McCLELLAND CONSULTING ENGINEERS, INC.
DESIGNED TO SERVE

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April 17, 2025

WDD ARCHITECTS
5050 Northshore Lane
North Little Rock, Arkansas 72118

CERTIFICATE OF AUTHORIZATION
McCLELLAND CONSULTING ENGINEERS, INC.
No. 24
ARKANSAS - ENGINEER

ATTN: Mr. Wallie Sprick
Executive Vice President, COO

RE: Preliminary Geotechnical Site Assessment for
DOC Franklin County Correctional Facility
Charleston, Arkansas
MCE Project Number: 25-9610

Dear Mr. Sprick:

We are submitting herewith the report for the Preliminary Geotechnical Site Assessment on the above-referenced project. We appreciate the opportunity to provide this service to you. If there are any questions regarding the Geotechnical Investigation, please contact us.

Sincerely yours,

McCLELLAND CONSULTING ENGINEERS, INC.
DESIGNED TO SERVE

STATE OF ARKANSAS
LICENSED PROFESSIONAL ENGINEER
No. 16434
STEVEN J. HEAD

04/17/2025

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Enclosure: Preliminary Geotechnical Site Assessment

PRELIMINARY GEOTECHNICAL SITE ASSESSMENT

DOC Franklin County Correctional Facility

MCE Project Number: 25-9610

Charleston, Arkansas

FOR

WD&D Architects
5050 Northshore Lane
North Little Rock, Arkansas 72118

Executive Summary

This is a report of findings of the Preliminary Geotechnical Site Assessment for the Department of Corrections (DOC) Franklin County Correctional Facility project near Charleston, Arkansas. This report includes detailed information on subsurface conditions and existing surface materials in addition to providing preliminary recommendations for site development, foundations, minimum pavement sections, and potential re-use of on-site materials. The significant findings listed below should not be used separately from the further discussion provided in the body of this report.

- MCE conducted a Geotechnical Investigation consisting of 27 project borings.
- Groundwater was encountered in a "perched" condition within two (2) project borings, B-01 and B-13, at depths of approximately 3.5 and 2.5 feet, respectively.
    - The Project Team should consider general dewatering measures to be required throughout the course of construction as well as the consideration towards groundwater mitigation beneath structure areas, such as waterproofing and underdrain systems. These items will also be dependent on final grading plans.
- Materials resulting auger refusal were encountered at all 27 project boring locations at depths ranging from approximately 1.5 to 9.75 feet below the existing surface elevations (auger refusal elevations ranging from approximately 570.98 to 687.51 feet).
    - Within 12 boring locations (B-01 through B-12), rock coring techniques were utilized, once auger refusal materials were encountered, to advance the borings to depths ranging from approximately 8.5 to 20 feet below the existing surface elevations.
- MCE recommends that the Contractor anticipates a minimum of 12 inches of initial stripping to be necessary across the project extents to fully remove the existing surface materials from the site.
    - Any remnants of previous developments (including, but not limited to, foundations, residential utilities, etc.), as well as all organics and otherwise deleterious materials, should be removed full-depth from beneath planned structure areas.
    - Additional stripping of up to two (2) feet may be needed in areas where mature trees and dense vegetation exist so that roots and other organic matter are properly removed.
- It is recommended that a shallow foundation system composed of individual (spread) and continuous footings be considered for the currently-expected structural features of the project.
    - This system is recommended as being adequate for the project scope when column and wall loads do not exceed 200 kips (unit of force equal to 1000 pounds) and four (4) kips per linear foot (klf) if foundation elements were to bear directly on in-situ rock materials.
    - Foundation elements may also bear directly on suitable in-situ Stratum II or Stratum III materials, or on properly placed and compacted select fill material. Suitable in-situ soils were generally encountered in the upper two (2) feet below the existing surface elevations within the "target areas".
        - Foundations bearing on select fill or suitable in-situ soils are recommended as being adequate for the project scope when column and wall loads do not exceed 100 kips and 2.5 klf, respectively.

        - It is recommended that the project carry an initial preliminary budget for the placement of a minimum of one (1) foot of select fill material below all structure foundation elements within the understood "target areas".

- Should the final structure loading conditions and/or site grading warrant the use of a deep foundation system, it is recommended that the project team consider the implementation of a drilled pier or micropile system.
    - More information regarding these items can be found in *Section 10.6* of this report.
- Based on the data obtained from this preliminary investigation, stable subgrade materials for project pavements are preliminary anticipated to be encountered within the upper two (2) feet below the existing surface elevations across the investigated area.
    - For preliminary planning purposes, it is recommended that the Design Team budget for the placement of two (2) feet of imported select fill materials to be placed beneath all project pavements.
- Any select fill material planned or required for the project is recommended to be a borrow material of locally available silty or clayey gravel or clayey sand meeting Unified Soils Classifications System (USCS) as a GC (Clayey Gravel), GM (Silty Gravel), SM (Silty Sand) or SC (Clayey Sand) material and having a Plasticity Index of 35 or less, a Liquid Limit of 45 or less, and a maximum of 40% passing the No. 200 sieve. Variations to this may be considered and representative select fill material samples should be submitted to the Geotechnical Engineer for approval prior to use on the project.
    - Based on the materials encountered during the investigation, existing on-site subgrade materials are anticipated to be suitable for use as select fill if they meet the parameters stated previously. Any material to be used as select fill on the project should be reviewed and approved by the Geotechnical Engineer. Reuse of the on-site rock material for either base course or "select fill" may be utilized through rock processing techniques. Should this be considered by the Design Team, further coordination with MCE should occur.
    - Some of the encountered on-site materials, particularly those with higher sand and gravel contents (SC, SM, and GM materials), will be more resilient and applicable as "select fill" below project structure and pavement features.
    - The on-site CL (Lean Clay) and ML (Silt) materials are recommended for use as general site fill. These materials are **not** recommended for use under any structures and pavement areas.
    - Alternatively, locally available shale materials may be utilized as select fill on the project provided that the shale satisfies the stipulations listed previously. Any shale material utilized as select fill should be compacted to 98% of the maximum dry density, as determined by ASTM D1557, at a moisture content within two (2) percent of optimum. Shale fill should not be used as an alternative to Class 7 base.
- ***A Final Geotechnical Investigation will be required to verify the recommendations and considerations stated in this report. The recommendations presented are based on the preliminary information available at the time of preparing this report.***

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Appendices

1.0 Introduction

McClelland Consulting Engineers, Inc. (MCE) conducted a Preliminary Geotechnical Site Assessment for the planned Department of Corrections (DOC) Franklin County Correctional Facility project near Charleston, Arkansas. The investigation was requested and authorized by Mr. Wallie Sprick with WD&D Architects and Ms. Lindsey Wallace with the DOC, with the intention of exploring the subsurface soil conditions within the planned project development area to provide preliminary recommendations for site development, foundations, minimum pavement sections, and the potential re-use of on-site materials.

It should be noted that a Final Geotechnical Investigation and Report should be conducted to provide construction-ready recommendations relating to the planned development.

2.0 Existing Site Description

The project site is located at 6310 South Highway 215, Charleston, Arkansas. The property is comprised of four (4) parcels with Franklin County Parcel IDs 002-01577-000, 002-01575-000, 002-00720-000, 002-00724-000, for a total area of approximately 828 acres.

It is understood that the site currently has three (3) natural gas wells, with Permit Numbers 35081, 13279, and 24586. Well number 35081 is understood to be *plugged and abandoned*, while the remaining two (2) are understood to be active wells. The remainder of the lot is understood to have largely been utilized for agricultural purposes, while also containing a single residence. An overhead high-voltage electric transmission line is understood to bisect the western portion of the site, from the northwest corner, through the south-central portion of the property

Based on observations made while on-site, it is known that one (1) single-family residence exists on the lot, as well as approximately six (6) ancillary structures. A gravel driveway with a length of approximately 0.8 miles is understood to connect the residence to Highway 215, while dirt paths were observed to connect throughout the remainder of the lot.

