Appendix K

Geotechnical Report & Seismic Study

Mountain Peak Energy Storage
Conditional Use Permit Application
September 2025

GEOTECHNICAL REPORT

Mountain Peak BESS Project
Mentor, Kansas

Dec. 23, 2024

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Kansas PE License No. PE30461

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1 Executive Summary

ANS Geo, Inc. is pleased to provide Geotechnical Report (Report) to Plus Power to summarize the results of our geotechnical investigation program in support of the proposed Mountain Peak Battery Energy Storage System (BESS) project located in Mentor, Kansas. ANS Geo has summarized, at a very high level, some of the critical geotechnical items and observations which may impact project design and construction within this Section from our observations during the completion of our geotechnical investigation at the project site.

1. ANS Geo advanced a total of eight (8) soil borings within the proposed BESS footprint.
2. The encountered soils observed were predominantly stiff to hard, low to high plasticity clays. Encountered subsurface conditions and laboratory test results are summarized in **Section 4**, and **Section 5**, respectively.
3. Water was observed within one (1) boring (B-07) at approximately 30 feet below grade at the time of our investigation.
4. Based on corrosivity lab testing and field soil resistivity measurements, the in-situ soil conditions generally indicate that soils are "corrosive". It is anticipated that appropriate protective measures for buried steel, such as increased galvanic coating or similar, will be employed.
5. Results of in-situ electrical resistivity testing yielded results as low as 927 ohm-cm and as high as 4,884 ohm-cm. The electrical designer should review the detailed data for the purpose of their design.
6. It is recommended that foundations be over-excavated by at least twelve (12) inches and backfilled with compacted structural fill over geotextile separation fabric to improve the bearing capacity within/under areas of proposed foundations (ie. concrete slabs, inverter pads, substation foundations), and to provide free-draining material beneath foundations to avoid softening during times of precipitation.
7. The frost depth within the project area is mapped as 36 inches below grade, which will govern embedment depths for shallow foundations.
8. Soil conditions observed within our investigation locations indicate that there is a low to moderate risk of expansive soils within the upper five (5) feet.
9. ANS Geo has provided Deep Foundation Recommendations in **Table A** directly preceding our provided Attachments.

2 Project Description

ANS Geo, Inc. is pleased to provide this Geotechnical Report to Plus Power to summarize the results of our geotechnical investigation program in support of the proposed Mountain Peak Battery Energy Storage System (BESS) project located in Mentor, Kansas. ANS Geo, in agreement with Plus Power, developed a geotechnical investigation program, implemented by ANS Geo intended to provide information to support the design and construction of the proposed BESS facility.

ANS Geo's geotechnical investigation program included a desktop study of local geologic conditions, soil borings, test pit excavations, in-situ electrical resistivity testing, laboratory thermal resistivity and corrosion testing, laboratory California Bearing Ratio (CBR) testing, and laboratory soil index testing. Our targeted test locations were focused within the proposed BESS footprint. A General Location Plan as well as as-completed Investigation Location Plan is provided as **Attachment A**.

2.1 Reference Files

Prior to commencing our geotechnical investigations, ANS Geo was provided multiple project files from Plus Power detailing the planned development. The referenced files are listed below:

- Mountain Peak ES.kmz – no date

In addition to these project files, site coordination, land access, and project details were shared and communicated between ANS Geo and Plus Power via e-mail and phone conversations.

2.2 Project Assumptions

ANS Geo understands that the proposed BESS facility development will include a BESS system, a project substation, interconnection facilities, and supporting structures and equipment. We understand the project footprint spans across multiple/contiguous parcels separated by existing county and state roads, and will include the development of unpaved, aggregate access roadways within the project parcels. ANS Geo assumes the project design life to be 35 years.

