified that the site layout has been changed with the proposed building moved to northeast and rotated to approximately 90 degrees. Only one of the pavement borings (P-02) was located within the new building layout. After approval from McDonalds team, ECS completed additional seven (7) test borings within the proposed building, drive-thru access, trash enclosure and high-rise pylon sign structure. ECS interpolated test boring surface elevation from the client provided topographical and grading Map. The approximate locations of the completed soil borings are shown on the **Boring Location Diagram** in Appendix A.

The drilling subcontractor performed the drilling services using an ATV drilling rig and utilizing continuous flight augers. Prior to drilling, the drilling subcontractor contacted the State of Missouri One-Call System to clear and mark public underground utilities in the vicinity of the project site. In addition, ECS engaged a private utility locator to scan limited areas around the proposed test borings, during initial and subsequent explorations, to assist in locating any underground utilities.

Standard penetration tests (SPTs) were conducted in the borings at regular intervals in general accordance with ASTM D1586. Additionally, the drilling subcontractor performed bedrock coring in five (5) of the test borings (B-01, B-03, B-04, S-01 and S-02). The drill crew backfilled the boreholes with cuttings at completion of drilling. Settlement or expansion of borehole backfill can occur over time resulting in a trip hazard. Monitoring the boreholes after initial drilling activities is not within our scope but should be done by the client or property owner.

3.1 SUBSURFACE CHARACTERIZATION

The subsurface conditions encountered were generally consistent with published geological mapping. The following Sections of this text provide generalized characterizations of the soil strata. Please refer to the boring logs in Appendix B.

GENERALIZED SUBSURFACE PROFILE					
Approximate Depth Range (feet)	Stratum No.	Material Description	Calibrated Penetrometer Resistance (tsf)	Natural Moisture Content (%)	SPT ⁽¹⁾ N-values (bpf)
Near-Surface Cover	NA	Topsoil: 4 to 18 inches	N/A	N/A	N/A
1 to 13½	I	(CL) Lean Clay, (CH) Gravelly Fat Clay, Fat Clay with sand, Fat Clay – trace sand and gravel, brown, reddish brown, moist, red and tan, moist, stiff to hard, with chert	1 to 4	22 to 54	6 to 50+
1 to 23½	II ²	Limestone Bedrock - fractured, slightly weathered, light gray, moderately hard to hard	N/A	N/A	50+

Notes:

- (1) Standard Penetration Testing
- (2) Cored bedrock at test borings B-01, B-03, B-04, S-01 and S-02

The soil/bedrock stratification shown on the boring logs represents the interpreted soil conditions at the actual boring locations. Variations in the stratification can occur between sample intervals and boring locations. The subsurface conditions at other times and locations on the site may differ from those found at the test boring locations. If different site conditions are encountered during construction, ECS should be contacted to review our recommendations related to the new information.

Because the split-spoon sampler has a 1½-inch inside diameter, the soil classifications noted on the boring logs may not be representative of the entire soil matrix. Material larger than the 1½-inch inside diameter of the split-spoon sampler cannot be collected and observed directly.

The Table below shows observed limestone bedrock depth with respect to existing grades, bedrock recovery, RQD and laboratory determined compressive strengths in general accordance with ASTM D7012 (*Method C*):

GENERALIZED LIMESTONE BEDROCK PROFILE						
Soil Boring No.	Apparent Depth to Rock below the Site Grade		Percent Recovery of Core Run	RQD, %	Unconfined Compressive Strength (Qu, psi)	Bedrock Types
	(feet)	(feet, MSL)				
B-01	4	1209	100	90 to 93	12,940	Weathered Limestone
B-02	2¾	1215¾	N/A	N/A	N/A	Weathered Limestone
B-03	5½	1208½	100	92	16,690	Weathered Limestone
B-04	1	1211	100	45 to 92	5,420; 10,000	Weathered Limestone
B-05	2¾	1214¾	N/A	N/A	N/A	Weathered Limestone
B-06	12½	1209½	N/A	N/A	N/A	Weathered Limestone
B-07	2½	1221½	N/A	N/A	N/A	Weathered Limestone
B-08	8	1212	N/A	N/A	N/A	Weathered Limestone
B-09	3	1216	N/A	N/A	N/A	Weathered Limestone
D-01	6	1193	N/A	N/A	N/A	Weathered Limestone
D-02	4	1200	N/A	N/A	N/A	Weathered Limestone
P-01	6	1219	N/A	N/A	N/A	Weathered Limestone
P-02	1½	1215½	N/A	N/A	N/A	Weathered Limestone
P-03	3½	1206½	N/A	N/A	N/A	Weathered Limestone
P-04	3½	1207½	N/A	N/A	N/A	Weathered Limestone
P-05	4	1206	N/A	N/A	N/A	Weathered Limestone
P-06	3	1205	N/A	N/A	N/A	Weathered Limestone
P-07	3¾	1211¾	N/A	N/A	N/A	Weathered Limestone
P-08	8	1218	N/A	N/A	N/A	Weathered Limestone
P-09	6	1219	N/A	N/A	N/A	Weathered Limestone
S-01	3½	1218½	92 to 100	56 to 92	10,170; 12,310	Weathered Limestone
S-02	13½	1211½	100	92 to 95	N/A	Weathered Limestone

3.2 GROUNDWATER OBSERVATIONS

The drillers observed the boreholes for the presence of measurable free ground water during drilling and at the completion of drilling. Free groundwater was observed at depths of about 10 and 13 feet in test borings B-06 and S-02, and perched water at test boring P-02 at 1 foot. However, groundwater was not

observed in the remaining test borings during drilling and immediately upon completion of the test borings.

Please note that in predominantly fine-grained soils, groundwater may take several days or weeks to stabilize in the boreholes. The drilling crew backfilled the open boreholes immediately upon completion of drilling. Soil in the Midwest frequently oxidize from gray to brown above the level where the soil remains saturated. This zone of change, which may be an indication of the seasonal high groundwater. Gray soils were not observed in the test borings. Gray soil were not observed in the test borings.

Variations in both groundwater types (perched and groundwater table aquifer) can occur as a result of changes in precipitation, evaporation, surface water runoff, construction activities, and other factors. The highest groundwater observations are normally encountered in late winter, and early spring and our current groundwater observations are not expected to be at the seasonal maximum water table. Perched water conditions may also develop and/or exist at shallower depths seasonally, particularly within more permeable soil underlain by less permeable soil and within existing building and utility backfill.

3.3 LABORATORY TESTING

The laboratory testing consisted of selected tests performed on samples obtained during our field exploration operations. Classification and index property tests were performed on representative soil samples. These tests included:

- Calibrated Penetrometer Resistance
- Natural Moisture Content (ASTM D2216)
- Atterberg Limits Test (ASTM D4318)
- Grain Size Analysis (ASTM D6913M)
- Compressive strength tests were performed on selected rock core samples in accordance with ASTM D7012 (*Method C*).

Each sample was visually classified on the basis of texture and plasticity in accordance with ASTM D2488 Standard Practice for Description and Identification of Soils (Visual-Manual Procedures) and including USCS classification symbols. After classification, the samples were grouped in the major zones noted on the boring logs in Appendix B. The group symbols for each soil type are indicated in parentheses along with the soil descriptions. The stratification lines between strata on the logs are approximate; in situ, the transitions may be gradual.

The soil samples will be retained in our laboratory for a period of 60 days, after which, they will be discarded, unless other instructions are received as to their disposal.

3.4 ELECTRICAL RESISTIVITY TOMOGRAPHY (ERT) - KARST MAPPING

ECS Midwest, LLC subcontracted the geophysical testing (Electrical Resistivity Tomography - ERT) to PPI at this property and the results of the field tests are included in Appendix D. Please note that the ERT tests were performed with the original site layout, and additional ERT tests were deemed not necessary during supplemental test boring phase and after the layout change. Therefore, the completed ERT test lines are located outside the revised building footprint. The purpose of an electrical resistivity survey is to estimate the electrical subsurface characteristics through measurements recorded on the ground surface. Based upon the results of the measured electrical properties, estimates of the true resistivities of subsurface features can be made. These estimated values can then be correlated to various geologic parameters including saturated soils, water interfaces, rock, and void spaces.

Resistivity lines were performed with the Dipole-Dipole Testing method. Electrode spacings were at 15-foot lengths for a total resistivity lines length of approximately 315 feet. The maximum depth of the resistivity testing was approximately 72 feet near the center of the resistivity lines. Resistivity testing was performed with an Abem Terrameter LS2 resistivity meter and the system uses a total of 64 electrode positions.

Based on the ERT test results and soil borings with bedrock cores, the subject site is underlain by potentially karstic bedrock conditions, which are common below Willard, MO and the surrounding areas. Apparent areas of active sinkhole formations, caves or conduits, or clay infillings were not observed near the proposed McDonald's building or along the resistivity lines or within bedrock cores performed at the subject site. Active sinkholes generally manifest surficial signs of settlement as closed contour depressions at the site (which were not visible during site testing). Additionally, active sinkholes can generally be identified via the resistivity data as cone shaped features from the movement of surficial soils into the subsurface materials. This type of feature was not noted in the resistivity cross-sections. Although active sinkholes were not observed, the potential for future sinkhole development is present in karst bedrock formations. Potential sources of sinkhole development include areas between bedrock pinnacles, crevices within the bedrock, weathered bedrock zones, and isolated pockets of low resistance clays.

4.0 DESIGN RECOMMENDATIONS

4.1 GEOLOGIC HAZARDS

Based on the review of publicly available *Maps from Missouri Geological Survey* in the general vicinity of the proposed project, ECS has identified the potential for this site to be underlain by calcium carbonate and calcium-magnesium carbonate bedrock (i.e., Burlington-Keokuk Limestone). Based on the Missouri DNR, there are approximately sixteen (16) sinkholes/points identified in the general vicinity of the proposed site, within a 2 miles radius. The presence of carbonate-rich bedrock indicates the potential for sinkhole development at this site. Karst development in this area occurs from the dissolution of the native limestone bedrock material. Over time, groundwater can transport the surrounding soil into bedrock voids causing visible surface features such as circular depressions or areas of drainage. However, some sinkholes may not be visible from surface because they are plugged or capped with a thin layer of rock. The extent of the sinkhole occurrence depends on the geologic, hydrogeologic and soil conditions of the site and what disturbances are being created by man-made features. The image below, provided by MDNR, depicts the development of sinkholes over time:



General Sinkhole Development (Source: Missouri Department of Natural Resources)

Based on the limited geophysical testing and soil borings with bedrock cores, potential karst or karst related anomalies were not identified around the proposed McDonalds facility. The subject site showed karstic bedrock conditions (which are common in the Willard, MO area) with widely varying degrees of weathering. However, the present state-of-the-art geotechnical engineering does not permit accurate predictions of where or when sinkholes will occur in the future. Site grading should be established to provide positive drainage both during and after construction, so as to minimize the potential for sinkhole development.

4.2 GEOTECHNICAL DISCUSSION

As discussed in **Section 2.2**, fill operations of up to 8 feet are anticipated within the southern and southwestern parts of the proposed building. In addition, fill operations of up to 12 feet are anticipated within the western, southern parts of the site and around the stormwater detention pond area.

With fill operations of approximately 8 to 12 feet necessary to reach final building, pavement and pond berm grades, the new fill will undergo self-weight compression. Generally, self-weight compression is considered $\frac{1}{4}$ to $\frac{3}{4}$ percent of cohesive fill height. Self-compression of the granular fill is generally less than $\frac{1}{4}$ percent and anticipated to be complete in a short duration of time such as 2 to 4 weeks after placement. Self-weight compression takes a longer duration for the cohesive soils, generally 2 to 6 months. In addition to self-weight compression of the newly placed fill, the underlying native soils may experience consolidation settlement due to the weight of new fill. With an average thickness of 8 to 12 feet of new fill, the underlying native cohesive soils will likely experience new fill weight of approximately 900 to 1,300 psf (pounds per square feet) over the southern and southwestern part of the lot. Based on our preliminary settlement analysis, total consolidation settlements of less than $\frac{1}{2}$ inch of in-situ soils are estimated under the weight of new engineered fill. ECS strongly recommends that granular soils (sand and gravel, or clayey sands) should be utilized as new engineered fill to expedite the self-compression of the newly placed fill. In case the client plans on using cohesive soils to raise the grade, ECS recommends utilizing settlement monitoring plates at the bottom and within the new engineered fill to monitor new fill self-compression settlement and when to start constructing the slabs and pavements.

Based on the limited laboratory tests (i.e. Atterberg limits) on existing soils in upper few feet, the soils appeared to be moderately to highly plastic clays. Based on our preliminary assessment, these moderately to highly plastic soils exhibit high shrink-swell potential, and estimated potential vertical rise (PVR) could be from $1\frac{1}{2}$ to $2\frac{1}{2}$ inches.

4.3 SHALLOW FOUNDATIONS - BUILDING AND LIGHT POLES

Based on the subsurface exploration and knowledge of the local geology, the soils encountered at this site are classified as medium to higher plasticity clays. Highly plastic 'fat' clays tend to shrink and swell (exhibit potential vertical rise - PVR) and could cause unacceptable heaving of the proposed lightly loaded grade supported slabs and pavement distress if supported on highly plastic soils. Also, ECS understands that the proposed structure will be lightly loaded and dead/live loads on the foundations may not counterbalance the swelling pressure of higher plasticity clays. Therefore, some undercutting may be required at the foundation, slab and pavement areas and replacing with low volume change material (LVC).

Provided subgrades and engineered fills are prepared as recommended in this report, the proposed building structure, high-rise pylon sign and light pole foundation can be supported by shallow foundations. We recommend the foundation design use the following parameters:

SHALLOW FOUNDATION DESIGN PARAMETERS		
Design Parameter	Column Footing	Wall Footing
Maximum Net Allowable Bearing Pressure ⁽¹⁾	2,500 psf	2,500 psf
Acceptable Bearing Soil Material	Compacted LVC engineered fill over Stratum I	Compacted LVC engineered fill over Stratum I
Minimum Width	30 inches	18 inches
Minimum Footing Embedment Depth (below slab or finished grade in heated area) ⁽²⁾	30 inches	30 inches
Minimum Exterior Frost Depth (below final exterior grade)	24 inches	24 inches
Estimated Total Settlement ⁽³⁾	Less than 1 inch	Less than 1 inch
Estimated Differential Settlement ⁽⁴⁾	Less than ½ inch between columns	Less than ½ inch

Notes:

- (1) Net allowable bearing pressure is the applied pressure in excess of the surrounding overburden soils above the base of the foundation.
- (2) For frost penetration requirements.
- (3) Based on client provided structural loads. If final loads are different, ECS must be contacted to update foundation recommendations and settlement calculations.
- (4) Based on maximum column/wall loads and variability in borings. Differential settlement can be re-evaluated once the foundation plans are available.

Suitable Bearing Materials: Footing pads are recommended to be supported on select engineered fill (low volume change material - LVC) or lean concrete overlying competent natural 'fat' clay soils. Soils suitable as the subgrade for engineered fill and indirect foundation support should have parameters as noted in the following Table or greater, unless otherwise approved by the geotechnical engineer:

SUITABLE BEARING SOIL CHARACTERISTICS BELOW SELECT FILL		
Maximum Net Allowable Bearing Capacity (psf)	Cohesive Soil (Clays)	
	Consistency	Unconfined Compressive Strength
2,500	Stiff	1½ tsf or higher

Estimated Undercuts: As discussed in the previous Sections of this report, fat' clays were encountered below the surficial cover or existing superficially thin crust of lean clays at this site. **Thus, ECS recommends undercutting of the higher plasticity fat clays to a depth of 24 inches below the lightly loaded shallow building/light poles foundations and slab area, if encountered at the foundation/slab subgrade elevation.** If soft, unsuitable and fill soils are observed within the building foundations, the unsuitable soil should be undercut and removed. Undercuts should be backfilled with low volume change (LVC) engineered fill or lean concrete up to the original design bottom of footing elevation; the original footing shall be constructed on top of the new engineered fill (LVC) or lean concrete.

Due to variable limestone bedrock depths across the site, the northern building foundations could be at or near the bedrock formation. ECS recommends that the building foundations should not be simultaneously supported on soil and bedrock due to high risk of differential settlement.

Cohesive soils encountered at or near the anticipated bearing elevations are sensitive to disturbance when exposed, especially when wet. The contractor should exercise care and minimize traffic and potential disturbance to the soil. If disturbed, the exposed surfaces should be compacted with a plate compacter or jumping jack prior to the placement of reinforcing steel and concrete.

During construction, observe and test the foundation bearing grade as recommended in the **Foundation and Slab Observations** section and backfill foundation undercuts as recommended in the **Earthwork Operations** section. ECS recommends hand auger probes and appropriate bearing capacity tests be conducted during construction. Cone penetrometer tests (static or dynamic, depending on soil type) should be conducted within the probe holes or other appropriate bearing capacity tests should be conducted to evaluate the support characteristics of the soil. The suitability of the actual bearing grade is recommended to be observed and tested to check that the soils are as indicated by the borings and are suitable to support the recommended maximum net allowable bearing pressure. The soil is recommended to be tested to a depth equivalent to half the footing width or a minimum of 3 feet, whichever is deeper, below the foundation bearing grade or undercut subgrade.

Uplift Considerations on Shallow Foundations: Where foundations will be subjected to uplift loads, it may be considered that the uplift resistance of the spread foundations be developed from the dead weight of the foundations plus the compression dead load of the columns and the weight of the overlying soil material above the foundations. A unit weight of 120 pounds per cubic foot could be used for compacted on-site soils as backfill around the foundations when determining the weight of the overlying soil prism. In connection with this, the geometry the soil prism for volume computations may be defined by diagonal planes extending up from the top of the perimeter of the foundation to the ground surface by lines drawn outward laterally ½ foot and 1 foot vertically.

Foundation Lateral Loading: The foundations are also expected to be subject to lateral forces. Lateral load resistance will be developed by friction or cohesion acting at the base of foundations, and the passive earth pressure developed by the footings below-grade. Passive pressure and sliding resistance (friction or cohesion) may be used in combination in determining the total resistance to lateral loads. The parameters in the following table are recommended for lateral foundation loading.

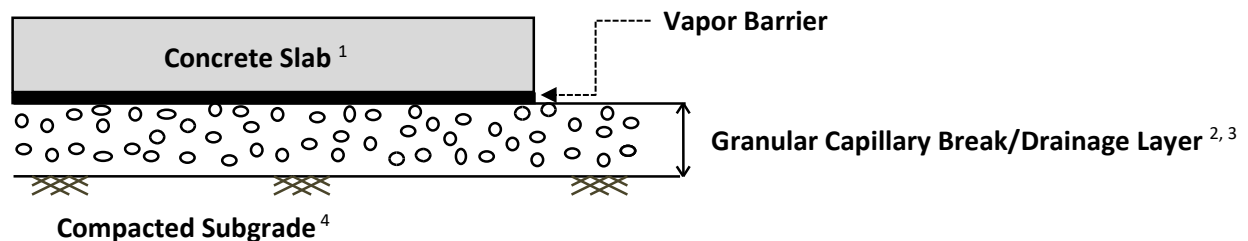
FOUNDATION LATERAL LOADING RESISTANCE PARAMETERS	
Soil Parameter	Estimated value
Coefficient of Passive Earth Pressure (K_p) - engineered fill	2.0
Soil Moist Unit Weight (γ)	120 pcf
Cohesion [<i>Foundations bearing on LVC Material</i>] (C) ⁽¹⁾	1,500
Interface Friction Angle [<i>Poured concrete on engineered fill</i>] (ϕ_i)	18°
Sliding Friction Coefficient [<i>Poured concrete on granular engineered fill</i>] (μ)	0.32
Passive Equivalent Fluid Pressure ⁽²⁾	240H (psf)

1. Cohesion may be used for lateral sliding resistance on the clay soil, but in no case should the lateral sliding resistance exceed half the dead load.
2. Passive Pressure is applicable for footings or shear key that are cast directly against natural soil or compacted engineered fill. If formwork is used for casting the footing and the footing excavation is backfilled later, passive resistance should be neglected. Wall drawings should clearly state that no form work is allowed to install wall if passive pressure is utilized for the design of the walls.

The final design of the foundation for lateral loads should be based on a minimum factor of safety against sliding of 1.5 and overturning of 2. Also, if the resultant force of the maximum vertical force does not act within the middle one-third (kern) of the footing, a smaller effective bearing area will occur and thereby result in a higher effective bearing pressure that must be accounted for in the design. The foundation could also be anchored to help resist overturning.

4.4 SLABS ON GRADE

For the construction of conventional ground-supported slabs, the recommendations provided in the sections of this report entitled **Subgrade Preparation** and **Earthwork Operations** should be followed. Based on the client provided finished floor elevation of EL. 1,224.5+ feet MSL for the proposed building structure, the building floor slab will likely be supported on new engineered fill (LVC). As discussed in the previous sections of this report, fat clays were encountered below the surficial pavement cover or undocumented fill. ECS recommends that the floor slab be supported on 24 inches of low volume change (LVC) material if highly plastic soils are encountered at the subgrade elevation. The following graphic depicts our soil-supported slab recommendations.



1. **Concrete Slab:** Minimum 4 inches thick
2. **Drainage Layer:** Minimum 6 inches thick
3. **Drainage Layer Material:** Gravel (GP, GW) having a maximum aggregate size of 1 inch and no more than 5 percent passing the No. 200 sieve.
4. **Compacted Subgrade:** 24 inches of Low Volume Change (LVC) Engineered Fill compacted to at least 98 percent of the maximum dry density per ASTM D698.

A thicker slab may be needed depending on the actual floor loads. The structural engineer should determine the actual slab thickness and steel reinforcing requirements. Provide adequate construction joints, contraction joints and isolation joints in the slab to reduce the impacts of cracking and shrinkage. Refer to ACI 302.1R04 *Guide for Concrete Floor and Slab Construction* for additional information regarding concrete slab joint design. Reinforce the slab with welded wire fabric or include an appropriate fiber mesh admixture to help control shrinkage cracking.

ECS recommends slabs-on-grade be underlain by a granular drainage layer placed on a properly prepared subgrade as recommended in the **Site Construction Recommendations** section. The granular material will serve as a capillary break, which if properly designed and installed can assist in more uniform curing of concrete.

The inclusion of a vapor retarder should be considered if the building contains moisture-sensitive floor coverings, equipment or materials. The vapor retarder will help reduce the potential of upward migration of water vapor from the soil into and through the concrete slab, which can contribute to excess humidity and microbial growth in the building. Where a vapor retarder is considered to help provide additional moisture protection, special attention should be given to the surface curing of the slabs to reduce uneven drying of the slabs and associated cracking and/or slab curling. The designer should consider the moisture

sensitivity of floor coverings and finishes, and the potential effects of slab curling and cracking when determining if the vapor retarder will be in direct contact with the slab or beneath a granular fill. The use of a blotter or cushion layer above the vapor retarder may be considered for project specific reasons. Refer to ACI 302.1R04 *Guide for Concrete Floor and Slab Construction* and ASTM E 1643 *Standard Practice for Installation of Water Vapor Retarders Used in Contact with Earth or Granular Fill under Concrete Slabs* for additional guidance on these issues.

Positive drainage around the perimeter of the proposed structures should be used to reduce the potential for water accumulation under the floor slab and foundation elements. Slope exterior grades adjacent to the building such that runoff is directed away from the building walls. Direct building downspouts away from the building walls/foundations. Direct slab and pavement surface runoff to appropriate stormwater infrastructure.

Building Slab Isolation: Isolate ground-supported slabs from the foundations and foundation-supported elements of the structure to help reduce shear and bending stresses in the floor caused by differential movement between the foundations and slab. Where the structural configuration prevents the use of a free-floating slab, design the slab with suitable reinforcement and load transfer devices to preclude overstressing of the slab.

Subgrade Modulus: Provided the subgrade is prepared, the new fill and granular drainage layer are placed as recommended in this report, and the final subgrade proofroll is satisfactory and fill is placed as recommended in the subgrade preparation of this report, design the slabs assuming a modulus of subgrade reaction, k of 150 psi/in (pounds per square inch per inch). This modulus of subgrade reaction value assumed is based on the recommended minimum drainage base thickness and correlation of index properties and soil type to historical 1 foot by 1 foot plate load tests. The modulus value used in design should be adjusted for areas larger than 1 foot by 1 foot.

Frost Susceptible Areas: Exterior sidewalks and exterior portions of the floor slab, such as at doorway entrance/exit slab may be susceptible to frost heave movement during colder weather. Additional insulation, installation of subgrade drainage, and/or replacement to the frost depth with non-frost-susceptible backfill should be considered for these areas. Pavement and ground surface grades are recommended to be sloped away from the building and flatwork, to reduce water infiltration and potential frost heave problems. In addition, all slab joints should be sealed to reduce surface water infiltration into the sub-base materials.

4.5 HIGH-RISE PYLON SIGN AND LIGHT POLE FOUNDATIONS - DRILLED SHAFTS

The proposed high-rise pylon sign and light pole structures can be supported on shallow foundations as discussed in Section 4.3. Alternatively, the proposed high-rise pylon sign and light pole structures can be supported on relatively shallow drilled shafts, bearing within the 'fat clays' and limestone bedrock. Pylon sign and Light pole structures specific information were not available at this time such as the pole heights and structural loads.