On-site vegetation was observed to include low to high-cut grass, prairie-type vegetation, and trees of varying size. Ponds are present in the center, east, northwest, and southwest parts of the site, with streams that generally connect to Onion Creek in the southern portion of the site.

Topographically, the site exhibits a rolling terrain, with a general slope from the north down to the south; maximum grade differentials are estimated to be on the order of 160 feet.

3.0 Preliminary Project Scope

It is understood that the subject property is planned to be developed as the new Franklin County Correctional Facility for the use of the Arkansas Department of Corrections. The project is in the very early stages; as such, site plans and other specific project documentation was not available at the time of preparing this report.

The correctional facility development is understood to include an approximately 120 to 250-acre main security area with housing, the control tower, food services, and primary facilities, while a wastewater treatment plant and water storage facility is also expected to be constructed on the property for use by the correctional facility. Additionally, a portion of the lot is planned to be utilized for agricultural use (cattle and a facility-use garden). At the time of preparing this proposal, two (2) general areas are understood to be in primary consideration for the main facility development. These areas are noted on the enclosed Boring Layout, which may be referenced on Plate 1 in Appendix A.

4.0 Field Investigation

Based on the understood project scope, MCE conducted a Preliminary Geotechnical Site Assessment consisting of 27 project borings. Each project boring had a planned target depth of 20.0 feet below the existing surface elevations, or until auger refusal materials were encountered, whichever was less. An additional budget of up to 80.0 feet of rock coring, to be utilized at the discretion of the Geotechnical Engineer, was also included in the project scope of work. As previously noted, a Boring Layout is provided on Plate 1 of Appendix A for reference.

4.1 Project Borings

The project borings were conducted using both a CME-45B truck-mounted drill rig and Diedrich D-50 Turbo track-mounted drill rig, each utilizing 4.5-inch diameter solid stem augers. Soil samples were obtained at the depths indicated on the boring logs with the use of a two (2) inch diameter split-spoon sampler. The split-spoon sampler was driven by blows from a 140-pound automatic hammer dropped from a fixed height of 30 inches.

The number of blows required to drive the split-spoon sampler the final 12 inches of an 18-inch drive, or portion thereof, is referred to as the Standard Penetration value, N, and is recorded on the boring logs in units of blows-per-foot. Water-based rock coring techniques utilizing a NQ-2 size bit were used to obtain samples of in-situ rock materials at project borings B-01 through B-12. Where rock coring operations were not conducted, final drilled depths are shown as the depths achieved by the split-spoon sampler. In addition to Standard Penetration Testing (SPT), the field tests performed included visual soil and rock classifications, groundwater observations, and rock recovery (REC) and rock quality designation (RQD) measurements.

The visual soil classifications are given on the boring logs, which can be referenced in Appendix B on Plates 2 through 28; a key to the terms and symbols on the boring logs is provided on Plate 29. Table 1 on the following page provides details of the project borings.

Table 1: Field Investigation Details

Boring ID	Existing Surface Elevations (feet)	Existing Surface Material and Thickness	Depth of Groundwater (feet)	Auger Refusal Depth (feet)	Auger Refusal Elevation (feet)	Total Amount of Rock Coring Conducted (feet)	Total Depth Investigated (feet)	End of Boring Elevation (feet)
B-01	648.50	Topsoil (9”)	3.5	4.0	644.50	6.0	10.0	638.50
B-02	648.49	Topsoil (8”)	-	4.5	643.99	8.0	12.5	635.99
B-03	624.32	Topsoil (4”)	-	4.5	619.82	5.0	9.5	614.82
B-04	609.85	Topsoil (8”)	-	3.0	606.85	8.5	11.5	598.35
B-05	624.20	Topsoil (7”)	-	2.5	621.70	9.0	11.5	612.70
B-06	628.07	Topsoil (10”)	-	6.0	622.07	10.0	16.0	612.07
B-07	637.33	Topsoil (6”)	-	4.5	632.83	10.0	14.5	622.83
B-08	663.57	Topsoil (7”)	-	3.0	660.57	17.0	20.0	643.57
B-09	685.03	Topsoil (5”)	-	4.0	681.00	11.0	15.0	670.03
B-10	648.56	Topsoil (5”)	-	3.5	645.06	5.5	8.5	640.06
B-11	596.87	Topsoil (6”)	-	3.5	593.37	6.0	9.5	587.37
B-12	632.37	Topsoil (7”)	-	2.5	629.87	7.5	10.0	622.37
B-13	622.35	Topsoil (8”)	2.5	9.75	612.60	-	9.75	612.60
B-14	636.71	Topsoil (10”)	-	2.0	634.71	-	2.0	634.71
B-15	663.41	Topsoil (9”)	-	7.0	656.41	-	7.0	656.41
B-16	685.08	Topsoil (7”)	-	2.5	682.58	-	2.5	682.58
B-17	692.01	Topsoil (3”)	-	4.5	687.51	-	4.5	687.51
B-18	661.75	Topsoil (7”)	-	4.5	657.25	-	4.5	657.25
B-19	637.87	Topsoil (2”)	-	4.5	633.37	-	4.5	633.37
B-20	643.20	Topsoil (4”)	-	4.5	638.70	-	4.5	638.70
B-21	608.55	Topsoil (7”)	-	4.0	604.55	-	4.0	604.55
B-22	587.78	Topsoil (5”)	-	2.0	585.78	-	2.0	585.78
B-23	576.35	Topsoil (9”)	-	1.5	574.85	-	1.5	574.85
B-24	573.98	Topsoil (4”)	-	3.0	570.98	-	3.0	570.98
B-25	592.39	Topsoil (4”)	-	3.5	588.89	-	3.5	588.89
B-26	627.34	Topsoil (8”)	-	5.5	621.84	-	5.5	621.84
B-27	622.77	Topsoil (10”)	-	2.5	620.27	-	2.5	620.27

NOTES: Surface Elevations shown in Table 1 are rounded to the nearest 0.01 foot and are based on MCE Topographic Survey Data. The corresponding end of boring elevations are based on these measurements.

4.2 Encountered Groundwater Conditions

Groundwater was encountered in a “perched” condition within two (2) project borings, B-01 and B-13, at depths of approximately 3.5 and 2.5 feet, respectively. Installation and periodic measurement of monitoring wells would be required to establish seasonal piezometric surfaces below the project site. Project grading should be designed to properly discharge any surface water that may develop following precipitation events.

Any groundwater or perched water must be removed prior to the placement of fill or construction materials, if encountered during construction. To help reduce the potential for issues related to groundwater, it is recommended that earthwork operations take place during typically drier portions of the calendar year (June through September). During earthwork operations it should be expected that typical dewatering measures will be required to maintain a desirable construction schedule. Should they be utilized under the final project scope, installation of drilled piers should expect dewatering measures to be necessary to complete the excavation of drilled shafts. Additional details are provided in *Section 10.6.1* of this report.

4.3 Encountered Auger Refusal Materials

Auger refusal is generally defined as the point at which a boring encounters material through which it can no longer be advanced using traditional auger drilling techniques. Refusal is somewhat subjective and is dependent on the type of drilling equipment used and the down pressures exerted by the drill rig.