ANS Geo's scope of work was completed based on our approved proposal dated October 10, 2024, and our recommendations herein are specific and based on our understanding of the project and communications with Plus Power. Should the configuration of the system differ from our stated understanding, it is imperative that ANS Geo is contacted to review, confirm, and/or update our recommendations to reflect the planned development. For example, recommendations such as pile design parameters will change if alternate pile installation techniques are considered, such as pre-drilling, screw piles, helical piles, compaction-and-backfilling, or other method, and these changes may cause a material change in the design of foundations. Similarly, should the location of access roadways and expected traffic volumes and loading, assumed facility design life, or other site condition change, our recommendations will need to be updated.

3 Methodology

3.1 Soil Boring Explorations

ANS Geo advanced eight (8) soil borings completed at select locations across the project area. A total of 8 borings (B-01 through B-08) were advanced within the proposed BESS footprint, The soil boring locations are depicted in the Investigation Location Plan, provided as **Attachment A**.

The eight (8) BESS footprint soil borings were advanced to approximately 40 feet below ground surface (BGS). An Acker Rebel XL track-mounted drill rig was used to collect soil samples using the Standard Penetration Test (SPT) Method through hollow-stem augers in accordance with ASTM Standard D1586. Soil samples were collected continuously within the upper 10 feet in each boring, then in five-foot intervals thereafter to the termination depth. Soil boring locations, proposed by ANS Geo and confirmed by Plus Power review, were located at relatively evenly spread locations throughout the project's BESS footprint. All soil borings were overseen and logged by an ANS Geo representative under the direction of a Professional Engineer licensed in the State of Kansas. Typed soil boring logs are presented as **Attachment C**.

3.2 Test Pit Excavations

ANS Geo advanced a total of eight (8) test pit excavations across the project area to evaluate the subsurface conditions. Test pit locations were located at relatively evenly spread locations throughout the project's array area(s). All test pits were overseen and documented by an ANS Geo geotechnical representative under the direction of a Professional Engineer licensed in the State of Kansas. Soil strata changes, soil classification, and excavation depths were documented during each test pit excavation and are presented within the test pit logs provided as **Attachment D**. Upon completion, each test pit excavation was backfilled with native soils, bucket-tamped, and driven over several times with the excavator to minimize any post-excavation settlement.

At select test pit locations, soil samples were collected between one (1) and five (5) feet below grade with the purpose of obtaining bulk soil samples for laboratory thermal resistivity testing (TRT), California Bearing Ratio (CBR) testing, and corrosivity testing. Upon completion, each test pit was backfilled to its existing grade with soil cuttings.

3.3 Electrical Resistivity Testing

As part of our field investigation program, ANS Geo performed field Electrical Resistivity Testing (ERT) at three (3) locations across the project site. In-situ soil resistivity measurements were obtained by utilizing the Wenner 4-Pin Method in accordance with ASTM G57 and IEEE Standard 81. Two (2) mutually perpendicular traverses were collected at each array area location utilizing electrode "a"-spacings of 1, 2, 3, 5, 10, 20, 50, 100, 200 and 300 feet.

4 Geology, Surface, and Subsurface Conditions

Prior to site mobilization, ANS Geo conducted a desktop review of publicly available geological maps and reports made available by the United States Geological Survey (USGS), the Kansas Geological Survey (KSGS) online mapping, and other available mapping. Our desktop review is summarized herein, along with our observed, site-specific subsurface conditions as identified through our field investigation.

4.1 Observed Site Conditions

In general, the project site and surroundings appeared to be consistent with considerably minor slopes (generally not exceeding 5%). Based on USGS topographic mapping, the average elevation across the site ranged between approximately 1310 feet and 1360 feet above mean sea level (AMSL).

4.2 Historic & Topographic Setting

ANS Geo reviewed historical satellite imagery made available via Google Earth. High-quality satellite imagery was available as far back as 1954, and our review indicated that the project area overall has not changed between 1954 and the time of the desktop study. The only major construction or alteration of the land that was visible was the construction of the nearby summit station approximately 400 feet northeast of the project site. The majority of land, however, has historically been used as farmland. In general, the project site and surroundings appeared to be consistent with considerably minor slopes (generally not exceeding 5%). Based on USGS topographic mapping, the average elevation across the site ranged between approximately 1310 feet and 1360 feet above mean sea level (AMSL).