The allowable axial load carrying capacity of a drilled shaft can be computed using the static method of analysis. According to this method, allowable axial capacity, Q_a , at a given penetration is taken as the sum of the skin friction on the side of the shaft, Q_{as} , and the end or point bearing at the shaft tip, Q_{ap} , so that:

$$Q_a = Q_{as} + Q_{ap} = f A_s + q A_p$$

where A_s and A_p represent, respectively, the embedded perimeter surface area and the end area of the shaft; f and q represent, respectively, the allowable unit skin friction and the allowable unit end or point bearing. The total allowable axial capacity in compression will be the summation of the allowable frictional capacity and the allowable end bearing capacity. The total allowable axial capacity in tension (uplift) will be the allowable frictional capacity of the shaft neglecting the end bearing component. Based on the project characteristics, the horizontal (shear) force and overturning moment will govern the design (depth and size) of the shaft. Steel reinforcing steel must be designed by the structural engineer to resist the referenced loads.

The Tables below summarize the soil design parameters (based on the soil conditions observed in soil borings S-01 and S-02) which should be used for uplift and lateral stability analyses based on the LPILE computer programs. Due to the disturbance during shaft installation, frost depth and wetting/drying cycles, ECS recommends that the upper 30 inches be ignored for the design of the foundation system.

ALLOWABLE SKIN FRICTION AND BEARING COMPONENTS IN COMPRESSION AND UPLIFT				
Soil Type	Approximate Depth below the Existing Grade (ft)	Allowable Unit End Bearing (psf)	Allowable Downward Axial Unit Skin Friction (psf)	Allowable Uplift Unit Skin Friction (psf)
Lean Clay - Fat Clay	0 to 2	N/A	N/A	N/A
Fat Clay	2 to 13	4,500	380	280
Limestone	3½ to >13	45,000	4,400	3,500

Note: Unit end bearing and side friction values were estimated using AASHTO LRFD Bridge Design Specifications 9th Edition

LPILE SOIL PARAMETERS FOR LATERAL LOAD ANALYSIS							
Soil Type	Depth Below the Existing Ground Surface (ft)	"p-y" criteria	Effective Unit Weight (pcf)	Insitu Shear Strength S_u (psf)	Unconfined Compressive Strength (psi)	Lateral Modulus K_s, K_c (pci)	Strain Factor $\epsilon_{50/K_{rm}}$
Fill - Lean Clay, Fat Clay	0 to 2	Neglect					
Fat Clay	2 to 13	Stiff	120/58 ²	1,500	N/A	$K_s=500$ $K_c=200$	0.007
Limestone	3½ to >13	Rock	140/78 ²	N/A	5,420 to 16,690	N/A	0.0005

Note: S_u -Undrained Shear Strength (psf); K_s/k_c -modulus of subgrade reaction (pci) for static/cyclic loading condition; and $\epsilon_{50/K_{rm}}$ - strain corresponding 50% of the maximum stress.
¹Neglect upper 2 feet of soil for lateral load analyses.
²Buoyant Unit Weight of soil. Groundwater was not encountered in test borings. However, if encountered, use buoyant unit weight.

4.5.1 Drilled Shaft Excavation/Installation Considerations

Groundwater was encountered at two (2) of the test borings (i.e., borings B-06 and S-02) during drilling operations. However, perched water may be encountered during drilling and shaft installation. The use of

a qualified contractor experienced in dewatering wet soils with shallow groundwater and installing the proposed type of drilled shaft foundations for this project is mandatory in order to reduce the potential for construction problems relative to cave-ins and 'running ground' conditions (ACI 304R, Chapter 8, Concrete Placed Under Water). A temporary steel casing is recommended to control these adverse ground issues and frequent gravel/chert beddings. It is also recommended that the geotechnical engineer who has knowledge of the design, and the geologic and subsurface conditions, should observe the drilled shaft excavation, construction and backfilling of concrete.

Weathered limestone was encountered in test borings ranging from approximately 1 to 13½ feet below existing grades. ECS recommends a minimum of 5 feet of bedrock socket within the weathered limestone bedrock to resist uplift, shear and moments. The bearing of the bedrock socket should be measured beginning at the point where continuous bedrock exists along the entire shaft perimeter. The bedrock socket should be observed by the third-part testing firm. The socket should be continuous around the drilled shaft diameter and should be clean. Where the required uplift loads cannot be safely resisted through side friction (i.e. shallow drilled shafts), rock anchors may be required.

Completed drilled shaft excavations should have a planar bottom for axial bearing considerations. Any loose material, disturbed material and water should be removed from the drilled shaft excavation prior to concrete backfilling. Drilled shaft excavations should not be allowed to stand open for any significant length of time because this may allow water to accumulate. Based on the expected drilled shaft depth, water may be encountered. The maximum depth of water at the bottom of the shaft excavation should not exceed 3 inches prior to concrete placement. Use of drilling mud, downhole dewatering, bentonite or polymer slurry and/or tremie concrete construction methods are expected because shallow groundwater was encountered at the boring location and cannot be managed using temporary casing only. Another alternative may be to use a larger diameter shaft to help reduce drilled shaft embedment depths needed. Sidewalls are recommended to be visually free of cuttings. A clean-out bucket should be provided to permit manual removal of all loose or disturbed soils within the drilled shaft excavation.

The potential for significant caving of shaft sidewalls in frequent chert/gravel beddings exists and temporary steel casing and/or slurry will be required during drilled shaft excavation. Care must be taken while removing liners or temporary casing during concrete placement so the head of concrete inside the casing is at all times greater than the earth pressure and hydrostatic head outside the casing. The concrete mix for the drilled shaft should be designed and placed to reduce the potential for arching of the concrete during removal of the casing. The temporary steel casing should be extended a minimum 3½ feet above the ground surface for safety and to reduce the potential risk of accidental fall-in of foreign materials and personnel into the excavation hole.

Concrete should consist of a Portland cement mixture properly air-entrained with an appropriate water to cement ratio for proper strength and durability. Because of the presence of water bearing granular soils, placement of concrete by tremie method is expected to be required. ECS recommends a slump of 5 to 7 inches if tremie placement is required. Care must be exercised so the concrete will not contact the reinforcing cages during placement otherwise segregation of the concrete will occur. If the mixture is too stiff, reinforcing cages may be pulled up when liners are withdrawn.

The drilled shaft should have a minimum shaft diameter of 24-inches to help reduce arching and the development of possible voids in placement of the insitu concrete. The shaft must also contain reinforcing steel (designed by the structural engineer) to resist all combinations of loadings and to satisfy the applicable building codes.

4.5.2 Competent Bedrock Verification Testing

The drilled shaft foundation should be evaluated by a third-party testing firm to determine the adequacy of the bearing material via a down-hole inspection or bedrock coring. The adequacy of the bearing level in bedrock formation will be based upon the absence of voids, highly weathered zones, clay infillings and seams within the first 4 feet of the test hole. Individual seams, if present below the first 4 feet should not be greater than ¼ inch, and the total of seams within the remainder of the test hole should not exceed 1 inch.

4.6 SEISMIC DESIGN CONSIDERATIONS

Seismic Site Classification: The International Building Code (IBC 2018) and ASCE 7-16 requires site classification for seismic design based on the upper 100 feet of a soil profile. The three parameters used to classify sites are shear wave velocity (v_s); undrained shear strength (S_u); and Standard Penetration Test (SPT) resistance (N-value). The seismic Site Class definitions for the weighted average of shear wave velocity, shear strength or SPT N-value in the upper 100 feet of the soil profile are listed below:

SEISMIC SITE CLASSIFICATION				
Site Class	Soil Profile Name	Shear Wave Velocity, V_s , (ft./s)	N value (bpf)	Undrained Shear Strength, S_u , (psf)
A	Hard Rock	$V_s > 5,000$ fps	N/A	N/A
B	Rock	$2,500 < V_s \leq 5,000$ fps	N/A	N/A
C	Very dense soil and soft rock	$1,200 < V_s \leq 2,500$ fps	>50	> 2,000
D	Stiff Soil Profile	$600 \leq V_s \leq 1,200$ fps	15 to 50	1,000 to 2,000
E	Soft Soil Profile	$V_s < 600$ fps	<15	< 1,000

The maximum explored depth in the present subsurface exploration was 23½ feet below current site grades. As such, based upon our interpretation of the subsurface conditions and our experience with similar geological settings, the appropriate Seismic Site Classification is "C" as shown in the preceding Table.

4.7 PAVEMENT RECOMMENDATIONS

Subgrade Characteristics: Given the client provided grading plan, cut operations of less than 3 feet and fill operations of up to 12 feet are anticipated for this site within the proposed pavement areas.

Extensive laboratory and field testing were not part of ECS scope for the pavement section design and recommendations. Assuming that the subgrade of the proposed pavements is prepared as recommended in this report, the rigid pavements may be designed for an estimated modulus of subgrade reaction, k_{v1} of 125 psi/in (pounds per square inch per inch) using 12 inches of low volume change material and flexible pavements may be designed using an estimated California Bearing Ratio (CBR) of 3.5.

As stated in the earlier Sections of this report, highly plastic 'fat' clays are encountered at the pavement subgrade elevation. Therefore, ECS recommends undercutting 12 inches of existing 'fat' clay soil below the pavement subgrade elevation, if encountered, and replacing it with low volume change material/low-plasticity soils (LVC).

Pavement Sections: The recommended minimum pavement sections presented in this report were based on the client provided traffic loading data. However, the project civil engineer should confirm the traffic loads and if changes are required, please provide ECS with the actual 18 kips ESAL loadings so that we can revise our pavement section recommendations. Given the client provided traffic data, ECS has **estimated** the following 18-kips ESAL's loads for the flexible and rigid pavement section design:

ESTIMATED 18-KIPS ESAL's	
Pavement Section	Total 18-kip design ESALs
Standard-Duty Auto Areas	11,000
Heavy-Duty Truck Stalls	30,000

The recommended minimum pavement sections listed in the Table below are based on the anticipated usage at the project site with a design life of 20 years. If the anticipated traffic will exceed that assumed, ECS should be contacted for revised pavement design recommendations; otherwise, increased pavement maintenance and a shortened pavement life should be expected. The new flexible and rigid pavement materials and construction should be in accordance with the Guidelines for AASHTO Pavement Design and applicable Missouri Department of Transportation (MoDOT) Standards.

PROPOSED PAVEMENT SECTIONS				
MATERIAL	FLEXIBLE PAVEMENT (CBR=3.5)		RIGID PAVEMENT (Kv=125 psi/in)	
	Light-Duty	Heavy-Duty	Light-Duty	Heavy-Duty
Portland Cement Concrete ($f'_c = 4000$ psi)	-	-	5 in.	6 in.
Bituminous Surface Course ¹	1½ in	2 in	-	-
Bituminous Binder Course ²	2½ in	3 in	-	-
Crushed Aggregate Base ³	8 in	10 in	6 in	6 in
TOTAL	12 in	15 in	11 in	12 in

1. According to Section 401 of MoDOT Standard Specification for Highway Construction.
2. According to Section 401 of MoDOT Standard Specification for Highway Construction.
3. According to Section 304 of MoDOT Standard Specification for Highway Construction. If crushed gravel or some other material is used in lieu of crushed stone, the material may have a lower structural coefficient, and a thicker base may be required.

Crushed Aggregate Base Course Compaction: The aggregate base course is recommended to be compacted to at least 98 percent of the maximum dry density obtained in accordance with ASTM D 698, Standard Proctor Method. Prior to placing the granular material, the pavement subgrade soil should be properly compacted, observed to be stable during a final proofroll and free of standing water, mud, and frozen soil.

Hot Mix Asphalt Compaction: The hot mix asphalt should be compacted to a minimum of 93 percent of the maximum theoretical density value.

Rigid Pavements: An air-entrained concrete mix (compressive strength of at least 4,000 pounds per square inch at 28 days) fiber-reinforced concrete is recommended for rigid pavements. Adequate

construction joints, contraction joints and isolation joints should be provided in the areas of rigid pavement to reduce the impacts of cracking and shrinkage. Please refer to ACI 330R-92 *Guide for Design of Concrete Parking Lots*. The Guide recommends an appropriate spacing strategy for the anticipated loads and pavement thickness. It has been our experience that joint spacing closer to the minimum values results in a pavement with less cracking and better long-term performance. Control joint spacing should be determined in accordance with the current ACI code. Expansion joints should be provided where pavement abuts fixed objects, such as the buildings and light poles.

Weather Restrictions: Daily temperatures from mid-November to April can often stay below 40°F, limiting the days that asphalt placement can occur. In this region, asphalt plants may close during the months of December, January, and/or February if particularly cold weather conditions prevail. However, this can change based on year-to-year temperature fluctuations.

Pavement Drainage: An important consideration with the design and construction of pavements is surface and subsurface drainage. Where standing water develops, either on the pavement surface or within the base course layer, softening of the subgrade and other problems related to the deterioration of the pavement can be expected. Based on our estimated groundwater level, ECS considers surface water infiltration to be the main source of water to be considered for pavement design on this project.

Shape or crown the final pavement surface to properly direct surface water to suitable on or off-site stormwater drainage infrastructure. Properly slope the clay pavement subgrade to avoid dips or pockets where water may become trapped. Dips in the clay subgrade could result in a “bathtub” effect, which may trap water and potentially soften the subgrade. The subgrade in areas requiring undercut and backfill with granular soils are recommended to be graded to drain toward a drain tile. The drain tile should be sloped a minimum of ½ to 1 percent to discharge to nearby storm sewers, drainage ditches or other appropriate drainage facilities. Install edge drains where site grades slope toward the pavement edge to reduce the potential for water to enter the base course layer. Slope edge drains to the nearest appropriate drainage facility. Water that ponds on the subgrade surface can lead to deterioration of the subgrade soils, reduction of the base course support characteristics and pavement heave. Good drainage should help reduce the possibility of the subgrade materials being wet over a long period of time. Water should not be allowed to pond behind curbs and saturate the pavement base stone. In downgrade areas, base stone should extend through the slope face to allow water entering the base stone a path to exit.

Pavement Maintenance: A sound maintenance program should be implemented to help maintain and enhance the performance of pavements and help attain the design service life. A preventative maintenance program should be started early in the pavement life to be effective. The “standard in the industry” supported by research indicates that preventative maintenance should typically begin within 2 to 5 years of the placement of pavement. Failure to perform preventative maintenance will reduce the service life of the pavement and increase the costs for corrective maintenance and full pavement rehabilitation. Seal joints and cracks with elastomeric caulk in a timely manner to help reduce water infiltration through the pavement section into the base course layer, which may result in softening of the subgrade and deterioration of the pavement. Observe pavements for distresses, such as cracks, depressions and poor drainage, at least twice a year, typically once in the spring and once in the fall.

Shrinkage cracking is common with asphalt and occurs with age. Development of cracks should be expected with normal exposure to weather, wear and age. These cracks may become larger when exposed to such things as weather and vegetation growth and should be treated promptly.

4.8 DETENTION POND RECOMMENDATIONS

One of the test borings (P-07), originally completed for the pavement areas, is located within the proposed stormwater detention pond. Based on the client provided grading plan, the soils at bottom the proposed stormwater detention ponds (EL. 1,213 feet MSL) are anticipated to consist of stiff to very stiff lean clays and highly plastic gravelly 'fat' clays with possible chert. A compacted clay liner (CCL) will likely be necessary due to the presence of frequent gravel and chert beds. The clay liner is recommended to be at least 2 feet in thickness. The onsite lean clay or fat clays (without large size gravel) encountered at this site may be feasible to use as liner but should be further evaluated and tested prior to its use. Clay soil used as liner is recommended to consist of material which has the following properties:

- A minimum of 50 percent, by dry weight, of soil passing the No. 200 sieve;
- An in-place saturated hydraulic conductivity of 1×10^{-7} cm/sec or less;
- An average liquid limit of 35 or greater, with no value less than 20;
- An average plasticity index of 15 or greater with no values less than 10;
- Compacted at a moisture content 2 to 4 percent above the optimum moisture content determined by Standard Proctor (ASTM D698) to a minimum of 95 percent of maximum dry density;
- Compacted with a sheep-foot compactor weighing at least 25,000 pounds, operated continuously, in loose lift thicknesses not to exceed the smaller of 6 inches or the length of the teeth on the footed compactor used;
- Less than 5 percent loss on ignition organic content per ASTM D2487; and
- The material must not contain deleterious materials.

For construction of the clay liners, it may be necessary to bench side walls of the ponds horizontally, with 1 to 3-foot vertical steps. This would allow horizontal placement and compaction of the liner section. However, adequate compaction for the purpose of detention is probably possible for cohesive fill placed in lifts parallel to the cut slope. Permanent slopes should be constructed at 3(H) on 1(V) or flatter, and erosion control measures should also be used. Where inlet and outlet pipes are constructed, measures are recommended to be taken to reduce seepage along conduits. Typical acceptable measures include the use of granular filters, anti-seepage collars or bentonite.

5.0 SITE CONSTRUCTION RECOMMENDATIONS

5.1 SUBGRADE PREPARATION

5.1.1 Stripping and Grubbing

The subgrade preparation should include stripping existing pavements, vegetation, and other soft or otherwise unsuitable materials from the 10-foot expanded building limits and 5-foot expanded pavement limits where feasible. A third-party testing firm should be retained to observe and document that existing pavements and other unsuitable surficial materials have been removed prior to the placement of engineered fill or construction of structures. Based on the test boring data, a stripping depth of approximately 4 to 18 inches is estimated. However, deeper depths may be anticipated in other areas not explored by the test borings. ECS does not recommend the subgrade soil remain exposed to the elements or construction traffic for a prolonged period of time as the subgrade may become disturbed and/or softened.

5.1.2 Highly Plastic Soils

Based on the visual classification and limited laboratory testing of the retrieved soil samples from the test borings and local knowledge of the area, ECS anticipates the soils at the foundation/slab/pavement subgrade elevation may consist of highly plastic fat clays. Due to the potential for expansive soils in the project site, ECS recommends removing 24 inches of existing 'fat clays' soils below the proposed lightly loaded foundations and floor slab subgrade elevation and 12 inches below the pavement subgrade elevations and replacing it with low volume change material/low-plastic engineered fill (LVC) to reduce the risk for movement due to subgrade soil shrink-swell behavior. If this option is deemed not feasible from a cost perspective within the pavement areas, the project team may choose to leave the existing highly plastic 'fat clay' soils in place, provided they proofroll satisfactorily. However, for this option, an increased risk of pavement distress and increased maintenance costs should be assumed. Based on FHWA Figure 5-34 (Geotechnical Aspects of Pavements Reference Manual Chapter 5.0), ECS estimated a potential vertical rise (PVR) in the range of 1½ to 2½ inches.

Alternatively, the existing highly plastic fat clays could be moisture conditioned at and above the optimum moisture content or lime stabilized to lessen the shrink-swell potential within the pavement areas.

5.1.3 Existing Utilities

Locate existing utilities. Relocate utilities planned to be maintained outside the proposed building area, if possible. For utilities not reused, cap-off and removed, or properly abandon in-place in accordance with applicable local codes and ordinances. Backfill excavations for utilities to be removed in the influence zone of new constructions with engineered fill. Grading operations must be done carefully so that existing utilities are not damaged or disturbed. Check utility invert elevations, depths, and sizes relative to the planned foundation elevations to determine what specific concerns are present.

5.1.4 Proofrolling

After removing all unsuitable surface materials, excavating to the proposed grade, and prior to the placement of any engineered fill or other construction materials, the exposed subgrade should be observed by a third-party testing firm. The exposed subgrade should be thoroughly proofrolled with previously approved construction equipment having a minimum axle load of 10 tons (e.g., fully loaded tandem-axle dump truck). The areas subject to proofrolling should be traversed by the equipment in two perpendicular (orthogonal) directions with overlapping passes of the vehicle under the observation of a

third-party testing firm. This procedure is intended to assist in the identification of yielding materials. In the event that unstable or "pumping" subgrade is identified during the proofroll, those areas should be marked for repair prior to the placement of any subsequent engineered fill or other construction materials.

If construction occurs during wet times of the year (such as during the spring or fall months) or immediately following extended periods of rain, then seasonal reduction of the near surface soil strength will occur. This may cause additional unstable or pumping subgrade areas for constructability concerns. The high moisture content fat clay materials present near the surface may not pass a proofroll and may need to be undercut or repaired. Some undercutting or repair of unstable subgrade soils should be anticipated during slab and pavement subgrade preparation. The actual quantity of the subgrade undercut, or stabilization should be determined at the time of construction.

Undercut or repair of unstable subgrades to establish a suitable support condition may be needed. The improvement method chosen may be influenced by several factors such as weather and schedule, as well as the area, depth and nature of the unstable subgrade soils. Depending on the aforementioned and other factors, subgrade repair methods may include:

Scarification and Compaction: Soils can be scarified, moisture conditioned (i.e., dried or wetted) to within a narrow range of the material's optimum moisture content and compacted. Scarification and compaction are generally most applicable where very shallow unstable conditions are encountered and at times when the soil can be properly dried or wetted to within a narrow range of the material's optimum moisture content.

Undercut and Replacement: ECS recommends soft or yielding soils be evaluated in approximately 6 to 12-inch intervals to help limit the volume of undercuts. If soft or yielding soils are identified, the contractor should remove only 6 to 12 inches of material at a time in the subject area and then proofroll/evaluate the undercut subgrade to determine if additional undercut is needed. This may take more time but could potentially reduce the removal of more soil than necessary.

Chemical Modification: Alternatively, if these soils cannot be stabilized by conventional methods, chemical modification of the subgrade soils, such as with lime, lime kiln dust, cement, cement kiln dust, or other materials, may be utilized to reduce the moisture content and/or provide additional stabilization. An experienced pre-qualified contractor that has successfully chemically modified similar-sized projects with similar soil conditions is recommended to be used. The soil modification procedure, such as determination of the type and quantity of additive, and mixing and curing procedures, should be evaluated before implementation. This evaluation may include testing the soil for pH, resistivity, sulfates, and chloride to check if an adverse chemical reaction could occur. The contractor should be required to minimize dusting or implement dust control measures. For preliminary estimating purposes, the approximate incorporation rate (based on dry weight of soil) is typically in the range of 6 to 7 percent for hydrated lime or lime by-products, and 5 to 7 percent for Portland cement. Typically, the percentage needed is less for hydrated lime than other lime byproducts because the available calcium oxide content of lime byproducts tend to be lower. Alteration of the pavement section to include additional drainage may be needed if a chemically stabilized subgrade is used.

5.1.5 Site Temporary Dewatering

Limited Excavation Dewatering: Based upon our subsurface exploration at this site, we believe construction dewatering at this site will be mainly limited to removing accumulated rainwater or perched water if it becomes an issue. The project team should consider contingencies during construction to deal with perched water during the construction of the proposed foundations in case it is encountered in unexplored areas of the project site.

5.2 EARTHWORK OPERATIONS

5.2.1 Shallow Bedrock

Based on the test boring data obtained during the exploration, relatively shallow limestone bedrock was encountered in test borings at depths ranging from approximately 1 to 13½ feet below the existing grades (less than 2½ feet near northeast/eastern part of the proposed building). Thus, the shallow bedrock excavation may be required during site grading and foundation excavation to planned subgrades. The excavation of weathered limestone rock can have a substantial impact on the cost and schedule of the proposed construction. ECS recommends that the contractor should make their own assessment of rock excavation. ECS believes that a larger excavator equipped with rock-teeth on the bucket can be used to excavate the weathered rock.

5.2.2 Engineered Fill

Product Submittals: Prior to placement of engineered fill, representative bulk samples (typically at least 50 to 100 pounds) of off-site borrow should be submitted to a third-party testing firm for laboratory testing, which may include natural moisture content, organic content, grain-size distribution, Atterberg limits, and moisture-density relationships for compaction. Imported material should be tested prior to being hauled to the site to determine if it complies with project specifications. Alternatively, Proctor data from other accredited laboratories can be submitted if the test results are within the last 90 days.

Satisfactory Engineered Fill Materials: Engineered fills should consist of materials free of debris with the following engineering properties:

ENGINEERED FILL INDEX PROPERTIES		
Subject		Property
Plasticity	Upper 4 feet in Building Areas and Upper 2 feet in Pavement Areas	LL ≤ 40, PI ≤ 15
	Below 4 feet in Building Areas and Below 2 feet in Pavement Areas	LL ≤ 50, PI ≤ 20
Max. Particle Size		3 inches
Max. Organic Content		5 percent

Open-graded materials, such as coarser sands, and gravels (SP and GP), which contain increased void space in their mass may need to be encapsulated within a filter geotextile. If the fill is to provide low-frost susceptible characteristics, it must be classified as a clean GW and GP, or coarser SW or SP per Unified Soil Classification System (ASTM D-2487) and must be properly drained.

Unsatisfactory Materials: Unsatisfactory fill materials, which do not satisfy the requirements for suitable materials, include topsoil and organic materials (PT, OH, and OL), frost susceptible silt (ML), and high plasticity soils elastic silt (MH) and fat clay (CH).