At the time of this investigation, materials resulting auger refusal were encountered at all 27 project boring locations at depths ranging from approximately 1.5 to 9.75 feet below the existing surface elevations (auger refusal elevations ranging from approximately 570.98 to 687.51 feet). Within 12 boring locations (B-01 through B-12), rock coring techniques were utilized once auger refusal materials were encountered to advance the borings to depths ranging from approximately 8.5 to 20 feet below the existing surface elevations. Additional information pertaining to the local geology and how it affects the project site can be found in the *Local Geology of the Project Site* section of this report (Section 7.0).

5.0 Laboratory Analysis

Laboratory tests were performed on the soil and rock samples recovered from the borings to determine the engineering properties of the project strata. The tests performed on soil samples collected from the borings included moisture content, Atterberg Limits, and sieve analyses, while the tests performed on the rock samples included unit weight and compressive strength measurements. Results of laboratory testing for the project borings are provided on the boring logs and in the Laboratory Testing Results in Appendix C.

Table 2 below shows the relevant test method specifications utilized on the project.

Table 2: Laboratory Test Method Specifications

Test Designation	Test Method
ASTM D2488	Standard Practice for Description and Identification of Soils (Visual)
ASTM D2487	Standard Practice for Classification of Soils for Engineering Purpose (USCS)
ASTM D2216	Standard Test Method for Lab Determination of Water Content of Soil
ASTM D6913	Standard Test Method for Particle-Size Distribution of Soils Using Sieve Analysis
ASTM D4318	Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils
ASTM D7012	Standard Test Methods for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens Under Varying States of Stress and Temperatures

Additional testing relating to soil chemical composition was conducted on samples collected from selected project boring locations at depths of approximately six (6) inches below the existing surface elevations in order to determine their suitability regarding landscaping on the project. The results of these analyses are provided in Table 3 on the following page.

Table 3: Landscape Soil Test Analysis

Boring ID	Estimated CEC - (ECEC) (cmolc/kg)	pH	P (ppm)	K (ppm)	Ca (ppm)	Mg (ppm)	Fe (ppm)	Mn (ppm)	Zn (ppm)	SO4-S (ppm)
B-04	10	6.7	134	82	1360	56	121	105	6.3	6
B-10	8	5.7	133	81	829	54	154	72	3.6	13
B-14	19	6.8	395	164	2957	215	280	47	23.6	12
B-16	8	6.3	41	47	1051	78	222	137	7.0	9
B-20	13	5.8	244	203	1304	177	333	100	26.1	13
B-22	8	5.4	84	164	544	72	191	71	5.1	7

CEC = Cation Exchange Capacity
ECEC = Effective Cation Exchange Capacity
cmolc/kg = centimol positive charge per kilogram of soil
pH = Potential of Hydrogen
P = Phosphorus
K = Potassium
Ca = Calcium
Mg = Magnesium
Fe = Iron
Mn = Manganese
Zn = Zinc
SO4-S = Sulfate-S
ppm = Parts per million

6.0 On-Site Soil Conditions

The following sections provide information regarding on-site conditions at the project location. This information includes descriptions of the existing soil types, imagery showing the approximate location of the existing soil types, and details about the local geology.

6.1 United States Department of Agriculture (USDA) Soil Types and Map

The following soil type exists in the project area according to current USDA soil maps, with the description from the Natural Resources Conservation Service (NRCS). The project site is located in Franklin County in western Arkansas. The soil types that exist in the project area according to current USDA soil maps are briefly detailed in Table 4 on the following page.

Table 4: USDA Local Soil Types

USDA Soil Type	USDA Symbol	USDA Descriptions
Nella Gravelly Fine Sandy Loam	AgC	The Nella series consists of very deep, well drained, moderately permeable soils. These soils formed in alluvium or colluvium and in residuum of limestone, sandstone and shale. They are on hillsides, benches, and foot slopes. Slopes range from three (3) to eight (8) percent.
Enders Gravelly Silt Loam	EnC2/EnD2	The Enders series consists of deep, well drained, very slowly permeable soils that formed in loamy and clayey residuum from shale, or interbedded shale and sandstone. These soils are on nearly level to moderately steep upland mountaintops and ridges and gently sloping to very steep mountain side slopes and foot slopes. Slopes range from three (3) to 20 percent.
Linker Fine Sandy Loam	LnC	The Linker series consists of moderately deep, well drained, moderately permeable soils that formed in loamy residuum from sandstone. These soils are on hills and mountains. Slopes range from three (3) to eight (8) percent.
Montevallo-Mountainburg Complex	MmD/MmE	The Montevallo series consists of shallow, well drained, moderately permeable soils that formed in loamy residuum from shale, siltstone, and sandstone. These soils are on hillslopes and ridges. The Mountainburg series consists of shallow, well drained, moderately permeable soils that formed in loamy residuum from sandstone. These soils are on hills and mountains. Slopes range from one (1) to 40 percent.
Mountainburg Gravelly Fine Sandy Loam	MtC	The Mountainburg series consists of shallow, well drained, moderately permeable soils that formed in loamy residuum from sandstone. These soils are on hills and mountains. Slopes range from three (3) to eight (8) percent.
Mountainburg Stony Fine Sandy Loam	MuD	The Mountainburg series consists of shallow, well drained, moderately permeable soils that formed in loamy residuum from sandstone. These soils are on hills and mountains. Slopes range from one (1) to 12 percent.
Pickwick Silt Loam	PsB2	The Pickwick series consists of very deep, well drained, moderately permeable soils on stream terraces. These soils formed in old alluvium or in a silty mantle one (1) to three (3) feet thick. Slopes range from one (1) to three (3) percent.

Figure 1 on the following page provides imagery of the approximate site location and how it relates to the existing soil type.