Based on USGS topographic mapping, the site has an average elevation of approximately 1310 to 1360 feet above mean sea level (ASML) and is generally flat, gently dipping down in the westward direction of the site. The project is situated within the Smokey Hills region of the Kansas Central Plains.

4.3 Surficial Geology

ANS Geo reviewed geological mapping made available by the Kansas Geological Survey (KGS) and the United Stage Geological Survey (USGS) which indicated the project area is mapped within Holocene- to Pleistocene-aged alluvium, terrace valley fill, and colluvium located in the Smoky Hills region of north-central Kansas. This region is characterized as a range of Cretaceous-aged hills that have been carved out by stream channels over the course of millions of years. As a result, unconsolidated sediments were carried in and deposited by streams in the river valleys, which are much younger than the rock which makes up the surrounding hills. These sediments include a range of sand, silt, and clay, with minor amounts of gravel that was deposited below floodplains, along major rivers, and in stream terraces.

The sediments that make up the geology of the project area were deposited in alluvial and colluvial environments on top of bedrock, resulting in materials that are highly variable in composition and thickness. This material is recorded to be approximately 60 feet thick on average but commonly extends to thicknesses of approximately 100 feet thick, especially within infilled bedrock valleys. This sediment is generally composed of poorly sorted sandy to clayey loam, containing clasts of residual bedrock-derived sandstone and shale that has been weathered out over time, especially along steep slopes and areas along small streams where upland material is being drained and transported downslope. Additionally, review of publicly available boring logs within the area indicated sand, silt, and clay underlain by shale observed within the upper 100 feet. A geologic unit map of the site vicinity is provided in **Attachment B**.

ANS Geo additionally reviewed surficial soil mapping available from the Natural Resource Conservation Service (NRCS) Web Soil Survey application. The NRCS survey was initially created for agricultural purposes and is generally limited to the upper five (5) to six (6) feet BGS; however, the resource provides generalized information pertaining to the soil chemistry and properties. The NRCS mapping identifies the project area to be primarily comprised of the Crete series, Irwin series, and Edalgo series, which are described as silt loams and clay loams that were deposited on hillslopes and fluvial valleys.

4.4 Bedrock Geology

ANS Geo reviewed geological mapping made available by the Kansas Geological Survey (KGS) and the United Stage Geological Survey (USGS) which indicated the project area is mapped within the early Cretaceous-aged Kiowa Shale and Cheyenne Sandstone Formations, which are located in the Smoky Hills region of north-central Kansas. This region is characterized as a range of Cretaceous-aged hills that are primarily composed of light gray to black fissile shale beds with basal fossiliferous limestone shell beds (coquinoidal), along with laterally discontinuous light gray to yellowish brown, cross bedded, medium- to fine-grained sandstones. Lenses of sandy shale, sandstone, and conglomerate are commonly found throughout this formation. Localized instances of calcite cemented sandstones, gypsum, iron carbonates, and pyrite are also recorded. These rocks formed from sediments that were deposited as the early Cretaceous Sea spread northeastward into modern day Kansas, covering this area in shallow seas, marine shales, and marginal marine sandstones. These rocks are a part of the broader Great Plains sedimentary rock system and reflect the dynamic processes of shallow marine activity over time. The rocks of this formation are often fossiliferous, containing primarily shallow marine fossils.

This formation has a variable thickness, typically ranging from 60 to 150 feet in Kansas, although it reaches a maximum subsurface thickness of 300 feet below grade in some regions. Publicly available depth to bedrock indicates that bedrock (primarily composed of interbedded shale and limestone) generally underlies approximately 30 feet of sediment on average. Additionally, shallow bedrock was not encountered across the site at the time of our investigation. A geologic unit map of the site vicinity is provided in **Attachment B**.

4.5 Observed Subsurface Conditions

ANS Geo has provided the generalized subsurface conditions within **Table 1** based observations recorded within our geotechnical investigation program. Soil boring logs and test pit photo logs have been provided as **Attachment C** and **Attachment D**, respectively, and should be reviewed for specific soil condition observations.