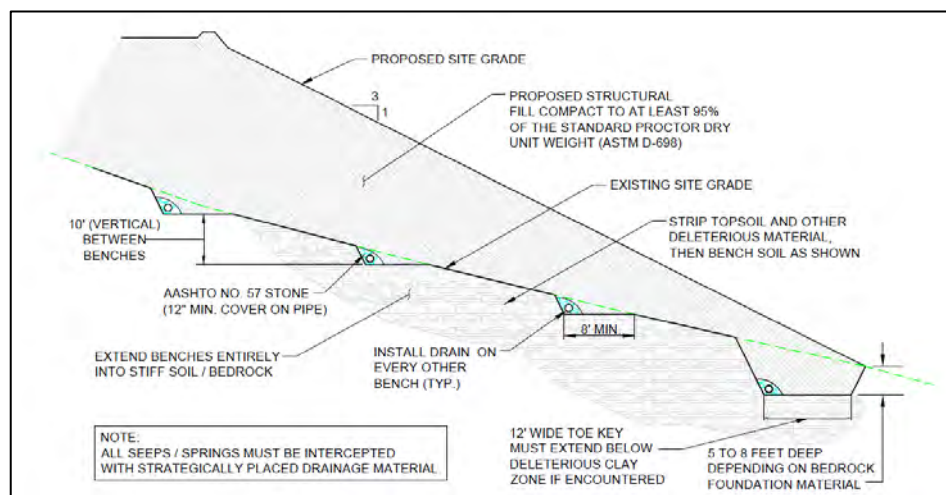
Pea gravel is not recommended to be used as engineered fill. Pea gravel has round/smooth characteristics, no fines and does not interlock when compacted, which makes it more susceptible to future movement and instability resulting in excessive and variable settlement.

On-Site Borrow Suitability: On-site soils consisting of fat, highly plastic soils (CH) are not recommended to be reused as an engineered fill due to their inherent high shrink-swell potential. However, clean lean clays can be used as engineered fill after testing. However, based on the amount of fill to raise the existing grades, ECS understands that imported fill (LVC) will be required.

5.2.3 Compaction

Subgrade Benching: Place fill material in horizontal lifts. Based on the client provided grading plan, new engineered fill up to 12 feet is anticipated (up to 8 feet in building area) at this site. Special benching should be used whenever the designer anticipates that there will be a stability problem and/or weak soils in an existing slope. Special benching is typically utilized to improve stability of fill placed on an existing slope or to remediate an unstable existing slope.

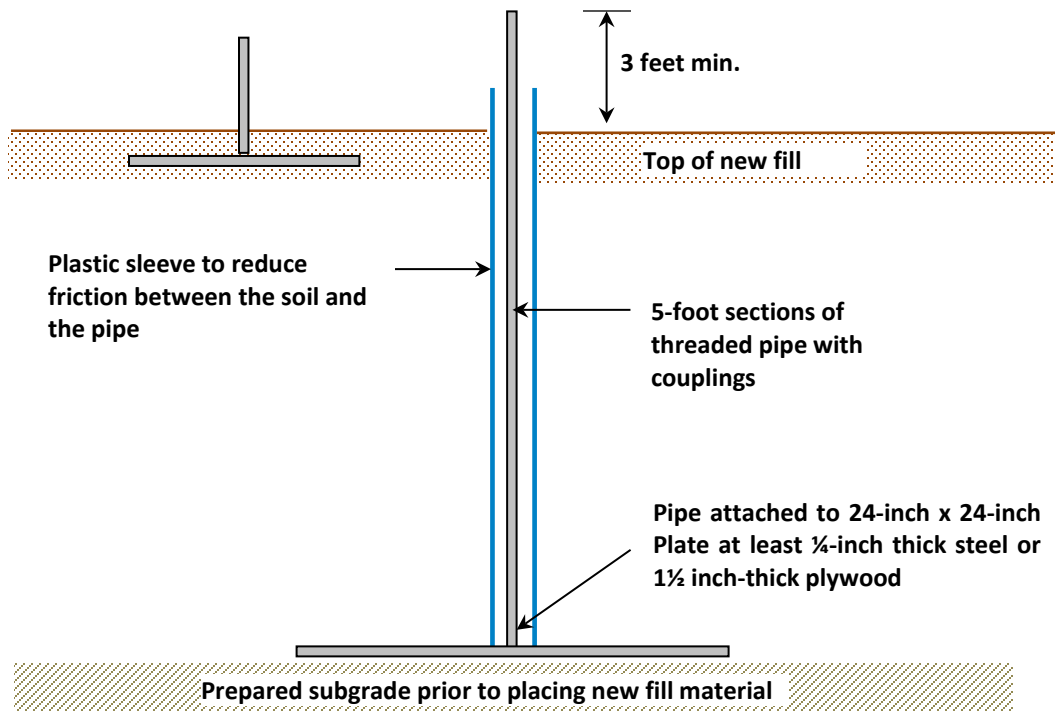
Where slope fills are planned on the face of an existing slope, it is typically recommended to use special benching to “knit” the embankment together. Compaction of fill soils placed upon an existing embankment is difficult. Conventional compaction equipment may not be used as on typical highway embankments, which means that fill placed directly on such a slope will generally be either poorly compacted or uncompacted, which leaves it highly susceptible to erosion or sliding failure. Even if compaction can be performed in such a fill, the sloping interface between the existing surface and the new fill provides a potentially weak continuous plane along which a shear failure may develop. Special benching creates a “stair-stepping” surface, which improves stability by inhibiting the development of a contiguous shear plane along the interface. Place and compact fill on a 3 (H):1 (V) or flatter slope, or step or bench as required to flatten.



General Benching Detail

Self-Weight Compression and Monitoring of New Fill: Relatively deep fill is expected to be required in a portion of the site. Engineered and compacted fill can still experience self-weight compression. In most cases, the amount of self-weight compression is negligible, but when fill heights exceed about 8 to 10 feet, the magnitude of self-weight compression can affect the performance of the proposed structure. For properly placed and compacted cohesive fill soils, self-weight compression can be approximately $\frac{1}{4}$ to $\frac{3}{4}$ percent of the new fill height, typically less than $\frac{1}{4}$ percent for granular fills. Self-weight compression settlement is in addition to the settlement induced by new structure load. Placement and compaction of new engineered fill should be performed as early as possible to allow as much of the self-weight compression of the fill to occur before construction of buildings, pavement, and other construction. For cohesive engineered fill 8 feet or greater, ECS recommends settlement plates and monuments be installed to observe movement and help determine when settlement induced by the new fill and the compression of the engineered fill has stabilized. The effects of self-weight compression can be both reduced and accelerated by using granular fill.

Settlement plates should be set on natural ground and within the new engineered fill to help estimating and differentiating the settlement of the in-situ soil strata and settlement of newly placed fill. Also, various types of settlement indicators are available for measuring settlement, consisting of mechanical, electrical or fluid instruments. One mechanical type is settlement plates constructed as conceptually shown below.



An initial reading of the first set of settlement plates installed on the existing grade should be made before filling placement. During fill placement readings are needed each time an extension is added to the settlement plate. Once the fill has been fully constructed, the second set of settlement plates would be installed at the new finished grade and then readings on both sets of settlement plates made. The settlement plate movement should be observed regularly by measuring the plate and ground surface elevations. The total duration of the plate readings would be a function of the observed movement, if any. If ECS is not scoped with observing the settlement plates, ECS should be provided with the settlement monitoring data for analysis and evaluation of when construction may proceed. We estimate at least 1

month of settlement data will be needed after fill placement to begin preliminary evaluation of settlement, and to refine estimates of future reading frequency and waiting time, if needed.

The elevations of the instruments and adjacent ground surface elevations should be referenced to a fixed datum outside the influence of the construction and should be established prior to filling. We recommend settlement plate elevations be measured to the nearest 0.01 foot, and the top of adjacent ground surface elevation be measured to the nearest 0.1 foot.

It is important that care be taken during construction so that the geotechnical instrumentation is not disturbed or damaged, since it plays an important part in determining when construction may proceed. It may be necessary to use hand-operated compaction equipment near settlement plates to achieve the required in-place density and reduce possible disturbance or damage to the instrumentation.

Engineered Fill Compaction: Place and compact engineered fill in appropriate thickness loose lifts as recommended below. Beyond these areas, achieve compaction of at least 90 percent. Give as much importance to the moisture content requirements of the material as the density requirements during placement and compaction considering the moisture sensitivity of the soil.

ENGINEERED FILL COMPACTION RECOMMENDATIONS		
Subject		Recommendation
Compaction Standard		Standard Proctor, ASTM D698
Recommended Compaction		≥ 98 percent of Max. Dry Density
Moisture Content	Fine-grained	-1 to +3 % points of the material's optimum value
	Coarse-grained	-3 to +3 % points of the material's optimum value

Compaction equipment suitable to the material type being compacted should be used. Sheepsfoot compaction equipment is typically suitable for fine-grained soils (clays). A vibratory steel drum roller is typically used for compaction of coarse-grained soils (sands and gravels) as well as to help seal compacted surfaces.

The maximum loose lift thickness depends upon the type of compaction equipment used. For isolated excavations around footing locations or within utility excavations, a hand tamper will likely be required. Listed below are generally recommended maximum loose lift thicknesses for compaction based on the utilized compaction equipment.

RECOMMENDED PRELIMINARY LOOSE LIFT THICKNESSES	
Equipment	Maximum Loose Lift Thickness (inches)
Large/Heavy, Self-Propelled Equipment	8
Small, Self-Propelled, or Remote Controlled (Rammax, etc.)	6 to 8
Hand Operated (Plate Tamps, Jumping Jacks, Wacker-Packers)	4 to 6

- Density testing during fill placement is important to check and document that the specified compaction is being achieved. Thinner lifts and/or more compactive energy may be needed to achieve the required degree of compaction.

In confined areas such as utility trenches, portable compaction equipment and thin lifts of 4 inches or less may be required to achieve specified degrees of compaction.

Fill Compaction Control: The expanded limits of the proposed construction areas should be well defined, including the limits of the fill zones for pavements, etc., at the time of fill placement. Maintain grade control throughout the filling operations. Filling operations are recommended to be observed on a full-time basis by a third-party testing firm to check and document that the minimum compaction requirements are being achieved. The recommended minimum frequency for field density testing of fills is listed in the Table below but should not be less than three tests per lift.

Location	Frequency of Tests
Expanded Building Limits	1 test per 2,500 sq. ft. per lift
Pavement Areas	1 test per 10,000 sq. ft. per lift
Utility Trenches	1 test per 200 linear ft. per lift
Other Non-Critical Areas	1 test per 10,000 sq. ft. per lift

Fill Placement Considerations: Do not place fill materials on frozen soils, on frost-heaved soils, on excessively wet soils, or soils that are otherwise unstable. Borrow fill materials should not contain frozen materials at the time of placement, and all frozen or frost-heaved soil should be removed prior to placement of engineered fill or other fill soils and aggregates. Excessively wet soils or aggregates should be scarified, aerated, and moisture conditioned.

Grade fill areas at the end of each workday to help facilitate drainage of precipitation and seal the surface by use of a smooth-drum roller to reduce infiltration of surface water. During placement and compaction of new fill at the beginning of each workday, the contractor may need to scarify existing subgrades to an approximate depth of 4 inches to reduce the potential for a weak plane to form between the new fill and the existing subgrade soils.

Drying and compaction of wet soil is typically difficult during the cold, winter months. Accordingly, earthwork should be performed during the warmer, drier times of the year, if practical. Proper drainage should be maintained during the earthwork phases of construction to prevent ponding of water which tends to degrade subgrade soils. Alternatively, if these soils cannot be stabilized by conventional methods as previously discussed, modification of the subgrade soils, such as with lime, Portland cement, or other materials may be utilized to adjust the moisture content.

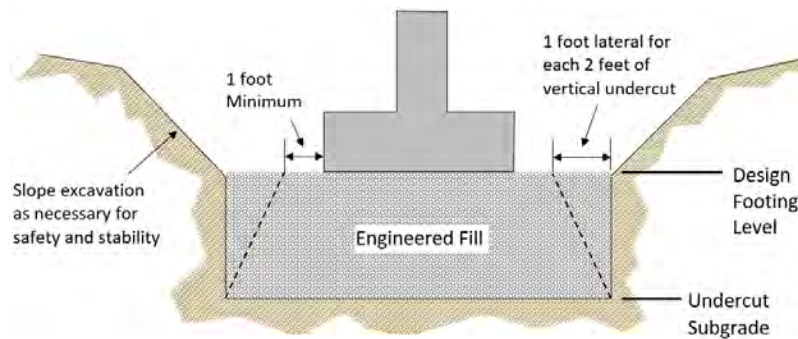
Have equipment readily available during earthwork for both drying and wetting of fill soils. We do not anticipate significant problems in controlling moisture within the fill during dry weather, but moisture control may be difficult during winter months or extended periods of rain. Controlling the moisture content of clayey and silty soils can be difficult. Further, these soils are easily degraded by construction traffic when the moisture content is elevated.

5.2.4 Weather Considerations

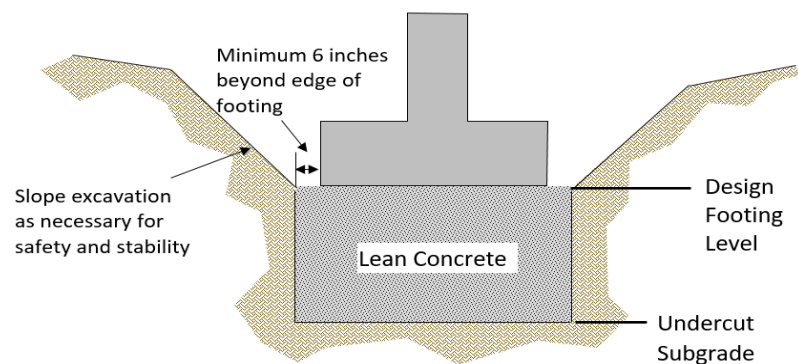
Conducting site work during periods of cool and/or wet weather (typically November to May) can be problematic for sites in the project region. Proper compaction of clay fill generally is very difficult to achieve during periods of cool and/or wet weather. Some drying, mixing, or chemical treatment of the soils would be necessary to obtain workable moisture contents for the on-site soils or proposed borrow materials if placed during the cool, wet seasons. If compaction of clay fill takes place under wet weather conditions, increased earthwork costs, an extended construction schedule, and soil improvement (likely chemical stabilization) likely would be required. In addition, reuse of the site soil may be severely limited.

Surface soils tend to be softer during wet weather conditions due to the excess moisture in the near surface soils. Weather-softened surface soils tend to result in more undercutting and/or stabilization than would be required during dry weather conditions, which increases site development costs. Project specifications should include definitions and require contractors to provide unit rates for subgrade stabilization, removal of inadequate soils, and replacement of inadequate soils with engineered fill appropriate for use during the anticipated construction season.

Engineered Fill Below Foundations: Unsuitable bearing soils encountered at the proposed foundation bearing grade or within the foundation influence zone are recommended to be removed to a suitable bearing subgrade and to a lateral extent, as conceptually shown below. Also, ECS recommends removing 24 inches of fat clays, if encountered, with low volume change material below the foundation bearing grades to minimize the shrink-swell of lightly loaded structure foundations. The zone of the engineered fill placed below the foundations is recommended to extend 1 foot beyond the outside edges of the footings and from that point, outward laterally 1 foot for every 2 feet of fill thickness below the footing.



Alternatively, backfill undercuts with lean concrete ($f'_c \geq 1,000$ psi at 28 days) up to the original design bottom of footing elevation. The original footing is recommended to be constructed on top of the hardened lean concrete. If lean concrete is utilized the excavation is recommended to be 1 foot wider than the footing (6 inches on each side), as conceptually shown below, and the lean concrete should be allowed to sufficiently harden prior to placement of the foundation concrete.



Fill Placement Considerations: Do not place fill materials on frozen soils, on frost-heaved soils, on excessively wet soils, or soils that are otherwise unstable. Borrow fill materials should not contain frozen materials at the time of placement, and all frozen or frost-heaved soils should be removed prior to placement of engineered fill or other fill soils and aggregates. Excessively wet soils or aggregates should be scarified, aerated, and moisture conditioned.

5.3 FOUNDATION OBSERVATIONS

Protection of Foundation Excavations: Exposure to the environment may weaken the soil at the footing bearing level if the foundation excavations remain open for too long a time. Therefore, foundation concrete should be placed on the same day that excavations are made. If the bearing soil is softened by surface water intrusion or exposure, the softened soils must be removed from the foundation excavation bottom immediately prior to placement of concrete. If the excavation must remain open overnight, or if rainfall becomes imminent while the bearing soils are exposed, a 1 to 3-inch thick "mud mat" of "lean" concrete should be placed on the bearing soils before the placement of reinforcing steel.

Footing Subgrade Observations: Most of the soils at the foundation bearing elevation are anticipated to be suitable for support of the proposed structure. It is important to have a third-party testing firm observe the foundation subgrade prior to placing foundation concrete, to confirm the bearing soils are what was anticipated.

5.4 UTILITY INSTALLATIONS

Utility Subgrades: The highly expansive 'fat' clay soils encountered in our exploration are not generally considered suitable for support of utility pipes due to higher shrink-swell potential. Therefore, ECS recommends undercutting the 'fat clays, if encountered, extending to a depth of about 12 inches and replacing with low volume change material. The pipe subgrades should be observed and probed for stability by a third-party testing firm. Any loose or unsuitable materials encountered should be removed and replaced with suitable compacted engineered fill, or pipe stone bedding material.

Utility Backfilling: The granular bedding material (AASHTO No. 57 stone) should be at least 4 inches thick, but not less than that specified by the civil engineer's project drawings and specifications. We recommend that the bedding materials be placed up to the spring line of the pipe. The fill placed for support of the utilities, as well as backfill over the utilities, should satisfy the requirements for engineered fill and Fill Placement.

Excavation Safety: All excavations and slopes should be constructed and maintained in accordance with OSHA excavation safety standards. The contractor is solely responsible for designing, constructing, and maintaining stable temporary excavations and slopes. The contractor's responsible person, as defined in 29 CFR Part 1926, should evaluate the soil exposed in the excavations as part of the contractor's safety procedures. In no case should slope height, slope inclination, or excavation depth, including utility trench excavation depth, exceed those specified in local, state, and federal safety regulations. ECS provides this information solely as a service to our client. ECS is not assuming responsibility for construction site safety or the contractor's activities; such responsibility is not being implied and should not be inferred.

5.5 GENERAL CONSTRUCTION CONSIDERATIONS

Existing Utilities: Prior to construction, ECS recommends all utilities in the proposed construction areas be positively identified and marked. Active utilities to remain in the construction areas should be exposed and protected during construction to reduce the potential for damage or interruption of service. Abandoned utilities should be removed and backfilled with compacted engineered fill or grouted full of lean concrete if left in-place.

Subgrade Protection: Measures should also be taken to limit site disturbance, especially from rubber-tired heavy construction equipment, and to control and remove surface water from development areas, including structural and pavement areas. It would be advisable to designate a haul road and construction

staging area to limit the areas of disturbance and to prevent construction traffic from excessively degrading sensitive subgrade soils and existing pavement areas. Haul roads and construction staging areas could be covered with excess depths of aggregate to protect those subgrades. The aggregate can later be removed and used in pavement areas.

Surface Drainage: Surface drainage conditions should be properly maintained. Surface water should be directed away from the construction area, and the work area should be sloped away from the construction area at a gradient of 1 percent or greater to reduce the potential of ponding water and the subsequent saturation of the surface soils. At the end of each workday, the subgrade soils should be sealed by rolling the surface with a smooth drum roller to minimize infiltration of surface water.

Bidding/Estimating Considerations: Contractors bidding or undertaking any work at the site should examine the results of the subsurface exploration, satisfy themselves as to the adequacy of the information for bidding and construction, make their own interpretation of the data, and consider the effect it may have on their cost proposal, construction techniques, schedule, and equipment capabilities. Furthermore, contractors should complete any additional fieldwork and investigation they deem necessary to properly prepare a cost proposal for the site work. Soil borings do not provide the same wide-scale view of the subsurface conditions that is obtained during site grading, excavation, or other aspects of earthwork construction. Additional scope may be required to obtain more detailed subsurface information needed for earthwork bid preparation, which could include test pits to better understand the lateral and vertical extents of the subsurface materials of concern such as existing undocumented fill. Even with this additional information, budget contingencies should be carried out in construction to help cover potential variations in subsurface conditions.

6.0 CLOSING

ECS has prepared this report to guide the geotechnical-related design and construction aspects of the project. We performed these services in accordance with the standard of care expected of professionals in the industry performing similar services on projects of like size and complexity at this time in the region. No other representation expressed or implied, and no warranty or guarantee is included or intended in this report.

The description of the proposed project is based on information provided to ECS by the Client. If any of this information is inaccurate or changes, either because of our interpretation of the documents provided or site or design changes that may occur later, ECS should be contacted so we can review our recommendations and provide additional or alternate recommendations that reflect the proposed construction.

We recommend that ECS review the project plans and specifications so we can confirm that those plans/specifications are in accordance with the recommendations of this geotechnical report.

Field observations, and quality assurance testing during earthwork and foundation installation are an extension of, and integral to, the geotechnical design. We understand a third-party testing firm will be retained to apply our expertise throughout the geotechnical phases of construction, and to provide consultation and recommendation should issues arise.

ECS is not responsible for the conclusions, opinions, or recommendations of others based on the data in this report.

Appendix A - Drawings and Reports

Site Location Diagram

Boring Location Diagram(s)

Subsurface Cross-Section(s)



SITE LOCATION DIAGRAM

24-1329 Proposed McDonald's Willard, MO 01.00.14

652 South Hunt Road, Willard, Missouri
McDonald's USA, LLC

ENGINEER
EI

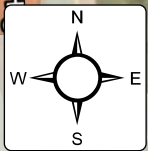
SCALE
1" = 400'

PROJECT NO.
16:16210

SHEET
1

DATE
6/23/2026

Service Layer Credits: Updated Plan_georeferenced:



Legend

-  Approximate Pavement Boring Locations - P
-  Approximate Building Boring Locations - B
-  Approximate Sign Boring Locations - S
-  Approximate Previously Proposed Pond Boring Locations - D
-  Approximate Cross-Section Location



BORING LOCATION DIAGRAM

24-1329 Proposed McDonald's Willard, MO 01.00.14

652 South Hunt Road, Willard, Missouri
McDonald's USA, LLC

ENGINEER EI
SCALE 1" = 60'
PROJECT NO. 16:16210
SHEET 1
DATE 6/23/2026

Generalized Subsurface Cross Section A-A'

Elevation (Ft)

Distance (Ft)

Anticipated FFE: EL. 1224.5 feet MSL

Borehole Data:

Borehole	Distance (Ft)	Elevation (Ft)	Soil Type	SPT	Notes
B-06	35	1221.5	CH	13	TS, CH
B-06	35	1219.5	CH	13	
B-06	35	1217.5	CH	12	
B-06	35	1215.5	CH	12	
B-06	35	1213.5	CH	12	
B-06	35	1211.5	WR		AR@8.7FT
B-06	35	1209.5	WR		EOB@12.7FT
B-07	55	1224.5	CH	50/11"	TS, CH
B-07	55	1222.5	CH		AR@2.7FT
B-08	85	1221.5	CH	24	TS, CH
B-08	85	1219.5	CH	25	
B-08	85	1217.5	CH	30	
B-08	85	1215.5	WR		AR@8.7FT
B-08	85	1213.5	WR		EOB@12.7FT
B-09	145	1221.5	CH	18	TS, CH
B-09	145	1219.5	CH		BR@3.8FT
B-09	145	1217.5	WR		
B-09	145	1215.5	WR		
B-09	145	1213.5	WR		
B-09	145	1211.5	WR		
B-09	145	1209.5	WR		
B-09	145	1207.5	WR		
B-09	145	1205.5	WR		
B-09	145	1203.5	WR		
B-09	145	1201.5	WR		
B-09	145	1199.5	WR		
B-09	145	1197.5	WR		
B-09	145	1195.5	WR		
B-09	145	1193.5	WR		
B-09	145	1191.5	WR		
B-09	145	1189.5	WR		
B-09	145	1187.5	WR		
B-09	145	1185.5	WR		
B-09	145	1183.5	WR		
B-09	145	1181.5	WR		
B-09	145	1179.5	WR		
B-09	145	1177.5	WR		
B-09	145	1175.5	WR		
B-09	145	1173.5	WR		
B-09	145	1171.5	WR		
B-09	145	1169.5	WR		
B-09	145	1167.5	WR		
B-09	145	1165.5	WR		
B-09	145	1163.5	WR		
B-09	145	1161.5	WR		
B-09	145	1159.5	WR		
B-09	145	1157.5	WR		
B-09	145	1155.5	WR		
B-09	145	1153.5	WR		
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B-09	145	1141.5	WR		
B-09	145	1139.5	WR		
B-09	145	1137.5	WR		
B-09	145	1135.5	WR		
B-09	145	1133.5	WR		
B-09	145	1131.5	WR		
B-09	145	1129.5	WR		
B-09	145	1127.5	WR		
B-09	145	1125.5	WR		
B-09	145	1123.5	WR		
B-09	145	1121.5	WR		
B-09	145	1119.5	WR		
B-09	145	1117.5	WR		
B-09	145	1115.5	WR		
B-09	145	1113.5	WR		

CLIENT:	McDonald's USA, LLC	PROJECT:	24-1329 Proposed McDonald's Willard, MO 01.00.14			
DRAWN DATE:	Jun 24, 2026	PROJECT NO.:	16:16210			
CHECKED DATE:	Jun 24, 2026	SCALE:	AS SHOWN			