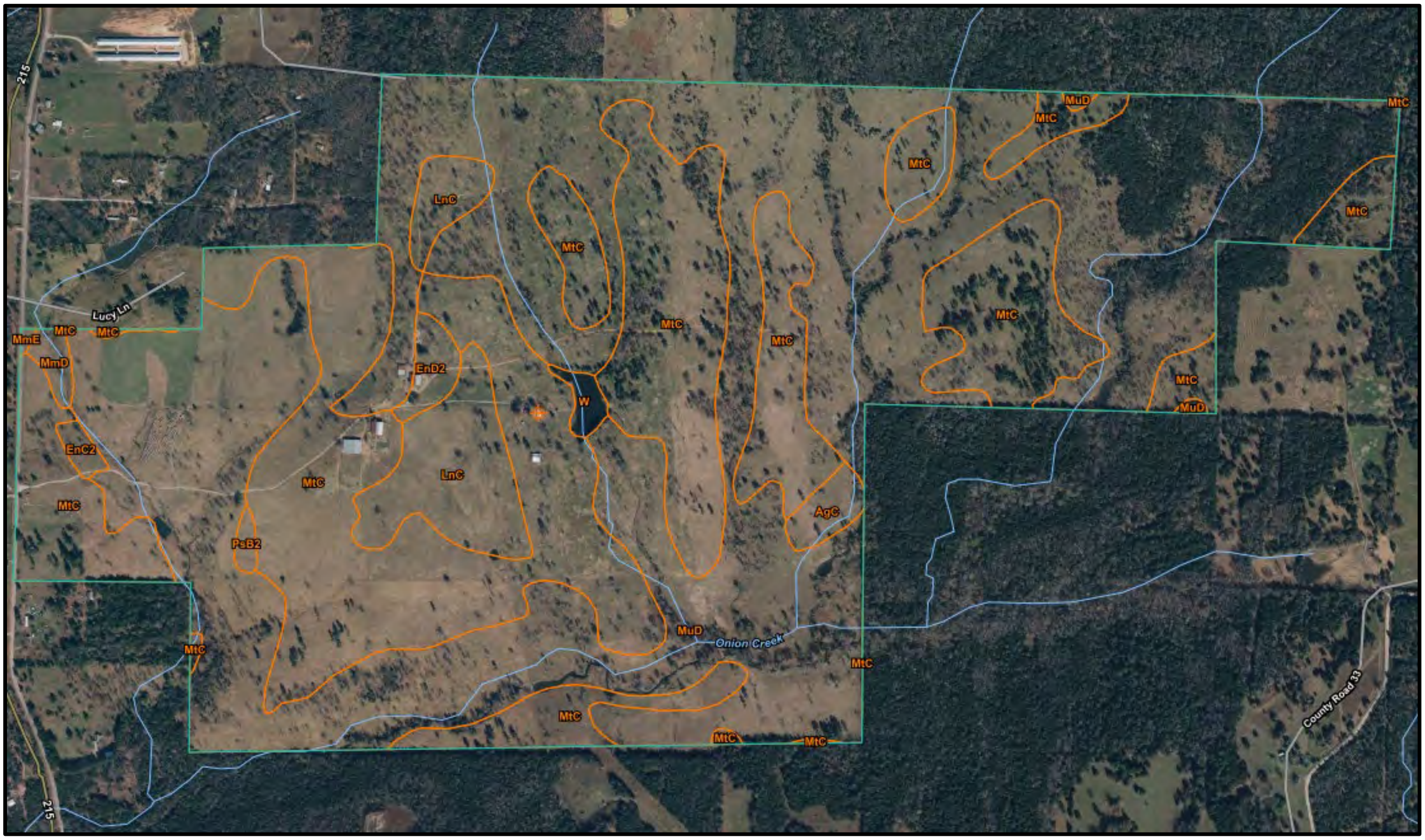

Figure 1: USDA Soil Survey Report Image
The image was produced by the United States Department of Agriculture.
The green outline represents the approximate project extent.

7.0 Local Geology of the Project Site

According to maps and literature published by the United States Geological Survey (USGS) and the Arkansas Geological Survey (AGS), the project site is underlain by the Middle Pennsylvanian aged (318 to 299 million years ago) Hartshorne Sandstone Formation.

A brief description from the Stratigraphic Summary of Arkansas – Information Circular 36 (IC-36) of the local geologic formations is provided on the following page.

7.1 Hartshorne Sandstone

The Hartshorne is normally a brown to light gray, massive, frequently cross-bedded, medium-grained sandstone. It is the first continuous sandstone underlying the Lower Hartshorne Coal. The formation is a prominent ledge-former under favorable structural conditions. A few fragmental plant fossils have been noted in the formation.

The Hartshorne rests with minor unconformity on the Atoka Formation. The unit's thickness ranges from about 10 to 300 feet. Named for exposures near Hartshorne in Pittsburg County, Oklahoma.

7.2 Structural Geology of the Project Site

It is prudent to mention that the project site is in close proximity to a many known faults and/or fault zones. These faults are known to trend from the east to the west. Modern literature indicates that these are not currently active.

Figure 2 on the following page provides a visual of the local geologic formations in relation to the project site. The heavier black lines represent the known faults in the surrounding areas.

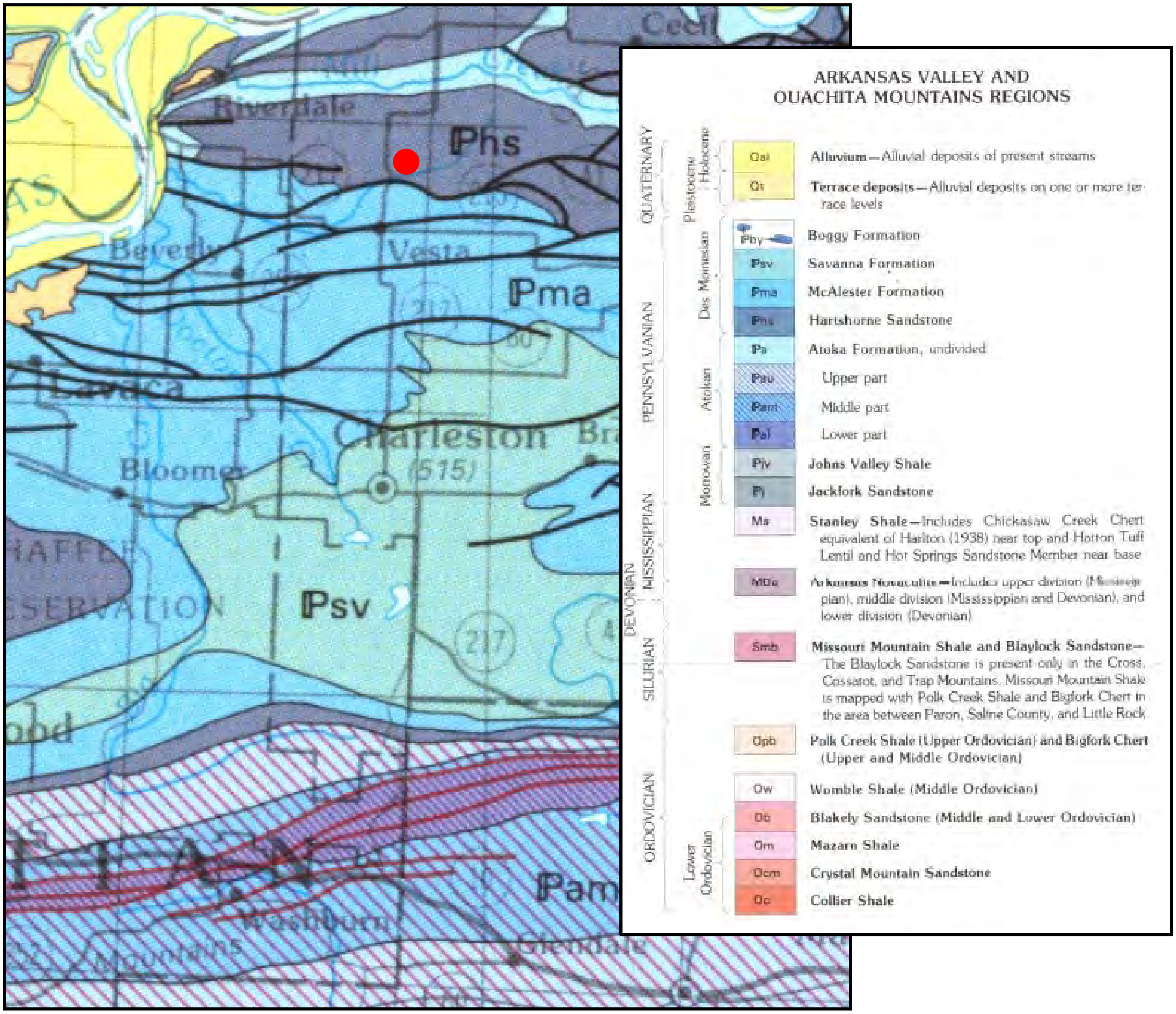

Figure 2: Image from the Geologic Map of Arkansas
The red dot represents the approximate location of the project site.

8.0 Seismic Site Classification & Liquefaction Considerations

The project site is recommended to be assigned either as a Risk Category III or IV according to Table 1604.5 of the 2021 International Building Code (IBC). Further coordination with the Design Team will be needed in order to determine the appropriate Risk Category based on occupancy and use of the facility. The site seismic classification determination may utilize the following values in Tables 5 and 6 on the following page, with reference to Section 1613 of the 2021 IBC and the American Society of Civil Engineers (ASCE) Standard 7-16, based on a Site Class B for the soil profile within the project area. A seismic site class is a classification system that categorizes soil and rock conditions at a site based on their stiffness and potential for amplification of earthquake ground motions. Meanwhile, a site class categorizes locations based on soil types and their engineering properties, typically used for structural design and safety measures. The "current" ASCE Standard 7-22 is not referenced in the 2021 IBC utilized by the state of Arkansas.