Table 1: Generalized Subsurface Profile

Stratum	Avg. Depth (ft)	Material (USGS Classification)	Avg. Consistency/ Density	Description
	Varies	Topsoil	-	A layer of topsoil was generally observed at grade, ranging from approximately 2 to 3 inches.
I	0 ~ 2	Clay (CL)	Stiff	Medium stiff to very stiff low plasticity clays were generally encountered near grade. Little amounts of silt were observed within this layer. Pocket penetrometer values ranged between 2.5 and exceeding 4.5 tons per square feet (tsf) with an average of 3.8 tsf in this layer. N-values generally ranged between 3 and 19 blows per foot (bpf) with an average N-value of 11 bpf in this layer.
II	2 ~ 20	Clay (CL)	Very Stiff	Stiff to hard low plasticity clays were predominantly encountered underlying the upper stiff clay layer and generally extended to approximately 20 feet in most borings. Varying amounts of silt and sand, along with trace amounts of gravel were also observed within this layer. Pocket penetrometer values exceeded 4.5 tsf in this layer. N-values ranged between 12 and exceeding 50 bpf with an

Stratum	Avg. Depth (ft)	Material (USGS Classification)	Avg. Consistency/ Density	Description
				average N-value of 42 bpf in this layer. Mottling was also commonly observed within this layer.
III	20 ~ 30	Clay (CL/CH)	Hard	Very stiff to hard low to high plasticity clays were generally encountered underlying the very stiff clay layer. Varying amounts of silt, sand, and gravel were also encountered within this layer. Pocket penetrometer values exceeded 4.5 tsf in this layer. N-values ranged between 15 and exceeding 50 bpf with an average N-value of 38 in this layer. Split spoon refusals were also commonly encountered within this layer.
IV	30 ~ 40+	Clay (CL)	Hard	Very stiff to hard low plasticity clays were generally encountered underlying the low to high plasticity clay layer. Varying amounts of silt, sand, and gravel were also encountered within this layer. Pocket penetrometer values exceeded 4.5 tsf in this layer. N-values ranged between 18 and exceeding 50 bpf with an average N-value of 46 in this layer. Split spoon refusals were also commonly encountered within this layer.

The mapped soil formations identified within our desktop study are generally consistent with the findings of our field investigations.

4.6 Groundwater

At the time of our investigation program, water was observed in 1 of the 16 investigated locations, at the depths indicated in **Table 2**. ANS Geo notes that it is possible the indicated water levels may represent perched conditions rather than static groundwater levels.

Table 2: Observed In-Borehole Water Levels

Location ID	Depth to Groundwater (ft)
B-07	30

4.7 Summary of Geohazards

ANS Geo assessed publicly available information, results of the geotechnical investigation and the site conditions during the investigation to evaluate any potential geotechnical or geological hazards.

The project site generally consists of flat farmlands; therefore, the slope stability risk is negligible.

According to FEMA flood maps, the project site is at minimal risk for floods, however it is mapped approximately 0.5 miles from a 100-year flood zone due to its proximity to the Smoky Hill River tributary. According to FEMA National Risk Index, there is a relatively high risk for tornadoes.

Based on karst hazard maps, the project site is partially located where bedrock susceptible to karstic features may be present. However, during the site investigation, bedrock was not encountered in the upper 50 feet below ground surface. Also, no geological features that might be associated with karstic formations were identified. Therefore, the risk of potential karst to the project is found to be low.

Since the project site is located in a low seismicity area and the encountered soils are generally fine-grained soils, the liquefaction risk is negligible.

The frost depth for the project site is about 30 inches.

The soils at the project site are generally low to high plasticity cohesive soils with low to moderate swell potential. A detailed discussion about the swelling soils and their swell potential is provided in **Section 7.3**.

5 Laboratory Results

Representative soil samples were collected during our investigation and submitted to ANS's accredited materials testing laboratory. Soil samples will be retained for a period of three (3) months following the initial submission of this Report.