Notes: 1-EOB: END OF BORING AR: AUGER REFUSALSR: SAMPLER REFUSAL 2-SEE INDIVIDUAL BORING LOG AND GEOTECHNICAL INFORMATION. 3-STANDARD PENETRATION TEST RESISTANCE (LEFT OF BORING) IN BLOWS PER FOOT (ASTM D1586). 4- TOPOGRAPHIC INFORMATION IS BASED ON PUBLICLY AVAILABLE DATA (GOOGLE OR CESIUM).THE TOPOGRAPHIC LINE SHOWN BETWEEN BORINGS IS FOR VISUAL REFERENCE ONLY.PLEASE REFER TO THE REFERENCE NOTES FOR BORING LOGS FOR SYMBOLOGY MEANING AND ADDITIONAL	Plastic Limit Water Content Liquid Limit X————●————Δ	▽	WL (First Encountered)		Fill	ECS
	[FINES CONTENT %]	▼	WL (Completion)		Possible Fill	
	BOTTOM OF CASING	▽	WL (Estimated Seasonal High Water)		Probable Fill	
	100% LOSS OF CIRCULATION					
	○ CALIBRATED PENETROMETER	▽	WL (Stabilized)		WR/Rock	

Appendix B - Field Operations

Reference Notes

Exploration Procedures

Boring Logs

Rock Core Photographs



REFERENCE NOTES FOR BORING LOGS

MATERIAL^{1,2}

	ASPHALT
	CONCRETE
	GRAVEL
	TOPSOIL
	VOID
	BRICK
	AGGREGATE BASE COURSE
	GW WELL-GRADED GRAVEL gravel-sand mixtures, little or no fines
	GP POORLY-GRADED GRAVEL gravel-sand mixtures, little or no fines
	GM SILTY GRAVEL gravel-sand-silt mixtures
	GC CLAYEY GRAVEL gravel-sand-clay mixtures
	SW WELL-GRADED SAND gravelly sand, little or no fines
	SP POORLY-GRADED SAND gravelly sand, little or no fines
	SM SILTY SAND sand-silt mixtures
	SC CLAYEY SAND sand-clay mixtures
	ML SILT non-plastic to medium plasticity
	MH ELASTIC SILT high plasticity
	CL LEAN CLAY low to medium plasticity
	CH FAT CLAY high plasticity
	OL ORGANIC SILT or CLAY non-plastic to low plasticity
	OH ORGANIC SILT or CLAY high plasticity
	PT PEAT highly organic soils

DRILLING SAMPLING SYMBOLS & ABBREVIATIONS

SS	Split Spoon Sampler	PM	Pressuremeter Test
ST	Shelby Tube Sampler	RD	Rock Bit Drilling
WS	Wash Sample	RC	Rock Core, NX, BX, AX
BS	Bulk Sample of Cuttings	REC	Rock Sample Recovery %
PA	Power Auger (no sample)	RQD	Rock Quality Designation %
HSA	Hollow Stem Auger		

PARTICLE SIZE IDENTIFICATION

DESIGNATION	PARTICLE SIZES
Boulders	12 inches (300 mm) or larger
Cobbles	3 inches to 12 inches (75 mm to 300 mm)
Gravel: Coarse	¾ inch to 3 inches (19 mm to 75 mm)
Fine	4.75 mm to 19 mm (No. 4 sieve to ¾ inch)
Sand: Coarse	2.00 mm to 4.75 mm (No. 10 to No. 4 sieve)
Medium	0.425 mm to 2.00 mm (No. 40 to No. 10 sieve)
Fine	0.074 mm to 0.425 mm (No. 200 to No. 40 sieve)
Silt & Clay ("Fines")	<0.074 mm (smaller than a No. 200 sieve)

COHESIVE SILTS & CLAYS

UNCONFINED COMPRESSIVE STRENGTH, QP ⁴	SPT ⁵ (BPF)	CONSISTENCY ⁷ (COHESIVE)
<0.25	<2	Very Soft
0.25 - <0.50	2 - 4	Soft
0.50 - <1.00	5 - 8	Firm
1.00 - <2.00	9 - 15	Stiff
2.00 - <4.00	16 - 30	Very Stiff
4.00 - 8.00	31 - 50	Hard
>8.00	>50	Very Hard

RELATIVE AMOUNT ⁷	COARSE GRAINED (%) ⁸	FINE GRAINED (%) ⁸
Trace	≤5	≤5
With	10 - 20	10 - 25
Adjective (ex: "Silty")	25 - 45	30 - 45

GRAVELS, SANDS & NON-COHESIVE SILTS

SPT ⁵	DENSITY
<5	Very Loose
5 - 10	Loose
11 - 30	Medium Dense
31 - 50	Dense
>50	Very Dense

WATER LEVELS⁶

	WL (First Encountered)
	WL (Completion)
	WL (Seasonal High Water)
	WL (Stabilized)

FILL AND ROCK

FILL	POSSIBLE FILL	PROBABLE FILL	ROCK

¹Classifications and symbols per ASTM D 2488-17 (Visual-Manual Procedure) unless noted otherwise.

²To be consistent with general practice, "POORLY GRADED" has been removed from GP, GP-GM, GP-GC, SP, SP-SM, SP-SC soil types on the boring logs.

³Non-ASTM designations are included in soil descriptions and symbols along with ASTM symbol [Ex: (SM-FILL)].

⁴Typically estimated via pocket penetrometer or Torvane shear test and expressed in tons per square foot (tsf).

⁵Standard Penetration Test (SPT) refers to the number of hammer blows (blow count) of a 140 lb. hammer falling 30 inches on a 2 inch OD split spoon sampler required to drive the sampler 12 inches (ASTM D 1586). "N-value" is another term for "blow count" and is expressed in blows per foot (bpf). SPT correlations per 7.4.2 Method B and need to be corrected if using an auto hammer.

⁶The water levels are those levels actually measured in the borehole at the times indicated by the symbol. The measurements are relatively reliable when augering, without adding fluids, in granular soils. In clay and cohesive silts, the determination of water levels may require several days for the water level to stabilize. In such cases, additional methods of measurement are generally employed.

⁷Minor deviation from ASTM D 2488-17 Note 14.

⁸Percentages are estimated to the nearest 5% per ASTM D 2488-17.



REFERENCE NOTES FOR ROCK CORES

ROCK CLASSIFICATION TYPES		
Igneous	Sedimentary	Metamorphic
Coarse Grained DIABASE DIORITE GABBRO GRANITE PEGMATITE PERIDOTITE SYENITE	Clastic (sediment) SHALE SILTSTONE SANDSTONE CONGLOMERATE LIMESTONE, OOLITIC	Foliated GNEISS PHYLLITE SCHIST SLATE
Fine Grained ANDESITE BASALT RHYOLITE TRACHYTE	Chemically Formed DOLOSTONE GYPSUM HALITE LIMESTONE	Non-Foliated AMPHIBOLITE HORNFELS MARBLE QUARTZITE
Pyroclastic OBSIDIAN PUMICE TUFF	Organic Remains CHALK COAL COQUINA	

HARDNESS	
Very Soft	Deformed by hand
Soft	Scratched with a fingernail
Moderately Hard	Scratched easily with a knife
Hard	Scratched with difficulty with a knife
Very Hard	Cannot be scratched with a knife

JOINT/FRACTURE SPACING	
Fractured/Jointed	Spacing
Very Widely	> 10 feet
Slightly	3 - 10 feet
Moderately	1 - 3 feet
Highly	2 inches - 1 foot
Intensely	< 2 inches

BEDDING	
Thinly	≤ 0.3 ft.
Medium	>0.3 ft. ≤ 1 ft.
Thickly	>1 ft. ≤ 3 ft.
Massive	>3 ft.

JOINT OR FRACTURE CONTINUITY
It shall be noted whether the joints or fractures are continuous or discontinuous. If continuity of joints is not discernable at the scale of the rock core, continuous joints or fractures shall be assumed.

JOINT/FRACTURE ORIENTATION	
The range or average orientation of each joint set or fracture trend shall be measured in degrees from a horizontal plane where possible. If no measurement is possible, the qualitative terms High, Moderate or Low-angle shall be used. Record whether the joints are present in Conjugate sets (i.e. having an opposite sense of dip)	
High	61-90 degree
Moderate	31-60 degree
Low-angle	0-30 degree
Dip-angle	(1-90) _____ degrees (if measured)

Description Sequence	Example Rock Classification Description
ROCK TYPE, [REC=_% ,RQD=_%], Weathering, Hardness, Bedding, Joint/Fracture Spacing, Joint/Fracture Surface Condition, Wall Rock Condition, Joint or Fracture Continuity, Joint/Fracture Orientation, Color, Additional Features	LIMESTONE, [REC=95%,RQD=60%], Highly Weathered, Hard, Thinly Bedded, Slightly Fractured/Jointed, Slightly Rough, Hard Wall Rock, Continuous, Moderate-angle Dip, Gray White

*ASTM D6032-17: RQD is performed on cores using BQ to PQ sized bits (1.433 to 3.345 inch diameter cores, respectively)

Recovery (REC(%))
$\frac{\text{Total rock recovered from run}}{\text{Total Run Length}}$

Rock Quality Designation (RQD(%))*	
$\frac{\text{Sum of core pieces } \geq 4 \text{ inches long}}{\text{Total Run Length}}$	
RQD%	Description of Rock Quality
0-25%	Very Poor
>25%-50%	Poor
>50%-75%	Fair
>75%-90%	Good
>90%	Excellent

WEATHERING	
Unweathered	No evidence of any chemical or mechanical alteration.
Slightly Weathered	Slight discoloration on surface, slight alteration along discontinuities, less than 10 percent of the rock volume altered.
Moderately Weathered	Discoloring evident, surface pitted and altered with alteration penetrating well below rock surfaces, weathering 'halos' evident. 10 to 50 percent of the rock altered.
Highly Weathered	Entire mass discolored, alteration pervading nearly all of the rock, with some pockets of slightly weathered rock noticeable, some minerals leached away.
Decomposed	Rock reduced to a soil with relict rock structure remaining (i.e. saprolite). Generally molded and crumbled by hand (friable).

JOINT/FRACTURE SURFACE CONDITION			
The following qualitative terms shall be used to describe surface condition of joints and fractures. Multiple terms can be used.			
Very rough	Slightly rough	Slickensided	Gouge

WALL ROCK CONDITION
The qualitative terms 'hard wall rock' or 'soft wall rock' shall be used to describe the condition of the parent rock on either side of the joint or fracture.



SUBSURFACE EXPLORATION PROCEDURE: STANDARD PENETRATION TESTING (SPT) ASTM D 1586 Split-Barrel Sampling

Standard Penetration Testing, or **SPT**, is the most frequently used subsurface exploration test performed worldwide. This test provides samples for identification purposes, as well as a measure of penetration resistance, or N-value. The N-Value, or blow counts, when corrected and correlated, can approximate engineering properties of soils used for geotechnical design and engineering purposes.

SPT Procedure:

- Involves driving a hollow tube (split-spoon) into the ground by dropping a 140-lb hammer a height of 30-inches at desired depth
- Recording the number of hammer blows required to drive split-spoon a distance of 18-24 inches (in 3 or 4 Increments of 6 inches each)
- Auger is advanced* and an additional SPT is performed
- One SPT typically performed for every two to five feet. An approximate 1.5 inch diameter soil sample is recovered.



**Drilling Methods May Vary—* The predominant drilling methods used for SPT are open hole fluid rotary drilling and hollow-stem auger drilling.

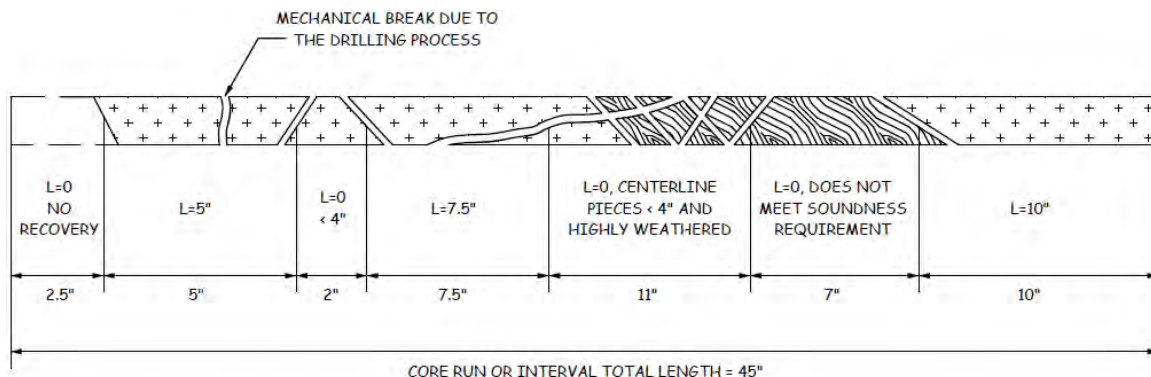


SUBSURFACE EXPLORATION PROCEDURE: ROCK CORE DRILLING AND SAMPLING ASTM D 2113

Rock core drilling is a method to obtain bedrock cores for geological data and to provide information about the engineering properties of rock formations at the drilled locations. Rock coring provides samples for characterization purposes, as well as a measure of the Recovery and Rock Quality Designation, or RQD, and various engineering laboratory tests. The geologic engineering properties determined from rock cores are advantageous in the design of geotechnical elements such as foundations.

Rock Core Procedure:

- Soil test borings are advanced through the soil medium to auger refusal, which is generally considered the top of rock.
- Rock recovery (REC) is the percentage of the total length of rock recovered divided by the length of the core run.
- Rock coring is accomplished with a diamond studded rock core barrel attached to a drill rig and utilized for sampling during the rotation and advancement of the hollow barrel.
- The Rock Quality Designation (RQD) is generally the percentage of the total length of the rock pieces 4-inches in length or greater divided by the length of the rock core run.



$$\begin{aligned} \text{REC}\% &= \frac{\text{TOTAL ROCK RECOVERED}}{\text{CORE RUN TOTAL LENGTH}} \times 100 \\ &= \frac{5" + 2" + 7.5" + 11" + 7" + 10"}{45"} \times 100 \\ &= \frac{42.5"}{45"} \times 100 = 94\% \end{aligned}$$

$$\begin{aligned} \text{RQD}\% &= \frac{\text{TOTAL SOUND CORE PIECES } > 4"}{\text{CORE RUN TOTAL LENGTH}} \\ &= \frac{5" + 7.5" + 10"}{45"} \\ &= \frac{22.5"}{45"} \times 100 = 50\% \end{aligned}$$

RQD (ROCK QUALITY
DESIGNATION)

CLASSIFICATION
OF ROCK QUALITY

0 - 25%	VERY POOR
>25 - 50%	POOR
>50 - 75%	FAIR
>75 - 90%	GOOD
>90 - 100%	EXCELLENT

LEGEND



SOUND ROCK



UN SOUND ROCK

NOTE: DIAGRAM ADAPTED FROM ASTM D6032/6032M-17

CLIENT: McDonald's USA, LLC				PROJECT NO.: 16:16210		BORING NO.: B-01		SHEET: 1 OF 1			
PROJECT NAME: 24-1329 Proposed McDonald's Willard, MO 01.00.14				DRILLER/CONTRACTOR: Palmerton And Parrish, Inc.							
SITE LOCATION: 652 South Hunt Road, Willard, Missouri, 65781								LOSS OF CIRCULATION			
LATITUDE: 37.292089		LONGITUDE: -93.411005		STRUCTURE:		SURFACE ELEVATION: 1213		BOTTOM OF CASING			

DEPTH (FT)	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE DISTANCE (IN)	SAMPLE RECOVERY (IN)	DESCRIPTION OF MATERIAL	STRATIGRAPHY	WATER LEVELS	ELEVATION (FT)	BLOWS/6" (TCP/RS/MC/SPT-N VALUE)*	RQD %	Rec %	QP (TS)	PL	MC	LL	Recovery %	RQD %	Fines% F	MC%
					Topsoil [Thickness=±17"].														
	S-1	SS	18	18	(CL) LEAN CLAY WITH SAND - reddish brown, moist, stiff, with chert.			1210	2-3-3 (6)										
	S-2	SS	7	7															
	S-3	RC	3		LIMESTONE, slightly weathered, Hard, Light gray.				4-50/1" (600)										
5.0	S-4	RC	60					1205											
					END OF BORING at 9.3 FT														

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL

 WL (First Encountered):		Not Observed		BORING STARTED: 04/24/2026		CAVE IN DEPTH: Not Observed	
 WL (Completion):		Not Observed		BORING COMPLETED: 04/24/2026		HAMMER TYPE: Automatic	
 WL (Seasonal High Water):				EQUIPMENT: Track		LOGGED BY: PW	
 WL (Stabilized):						DRILLING METHOD: Hollow Stem Auger (0'-9.33')	

GEOTECHNICAL BOREHOLE LOG

CLIENT: McDonald's USA, LLC				PROJECT NO.: 16:16210		BORING NO.: B-02		SHEET: 1 OF 1			
PROJECT NAME: 24-1329 Proposed McDonald's Willard, MO 01.00.14				DRILLER/CONTRACTOR: Palmerton And Parrish, Inc.							
SITE LOCATION: 652 South Hunt Road, Willard, Missouri, 65781								LOSS OF CIRCULATION			
LATITUDE: 37.292081		LONGITUDE: -93.410692		STRUCTURE:		SURFACE ELEVATION: 1218		BOTTOM OF CASING			
DEPTH (FT)	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE DISTANCE (IN)	SAMPLE RECOVERY (IN)	DESCRIPTION OF MATERIAL	STRATIGRAPHY	WATER LEVELS	ELEVATION (FT)	BLOWS/6" (TCP/RS/MC/SPT-N VALUE)*	SPT  Recovery %  RQD %  01020304050 020406080100 020406080100 QP  PL  MC  LL  012345 020406080100 020406080100 01020304050 020406080100 020406080100	
					Topsoil [Thickness=±12"].						
	S-1	SS	18	18	(CH) FAT CLAY WITH GRAVEL - reddish brown, moist, very stiff, with chert.				5-5-6 (11)		
	S-2	SS	1	1					50/1" (50/1")		
					AUGER REFUSAL at 2.8 FT						
THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL											
 WL (First Encountered):				Not Observed		BORING STARTED:				04/24/2026	
 WL (Completion):				Not Observed		BORING COMPLETED:				04/24/2026	
 WL (Seasonal High Water):						EQUIPMENT:		LOGGED BY:		DRILLING METHOD:	
 WL (Stabilized):						Track		PW		Hollow Stem Auger (0'-2.75')	
GEOTECHNICAL BOREHOLE LOG											

CLIENT: McDonald's USA, LLC				PROJECT NO.: 16:16210		BORING NO.: B-03		SHEET: 1 OF 1			
PROJECT NAME: 24-1329 Proposed McDonald's Willard, MO 01.00.14				DRILLER/CONTRACTOR: Palmerton And Parrish, Inc.							
SITE LOCATION: 652 South Hunt Road, Willard, Missouri, 65781								LOSS OF CIRCULATION			
LATITUDE: 37.292023		LONGITUDE: -93.410848		STRUCTURE:		SURFACE ELEVATION: 1214		BOTTOM OF CASING			

DEPTH (FT)	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE DISTANCE (IN)	SAMPLE RECOVERY (IN)	DESCRIPTION OF MATERIAL	STRATIGRAPHY	WATER LEVELS	ELEVATION (FT)	BLOWS/6" (TCP/RS/MC/SPT-N VALUE)*	RQD %		Rec %		QP - Q			
										SPT	Recovery %	RQD %	Fines%	MC%	LL	PL	MC
5.0					Topsoil [Thickness=±12"].												
	S-1	SS	18	18	(CH) GRAVELLY FAT CLAY - reddish brown, moist, stiff, with chert.				7-10-6 (16)								
	S-2	SS	18	18	(CH) FAT CLAY - reddish brown, moist, stiff, with chert.			1210	2-4-3 (7)								
10.0					LIMESTONE, slightly weathered, Moderately hard to hard, Light gray.												
	S-3	RC	60					1205									
					END OF BORING at 10.8 FT												

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL

WL (First Encountered):	Not Observed	BORING STARTED:	04/24/2026	CAVE IN DEPTH:	Not Observed
WL (Completion):	Not Observed	BORING COMPLETED:	04/24/2026	HAMMER TYPE:	Automatic
WL (Seasonal High Water):		EQUIPMENT:	LOGGED BY:	DRILLING METHOD:	
WL (Stabilized):		Track	PW	Hollow Stem Auger (0'-10.83')	

GEOTECHNICAL BOREHOLE LOG

CLIENT: McDonald's USA, LLC				PROJECT NO.: 16:16210		BORING NO.: B-04		SHEET: 1 OF 1		
PROJECT NAME: 24-1329 Proposed McDonald's Willard, MO 01.00.14				DRILLER/CONTRACTOR: Palmerton And Parrish, Inc.						
SITE LOCATION: 652 South Hunt Road, Willard, Missouri, 65781								LOSS OF CIRCULATION		
LATITUDE: 37.291967		LONGITUDE: -93.411011		STRUCTURE:		SURFACE ELEVATION: 1212		BOTTOM OF CASING		

DEPTH (FT)	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE DISTANCE (IN)	SAMPLE RECOVERY (IN)	DESCRIPTION OF MATERIAL	STRATIGRAPHY	WATER LEVELS	ELEVATION (FT)	BLOWS/6" (TCP/RS/MC/SPT-N VALUE)*	RQD % Rec % 	020406080100 Recovery % 020406080100 RQD %	020406080100 Fines% 020406080100 MC%
					Topsoil [Thickness=±9"].							
	S-1	RC	60		LIMESTONE, [REC=100%, RQD=45%], slightly weathered, Hard, Light gray.			1210		45100		
5.0												
	S-2	RC	60					1205		92100		
10.0												
					END OF BORING at 10.8 FT							

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL			
WL (First Encountered):	Not Observed	BORING STARTED:	04/24/2026
WL (Completion):	Not Observed	BORING COMPLETED:	04/24/2026
WL (Seasonal High Water):		EQUIPMENT:	LOGGED BY:
WL (Stabilized):		Track	PW
		DRILLING METHOD: Hollow Stem Auger (0'-10.75')	

GEOTECHNICAL BOREHOLE LOG

CLIENT: McDonald's USA, LLC				PROJECT NO.: 16:16210		BORING NO.: B-05		SHEET: 1 OF 1										
PROJECT NAME: 24-1329 Proposed McDonald's Willard, MO 01.00.14				DRILLER/CONTRACTOR: Palmerton And Parrish, Inc.														
SITE LOCATION: 652 South Hunt Road, Willard, Missouri, 65781								LOSS OF CIRCULATION										
LATITUDE: 37.291957		LONGITUDE: -93.41068		STRUCTURE:		SURFACE ELEVATION: 1217		BOTTOM OF CASING										
DEPTH (FT)	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE DISTANCE (IN)	SAMPLE RECOVERY (IN)	DESCRIPTION OF MATERIAL	STRATIGRAPHY	WATER LEVELS	ELEVATION (FT)	BLOWS/6" (TCP/RS/MC/SPT-N VALUE)*	SPT	Recovery %	RQD %	QP	PL	MC	LL	Fines%	MC%
					Topsoil [Thickness=±12"].													
	S-1	SS	18	18	(CH) FAT CLAY - trace gravel, reddish brown, moist, very stiff.			1215	3-4-6 (10)	10					21.6	37	3	97.9
					AUGER REFUSAL at 2.8 FT													
THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL																		
WL (First Encountered):		Not Observed		BORING STARTED:		04/24/2026		CAVE IN DEPTH:		Not Observed								
WL (Completion):		Not Observed		BORING COMPLETED:		04/24/2026		HAMMER TYPE:		Automatic								
WL (Seasonal High Water):				EQUIPMENT:		LOGGED BY:		DRILLING METHOD:										
WL (Stabilized):				Track		PW		Hollow Stem Auger (0'-2.83')										
GEOTECHNICAL BOREHOLE LOG																		

CLIENT: McDonald's USA, LLC				PROJECT NO.: 16:16210		BORING NO.: B-06		SHEET: 1 OF 1		
PROJECT NAME: 24-1329 Proposed McDonald's Willard, MO 01.00.14				DRILLER/CONTRACTOR: Palmerton And Parrish, Inc						
SITE LOCATION: 652 South Hunt Road, Willard, Missouri, 65781								LOSS OF CIRCULATION		
LATITUDE: 37.29246		LONGITUDE: -93.410631		STRUCTURE:		SURFACE ELEVATION: 1222		BOTTOM OF CASING		

DEPTH (FT)	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE DISTANCE (IN)	SAMPLE RECOVERY (IN)	DESCRIPTION OF MATERIAL	STRATIGRAPHY	WATER LEVELS	ELEVATION (FT)	BLOWS/6" (TCP/RS/MC/SPT-N VALUE)*	SPT	Recovery %	RQD %	Fines% F	MC%
					Topsoil [Thickness=4"].									
	S-1	SS	18	18	(CH) GRAVELLY FAT CLAY - reddish brown, moist, stiff.			1220	9-8-5 (13)	13				
5.0	S-2	SS	18	18	(CH) FAT CLAY - reddish brown, moist, stiff to very stiff.				5-6-7 (13)	13			2.5 48.7	
	S-3	SS	18	18				1215	4-5-7 (12)	12			1.25 41.1	
	S-4	SS	18	18					4-5-7 (12)	12			1.5 55.3	
10.0								1210						
					END OF BORING at 12.7 FT									