Table 5: ASCE Seismic Design Values – Risk Category III

Seismic Values	
S_S	0.173
S_1	0.094
F_a	0.9
F_v	0.8
S_{MS}	0.156
S_{M1}	0.075
S_{DS}	0.104
S_{D1}	0.05
T_L	12
PGA	0.083
PGA_M	0.075
F_{PGA}	0.9
I_e	1.25
C_V	0.7
Seismic Design Category	A

Table 6: ASCE Seismic Design Values – Risk Category IV

Seismic Values	
S_S	0.173
S_1	0.094
F_a	0.9
F_v	0.8
S_{MS}	0.156
S_{M1}	0.075
S_{DS}	0.104
S_{D1}	0.05
T_L	12
PGA	0.083
PGA_M	0.075
F_{PGA}	0.9
I_e	1.5
C_V	0.7
Seismic Design Category	A

The Federal Emergency Management Agency (FEMA) HAZUS software identifies areas with high risks for natural hazards and estimated the physical, economic, and social impacts of earthquakes, hurricanes, floods, and tsunamis. This software assigns a number ranging from 0 to five (5), which ranges the susceptibility to liquefaction from none to very high, respectively. This project site is assigned a HAZUS number of one (1), indicating a very low site susceptibility to liquefaction.

Liquefaction is the transformation of a granular material from a solid state into a liquefied state as a consequence of increased pore pressures and decreased effective stress. Types of ground failures resulting from liquefaction can include sand boils, lateral spreads, ground settlement, ground cracking, and ground warping.

9.0 On-Site Soil Stratum Summary

This summary is based on a collection of field notes and field-testing values recorded during the on-site investigation, notes recorded during the laboratory analysis, and results from the laboratory testing. The encountered subsurface soil conditions are summarized below and on the following page.

9.1 Stratum I – Surface Materials

The materials that make up Stratum I consist of topsoil, with thicknesses ranging from approximately two (2) to 10 inches. These thicknesses are only valid for the project boring locations and could fluctuate in the unexplored portions of the project site.

9.2 Stratum II – Subgrade Materials

The materials that make up Stratum II consist of Silty Sand (SM), Silty Sand with Gravel (SM), Silt (ML), Sandy Silt (ML), Lean Clay (CL), Lean Clay with Sand (CL), Sandy Lean Clay (CL), Silty Gravel with Sand (GM), and Clayey Sand (SC). These materials were encountered in various colors and contained various amounts of gravel, sand, and fines.

Consistency values for the Stratum II SM materials ranged from medium-dense to very dense, with corresponding N-values ranging from nine (9) to greater than 50. The natural soil moisture content for these materials ranged from 2.6 to 16.5 percent. The Liquid Limit (LL) and Plasticity Index (PI) of these materials were determined to be non-plastic (NP). The fine fraction of these materials exhibited negligible plasticity characteristics. The fine fraction of these materials makes up between 14 and 43 percent of the overall soil mass, as indicated by the results of gradation analysis from the borings.

Consistency values for the Stratum II ML materials ranged from very soft to hard, with corresponding N-values ranging from zero (0) to greater than 50. The natural soil moisture content for these materials ranged from 8.5 to 20.9 percent. The LL and PI of these materials were determined to be NP. The fine fraction of these materials exhibited negligible plasticity characteristics. The fine fraction of these materials makes up between 51 and 66 percent of the overall soil mass, as indicated by the results of gradation analysis from the borings.

Consistency values for the Stratum II CL materials ranged from medium-stiff to hard, with corresponding N-values ranging from four (4) to greater than 50. The natural soil moisture content for these materials ranged from 7.5 to 31.6 percent. The LL was determined to range from 27 to 42, with PI values ranging from eight (8) to 23. The fine fraction of these materials exhibited low plasticity characteristics. The fine fraction of these materials makes up between 57 and 77 percent of the overall soil mass, as indicated by the results of gradation analysis from the borings.

Consistency values for the Stratum II GM materials ranged from medium-dense to very dense, with corresponding N-values ranging from 10 to greater than 50. The natural soil moisture content for these materials ranged from 2.6 to 15.1 percent. The LL and PI of these materials were determined to be NP. The fine fraction of these materials exhibited negligible plasticity characteristics. The fine fraction of these materials makes up between 28 and 48 percent of the overall soil mass, as indicated by the results of gradation analysis from the borings.

Consistency values for the Stratum II SC materials ranged from medium-dense to very dense, with corresponding N-values ranging from 12 to greater than 50. The natural soil moisture content for these materials ranged from 3.4 to 12.5 percent. The LL of this material was determined to be 33, with a PI value of 16. The fine fraction of these materials exhibited low plasticity characteristics. The fine fraction of these materials makes up approximately 43 percent of the overall soil mass, as indicated by the results of gradation analysis from the borings.

Figures 3 through 7 below provide examples of the soils encountered within Stratum II.





Figure 3: (Top Left) Stratum II SM material from B-01; approximately two (2) feet below the existing surface elevation.
Figure 4: (Top Middle) Stratum II ML material from B-23; approximately 0.5 feet below the existing surface elevation.
Figure 5: (Top Right) Stratum II CL material from B-06; approximately 3.5 feet below the existing surface elevation.
Figure 6: (Bottom Left) Stratum II GM material from B-09; approximately two (2) feet below the existing surface elevation.
Figure 7: (Bottom Right) Stratum II SC material from B-17; approximately 0.5 feet below the existing surface elevation.

9.3 Stratum III – Competent Rock Materials

The materials that make up Stratum III consist of competent rock materials, identified during this investigation as sandstone indicative of the underlying Hartshorne Sandstone Formation. The consistency of these materials was determined to range from soft to hard. As previously noted, core samples were collected using a two (2) inch diameter NQ2 sized core barrel in project borings B-01 through B-12 once auger refusal materials were encountered at depths beginning between 2.5 and six (6) feet below the existing surface elevations.

The rock core samples collected were subjected to further testing to determine the percent recovery (REC), percent of rock quality designation (RQD), the unit weight, and ultimate compressive strength. The results of these analyses are provided in Table 7 on the following page.

Table 7: Rock Core Data

Boring ID	Core Run	Depth (feet)	Description of Material	Recovery (%)	RQD (%)	Unit Weight (pcf)	Ultimate Compressive Strength (tsf)
B-01	1	4.4-5.0	Sandstone	100	41	152	335
	2	5.0-10.0	Sandstone	100	93	156	1227
	3	10.0-15.0	Sandstone	100	95	154	594
B-02	1	4.5-7.5	Sandstone	90	50	154	281
	2	7.5-12.5	Sandstone	100	100	156	355
B-03	1	4.5-9.5	Sandstone	95	50	158	554
B-04	1	3.0-6.5	Sandstone	83	46	158	670
	2	6.5-11.5	Sandstone	100	57	157	419
B-05	1	2.5-6.5	Sandstone	95	57	155	260
	2	6.5-11.5	Sandstone	100	52	156	475
B-06	1	6.0-11.0	Sandstone	70	27	-	-
	2	11.0-16.0	Sandstone	80	23	155	359
B-07	1	4.5-9.5	Sandstone	96	64	154	487
	2	9.5-14.5	Sandstone	83	83	156	1,534
B-08	1	3.0-5.0	Sandstone	100	61	155	634
	2	5.0-10.0	Sandstone	100	77	157	605
	3	10.0-15.0	Sandstone	100	100	156	467
B-09	1	4.0-5.0	Sandstone	100	0	-	-
	2	5.0-10.0	Sandstone	94	70	153	458
	3	10.0-15.0	Sandstone	82	71	156	723
B-10	1	3.5-8.5	Sandstone	83	50	153	299
B-11	1	3.5-5.5	Sandstone	94	40	153	399
	2	5.5-10.5	Sandstone	98	65	158	949
	3	10.5-15.5	Sandstone	98	98	157	763
B-12	1	2.5-5.0	Sandstone	100	32	149	568
	2	5.0-10.0	Sandstone	97	80	156	791

10.0 Preliminary Engineer's Analysis and Recommendations

At the time of preparing this report, it is understood that the subject property is planned to be developed as the new Franklin County Correctional Facility for the use of the Arkansas Department of Corrections. The project is in the very early stages and site plans and other project documents were not yet available at the time of preparing this report.