5.1 Soil Index Testing

A summary of the index laboratory test results has been provided within **Table 3** and **Table 4**. As-received laboratory test results are included within **Attachment E**.

Table 3: Soil Index Testing Summary (Sieve Analysis, ASTM D6913)

Location ID	Sample ID	Depth (ft)	% Gravel	% Sand	% Fines	% Moisture
B-06	S-5	8 – 10	0.0	2.5	97.5	14.4
B-07	S-3	4 – 6	0.0	8.1	91.9	7.1

Table 4: Soil Index Testing Summary (Atterberg Limits, ASTM D4318)

Location ID	Sample ID	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	% Moisture	USCS
B-01	S-1	0 – 2	49	22	27	27.4	CL
B-01	S-8	23 – 25	75	27	48	28.1	CH
B-02	S-2	2 – 4	49	22	27	14.9	CL
B-02	S-9	28 – 30	55	23	32	27.6	CH
B-03	S-3	4 – 6	57	24	33	20.3	CH
B-03	S-8	23 – 25	61	24	37	21.4	CH
B-04	S-4	6 – 8	47	21	26	15.9	CL
B-04	S-8	23 – 25	73	27	46	29.5	CH
B-05	S-3	4 – 6	46	21	25	14.3	CL
B-05	S-9	28 – 30	53	22	31	17.3	CH
B-06	S-8	23 – 25	58	24	34	19.9	CH
B-07	S-9	28 – 30	54	22	32	18.3	CH
B-08	S-5	8 – 10	38	20	18	13.9	CL
B-08	S-9	28 – 30	65	25	40	23.3	CH

5.2 Thermal Resistivity Testing

ANS Geo collected bulk samples from three (3) locations within the project area from one (1) to four (4) feet below grade for laboratory testing of Thermal Resistivity. Soils were collected in a five-gallon bucket and delivered to ANS's accredited laboratory for testing. The soil was compacted to 85 and 95 percent of its Standard Proctor Density in accordance with ASTM D698, and Thermal Resistivity Testing was conducted in accordance with IEEE Standard 442-2017 and ASTM D5334. Results of the thermal testing summarized within **Table 5.** Complete, as-received results are provided within **Attachment E**.

Table 5: Thermal Resistivity Testing Summary (ASTM D5334)

Location ID	Material Type	Standard Proctor Density (%)	Thermal Resistivity Values at Various Moisture Contents					Received Moisture Content (%)	Re-Molded Dry Density (lb/ft^3)
			% water	% water	% water	% water	% water		
			(°C-cm/W)	(°C-cm/W)	(°C-cm/W)	(°C-cm/W)	(°C-cm/W)		
TP-02	Clay	85	0.0	5.6	11.2	16.7	22.3	18.9	81.2
			290.4	**187.2**	**83.3**	**69.4**	**64.1**		
		95	0.0	5.6	11.2	16.7	22.3		90.7
			242.7	**147.4**	**73.3**	**63.5**	**58.4**		
TP-05	Clay	85	0.0	6.6	13.3	19.9	26.5	10.8	80.2
			283.8	**164.4**	**85.5**	**74.5**	**68.3**		
		95	0.0	6.6	13.3	19.9	26.5		89.6
			233.9	**131.9**	**79.6**	**69.9**	**63.3**		
TP-07	Clay	85	0.0	7.6	15.1	22.7	30.2	21.8	81.6
			290.4	**154.2**	**94.9**	**82.6**	**74.1**		
		95	0.0	7.6	15.1	22.7	30.2		91.2
			238.2	**130.4**	**90.7**	**78.0**	**68.8**		

5.3 Corrosivity Testing

ANS Geo collected bulk samples from zero (0) to five (5) feet grade at three (3) locations for corrosivity testing. The results of the testing, completed by ANS, are summarized within **Table 6** and detailed within **Attachment E**.