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL

WL (First Encountered): 10 FT	BORING STARTED: 06/11/2026	CAVE IN DEPTH: Not Observed
WL (Completion): Not Observed	BORING COMPLETED: 06/11/2026	HAMMER TYPE: Automatic
WL (Seasonal High Water):	EQUIPMENT: Track	LOGGED BY: JGC
WL (Stabilized):		DRILLING METHOD: CFA (0'-12.67')

GEOTECHNICAL BOREHOLE LOG

CLIENT: McDonald's USA, LLC				PROJECT NO.: 16:16210		BORING NO.: B-07		SHEET: 1 OF 1														
PROJECT NAME: 24-1329 Proposed McDonald's Willard, MO 01.00.14				DRILLER/CONTRACTOR: Palmerton And Parrish, Inc																		
SITE LOCATION: 652 South Hunt Road, Willard, Missouri, 65781								LOSS OF CIRCULATION														
LATITUDE: 37.292412		LONGITUDE: -93.410508		STRUCTURE:		SURFACE ELEVATION: 1224		BOTTOM OF CASING														
DEPTH (FT)	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE DISTANCE (IN)	SAMPLE RECOVERY (IN)	DESCRIPTION OF MATERIAL	STRATIGRAPHY	WATER LEVELS	ELEVATION (FT)	BLOWS/6" (TCP/RS/MC/SPT-N VALUE)*	SPT  Recovery %  RQD %  0 10 20 30 40 50 0 20 40 60 80 100 0 20 40 60 80 100												
					Topsoil [Thickness=4"].																	
	S-1	SS	11	11	(CH) GRAVELLY FAT CLAY - brown and reddish brown, moist, very stiff.					21-50/5" (50/11")												
	S-2	SS	0	0						50/0"												
					AUGER REFUSAL at 2.7 FT																	
THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL																						
 WL (First Encountered):					Not Observed		BORING STARTED:					06/11/2026		CAVE IN DEPTH: Not Observed								
 WL (Completion):					Not Observed		BORING COMPLETED:					06/11/2026		HAMMER TYPE: Automatic								
 WL (Seasonal High Water):							EQUIPMENT:		LOGGED BY:		DRILLING METHOD:											
 WL (Stabilized):							Track		JGC		CFA (0'-2.67')											
GEOTECHNICAL BOREHOLE LOG																						

CLIENT: McDonald's USA, LLC				PROJECT NO.: 16:16210		BORING NO.: B-08		SHEET: 1 OF 1		
PROJECT NAME: 24-1329 Proposed McDonald's Willard, MO 01.00.14				DRILLER/CONTRACTOR: Palmerton And Parrish, Inc.						
SITE LOCATION: 652 South Hunt Road, Willard, Missouri, 65781								LOSS OF CIRCULATION		
LATITUDE: 37.292316		LONGITUDE: -93.410642		STRUCTURE:		SURFACE ELEVATION: 1220		BOTTOM OF CASING		

DEPTH (FT)	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE DISTANCE (IN)	SAMPLE RECOVERY (IN)	DESCRIPTION OF MATERIAL	STRATIGRAPHY	WATER LEVELS	ELEVATION (FT)	BLOWS/6" (TCP/RS/MC/SPT-N VALUE)*	SPT	Recovery %	RQD %	Fines% F	MC%
5.0					Topsoil [Thickness=4"].									
	S-1	SS	18	18	(CH) GRAVELLY FAT CLAY - brown and dark brown, moist, very stiff.				10-12-12 (24)	24				
	S-2	SS	18	18	(CH) FAT CLAY - trace gravel, reddish brown, moist, very stiff.				4-7-18 (25)	25			2	51.7
	S-3	SS	18	18					4-5-25 (30)	30			56.3	6.3
	S-4	SS	2	2	(WR) WEATHERED ROCK SAMPLED AS [WEATHERED LIMESTONE].				50/2"					
AUGER REFUSAL at 8.7 FT														

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL

WL (First Encountered):	Not Observed	BORING STARTED: 06/11/2026	CAVE IN DEPTH: Not Observed
WL (Completion):	Not Observed	BORING COMPLETED: 06/11/2026	HAMMER TYPE: Automatic
WL (Seasonal High Water):		EQUIPMENT: Track	LOGGED BY: JGC
WL (Stabilized):			DRILLING METHOD: CFA (0'-8.67')

GEOTECHNICAL BOREHOLE LOG

CLIENT: McDonald's USA, LLC				PROJECT NO.: 16:16210		BORING NO.: B-09		SHEET: 1 OF 1			
PROJECT NAME: 24-1329 Proposed McDonald's Willard, MO 01.00.14				DRILLER/CONTRACTOR: Palmerton And Parrish, Inc.							
SITE LOCATION: 652 South Hunt Road, Willard, Missouri, 65781								LOSS OF CIRCULATION			
LATITUDE: 37.292193		LONGITUDE: -93.410649		STRUCTURE:		SURFACE ELEVATION: 1219		BOTTOM OF CASING			
DEPTH (FT)	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE DISTANCE (IN)	SAMPLE RECOVERY (IN)	DESCRIPTION OF MATERIAL	STRATIGRAPHY	WATER LEVELS	ELEVATION (FT)	BLOWS/6" (TCP/RS/MC/SPT-N VALUE)*	SPT  Recovery %  RQD %  0 1 2 3 4 5 PL MC LL 0 20 40 60 80 100 Fines% F MC%  0 20 40 60 80 100	
	S-1	SS	18	18	Topsoil [Thickness=4"]. (CH) GRAVELLY FAT CLAY - brown and reddish brown, moist, stiff.				8-9-9 (18)	18  27  2 	
	S-2	SS	3	3	(WR) WEATHERED ROCK SAMPLED AS [WEATHERED LIMESTONE].				50/3"	30.2 	
					BUCKET REFUSAL at 3.8 FT						
THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL											
 WL (First Encountered):		Not Observed		BORING STARTED:		06/11/2026		CAVE IN DEPTH:		Not Observed	
 WL (Completion):		Not Observed		BORING COMPLETED:		06/11/2026		HAMMER TYPE:		Automatic	
 WL (Seasonal High Water):				EQUIPMENT:		LOGGED BY:		DRILLING METHOD:		CFA (0'-3.8')	
 WL (Stabilized):				Track		JGC					
GEOTECHNICAL BOREHOLE LOG											

CLIENT: McDonald's USA, LLC				PROJECT NO.: 16:16210		BORING NO.: D-01		SHEET: 1 OF 1			
PROJECT NAME: 24-1329 Proposed McDonald's Willard, MO 01.00.14				DRILLER/CONTRACTOR: Palmerton And Parrish, Inc							
SITE LOCATION: 652 South Hunt Road, Willard, Missouri, 65781								LOSS OF CIRCULATION			
LATITUDE: 37.292036		LONGITUDE: -93.412319		STRUCTURE:		SURFACE ELEVATION: 1199		BOTTOM OF CASING			

DEPTH (FT)	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE DISTANCE (IN)	SAMPLE RECOVERY (IN)	DESCRIPTION OF MATERIAL	STRATIGRAPHY	WATER LEVELS	ELEVATION (FT)	BLOWS/6" (TCP/RS/MC/SPT-N VALUE)*	SPT	Recovery %	RQD %	Fines% F	MC%
					Topsoil [Thickness=±8"].									
	S-1	SS	18	18	(CL) LEAN CLAY - brown, moist, very stiff.				1-2-5 (7)					
	S-2	SS	18	18	(CH) FAT CLAY - trace gravel, reddish brown, moist, very stiff.			1195	2-6-12 (18)					
5.0	S-3	SS	0	0					50/0" (50/0")					
	AUGER REFUSAL at 6.0 FT													

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL

WL (First Encountered):	Not Observed	BORING STARTED: 04/29/2026	CAVE IN DEPTH: Not Observed
WL (Completion):	Not Observed	BORING COMPLETED: 04/29/2026	HAMMER TYPE: Automatic
WL (Seasonal High Water):		EQUIPMENT: Track	DRILLING METHOD: Hollow Stem Auger (0'-6')
WL (Stabilized):		LOGGED BY: PW	

GEOTECHNICAL BOREHOLE LOG

CLIENT: McDonald's USA, LLC				PROJECT NO.: 16:16210		BORING NO.: D-02		SHEET: 1 OF 1			
PROJECT NAME: 24-1329 Proposed McDonald's Willard, MO 01.00.14				DRILLER/CONTRACTOR: Palmerton And Parrish, Inc							
SITE LOCATION: 652 South Hunt Road, Willard, Missouri, 65781								LOSS OF CIRCULATION			
LATITUDE: 37.292018		LONGITUDE: -93.411719		STRUCTURE:		SURFACE ELEVATION: 1204		BOTTOM OF CASING			

DEPTH (FT)	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE DISTANCE (IN)	SAMPLE RECOVERY (IN)	DESCRIPTION OF MATERIAL	STRATIGRAPHY	WATER LEVELS	ELEVATION (FT)	BLOWS/6" (TCP/RS/MC/SPT-N VALUE)*	SPT	Recovery %	RQD %	QP	PL	MC	LL	Fines%	MC%
					Topsoil [Thickness=±12"].													
	S-1	SS	18	18	(CH) FAT CLAY - trace gravel, reddish brown, moist, stiff.				1-4-5 (9)	9					25	3		
	S-2	SS	11	11	(WR) WEATHERED ROCK SAMPLED AS [WEATHERED LIMESTONE].			1200	8-50/5" (120)	120								
					AUGER REFUSAL at 4.5 FT													

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL

 WL (First Encountered):		Not Observed		BORING STARTED:		04/29/2026		CAVE IN DEPTH:		Not Observed	
 WL (Completion):		Not Observed		BORING COMPLETED:		04/29/2026		HAMMER TYPE:		Automatic	
 WL (Seasonal High Water):				EQUIPMENT:		LOGGED BY:		DRILLING METHOD:			
 WL (Stabilized):				Track		PW		Hollow Stem Auger (0'-4.5')			

GEOTECHNICAL BOREHOLE LOG

CLIENT: McDonald's USA, LLC				PROJECT NO.: 16:16210		BORING NO.: P-01		SHEET: 1 OF 1		
PROJECT NAME: 24-1329 Proposed McDonald's Willard, MO 01.00.14				DRILLER/CONTRACTOR: Palmerton And Parrish, Inc.						
SITE LOCATION: 652 South Hunt Road, Willard, Missouri, 65781								LOSS OF CIRCULATION		
LATITUDE: 37.292175		LONGITUDE: -93.410321		STRUCTURE:		SURFACE ELEVATION: 1225		BOTTOM OF CASING		

DEPTH (FT)	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE DISTANCE (IN)	SAMPLE RECOVERY (IN)	DESCRIPTION OF MATERIAL	STRATIGRAPHY	WATER LEVELS	ELEVATION (FT)	BLOWS/6" (TCP/RS/MC/SPT-N VALUE)*	SPT	Recovery %	RQD %	Fines% F	MC%
					Topsoil [Thickness=±18"].									
	S-1	SS	18	18	(CH) GRAVELLY FAT CLAY - brown, moist, stiff.				3-6-3 (9)					
					(CH) FAT CLAY - reddish brown, moist, stiff.									
	S-2	SS	18	18					2-3-5 (8)					
5.0								1220						
	S-3	SS	18	18	(WR) WEATHERED ROCK SAMPLED AS [HIGHLY WEATHERED LIMESTONE] (possible pinnacle).				10-8-2 (10)					
END OF BORING at 7.5 FT														

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL

WL (First Encountered):	Not Observed	BORING STARTED: 04/24/2026	CAVE IN DEPTH: Not Observed
WL (Completion):	Not Observed	BORING COMPLETED: 04/24/2026	HAMMER TYPE: Automatic
WL (Seasonal High Water):		EQUIPMENT: Track	DRILLING METHOD: Hollow Stem Auger (0'-7.5')
WL (Stabilized):		LOGGED BY: PW	

GEOTECHNICAL BOREHOLE LOG

CLIENT: McDonald's USA, LLC					PROJECT NO.: 16:16210		BORING NO.: P-02		SHEET: 1 OF 1		
PROJECT NAME: 24-1329 Proposed McDonald's Willard, MO 01.00.14					DRILLER/CONTRACTOR: Palmerton And Parrish, Inc.						
SITE LOCATION: 652 South Hunt Road, Willard, Missouri, 65781									LOSS OF CIRCULATION		
LATITUDE: 37.292259			LONGITUDE: -93.410762		STRUCTURE:		SURFACE ELEVATION: 1217		BOTTOM OF CASING		

DEPTH (FT)	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE DISTANCE (IN)	SAMPLE RECOVERY (IN)	DESCRIPTION OF MATERIAL	STRATIGRAPHY	WATER LEVELS	ELEVATION (FT)	BLOWS/6" (TCP/RS/MC/SPT-N VALUE)*	SPT	Recovery %	RQD %	QP	PL	MC	LL	Fines%	MC%
					Topsoil [Thickness=±18"].													
	S-1	SS	8	8	(CH) FAT CLAY - reddish brown, stiff, with chert and limestone fragments.				3-50/2" (300)	300					46.9			
					END OF BORING at 1.7 FT													

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL

 WL (First Encountered):	Not Observed	BORING STARTED:	04/24/2026	CAVE IN DEPTH:	Not Observed
 WL (Completion):	1 FT	BORING COMPLETED:	04/24/2026	HAMMER TYPE:	Automatic
 WL (Seasonal High Water):		EQUIPMENT:	LOGGED BY:	DRILLING METHOD:	
 WL (Stabilized):		Track	PW	Hollow Stem Auger (0'-1.67')	

GEOTECHNICAL BOREHOLE LOG

CLIENT: McDonald's USA, LLC				PROJECT NO.: 16:16210		BORING NO.: P-03		SHEET: 1 OF 1		
PROJECT NAME: 24-1329 Proposed McDonald's Willard, MO 01.00.14				DRILLER/CONTRACTOR: Palmerton And Parrish, Inc.						
SITE LOCATION: 652 South Hunt Road, Willard, Missouri, 65781								LOSS OF CIRCULATION		100%
LATITUDE: 37.292275		LONGITUDE: -93.411323		STRUCTURE:		SURFACE ELEVATION: 1210		BOTTOM OF CASING		100%

DEPTH (FT)	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE DISTANCE (IN)	SAMPLE RECOVERY (IN)	DESCRIPTION OF MATERIAL	STRATIGRAPHY	WATER LEVELS	ELEVATION (FT)	BLOWS/6" (TCP/RS/MC/SPT-N VALUE)*	SPT	Recovery %	RQD %	QP	PL	MC	LL
					Topsoil [Thickness=±3"].											
	S-1	SS	18	18	(CL) LEAN CLAY - brown, moist, stiff.				5-5-6 (11)	11						
					(CH) GRAVELLY FAT CLAY - reddish brown, moist, very stiff.											
	S-2	SS	1	0	END OF BORING at 3.6 FT				50/1" (50/1")							

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL

WL (First Encountered):	Not Observed	BORING STARTED:	04/24/2026	CAVE IN DEPTH:	Not Observed
WL (Completion):	Not Observed	BORING COMPLETED:	04/24/2026	HAMMER TYPE:	Automatic
WL (Seasonal High Water):		EQUIPMENT:	Track	LOGGED BY:	PW
WL (Stabilized):				DRILLING METHOD:	Hollow Stem Auger (0'-3.58')

GEOTECHNICAL BOREHOLE LOG

CLIENT: McDonald's USA, LLC				PROJECT NO.: 16:16210		BORING NO.: P-04		SHEET: 1 OF 1							
PROJECT NAME: 24-1329 Proposed McDonald's Willard, MO 01.00.14				DRILLER/CONTRACTOR: Palmerton And Parrish, Inc.											
SITE LOCATION: 652 South Hunt Road, Willard, Missouri, 65781								LOSS OF CIRCULATION							
LATITUDE: 37.292166		LONGITUDE: -93.411147		STRUCTURE:		SURFACE ELEVATION: 1211		BOTTOM OF CASING							
DEPTH (FT)	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE DISTANCE (IN)	SAMPLE RECOVERY (IN)	DESCRIPTION OF MATERIAL	STRATIGRAPHY	WATER LEVELS	ELEVATION (FT)	BLOWS/6" (TCP/RS/MC/SPT-N VALUE)*	SPT	Recovery %	RQD %	Fines%	MC%	
					Topsoil [Thickness=±3"]. (CL) LEAN CLAY - brown, moist, stiff.										
	S-1	SS	18	18	(CH) GRAVELLY FAT CLAY - reddish brown, moist, very stiff.			1210	3-7-11 (18)	18				25.4	2.5
					AUGER REFUSAL at 3.4 FT										
THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL															
 WL (First Encountered):		Not Observed		BORING STARTED:		04/24/2026		CAVE IN DEPTH:		Not Observed					
 WL (Completion):		Not Observed		BORING COMPLETED:		04/24/2026		HAMMER TYPE:		Automatic					
 WL (Seasonal High Water):				EQUIPMENT:		LOGGED BY:		DRILLING METHOD:		Hollow Stem Auger (0'-3.42')					
 WL (Stabilized):				Track		PW									
GEOTECHNICAL BOREHOLE LOG															

CLIENT: McDonald's USA, LLC				PROJECT NO.: 16:16210		BORING NO.: P-05		SHEET: 1 OF 1			
PROJECT NAME: 24-1329 Proposed McDonald's Willard, MO 01.00.14				DRILLER/CONTRACTOR: Palmerton And Parrish, Inc.							
SITE LOCATION: 652 South Hunt Road, Willard, Missouri, 65781								LOSS OF CIRCULATION			
LATITUDE: 37.292038		LONGITUDE: -93.411326		STRUCTURE:		SURFACE ELEVATION: 1210		BOTTOM OF CASING			

DEPTH (FT)	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE DISTANCE (IN)	SAMPLE RECOVERY (IN)	DESCRIPTION OF MATERIAL	STRATIGRAPHY	WATER LEVELS	ELEVATION (FT)	BLOWS/6" (TCP/RS/MC/SPT-N VALUE)*	SPT	Recovery %	RQD %	QP	PL	MC	LL	Fines%	MC%
					Topsoil [Thickness=±17"].													
	S-1	SS	18	18	(CH) FAT CLAY - trace gravel, reddish brown, moist, stiff, with chert.				2-3-7 (10)	10					26.6	2.25		
	S-2	SS	11	11	(WR) WEATHERED ROCK SAMPLED AS [WEATHERED LIMESTONE].				15-50/5" (120)	120					1.5	39.2		
					AUGER REFUSAL at 4.4 FT													

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL

 WL (First Encountered):		Not Observed		BORING STARTED:		04/24/2026		CAVE IN DEPTH:		Not Observed	
 WL (Completion):		Not Observed		BORING COMPLETED:		04/24/2026		HAMMER TYPE:		Automatic	
 WL (Seasonal High Water):				EQUIPMENT:		LOGGED BY:		DRILLING METHOD:			
 WL (Stabilized):				Track		PW		Hollow Stem Auger (0'-4.42')			

GEOTECHNICAL BOREHOLE LOG

CLIENT: McDonald's USA, LLC				PROJECT NO.: 16:16210		BORING NO.: P-06		SHEET: 1 OF 1			
PROJECT NAME: 24-1329 Proposed McDonald's Willard, MO 01.00.14				DRILLER/CONTRACTOR: Palmerton And Parrish, Inc.							
SITE LOCATION: 652 South Hunt Road, Willard, Missouri, 65781								LOSS OF CIRCULATION			
LATITUDE: 37.291805		LONGITUDE: -93.411339		STRUCTURE:		SURFACE ELEVATION: 1208		BOTTOM OF CASING			

DEPTH (FT)	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE DISTANCE (IN)	SAMPLE RECOVERY (IN)	DESCRIPTION OF MATERIAL	STRATIGRAPHY	WATER LEVELS	ELEVATION (FT)	BLOWS/6" (TCP/RS/MC/SPT-N VALUE)*	SPT	Recovery %	RQD %	Fines%	MC%
					Topsoil [Thickness=±12"].									
	S-1	SS	18	18	(CH) FAT CLAY - trace gravel, reddish brown, moist, very stiff.				2-3-4 (7)	7				
	S-2	SS	0	0	(WR) WEATHERED ROCK SAMPLED AS [WEATHERED LIMESTONE]. AUGER REFUSAL at 3.5 FT			1205	50/0" (50/0")	50/0"				

0 10 20 30 40 50

0 20 40 60 80 100

0 1 2 3 4 5

0 20 40 60 80 100

PL MC LL

QP -○-

33.9 3

7

50/0"

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL

 WL (First Encountered):	Not Observed	BORING STARTED:	04/24/2026	CAVE IN DEPTH:	Not Observed
 WL (Completion):	Not Observed	BORING COMPLETED:	04/24/2026	HAMMER TYPE:	Automatic
 WL (Seasonal High Water):		EQUIPMENT:	LOGGED BY:	DRILLING METHOD:	
 WL (Stabilized):		Track	PW	Hollow Stem Auger (0'-3.5')	

GEOTECHNICAL BOREHOLE LOG

CLIENT: McDonald's USA, LLC				PROJECT NO.: 16:16210		BORING NO.: P-07		SHEET: 1 OF 1			
PROJECT NAME: 24-1329 Proposed McDonald's Willard, MO 01.00.14				DRILLER/CONTRACTOR: Palmerton And Parrish, Inc.							
SITE LOCATION: 652 South Hunt Road, Willard, Missouri, 65781								LOSS OF CIRCULATION			
LATITUDE: 37.291853		LONGITUDE: -93.410866		STRUCTURE:		SURFACE ELEVATION: 1215		BOTTOM OF CASING			

DEPTH (FT)	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE DISTANCE (IN)	SAMPLE RECOVERY (IN)	DESCRIPTION OF MATERIAL	STRATIGRAPHY	WATER LEVELS	ELEVATION (FT)	BLOWS/6" (TCP/RS/MC/SPT-N VALUE)*	SPT	Recovery %	RQD %	QP	PL	MC	LL	Fines%	MC%
					Topsoil [Thickness=±3"].													
	S-1	SS	18	18	(CL) LEAN CLAY - brown, moist, stiff.					3-8-8 (16)	16					25.8		
					(CH) GRAVELLY FAT CLAY - reddish brown, moist, very stiff.											2.5		
	S-2	SS	2	2					50/2" (50/2")	50/2"								
					END OF BORING at 3.8 FT													

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL

WL (First Encountered):	Not Observed	BORING STARTED: 04/24/2026	CAVE IN DEPTH: Not Observed
WL (Completion):	Not Observed	BORING COMPLETED: 04/24/2026	HAMMER TYPE: Automatic
WL (Seasonal High Water):		EQUIPMENT: Track	LOGGED BY: PW
WL (Stabilized):			DRILLING METHOD: Hollow Stem Auger (0'-3.75')

GEOTECHNICAL BOREHOLE LOG

CLIENT: McDonald's USA, LLC				PROJECT NO.: 16:16210		BORING NO.: P-09		SHEET: 1 OF 1		
PROJECT NAME: 24-1329 Proposed McDonald's Willard, MO 01.00.14				DRILLER/CONTRACTOR: Palmerton And Parrish, Inc.						
SITE LOCATION: 652 South Hunt Road, Willard, Missouri, 65781								LOSS OF CIRCULATION		
LATITUDE: 37.292681		LONGITUDE: -93.410551		STRUCTURE:		SURFACE ELEVATION: 1225		BOTTOM OF CASING		

DEPTH (FT)	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE DISTANCE (IN)	SAMPLE RECOVERY (IN)	DESCRIPTION OF MATERIAL	STRATIGRAPHY	WATER LEVELS	ELEVATION (FT)	BLOWS/6" (TCP/RS/MC/SPT-N VALUE)*	SPT	Recovery %	RQD %	Fines% F	MC%	
5.0					Topsoil [Thickness=4"].										
	S-1	SS	18	18	(CH) GRAVELLY FAT CLAY - brown and reddish brown, moist, very stiff.				13-19-16 (35)	35				14.5	3
	S-2	SS	18	18	(CH) FAT CLAY - trace gravel, reddish brown, moist, stiff.				5-8-9 (17)	17				1.5	51.6
	S-3	SS	7	7	(WR) WEATHERED ROCK SAMPLED AS [WEATHERED LIMESTONE].				12-50/1" (50/7")	50/7"				58.1	
					AUGER REFUSAL at 6.7 FT										

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL

WL (First Encountered):	Not Observed	BORING STARTED: 06/11/2026	CAVE IN DEPTH: Not Observed
WL (Completion):	Not Observed	BORING COMPLETED: 06/11/2026	HAMMER TYPE: Automatic
WL (Seasonal High Water):		EQUIPMENT: Track	LOGGED BY: JGC
WL (Stabilized):			DRILLING METHOD: CFA (0'-6.67')

GEOTECHNICAL BOREHOLE LOG

CLIENT: McDonald's USA, LLC				PROJECT NO.: 16:16210		BORING NO.: S-01		SHEET: 1 OF 1		
PROJECT NAME: 24-1329 Proposed McDonald's Willard, MO 01.00.14				DRILLER/CONTRACTOR: Palmerton And Parrish, Inc.						
SITE LOCATION: 652 South Hunt Road, Willard, Missouri, 65781								LOSS OF CIRCULATION		
LATITUDE: 37.291779		LONGITUDE: -93.41056		STRUCTURE:		SURFACE ELEVATION: 1222		BOTTOM OF CASING		