The correctional facility is understood to include an approximately 120 to 250-acre main security area with housing, the control tower, food services, and primary facilities, while a wastewater treatment plant and water tank will also be located on the property for use by the correctional facility. Additionally, a portion of the lot is planned to be utilized for agricultural use. At the time of preparing this report, two (2) areas are understood to be in primary consideration for the development of the primary facility. These areas are noted on the enclosed exploration plan in Appendix A.

This investigation was intended to the provide the Client with preliminary geotechnical recommendations and considerations to the encountered subsurface conditions and their suitability in regards to the planned development. Those recommendations and considerations are presented in the following subsections of this report. These recommendations and considerations should be considered preliminary and are **not** to be considered construction-ready. A Final Geotechnical Investigation and Report will be required to provide final recommendations for any development features at the project site.

10.1 Initial Site Preparation

As noted in *Section 9.1*, the surface materials encountered during this investigation consisted of topsoil materials ranging in thickness from approximately two (2) to 10 inches. These materials were encountered in each of the project borings. These thicknesses are only valid for the project boring locations and could fluctuate in the unexplored portions of the project site.

Additionally, several structures including a single-family residence and approximately six (6) ancillary structures (barns, workshops, riding arena, etc.) were also observed across the site. The depth of the foundations and other elements relating to these structures are not known at the time of preparing this report.

MCE recommends that the Contractor anticipates a minimum of 12 inches of initial stripping to be necessary across the project extents to fully remove the existing surface materials from the site. Any remnants of previous developments (including, but not limited to, foundations, residential utilities, etc.), as well as all organics and otherwise deleterious materials, should be removed full-depth prior to the construction of the correctional facility. Additional stripping of up to two (2) feet may be needed in areas where mature trees and dense vegetation exist.

10.2 Preliminary Site Grading Considerations

It is anticipated that both cuts and fills will be required to achieve finished site elevations. It is expected that most of the "target development areas" will require more fills than cuts. This may require retaining walls to be implemented across the site. MCE recommends that "suitable" Stratum II soils or competent rock (Stratum III) be exposed prior to the placement of structural elements or select fill needed to achieve the planned finished subgrade (FSG) elevation(s).

Additional care should be taken by the Contractor to prevent saturation of the subgrade soils, as these materials are known to lose significant strength following precipitation events or other conditions that may lead to increased moisture within the soil stratum. This can be achieved by providing positive drainage during construction and covering with select fill material soon after excavation, if applicable. The on-site subgrade soils will be especially susceptible to reduced shear strengths if construction occurs during historically wet portions of the calendar year, generally occurring between October and May.

The anticipated depths and elevations to the suitable subgrade materials described herein are based on the conditions encountered at the time of this investigation and may vary based on site conditions at the time of construction.

10.2.1 Site Grading Considerations – Excavated Slopes/Vertical Trenching

Excavations should be performed in accordance with the requirements outlined by the Occupational Safety and Health Administration (OSHA) 1926 – Subpart P – Appendix B. Excavated slopes during construction with depths less than 20 feet should be benched or sloped to provide the minimum horizontal-to-vertical (H:V) ratios as noted in Table 8 below.

Table 8: Temporary Slopes During Construction

On-site Soil Stratum	Material Description	OSHA Soil Type	Maximum Allowable Slopes (H:V)
Stratum II	Subgrade Materials	Type C	$1^1/_2$:1 (34°)
Stratum III	Competent Rock Materials	Stable Rock	Vertical (90°)

Note: OSHA Soil Type assignments should be considered preliminary and should be verified at the time of construction, if applicable, by an OSHA-competent person.

Sloping or benching of excavations greater than 20 feet deep shall be designed by a licensed Professional Engineer (PE) prior to excavation. Construction slopes steeper than recommended may be unstable, particularly when introduced to moisture increases during precipitation events. If excavation efforts require deep vertical trenching (deeper than five (5) feet), and the minimum allowable slope ratio is not achievable, then the Contractor must establish a comprehensive Shoring Plan. That Shoring Plan should be reviewed and stamped by a license PE prior to excavation.

10.3 Subgrade Verification

Following stripping and initial grading within the project dimensions, the subgrade should be initially evaluated by the Geotechnical Engineer or his/her representative through proof-rolling operations. In the event that proof-rolling may not be the most effective means of evaluating the state of stability of the subgrade materials, alternative means of verification may be conducted under the direction of the Geotechnical Engineer.

Any soft and/or yielding subgrade areas encountered should be repaired by undercutting and backfilling with select fill material and then subsequently evaluated by the Geotechnical Engineer or his/her representative for approval. Recommendations for undercut should only occur following the subgrade evaluation process. The frequency of and the total depth of required undercut may increase based on site conditions at the time of earthwork operations, particularly if construction occurs during a wet weather pattern.

10.4 Preliminary General Foundation Recommendations

Any foundations relevant to the planned Correctional Facility development should be sized to meet three (3) conditions. First, the maximum stresses imposed on the foundation strata should not exceed the allowable bearing pressures as determined by the shear strength properties of the bearing strata. Secondly, foundations should be designed to limit the maximum anticipated total and differential settlement to magnitudes that will neither damage nor impair the use of the structures.

Finally, the foundation systems must also be designed to resist the anticipated lateral or overturning forces during the most critical loading conditions, including earthquake loadings. These factors, as well as construction considerations related to the existing soil and ground conditions, were influential in the preparation of the recommendations presented hereinafter.

Table 9 below helps line out the primarily considered foundation types and the general maximum loading parameters set for the bearing layers for the foundations. At this time, these should be considered general guidelines and may vary through additional coordination with the Design Team. This information is discussed further in *Sections 10.5 through 10.6*.

Table 9: Foundations & Bearing Correlations

Foundation Type	Foundation Bearing	Max. Column Loading (kips)	Max. Wall Loading (klf)
Shallow Foundations	In-Situ Soils	100	2.5
	Select Fill	100	2.5
	Rock	200	4.0
Deep Foundations – Drilled Piers	Rock	N/A*	N/A*
Deep Foundation - Micropiles	Rock	250	4.0

***A Drilled Pier Foundation System is Anticipated to Satisfy Any Required Maximum Loading Conditions Applicable to the Correctional Facility Development**. Further Details are Provided in *Section 10.6.1.*

10.5 Preliminary Shallow Foundation Recommendations

As the final scope was not yet available for the project structures, considerations for the use of both shallow and deep foundation elements have been provided for the consideration of the Design Team.

Based on the preliminary information regarding the planned structures, it is recommended that a shallow foundation system composed of individual (spread) and continuous footings be considered.

Current planning and budgeting may operate under the anticipation that the foundation elements may bear directly on suitable in-situ Stratum II or Stratum III materials, generally encountered in the upper two (2) feet below the existing surface elevations within the “target areas”. However, it is recommended that the project carry an initial preliminary budget for the placement of a minimum of one (1) foot of select fill material below all structure foundation elements within the understood “target areas”.