Table 6: Corrosivity Testing Summary

Location ID	pH	Sulfate (mg/kg)	Chloride (mg/kg)	Redox Potential (average) (mV)	Soil Box (Calculated Resistivity) (Ω-cm)
TP-03	7.9	<15	51	302	2,780
TP-06	7.6	480	54	305	33,300
TP-08	7.2	30	66	312	45,400

5.4 California Bearing Ratio

ANS Geo collected bulk samples from one (1) to three (3) feet below grade at two (2) locations for testing of California Bearing Ratio (CBR) in accordance with ASTM D1883 at approximately 95 percent of its Standard Proctor Density (ASTM D698). The results of the testing have been summarized within **Table 7** and are detailed within **Attachment E**.

Table 7: California Bearing Ratio Summary

Boring ID	CBR Ratio (%)
TP-01	1.6
TP-04	1.8

6 Seismic Considerations

6.1 Site Classification

Based on the observations recorded within our subsurface investigation program and our familiarity with the project area, Site Class D is assumed as the average condition across the project site for Risk Category II.

The seismic ground motion values for this this were obtained from the USGS Seismic Hazard Maps, referenced in ASCE 7-16 Standard and provided as **Attachment G,** and are as follows:

- 0.2 second spectral response acceleration, S_S= 0.077 g
- 1 second spectral response acceleration, S_1= 0.049 g
- Maximum spectral acceleration for short periods, S_{MS}= 0.123 g
- Maximum spectral acceleration for a 1-second period, S_{M1}= 0.118 g
- 5% damped design spectral acceleration at short periods, S_{DS}= 0.082 g
- 5% damped design spectral acceleration at 1-second period, S_{D1}= 0.079 g

6.2 Historic Seismic Events

According to the United States Geological Survey (USGS) earthquake catalog, several minor and micro earthquakes (categorized on the Richter scale as having a magnitude of less than 4) have occurred within a 50-mile radius of the project site within the past 50 years. Additionally, three (3) light earthquakes (categorized on the Richter scale as having a magnitude of approximately 4) have occurred within a 50-mile radius of the project site within the past 50 years. These earthquakes include a magnitude 4.3 earthquake and a magnitude 4.0 earthquake that occurred on December 8th and December 15th, 2021, respectively, near the town of Gypsum, which is approximately 7 miles southeast of the site. Another magnitude 4.0 earthquake occurred on November 11th, 2023 near the town of Bennington, which is approximately 23 miles northwest of the site.

6.3 Preliminary Seismic Evaluation

The designated seismic site class is anticipated based on results from our investigation program and using select areas of the site which have been investigated by ANS Geo. Based on our observation of subsurface conditions, estimated Site Class ratings, and review of USGS's 2023 National Seismic Hazard Map and publicly available information, ANS Geo concludes that the area is generally considered to be a low seismic hazard zone.

7 Foundation Design Considerations

ANS Geo anticipates that, as typical with BESS facility construction, embedded posts, such as W6x9 H-piles, will be used to support the proposed BESS facility. Conventional shallow foundations such as sonotubes, spread footings, or systems may also be utilized for equipment pads and associated support structures. Deep foundations such as helical piles, driven piles, or drilled cast-in-place concrete piers may be considered to where increased lateral or bearing capacities are required, or where structures are particularly sensitive to vertical soil movements.

7.1 Corrosion Considerations

7.1.1 Buried Steel

Given limited testing results measuring the soil pH level, sulfate and chloride concentrations, resistivity, and redox potential summarized in **Section 5.3** (Table 6), in consideration with the soil and moisture conditions observed, the in-situ soil conditions generally indicate soils that are "corrosive". It is anticipated that hot dipped galvanized steel with a minimum zinc coating thickness in accordance with ASTM A123 may require a significant increase in sacrificial steel thickness to be able to accommodate the expected corrosion loss over the project design life. Therefore, we strongly recommend evaluating the need for an increased zinc coating thickness of 5-mil to provide some protection prior to the increased rate of expected bare-steel corrosion loss over the project design life. For structural steel shapes, a minimum zinc coating thickness typically ranges from 3-mil to 4-mil depending on the steel section size as specified by ASTM A123. For example, a W6x9 shall contain a minimum zinc coating grade of 75 micrometers, or a 3-mil thick coating.