DEPTH (FT)	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE DISTANCE (IN)	SAMPLE RECOVERY (IN)	DESCRIPTION OF MATERIAL	STRATIGRAPHY	WATER LEVELS	ELEVATION (FT)	BLOWS/6" (TCP/RS/MC/SPT-N VALUE)*	RQD %		Rec %		QP - Q			
										SPT	Recovery %	RQD %	PL	MC	LL		
					Topsoil [Thickness=±18"].												
	S-1	SS	18	18	(CH) FAT CLAY - dark brown, moist, very stiff, with chert.			1220	5-7-6 (13)								
	S-2	SS	3	3	LIMESTONE, slightly weathered, Moderately hard to hard, Light gray.				50/3" (50/3")								
5.0	S-3	RC	12														
	S-4	RC	48					1215									
10.0	S-5	RC	60					1210									
END OF BORING at 14.6 FT																	

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL			
WL (First Encountered):	Not Observed	BORING STARTED: 04/24/2026	CAVE IN DEPTH: Not Observed
WL (Completion):	Not Observed	BORING COMPLETED: 04/24/2026	HAMMER TYPE: Automatic
WL (Seasonal High Water):		EQUIPMENT: Track	DRILLING METHOD: Hollow Stem Auger (0'-14.58')
WL (Stabilized):		LOGGED BY: PW	

GEOTECHNICAL BOREHOLE LOG

CLIENT: McDonald's USA, LLC				PROJECT NO.: 16:16210		BORING NO.: S-02		SHEET: 1 OF 2		
PROJECT NAME: 24-1329 Proposed McDonald's Willard, MO 01.00.14				DRILLER/CONTRACTOR: Palmerton And Parrish, Inc.						
SITE LOCATION: 652 South Hunt Road, Willard, Missouri, 65781								LOSS OF CIRCULATION		100%
LATITUDE: 37.29277		LONGITUDE: -93.410583		STRUCTURE:		SURFACE ELEVATION: 1225		BOTTOM OF CASING		100%

DEPTH (FT)	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE DISTANCE (IN)	SAMPLE RECOVERY (IN)	DESCRIPTION OF MATERIAL	STRATIGRAPHY	WATER LEVELS	ELEVATION (FT)	BLOWS/6" (TCP/RS/MC/SPT-N VALUE)*	RQD %		Rec %		QP - Q				PL				MC				LL			
										Recovery %	RQD %	Fines%	MC%	0	1	2	3	4	5	0	1	2	3	4	5				
					Topsoil [Thickness=4"].																								
	S-1	SS	18	18	(CH) GRAVELLY FAT CLAY - reddish brown, moist, stiff to hard.				2-2-5 (7)																				
5.0	S-2	SS	18	18					5-13-14 (27)																				
	S-3	SS	18	18					15-18-14 (32)																				
10.0	S-4	SS	18	18					25-15-18 (33)																				
	S-5	SS	1	1	LIMESTONE, [REC=100%, RQD=95.24%], slightly weathered, Light Gray.				50/1"																				
	S-6	RS	21	21																									

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL			
WL (First Encountered):	13 FT	BORING STARTED:	06/11/2026
WL (Completion):	Not Observed	BORING COMPLETED:	06/11/2026
WL (Seasonal High Water):		EQUIPMENT:	Track
WL (Stabilized):		LOGGED BY:	JGC
		CAVE IN DEPTH:	Not Observed
		HAMMER TYPE:	Automatic
		DRILLING METHOD:	CFA/NX CORE (0'-23.6')

GEOTECHNICAL BOREHOLE LOG

CLIENT: McDonald's USA, LLC				PROJECT NO.: 16:16210		BORING NO.: S-02		SHEET: 2 OF 2			
PROJECT NAME: 24-1329 Proposed McDonald's Willard, MO 01.00.14				DRILLER/CONTRACTOR: Palmerton And Parrish, Inc.							
SITE LOCATION: 652 South Hunt Road, Willard, Missouri, 65781								LOSS OF CIRCULATION			
LATITUDE: 37.29277		LONGITUDE: -93.410583		STRUCTURE:		SURFACE ELEVATION: 1225		BOTTOM OF CASING			

DEPTH (FT)	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE DISTANCE (IN)	SAMPLE RECOVERY (IN)	DESCRIPTION OF MATERIAL	STRATIGRAPHY	WATER LEVELS	ELEVATION (FT)	BLOWS/6" (TCP/RS/MC/SPT-N VALUE)*	RQD %		Rec %		QP							
										SPT	Recovery %	RQD %	PL	MC	LL						
20.0	S-6	RS	21	21	LIMESTONE, [REC=100%, RQD=95.24%], slightly weathered, Light Gray. (continued)			1205													
	S-7	RS	60	60																	
	S-8	RS	39	39																	
					END OF BORING at 23.6 FT																

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL

 WL (First Encountered):	13 FT	BORING STARTED:	06/11/2026	CAVE IN DEPTH:	Not Observed
 WL (Completion):	Not Observed	BORING COMPLETED:	06/11/2026	HAMMER TYPE:	Automatic
 WL (Seasonal High Water):		EQUIPMENT:	LOGGED BY:	DRILLING METHOD: CFA/NX CORE (0'-23.6')	
 WL (Stabilized):		Track	JGC		

GEOTECHNICAL BOREHOLE LOG



Rock Core B-01

Run 1: From 4.1 feet to 4.3 feet Below Existing Grade; REC = 100%, RQD = 90%

Run 2: From 4.3 feet to 9.3 feet Below Existing Grade; REC = 100%, RQD = 93%



Rock Core B-03

Run 1: From 5.8 feet to 10.8 feet Below Existing Grade; REC = 100%, RQD = 92

Rock Cores Photographs
June 23, 2026



24-1329 Proposed McDonald's
652 S. Hunt Road, Willard, MO 65781
ECS Project No. 16:16210



Rock Core B-04

Run 1: From 0.7 feet to 5.7 feet Below Existing Grade; REC = 100%, RQD = 45%
 Run 2: From 5.7 feet to 10.7 feet Below Existing Grade; REC = 100%, RQD = 92%



Rock Core S-01

Run 1: From 4.6 feet to 5.6 feet Below Existing Grade; REC = 100%, RQD = 92
 Run 2: From 5.6 feet to 9.6 feet Below Existing Grade; REC = 92%, RQD = 56
 Run 3: From 9.6 feet to 14.6 feet Below Existing Grade; REC = 100%, RQD = 82

Rock Cores Photographs
 June 23, 2026



24-1329 Proposed McDonald's
 652 S. Hunt Road, Willard, MO 65781
 ECS Project No. 16:16210



Rock Core S-02

Run 1: From 13.6 feet to 15.3 feet Below Existing Grade; REC = 100%, RQD = 95%

Run 2: From 15.3 feet to 20.3 feet Below Existing Grade; REC = 100%, RQD = 93%

Run 3: From 20.3 feet to 23.6 feet Below Existing Grade; REC = 100%, RQD = 92%

Rock Cores Photographs
June 23, 2026



24-1329 Proposed McDonald's
652 S. Hunt Road, Willard, MO 65781
ECS Project No. 16:16210

Appendix C - Laboratory Testing

Laboratory Testing Procedures

Grain Size Analysis/

Plasticity Chart(s)



LABORATORY PROCEDURES:

Moisture content determination was performed on select fine-grained soil samples in accordance with ASTM D 2216.

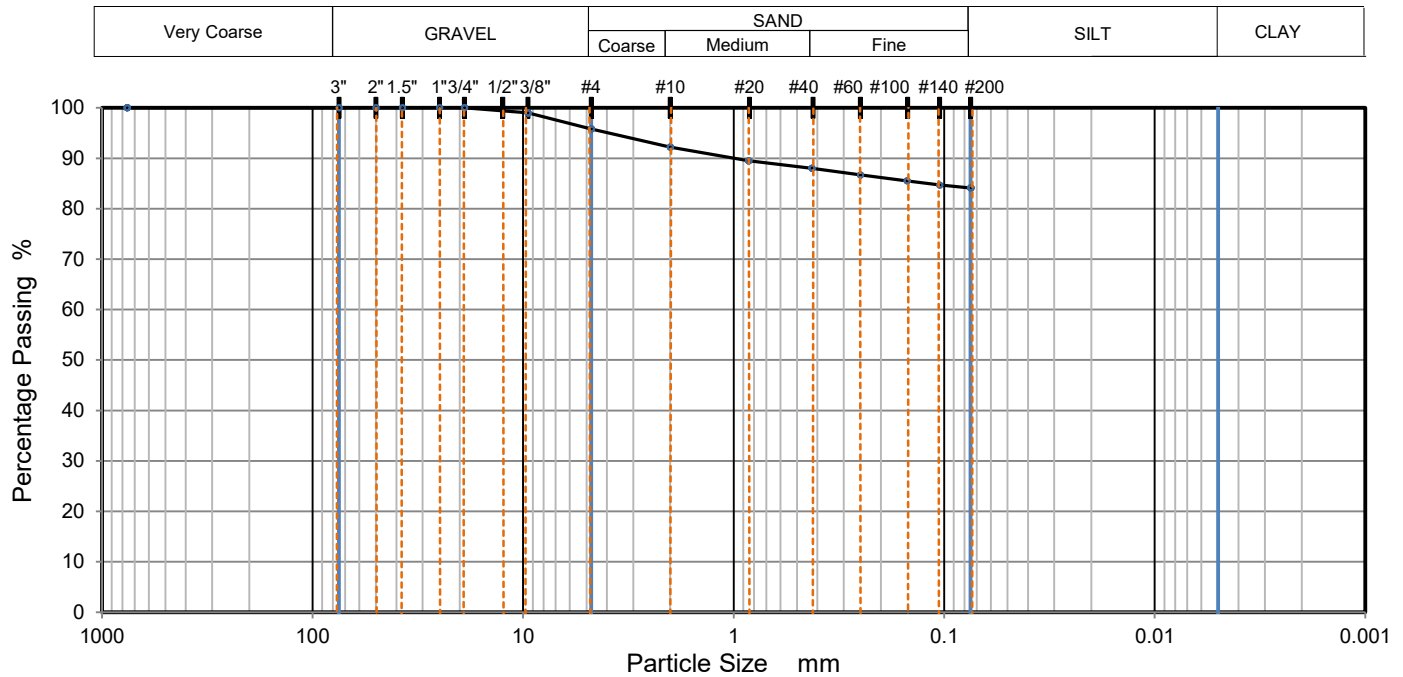
Calibrated hand penetrometer tests (Qp) were performed on select cohesive soil samples. In the hand penetrometer test, the unconfined compressive strength of a soil sample is estimated, to a maximum of 4.5 or 6 tons per square foot (tsf), depending on the penetrometer model, by measuring the resistance of a soil sample to penetration by a small, calibrated, spring-loaded cylinder.

Atterberg limits determination was performed on select fine-grained soil samples in accordance with ASTM D 4319. The Atterberg limits are a basic measure of the critical water contents of a fine-grained soil: its liquid limit, plastic limit, and shrinkage limit. Depending on its water content, a soil may appear in one of four states: solid, semi-solid, plastic and liquid. In each state, the consistency and behavior of a soil is different and consequently so are its engineering properties. Atterberg limits can also be used to help distinguish between silt and clay, and to distinguish between different types of silts and clays.

Particle size distribution, also referred to as gradation or sieve analysis, refers to the proportions by dry mass of a soil particles distributed over specified particle-size ranges. The particle size distribution is used to help classify soils for engineering purposes. Particle size distribution determination was performed on select soil samples in accordance with ASTM D 6913M-17.

Unconfined Compression tests (bedrock core sampels) were performed on selected rock core samples in accordance with ASTM D7012 (Method C).

PARTICLE SIZE DISTRIBUTION



TEST RESULTS (ASTM D6913M-17-METHOD A)

Sieving		Hydrometer Sedimentation	
Particle Size	% Passing	Particle Size mm	% Passing
3"	100		
2"	100		
1 1/2"	100		
1"	100		
3/4"	100		
3/8"	99		
#4	96		
#10	92		
#20	90		
#40	88		
#60	87		
#100	86		
#140	85		
#200	84		

Dry Mass of sample, g

191.1

Sample Proportions	% dry mass
Very coarse, >3" sieve	0
Gravel, 3" to #4 sieve	4
Coarse Sand, #4 to #10 sieve	4
Medium Sand, #10 to #40	4
Fine Sand, #40 to #200	4
Fines <#200	84

USCS	CH	Liquid Limit	57	D90	0.996	D50		D10	
AASHTO	A-7-6	Plastic Limit	22	D85	0.120	D30		Cu	
USCS Group Name	Fat clay with sand	Plasticity Index	35	D60		D15		Cc	

Project: 24-1329 Proposed McDonald's Willard, MO 01.00.14

Client: McDonald's USA, LLC

Sample Description: Fat Clay with Sand

Sample Source: B-01

Project No.: 16:16210

Depth (ft): 1.0 - 2.5

Sample No.: S-1

Date Reported: 6/3/2026



Office / Lab

Address

Office Number / Fax

ECS Midwest LLC - Chicago

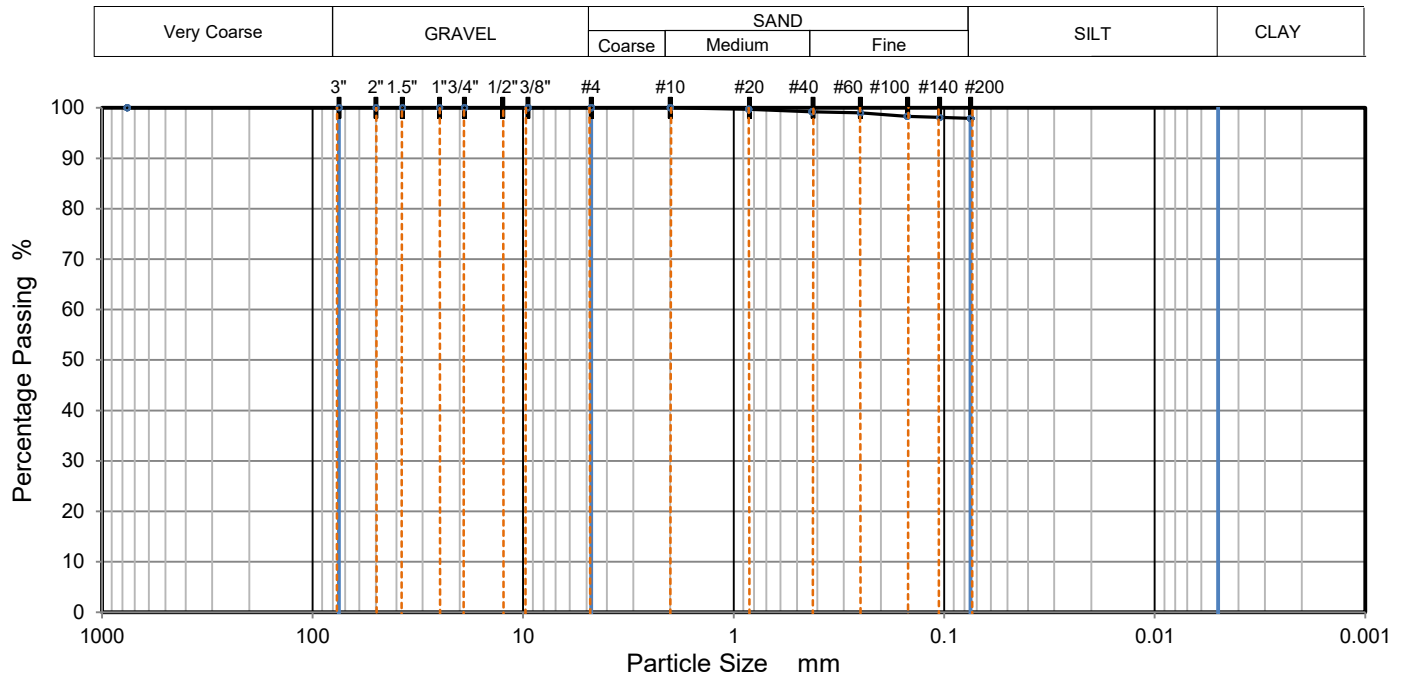
1575 Barclay Boulevard
Buffalo Grove, IL 60089

(847)279-0366

(847)279-0369

Tested by	Checked by	Approved by	Date Received	Remarks
OTorres	JSutton	JSutton		

PARTICLE SIZE DISTRIBUTION



TEST RESULTS (ASTM D6913M-17-METHOD A)

Sieving		Hydrometer Sedimentation	
Particle Size	% Passing	Particle Size mm	% Passing
3"	100		
2"	100		
1 1/2"	100		
1"	100		
3/4"	100		
3/8"	100		
#4	100		
#10	100		
#20	100		
#40	99		
#60	99		
#100	98		
#140	98		
#200	98		

Dry Mass of sample, g

77.8

Sample Proportions	% dry mass
Very coarse, >3" sieve	0
Gravel, 3" to # 4 sieve	0
Coarse Sand, #4 to #10 sieve	0
Medium Sand, #10 to #40	1
Fine Sand, #40 to #200	1
Fines <#200	98

USCS	CH	Liquid Limit	90	D90		D50		D10	
AASHTO	A-7-5	Plastic Limit	37	D85		D30		Cu	
USCS Group Name	Fat clay	Plasticity Index	53	D60		D15		Cc	

Project: 24-1329 Proposed McDonald's Willard, MO 01.00.14

Project No.: 16:16210

Client: McDonald's USA, LLC

Depth (ft): 1.0 - 2.5

Sample Description: Elastic Silt

Sample No.: S-1

Sample Source: B-05

Date Reported: 6/3/2026



Office / Lab

Address

Office Number / Fax

ECS Midwest LLC - Chicago

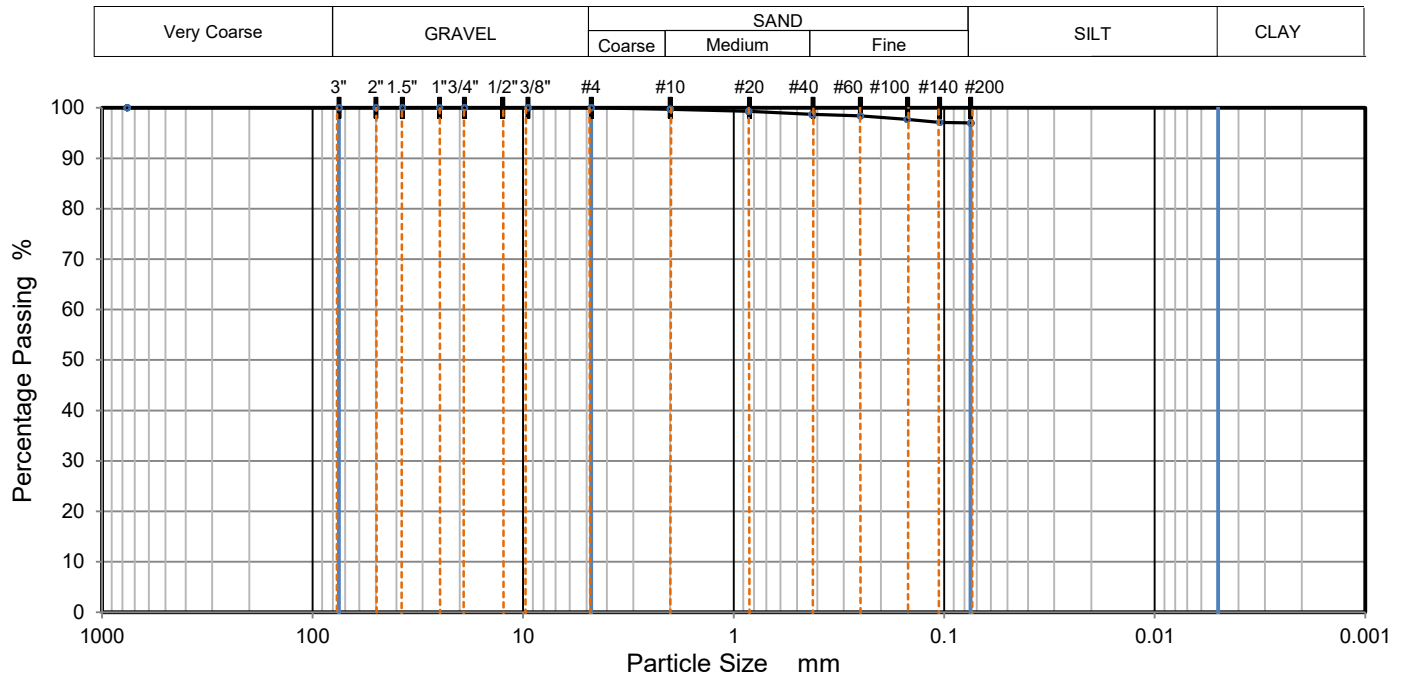
1575 Barclay Boulevard
Buffalo Grove, IL 60089

(847)279-0366

(847)279-0369

Tested by	Checked by	Approved by	Date Received	Remarks
OTorres	JSutton	JSutton		

PARTICLE SIZE DISTRIBUTION



TEST RESULTS (ASTM D6913M-17-METHOD A)

Sieving		Hydrometer Sedimentation	
Particle Size	% Passing	Particle Size mm	% Passing
3"	100		
2"	100		
1 1/2"	100		
1"	100		
3/4"	100		
3/8"	100		
#4	100		
#10	100		
#20	99		
#40	99		
#60	98		
#100	98		
#140	97		
#200	97		

Dry Mass of sample, g

164.6

Sample Proportions	% dry mass
Very coarse, >3" sieve	0
Gravel, 3" to # 4 sieve	0
Coarse Sand, #4 to #10 sieve	0
Medium Sand, #10 to #40	1
Fine Sand, #40 to #200	2
Fines <#200	97

USCS	CL	Liquid Limit	43	D90		D50		D10	
AASHTO	A-7-6	Plastic Limit	20	D85		D30		Cu	
USCS Group Name	Lean clay	Plasticity Index	23	D60		D15		Cc	

Project: 24-1329 Proposed McDonald's Willard, MO 01.00.14

Project No.: 16:16210

Client: McDonald's USA, LLC

Depth (ft): 3.5 - 5.0

Sample Description: Lean Clay

Sample No.: S-2

Sample Source: D-01

Date Reported: 6/3/2026



Office / Lab

Address

Office Number / Fax

ECS Midwest LLC - Chicago

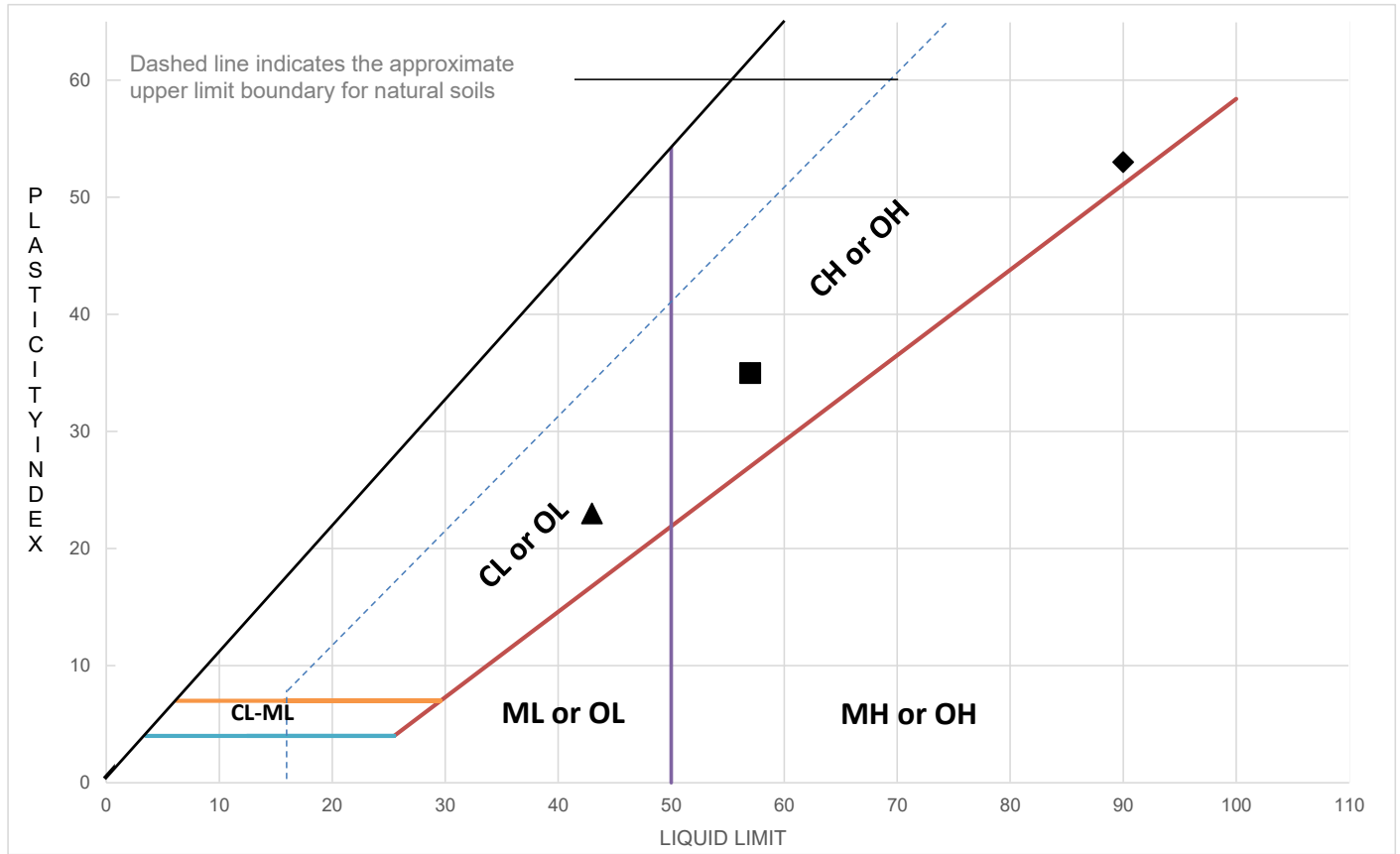
1575 Barclay Boulevard
Buffalo Grove, IL 60089

(847)279-0366

(847)279-0369

Tested by	Checked by	Approved by	Date Received	Remarks
OTorres	JSutton	JSutton		

LIQUID AND PLASTIC LIMITS TEST REPORT



TEST RESULTS (ASTM D4318-10 (MULTIPOINT TEST))

	Sample Location	Sample Number	Sample Depth (ft)	LL	PL	PI	%<#40	%<#200	AASHTO	USCS	Material Description
■	B-01	S-1	1.00-2.50	57	22	35	88.0	84.1	A-7-6	CH	Fat Clay with Sand
◆	B-05	S-1	1.00-2.50	90	37	53	99.2	97.9	A-7-5	CH	Fat Clay
▲	D-01	S-2	3.50-5.00	43	20	23	98.7	97.0	A-7-6	CL	Lean Clay

Project: 24-1329 Proposed McDonald's Willard, MO 01.00.14
Client: McDonald's USA, LLC

Project No.: 16:16210
Date Reported: 6/3/2026



Office / Lab
ECS Midwest LLC - Chicago

Address
1575 Barclay Boulevard
Buffalo Grove, IL 60089

Office Number / Fax
(847)279-0366
(847)279-0369

Tested by	Checked by	Approved by	Date Received
OTorres	JSutton	JSutton	

Appendix D - Supplemental Documents

Geophysical Testing/Mapping Report

Important Information about This Geotechnical Engineering Report

May 8, 2026

ECS Midwest, LLC
1575 Barclay Boulevard
Buffalo Grove, Illinois 60089

Attn: Mr. Eram Iqbal, P.E.
Email: eiqbal@ecslimited.com

RE: Geophysics Summary Letter
S. Hunt Road, Willard, MO
PPI Project Number: 26-1943

Dear Mr. Iqbal:

Palmerton and Parrish, Inc. (PPI) appreciates the opportunity to submit this Geophysical Testing Letter for the vacant lot off S. Hunt Road in Willard, Missouri. The purpose of this letter is to provide geophysical testing results and interpretation.