This is due to the presence of unsuitable ML materials and this allowance should be confirmed or altered through by the Final Geotechnical Investigation.

For preliminary planning purposes, it is recommended that the foundation elements bearing directly on Stratum II materials, or on properly placed select fill bearing on suitable Stratum II/III materials may utilize safe allowable bearing pressures of 2,000 pounds per square foot (psf) for continuous footings and 2,200 psf for spread footings. For foundation elements bearing directly on Stratum III sandstone materials, it is preliminarily recommended that Design Team utilize safe allowable bearing capacities of 3,000 psf for continuous footings and 3,500 psf for spread footings in that conditon. Again, these preliminary recommendations should be confirmed through the Final Geotechnical Investigation.

The Design Team should consider the potential for water intrusion and the potential necessity for erosion protection, should the structure bear directly on or within proximity of Stratum III rock materials, given the frequency of water flow across the site.

As an initial procedural operation, the structure footprints should be proof-rolled or otherwise evaluated for stability (or competent rock, if relevant) following initial stripping and grading operations.

The stability of soils beneath the foundation footprint can also be evaluated by alternate means if proof-rolling is not feasible, provided that it is verified by a representative of the Geotechnical Engineer. These recommendations assume that the weather conditions at the time of construction are similar to those experienced at the time of our investigation.

As with other elements of this report, final foundation recommendations would require additional information, as well as a more finalized project scope including structure locations, loading conditions, and preferably, though not necessary, finished floor elevations (FFE). ***A Final Geotechnical Investigation will be required to determine the recommended and necessary foundation system for the planned structures. The information provided in this section is for the preliminary consideration of the project Design Team based on the information available at the time of preparing this report.***

10.6 Preliminary Deep Foundation Recommendations

Due to the encountered subgrade materials and potential need for increased bearing capacities than those provided in the previous report section, it is recommended that the Design Team consider the implementation of deep foundation system to increase the allowable bearing capacity for shallow foundation elements. For the consideration of the Design Team, preliminary recommendations and considerations have been provided for the implementation of both drilled pier and micropile foundation systems. ***It is recommended that lateral and axial loading analyses be conducted at part of the Final Geotechnical Investigation in order to adequately provide design criteria for the selected methods.***

As with the recommended shallow foundation system, these recommendations should be considered preliminary and are to be confirmed or revised by a Final Geotechnical Investigation/Report.

10.6.1 Drilled Piers

Should the final structure loading conditions warrant the use of a deep foundation system, it is recommended that a drilled pier and grade beam foundation system founded into the encountered competent rock materials (Stratum III) will be suitable for the support of the on-site structures. A drilled pier foundation system is anticipated to be largely applicable when maximum column loads exceed 200 kips and maximum wall loads exceed four (4) klf.

The exact lengths of the rock sockets should be determined based on the specific loading conditions; however, for preliminary planning purposes, the Design Team should anticipate a minimum rock socket length of five (5) feet into the competent rock materials, or a minimum of one (1) pier diameter (1L:1D), whichever is greater. Drilled piers should have a minimum length-to-diameter ratio of 2L:1D.

The drilled piers should be designed for end-bearing support. It is recommended that the design may utilize a preliminary safe allowable end bearing capacity of 85 kips per square foot (ksf), based on a minimum factor of safety of 2.0, regarding the encountered materials.

Within the “target areas”, competent rock materials are expected to be first encountered at depths ranging from approximately 2.5 to six (6) feet below the existing surface elevations (elevations ranging from approximately 593 to 681 feet).

Drilling operations should expect hard rock drilling to be required to complete the recommended rock sockets. The use of temporary casing may be required to retain the Stratum II surface materials during drilling; however, permanent casing is not anticipated to be required during drilled pier installation.

The bearing stratum should be verified for competency and consistency by the inspection of probe holes drilled beyond the bottom-of-pier elevations. A determination towards the minimum extent of the probing operations should be recommended during the Final Geotechnical Investigation.

Prior to the placement of concrete within pier shafts, the Contractor should make a reasonable effort to remove all water and other deleterious materials from the drilled pier excavation. In the event that groundwater cannot be adequately removed using conventional pumping methods, alternative means such as the tremie method should be utilized to displace groundwater during concrete placement.

As a general consideration, drilled piers should **not** be positioned within one (1) pier diameter of each other end-to-end. For piers within three (3) pier diameters end-to-end, it is recommended that a minimum of three (3) days of concrete curing occur prior to subsequent rock coring. Alternatively, piers within this condition may be constructed consecutively, provided that measures such as shoring are available if required.

Further, when installing multiple piers within this proximity, it is recommended that these piers have similar bearing depths, regardless of size or loading conditions, further coordination is recommended on a case-by-case basis and may be directed by the results of the performed probing operations.

Preliminarily, total and/or differential foundation settlement under the elements supported by the drilled pier foundation system can be anticipated to range from negligible to one-eighth (1/8) inch, due to the settlement necessary to active the skin friction bearing as well as the structure loading.

Drilled piers in an uplift condition may preliminarily utilize an allowable uplift skin friction value of 600 psf for the overburden soils (Stratum II) and 2,000 psf for the competent sandstone materials (Stratum III). Skin friction calculations should negate the top two (2) feet of the drilled pier length, regardless of the subgrade material at this depth.

10.6.2 Micropiles

As an alternative deep foundation system, it is recommended that a hollow-stem micropile foundation system should be preliminarily considered for the support of the development structures. Micropiles may generally be considered for use in areas with maximum column loads up to 350 kips; this load may be increased through the design process through coordination with the installation Contractor. The preliminary design of the micropile foundation system should utilize end-bearing on competent rock materials. It is imperative that the micropiles bear on consistent rock material as opposed to rock shelves. The design and performance of these foundation systems are typically provided by the installation Contractor. Micropiles produce lateral stability by drilling into the rock formation, creating an advantage over alternative pile systems such as helical piles.

The micropiles may be preliminarily designed for an ultimate grout-to-ground bond strength of 175 pounds per square inch (psi), with an allowable strength of 100 psi.

For micropiles in an uplift condition, an allowable skin friction value of 600 psf may be preliminarily utilized for the overburden (Stratum II) materials, and a value of 2,000 psf may be preliminarily utilized for the competent sandstone materials (Stratum III). Skin friction calculations should neglect the upper two (2) feet of the micropile length, regardless of the subgrade material at this depth.

The battering of micropiles may be required to resist project lateral loading, particularly for use beneath project retaining walls. The design of the system should neglect lateral forces in the upper two (2) feet of the micropiles.

10.7 Preliminary Slab-on-Grade Recommendations

Slab-on-grade (SOG) construction may be utilized for the planned structures provided a minimum four (4) inch cushion of sand, crushed stone, or gravel is placed beneath the slab areas with a vapor barrier directly below the concrete. It is preliminary recommended that stable subgrade material be exposed beneath the SOGs and a minimum of two (2) feet of select fill material is properly placed beneath the slab to provide adequate subgrade support and stable under slab conditions.

It is recommended that all below-grade and partially below-grade levels should incorporate an underdrain system due to the possibility for groundwater to accumulate above the Stratum III materials. A French-drain system should be constructed utilizing clean crushed stone or gravel that can exhibit positive drainage.

If positive drainage from the underdrain system cannot be accomplished, it is recommended that a sump pump be utilized in order to ensure that the water drains out and away from the structure. The project team should consider an enveloped waterproofing system for each of the structures anticipated onsite.