Steel section loss in piles decreases the structural load carrying capacity of the member as well as increases the member deflections. Therefore, it is recommended that the final structural design considers the useful life of galvanized (zinc) coating, followed by the anticipated loss of steel due to corrosion to ensure the structural integrity is maintained throughout the service life. Thicker pile sections, increased zinc coating thickness, or other corrosion protection measures may be necessary to accommodate any reduction in structural capacity. For example, it is possible that a W6x12 pile with a standard zinc coating thickness could corrode to W6x9-equivalent section throughout the service life depending on the corrosion-related soil properties.

Based on the limited corrosivity and resistivity testing results, it is our professional opinion that a 75-micrometer (3-mil) zinc coating would maintain an approximate lifespan of approximately 6.26 years prior to full depletion. Upon depletion of the zinc coating, bare steel loss would occur at an estimated rate of 1.55 mils (0.00155 inches) per year. For context, under these assumed conditions, a 3-mil coated W6x9 steel pile would experience approximately 44.68 mils (0.04468 inches) of steel loss (per side) within a 35-year lifespan. Considering that steel with this minimum zinc coating thickness may require a significant increase in sacrificial steel thickness to be able to accommodate the expected corrosion loss over the project design life, ANS Geo recommends considering an increased zinc coating thickness of 5-mil to provide additional corrosion protection for the intended design life and use. It is our professional opinion that a 5-mil zinc coating would maintain an approximate lifespan of approximately 20.27 prior to full depletion. Upon depletion of the zinc coating, bare steel loss would occur at the same estimated rate previously noted. Under these assumed conditions, a 5-mil coated steel pile would experience approximately 22.91 mils (0.02291 inches) of steel loss (per side) within a 35-year lifespan.

It is recommended that a separate, detailed corrosion evaluation report be developed by ANS Geo, or others, to interpret the soil corrosivity test results and estimate the rate of corrosion for zinc and bare steel resulting from exposure to the surrounding environment. This detailed corrosion evaluation may be provided to the Owner and/or a foundation engineer to incorporate the test results into the design and selection of pile foundations, or other buried steel across the site.

7.1.2 Buried Concrete

Corrosive soils can have a significant impact on below-grade concrete foundations by potentially damaging or weakening the concrete. One of the primary forms of concrete deterioration due to exposure to corrosive soils is sulfate attack. Sulfate attack is a common form of concrete deterioration which occurs when concrete comes in contact with water or soil containing sulfates. Sulfates are typically found in some soils, in seawater, and in wastewater treatment plants. The principal factors which affect the rate and severity of sulfate attack are permeability of concrete, concentration of sulfates, tricalcium aluminate (C_3A) content, and calcium hydroxide content. When sulfates react with C_3A, it will form ettringite which will expand and create internal tension within the concrete that eventually leads to cracking. Therefore, a low C_3A content is one of the main considerations when selecting cement for sulfate resistance.

Recommended concrete properties, including cement type, to resist sulfate attack are based on the site-specific sulfate exposure class, as per ACI 318-19, Table 19.3.1.1. The severity of the exposure of concrete to sulfate is divided into four classes (S0 through S3) depending on the water-soluble sulfate in soil (percent by mass) or dissolved sulfates in water (ppm).

Results of ANS Geo's laboratory testing indicate that a sulfate exposure classes of **S0** may be appropriate for concrete bearing in soil similar to those assayed for this investigation. For sulfate exposure class **S0**, external sulfate attack is likely not a concern and there are no recommended restrictions on cement type.

ANS Geo recommends that concrete adheres to the requirements of ACI 318-19, Table 19.3.2.1 for concrete properties including maximum water-cement ratio, minimum compressive strength (psi), and cement type for the site-specific sulfate exposure class. These recommendations do not consider acidic or basic soils, which should additionally be considered during cement design.

7.2 Frost Depth Considerations

According to the US Department of Commerce frost mapping, within Saline County, Kansas, the local frost depth is mapped to exist at approximately 36 inches below grade. ANS Geo recommends that all shallow (non-pile) foundations should be embedded at least to this depth. Shallower foundation depths may also be accommodated, provided they are appropriately frost-protected by way of appropriately designed haunched edges, foam insulation, and/or free-draining structural fill extending to the frost depth.