Testing Performed

PPI has performed a total of three (3) Electrical Resistivity Tomography (ERT) lines for the subject site across the proposed McDonalds location as requested by the Client. The ERT layout is shown in the attached Figure 1. Resistivity lines were performed with the Dipole-Dipole Testing method. Electrode spacings were at 5-foot lengths for a total resistivity line length of approximately 315 feet. The maximum depth of the resistivity testing was approximately 72 feet near the center of the resistivity lines.

Resistivity testing was performed with an Abem Terrameter LS2 resistivity meter. PPI's system uses a total of 64 electrode positions. Data was reduced utilizing ZondRes2D and individual resistivity sections are attached as Figures R1 through R3.

Interpretation Challenges

When reviewing ERT data it is important to understand the limits of resistivity testing. Below is a list of important interpretive challenges that should be considered:

- Resistivity is not a direct test of material type and values cannot be directly correlated to a specific material. For instance, dry soils may have the same resistance as that of intact bedrock or saturated porous bedrock may have the same resistance as very moist clays;
- ERT tests have resolutions of the area tested, meaning that items smaller than the resolution of the test may not appear in the test results;

- ERT resolutions are based on the electrode spacing. For this project electrode spacing was at 5-feet and items significantly smaller than 5 feet will likely not appear in the resistivity tests;
- Resolution of the ERT lines also slightly decrease with depth; and
- Data is generally collected and plotted in a triangular shape (as shown in the ZondRes2D models on the attached Figures R1 through R3) and extrapolated to the rectangular cross-section for the Inverse Model Resistivity Section. Accordingly, data in the lower right and left corners of the Inverse Models is less reliable than from data in the areas of the Measured Apparent Resistivity Sections and has been removed. Additionally, due to the calculations performed to establish the Inverse Model, the far sides of the model are highly influenced from the edges of the data collections and should also be considered as less reliable data.

Collected Data

As noted above, PPI collected 3-lines of data noted as Resistivity Lines 1, 2, and 3. Resistivity line directions ran from northwest to southeast and southwest to northeast and electrode locations were collected with PPI's RTK GPS system. Collected data was returned to PPI's office for inversion. Prior to inversion, PPI corrected the electrode elevations based on measurements made in the field. Additionally, PPI reviewed the field measurements and removed negative resistivity data and data that was suspect and generally outside the capabilities of the equipment used. PPI then inverted the collected data to develop the Inverse Model Resistivity Sections.

Root Mean Square Error (RMSE) of inverted sections show how the inverted data model matches with the measured values. Generally, the RMSE is desired to be low and it is considered industry standard to be below 15 percent. PPI's RMSE of the inverse models shown in the attached Figures R1, R2, and R3 were 2.6%, 2.7% and 2.9%.

PPI has provided color balanced and topographically adjusted inverse models in the attached Figures R1, R2, and R3. Additionally, PPI has also included Interpretation of the resistivity testing and the ZondRes2D outputs showing the Measured Apparent Resistivity, Calculated Apparent Resistivity, and Raw Inverse Model Sections of each Resistivity Line.

Interpretation of Data

PPI has included some notes of interpretation of the resistivity data in the attached Figures R1, R2, and R3. PPI interpreted the data based on what is anticipated to be intact bedrock, weathered bedrock, fat clay soil, and surficial soil areas. Interpretations are based on the amount of resistance noted in the final inverse model, PPI's experience in the area, and limited boring information.

Based on the results of the resistivity testing, the subject site showed some karstic bedrock conditions in the form of pinnacled and irregular bedrock which is common in the Willard area. Based on the resistivity testing, PPI anticipates that the subject site contains lean clay soils followed by fat clay soils with chert gravel and bedrock pinnacles over variably weathered limestone bedrock materials. Large floating boulders/residual bedrock (often mistaken for intact bedrock) is also anticipated at the subject site based on the resistivity results. Areas of shallow pinnacles and/or shallow floating boulders should be anticipated and may be encountered during excavations. Some bedrock is exposed at the surface at the east end of the site.

Based on PPI's experience with karst in this area, karst features such as sinkholes often form between or on the sides of bedrock pinnacles. This is likely due to bedrock jointing forming the pinnacles and troughs within the bedrock. Bedrock joints often contain increased weathering due to the fracture in bedrock allowing water, soil, and other sediments to move through these features. PPI identified a likely bedrock joint or possible karst conduit along Resistivity Lines 1, 2, and 3.

Interpretation of Resistivity Line 1

Resistivity Line 1 generally runs from the northwest to southeast direction at the subject site as shown in Figure 1 and Figure R1. Boring B-01 is located on the resistivity line and extended to bedrock at 4.08 feet and then cored rock to 9.33 feet.

Resistivity testing within this location indicated an undulating bedrock surface. Pinnacled and weathered rock was noted at the surface near the eastern end of the line. Isolated areas of gravel and boulders consisting of chert are anticipated within the soils. Fat clays and lean clays are anticipated to be the primary soil type at the subject site with variable amounts of gravel. Where soils are noted to be more conductive (as noted by the blue colors), a decrease in gravel content is anticipated, or moisture content is higher, or both.

As noted in Figure R1, an area of increased conductivity may indicate karst activity within this location; however, no indication of a sinkhole was apparent at the surface. These features could be soil filled bedrock joints but could also have conduits that allow soil to be transported into the subsurface. These conduits tend to become active when large amounts of water infiltrate the soils down to bedrock. Based on knowledge

of bedrock jointing in the area, this feature could extend to the northeast and southwest.

Interpretation of Resistivity Line 2

Resistivity line 2 runs from the southwest to northeast direction at the subject site as shown in Figure 1 and Figure R2.

Borings B-02, B-03, and B-04 were noted to be within this resistivity line. B-02 extend to bedrock at 2.75 feet. B-03 extended to bedrock at 5.83 feet and cored rock to 10.83 feet. B-04 extended to bedrock at 0.8 feet and cored rock to 10.75 feet.

Resistivity testing within this location indicated an undulating bedrock surface. Pinnacled and weathered rock was noted at the surface near the eastern end of the line. Pinnacled and weathered rock may be present at or near the surface along the entirety of the line. Isolated areas of gravel and boulders consisting of chert are anticipated within the soils. Fat clays and lean clays are anticipated to be the primary soil type at the subject site with variable amounts of gravel. Where soils are noted to be more conductive (as noted by the blue colors), a decrease in gravel content is anticipated, or moisture content is higher, or both.

As noted in Figure R2 two areas of increased conductivity may indicate karst activity within this location; however, no indication of a sinkhole was apparent at the surface. These features could be soil filled bedrock joints but could also have conduits that allow soil to be transported into the subsurface. These conduits tend to become active when large amounts of water infiltrate the soils down to bedrock. Based on knowledge of bedrock jointing in the area, this feature could extend to the northeast and southwest.

Interpretation of Resistivity Line 3

Resistivity line 3 runs from the northwest to the southeast direction at the subject site as shown in Figure 1 and Figure R3.

Borings P-03, P-04, B-03 and B-05 were noted to be within this resistivity line. P-03 extended to bedrock at 3.58 feet. P-04 extended to bedrock at 3.4 feet. B-03 extended to bedrock at 5.83 feet and cored rock to 10.83 feet. B-05 extended to bedrock at 4.4 feet.

Resistivity testing within this location indicated an undulating bedrock surface. Pinnacled and weathered rock was noted at the surface near the eastern end of the line. Isolated areas of gravel and boulders consisting of chert are anticipated within the soils. Fat clays and lean clays are anticipated to be the primary soil type at the subject site with variable amounts of gravel. Where soils are noted to be more

conductive (as noted by the blue colors), a decrease in gravel content is anticipated, or moisture content is higher, or both.

As noted in Figure R3 an areas of increased conductivity may indicate karst activity within this location; however, no indication of a sinkhole was apparent at the surface. These features could be soil filled bedrock joints but could also have conduits that allow soil to be transported into the subsurface. These conduits tend to become active when large amounts of water infiltrate the soils down to bedrock. Based on knowledge of bedrock jointing in the area, this feature could extend to the northeast and southwest.

Closure

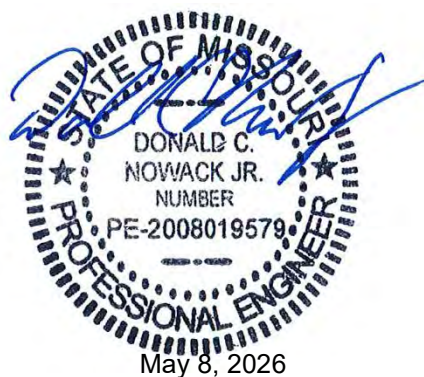
Should you have any questions or need additional information, please feel free to call our office (417-864-6000).

PALMERTON & PARRISH, INC.

By:



Donald C. Nowack, P.E., R.G.
Geological Engineer

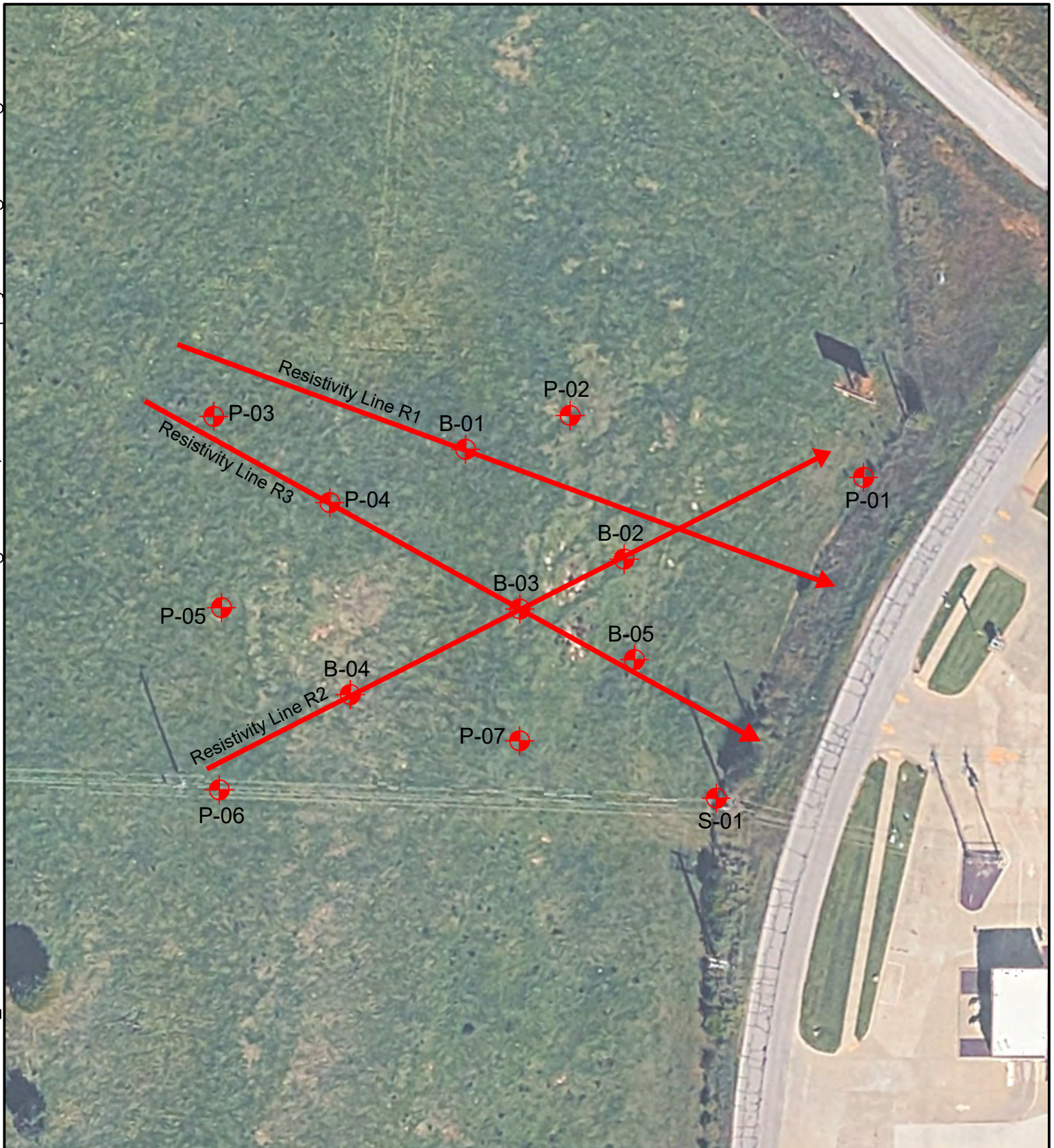


Attachments – Appendix I

Figure 1 – Site Plan
Figure R1 – Resistivity Line 1
Figure R2 – Resistivity Line 2
Figure R3 – Resistivity Line 3

APPENDIX I

FIGURES



LEGEND



Boring Location

SCALE
1" = 60'

Project: S. Hunt Road, Willard, MO
Client: ECS Midwest, LLC

Boring & Resistivity Line Location Plan

Date: May 4, 2026

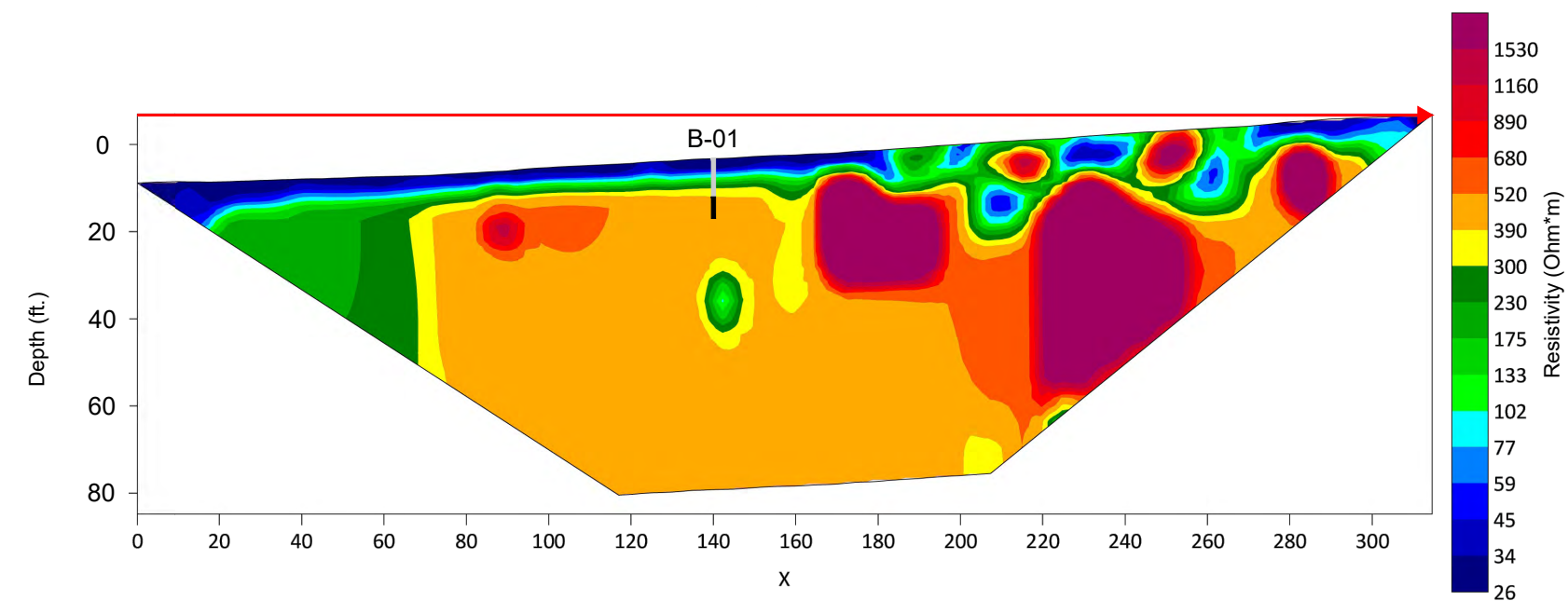
Project Number: 26-1943



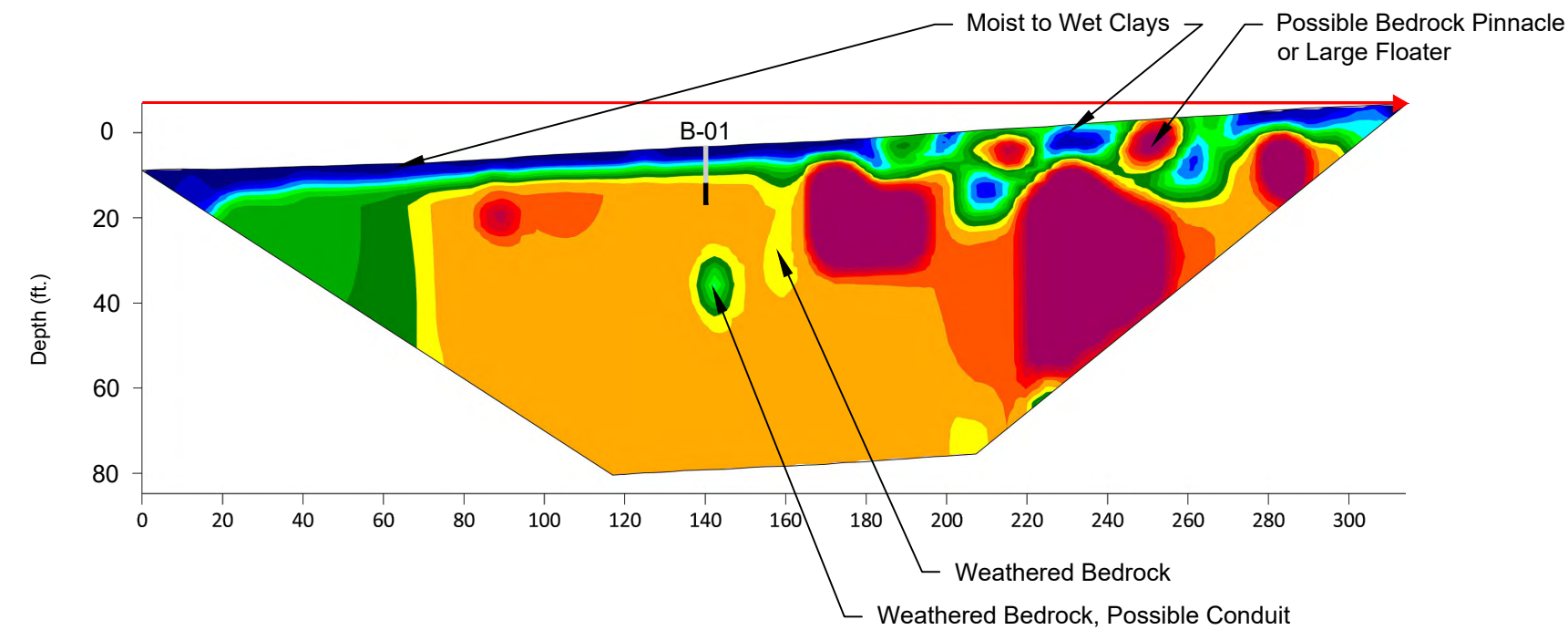
PALMERTON & PARRISH, INC.
GEOTECHNICAL AND MATERIALS ENGINEERS/MATERIALS TESTING LABORATORIES/ENVIRONMENTAL SERVICES

FIGURE 1

Resistivity Cross Section - Line R1



Resistivity Interpretation - Line R1

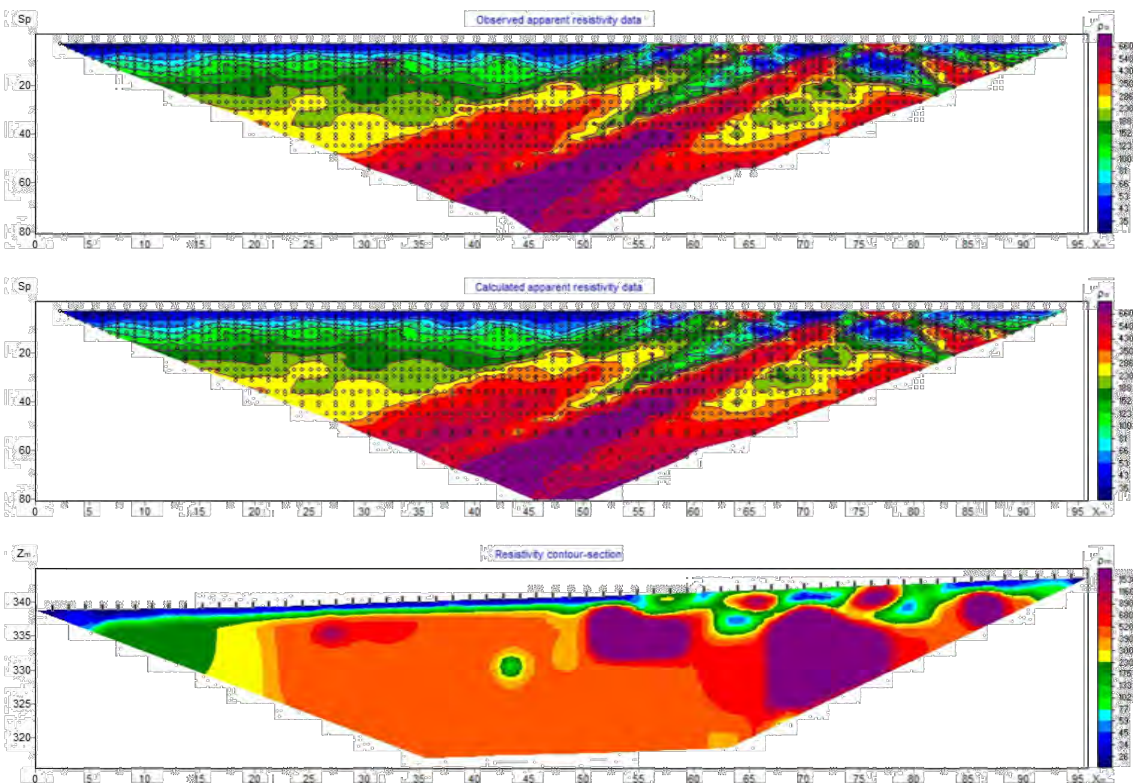


INTERPRETATIONS

- Pinnaced and weathered limestone expected from 102 to 390 Ohm*m (dark green through yellow).
- Highly conductive areas (dark blue) on edges likely due to edge effects.

LIMITATIONS

- Resistivity testing has a limited resolution that is based on the electrode spacing.
- Resistivity resolution also decreases with depth of penetration.
- Electrode spacing at the subject site was 5 feet.
- Features significantly smaller than 5 feet are not anticipated to show in the resistivity testing.