10.8 Preliminary Lateral Earth Pressures

Any earth-retaining structures implemented as part of the Correctional Facility should be preliminarily designed to resist the minimum equivalent fluid pressures provided in Table 10 below. The recommended minimum factor of safety against sliding and overturning is 1.5 and 2.0, respectively. The provided lateral earth pressures assume a drained condition for the backfill material.

To achieve a drained condition, the retaining structures should be backfilled using a free-draining granular material and provided thru-wall drains or a gravity trench drain system graded to daylight for the release of any hydrostatic pressure that may develop. Alternative means of drainage may be required if daylighting is not an option, those alternate means would need to be discussed and approved by the Design Team.

The values provided in Table 10 for No. 57 or No. 67 crushed stone gravel assume a 1H:1V maximum backfill slope from the heel of the retaining wall foundation. If a vertical “chimney drained” is provided by the No. 57 or No. 67 stone, then the values for on-site soils should be used based on proximity and relevancy to the material behind the gravel.

Table 10: Estimated Lateral Earth Pressures – Drained Condition

Soil/Backfill Type	Moist Unit Weight (lbs/ft^3)	Friction Angle φ (^{0})	Equivalent Fluid Pressure (lbs/ft^3)		
			Active	Passive	At-Rest
On-site Soils Stratum II	95	29	33	274	49
Select Fill Material (GC, GM, or SC)	125	25	51	308	72
No. 57 or No. 67 Stone	100	35	27	369	43

A coefficient of friction of 0.40 may be used provided the retaining structure is supported on a minimum of four (4) inches of placed and compacted Class 7 Base Course material. A friction value of 0.35 may be used provided the retaining structures are supported directly on select fill material or on-site soils.

10.9 Preliminary Project Pavement Recommendations

Site grading for the planned pavement improvement areas should initially consist of removing all Stratum I materials, followed by proof-rolling as previously described. Where finished subgrade elevations are planned to be below the existing surface elevations, the subgrade should be excavated to the planned finished subgrade elevation prior to proof-rolling operations. Subgrade preparation and proof-rolling should follow the same procedure as described in the *Subgrade Verification* section (*Section 10.3*) of this report. Based on the data obtained from this preliminary investigation, stable subgrade materials for project pavements are preliminary anticipated to be encountered within the upper two (2) feet below the existing surface elevations across the investigated area.

For preliminary planning purposes, it is recommended that the Design Team budget for the placement of two (2) feet of imported select fill materials to be placed beneath all project pavements.

Thickened lifts or "bridging" lifts may be utilized to reduce the amount of undercut necessary in areas of low-consistency materials. These operations should only be implemented under the direction of the Geotechnical Engineer. The top eight (8) inches of any thickened lift should be compacted and tested per project specifications. A minimum of one (1) standard lift should be placed above any thickened lift.

As with the planned project structure(s), these recommendations should be verified or altered through a Final Geotechnical Investigation.

10.10 Preliminary Minimum Pavement Section Recommendations

The following pavement recommendations provided in this section are based on stable subgrade material and/or select fill materials existing beneath the recommended pavement sections. This requirement would be provided by proper placement of approved select fill material and/or stable on-site material being verified by proof-rolling within the pavement dimensions. Minimum pavement sections are recommended to be shown in Tables 11 and 12 below.

For the recommendations provided in Tables 11 and 12, light-duty pavements are considered to be those pavements with low-volume traffic areas such as pedestrian walkways, parking, and staging areas. The standard duty pavements are recommended as performing similarly to a typical city street pavement section with a residential classification. Heavy duty pavement recommendations are intended to apply to areas subjected to frequently heavy truck traffic, such as dumpster pads and loading docks.

Table 11: Minimum Project Pavement Sections - Asphalt

Pavement Type	Pavement Materials	Light Duty	Standard Duty	Heavy Duty
Asphalt Pavement	ACHM Surface Course ($^1/_2$")	3"	3"	2"
	ACHM Binder Course (1")	N/A	N/A	3"
	Class 7 Base Course (95% MPD)	6"	8"	12"

Table 12: Minimum Project Pavement Sections - Concrete

Pavement Type	Pavement Materials	Light Duty	Standard Duty	Heavy Duty
Concrete Pavement	Portland Cement Concrete	5"	5"	6"
	Class 7 Base Course (95% MPD)	4"	6"	8"

The project pavement sections provided in Tables 11 and 12 should be utilized minimum preliminary recommendations and may be increased at the discretion of the project Design Team. MCE can assist in providing a more formalized pavement design should it be requested.

10.11 Select Fill Material

Any select fill material planned or required for the project is recommended to be an off-site borrow material of locally available silty or clayey gravel or clayey sand meeting Unified Soils Classifications System (USCS) as a GC, GM, SM or SC material and having a Plasticity Index of 35 or less, a Liquid Limit of 45 or less, and a maximum of 40% passing the No. 200 sieve. Variations to this may be considered and representative select fill material samples should be submitted to the Geotechnical Engineer for approval prior to use on the project.

Based on the materials encountered during the investigation, existing on-site subgrade materials are anticipated to be suitable for use as select fill if they meet the parameters stated previously. Any material to be used as select fill on the project should be reviewed and approved by the Geotechnical Engineer. Reuse of the on-site rock material for either base course or "select fill" may be utilized through rock processing techniques. Should this be considered by the Design Team, further coordination with MCE should happen.

Some of the encountered on-site materials, particularly those with higher sand and gravel contents (SC, SM, and GM materials), will be more resilient and applicable as "select fill" below project structure and pavement features "Stockpiling" and mixing of these materials will assist with consistency across the structural fill areas. The on-site CL and ML materials should not be utilized beneath any structures or pavement dimensions but are acceptable for general site fill for use in areas such as greenspace.

Additionally, locally available shale materials may be utilized as select fill on the project provided that the shale satisfies the stipulations listed previously. Any shale material utilized as select fill should be compacted to 98% of the maximum dry density, as determined by ASTM D1557, at a moisture content within two (2) percent of optimum. Shale fill should not be used as an alternative to Class 7 base.

When placing fill next to existing slopes, the slope face should be stripped of all vegetation and the face "benched" to allow the placement of horizontal lifts and bonding to the slope face. Table 13 below provides the recommended compaction parameters for select fill and Class 7 base course to be used on the project.

Table 13: Compaction Requirements

Material Type	Test Standard	Minimum Dry Density (%)	Optimum Moisture Range (%)
Select Fill (Non-Shale - Pavements)	ASTM D698 / AASHTO T99	95	-3% to +3%
Select Fill (Non-Shale - Structures)	ASTM D698 / AASHTO T99	98	-3% to +3%
Select Fill (Shale)	ASTM D1557 / AASHTO T180	98	-2% to +2%
Class 7 Base Course	ASTM D1557 / AASHTO T180	95	Near Optimum

11.0 Construction Materials Testing and Special Inspections

Construction materials testing and special inspection services should be provided by MCE to provide consistency with the recommendations in this report and the documentation of those recommendations being implemented during construction.

Testing of the earthwork, concrete, structure, and other phases should be conducted and documented during construction to assure the Owner and Engineer that the construction complies with the specifications. Field verification of earthwork operations will be required to confirm the recommendations contained herein. Additionally, all trenching and excavations should be conducted following the current Arkansas State Law and Occupational Safety and Health Administration (OSHA) guidelines and requirements.

12.0 Limitations and Reserved Rights

The preliminary recommendations and conclusions made in this report are based on the assumption that the subsoil conditions do not deviate appreciable from those disclosed in the subsurface exploration and that a Final Geotechnical Investigation will be conducted at a later date. Following the Final Geotechnical Investigation, the recommendations provided by this report should be considered insufficient, owing to the new subsurface data. A review of the final construction plans and specification by this office is encouraged to ensure compliance with the intent of these recommendations.