Instrument: ABEM Terrameter LS2	ABEM Terrameter LS2
ECS Midwest, LLC	Z. Wormington
S. Hunt Road, Willard, MO	RMS Error = 2.6%

Resistivity Line R1 Layout



Project: S. Hunt Road, Willard, MO
Client: ECS Midwest, LLC

Resistivity Line R1

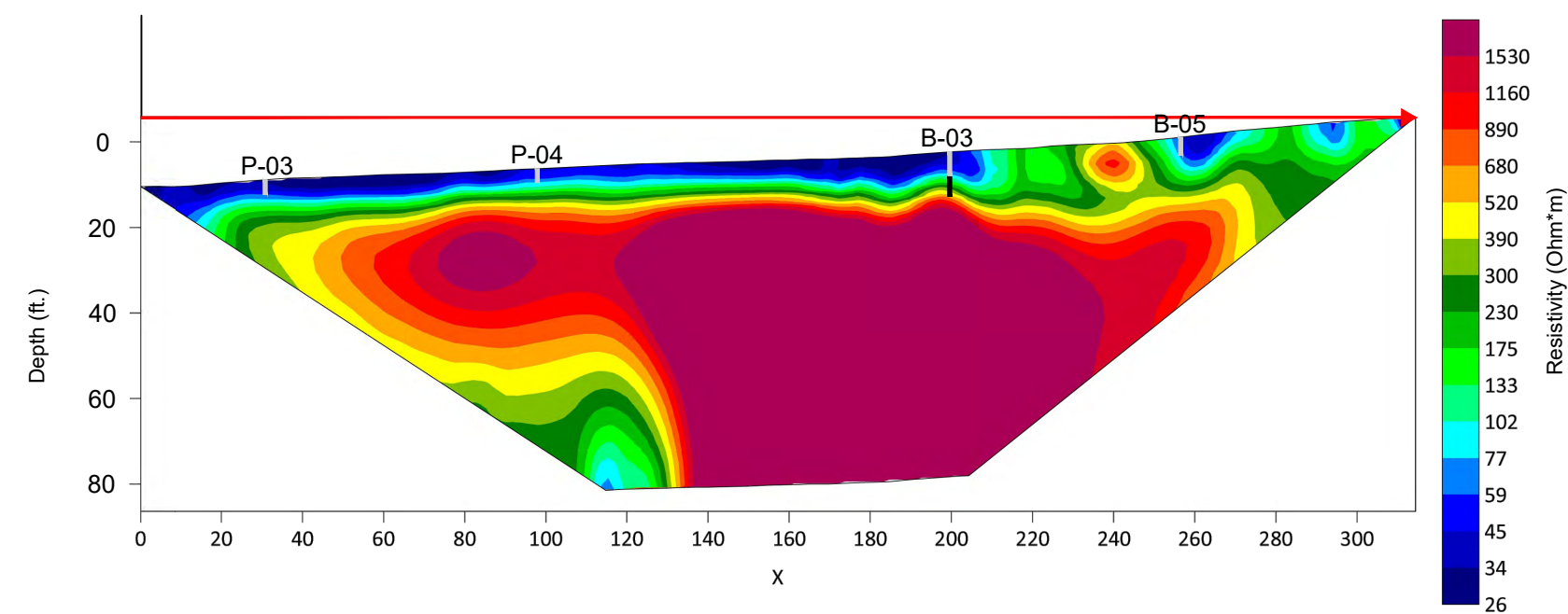
Date: May 4, 2026

Project Number: 26-1943

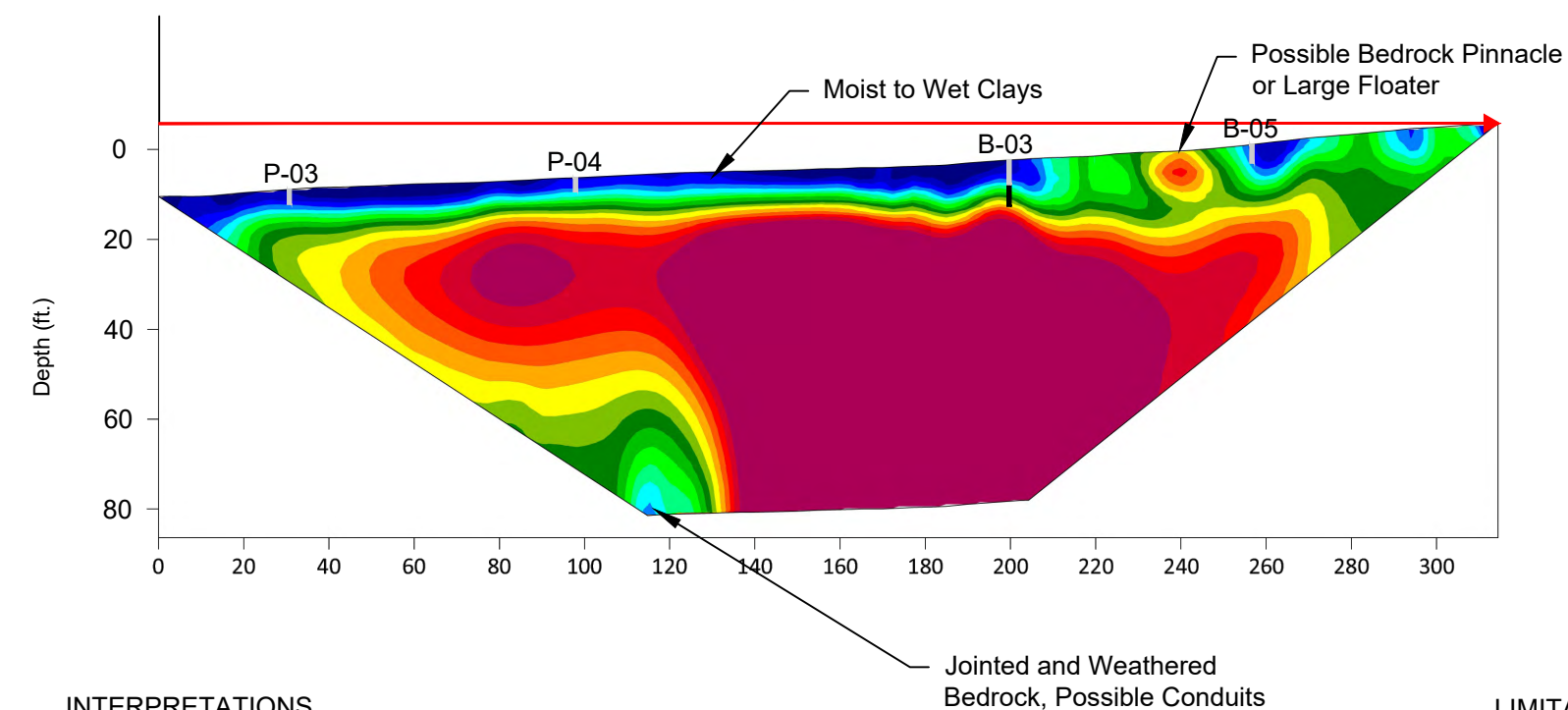
PPI PALMERTON & PARRISH, INC.
GEOTECHNICAL AND MATERIALS ENGINEERS/MATERIALS TESTING LABORATORIES/ENVIRONMENTAL SERVICES

FIGURE R1

Resistivity Cross Section - Line R3



Resistivity Interpretation - Line R3

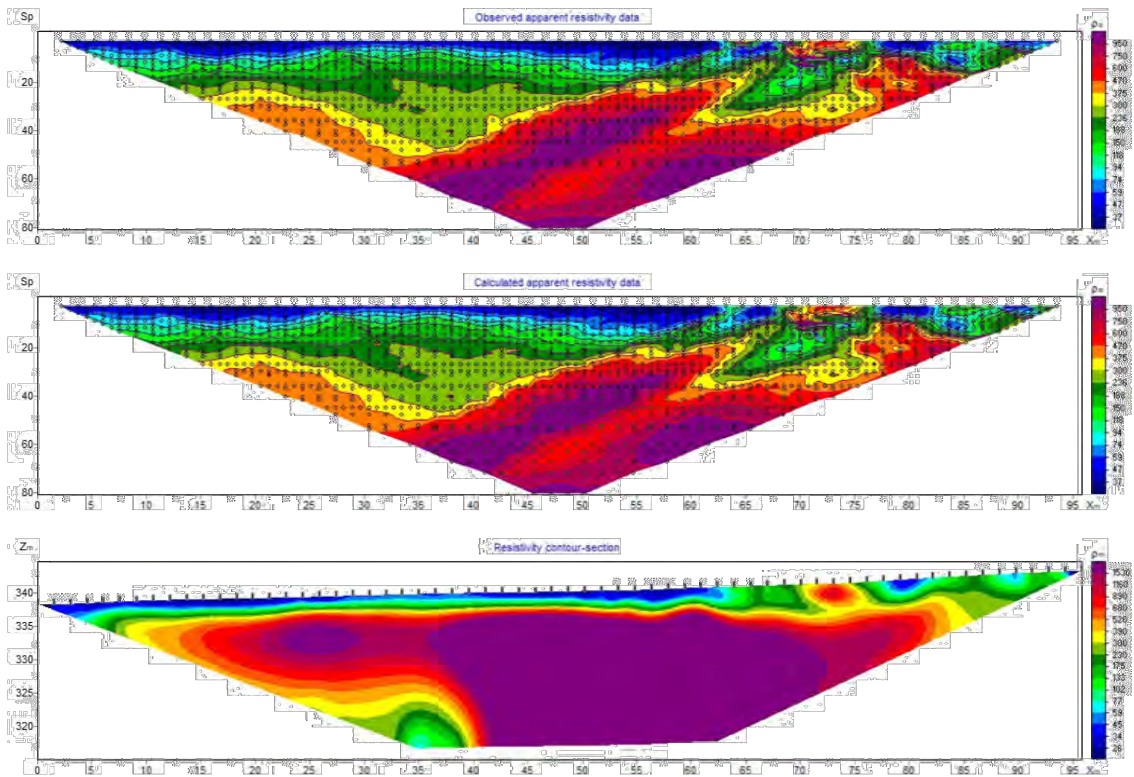


INTERPRETATIONS

- Pinnaced and weathered limestone expected from 102 to 390 Ohm*m (dark green through yellow).
- Highly conductive areas (dark blue) on edges likely due to edge effects.

LIMITATIONS

- Resistivity testing has a limited resolution that is based on the electrode spacing.
- Resistivity resolution also decreases with depth of penetration.
- Electrode spacing at the subject site was 5 feet.
- Features significantly smaller than 5 feet are not anticipated to show in the resistivity testing.



Instrument: ABEM Terrameter LS2	ABEM Terrameter LS2
ECS Midwest, LLC	Z. Wormington
S. Hunt Road, Willard, MO	RMS Error = 2.9%

Resistivity Line R3 Layout



Project: S. Hunt Road, Willard, MO
Client: ECS Midwest, LLC

Resistivity Line R3

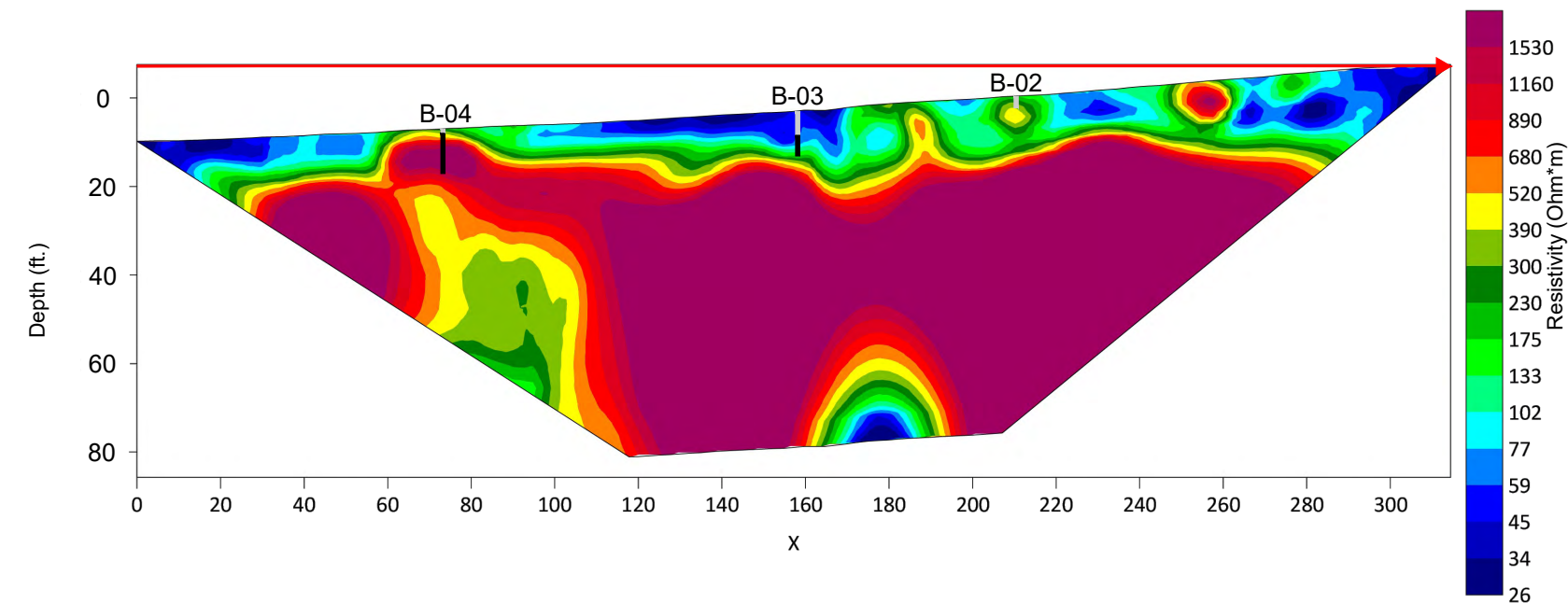
Date: May 4, 2026

Project Number: 26-1943

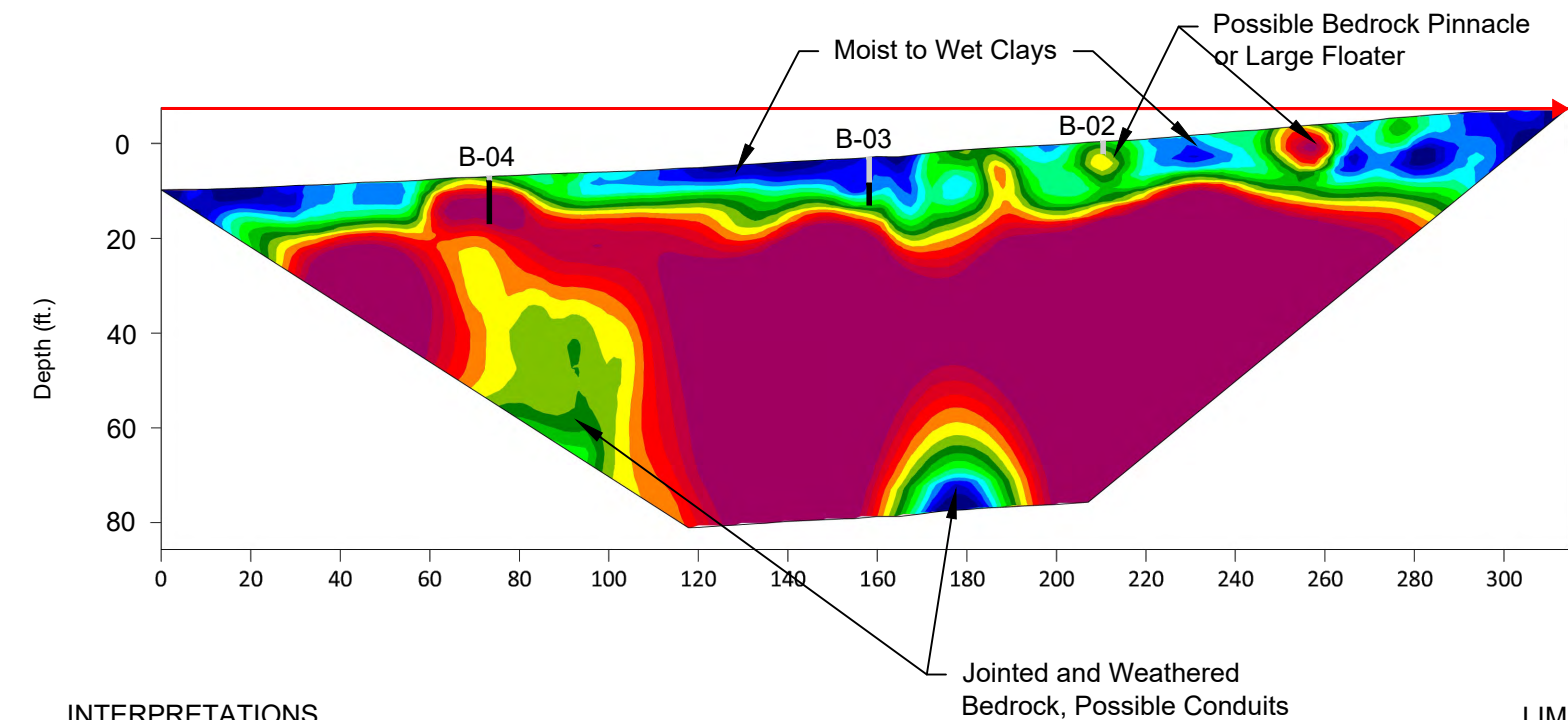
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FIGURE R3

Resistivity Cross Section - Line R2



Resistivity Interpretation - Line R2

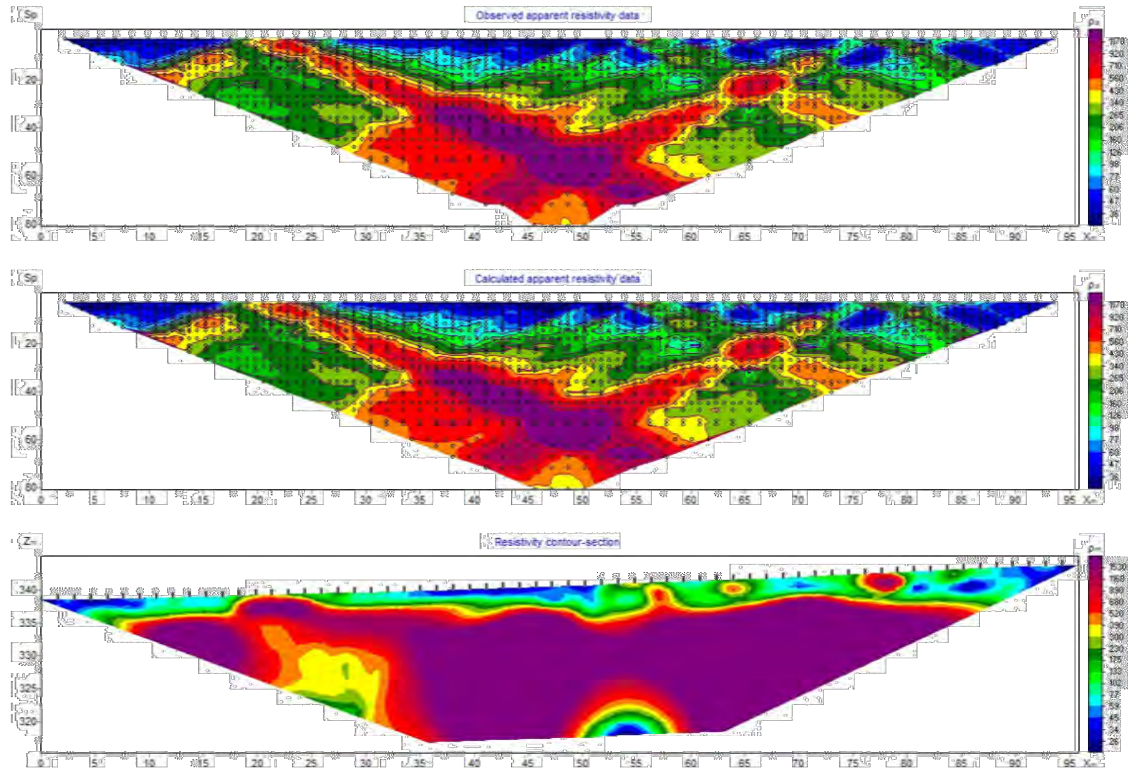


INTERPRETATIONS

- Pinnaced and weathered limestone expected from 102 to 390 Ohm*m (dark green through yellow).
- Highly conductive areas (dark blue) on edges likely due to edge effects.

LIMITATIONS

- Resistivity testing has a limited resolution that is based on the electrode spacing.
- Resistivity resolution also decreases with depth of penetration.
- Electrode spacing at the subject site was 5 feet.
- Features significantly smaller than 5 feet are not anticipated to show in the resistivity testing.



Instrument: ABEM Terrameter LS2	ABEM Terrameter LS2
ECS Midwest, LLC	Z. Wormington
S. Hunt Road, Willard, MO	RMS Error = 2.7%

Resistivity Line R2 Layout



Project: S. Hunt Road, Willard, MO
Client: ECS Midwest, LLC

Resistivity Line R2

Date: May 4, 2026

Project Number: 26-1943

PPI PALMERTON & PARRISH, INC.
GEOTECHNICAL AND MATERIALS ENGINEERS/MATERIALS TESTING LABORATORIES/ENVIRONMENTAL SERVICES

FIGURE R2

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.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, you can benefit from a lowered exposure to problems associated with subsurface conditions at project sites and development of them that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed herein, contact your GBA-member geotechnical engineer. Active engagement in GBA exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Understand the Geotechnical-Engineering Services Provided for this Report

Geotechnical-engineering services typically include the planning, collection, interpretation, and analysis of exploratory data from widely spaced borings and/or test pits. Field data are combined with results from laboratory tests of soil and rock samples obtained from field exploration (if applicable), observations made during site reconnaissance, and historical information to form one or more models of the expected subsurface conditions beneath the site. Local geology and alterations of the site surface and subsurface by previous and proposed construction are also important considerations. Geotechnical engineers apply their engineering training, experience, and judgment to adapt the requirements of the prospective project to the subsurface model(s). Estimates are made of the subsurface conditions that will likely be exposed during construction as well as the expected performance of foundations and other structures being planned and/or affected by construction activities.

The culmination of these geotechnical-engineering services is typically a geotechnical-engineering report providing the data obtained, a discussion of the subsurface model(s), the engineering and geologic engineering assessments and analyses made, and the recommendations developed to satisfy the given requirements of the project. These reports may be titled investigations, explorations, studies, assessments, or evaluations. Regardless of the title used, the geotechnical-engineering report is an engineering interpretation of the subsurface conditions within the context of the project and does not represent a close examination, systematic inquiry, or thorough investigation of all site and subsurface conditions.

Geotechnical-Engineering Services are Performed for Specific Purposes, Persons, and Projects, and At Specific Times

Geotechnical engineers structure their services to meet the specific needs, goals, and risk management preferences of their clients. A geotechnical-engineering study conducted for a given civil engineer

will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client.

Likewise, geotechnical-engineering services are performed for a specific project and purpose. For example, it is unlikely that a geotechnical-engineering study for a refrigerated warehouse will be the same as one prepared for a parking garage; and a few borings drilled during a preliminary study to evaluate site feasibility will not be adequate to develop geotechnical design recommendations for the project.

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project or purpose;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, the reliability of a geotechnical-engineering report can be affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If you are the least bit uncertain about the continued reliability of this report, contact your geotechnical engineer before applying the recommendations in it. A minor amount of additional testing or analysis after the passage of time – if any is required at all – could prevent major problems.*

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read the report in its entirety. Do not rely on an executive summary. Do not read selective elements only. *Read and refer to the report in full.*

You Need to Inform Your Geotechnical Engineer About Change

Your geotechnical engineer considered unique, project-specific factors when developing the scope of study behind this report and developing the confirmation-dependent recommendations the report conveys. Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the elevation, configuration, location, orientation, function or weight of the proposed structure and the desired performance criteria;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project or site changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept*

responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.

Most of the “Findings” Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site’s subsurface using various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing is performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgement to form opinions about subsurface conditions throughout the site. Actual site-wide subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team through project completion to obtain informed guidance quickly, whenever needed.

This Report’s Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, they are not final, because the geotechnical engineer who developed them relied heavily on judgement and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* exposed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals’ misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a continuing member of the design team, to:

- confer with other design-team members;
- help develop specifications;
- review pertinent elements of other design professionals’ plans and specifications; and
- be available whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction-phase observations.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note*

conspicuously that you’ve included the material for information purposes only. To avoid misunderstanding, you may also want to note that “informational purposes” means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. This happens in part because soil and rock on project sites are typically heterogeneous and not manufactured materials with well-defined engineering properties like steel and concrete. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include 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.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a “phase-one” or “phase-two” environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually provide environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures.* If you have not obtained your own environmental information about the project site, ask your geotechnical consultant for a recommendation on how to find environmental risk-management guidance.

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, the engineer’s services were not designed, conducted, or intended to prevent migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, *proper implementation of the geotechnical engineer’s recommendations will not of itself be sufficient to prevent moisture infiltration.* Confront the risk of moisture infiltration by including building-envelope or mold specialists on the design team. *Geotechnical engineers are not building-envelope or mold specialists.*



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