For shallow foundations which are not load-bearing or sensitive to movement, such foundations may be able to be founded at shallower depths. ANS Geo should be contacted to provide recommendations for minimum embedment depth in this scenario.

7.3 Soil Shrink & Swell Potential

Shrinkage and swelling of soils refer to the volumetric change (decrease and increase) exhibited in primarily fine-grained soils due to a change in moisture conditions. The extent of shrinking and swelling is largely influenced by the type and amount of clay present in the native near-surface soils. Higher-risk soils generally include fine-grained material with a high clay content, greater than 50 percent by weight, and liquid limits of 50 percent or higher (fat clays).

7.3.1 Site-Specific Evaluation

Given the location of the project and soils encountered, the potential for shrinkage and swelling of soils was considered. The results of laboratory plasticity tests in the borings indicate that generally high plasticity clay soils are present within the upper four to six feet below grade with liquid limit up to 57, and an average liquid limit of 48.

Since soil expansion and swell were of concern, ANS Geo performed moisture content testing at natural (in-situ) moisture content of recovered soil samples within the upper 10-feet. The purpose of this testing was to develop a moisture content with depth profile, to establish the variation of moisture content through the soil profile. This data was used to calculate the active zone depth, which is the depth at which potential swelling and shrinkage is expected to act.

7.3.2 Active Zone Depth

Based on ANS Geo's observations on site, and plotting the moisture content with depth, we have characterized the site to have a zone of moisture variation of approximately 4 to 6-feet below grade. Therefore, for design purposes, ANS Geo recommends an active zone depth of 5-feet. This depth has been recommended considering the understanding that moisture content may fluctuate throughout the profile, and that expansion may not occur equally and simultaneously at any given time along the full depth of the soil profile within this zone.

ANS Geo also utilized our regional experience and the *Design Procedure for Drilled Concrete Piers in Expansive Soil* compiled by the Structural Committee of the Foundation Performance Association (Document No. FPA-SC-16-0) to cross-reference our estimation of the active zone depth. In addition, our evaluation is based on our understanding that the array tracker system, through installation and tolerance, can accommodate up to one inch of movement between piles which will allow for some minor amount of heave. It is noted that the active depth assumed herein may not represent the moisture variations that can occur at greater depths due to the presence of large tree root systems that could desiccate the soils, the presence of other heating units, or soil wetting due to pipe leaks, poor drainage, etc.

For expansive and shrinking soils, the soil active zone is defined at the depth at which the moisture content of the natural soils will fluctuate. As moisture and water are absorbed or released by the clay molecules, the expansion and contraction of the clay occurs. The *amount* of expansion and contraction are typically referred to as Potential Vertical Rise, or "PVR".

7.3.3 Potential Vertical Rise (PVR)

The amount of potential soil movement due to shrinking and swelling with soil moisture variations is represented or indicated by Potential Vertical Rise (PVR). Regardless of the project site being within Texas or another state, the industry-standard method for evaluating PVR is by use of the Texas Department of Transportation (TXDOT) TEX-124-E method. Using the TXDOT method, the estimated PVR value is on the order of approximately 1 inch or less.

The structural/civil engineer should take the expected PVR into account during structural design, raising of site fill, designing and construction foundation and roads, and any other site elements which may be intolerant of movement. It should be noted that it is very difficult to predict the moisture variations under the structure during its service life. Therefore, the PVR estimates provided herein should be considered approximate probable estimates based on industry standard practice and experience, and the movements predicted herein should not be construed as absolute values that could occur in the field. Poor drainage and water infiltration into the foundation soils can be detrimental to the ground supported structures. Excessive wetting of soil (due to accumulation of water), or, excessive drying (due to the presence of large trees, removal of vegetation and exposure to atmosphere, change in topography and drainage, etc.) could possibly result in greater PVR values than those estimated herein.

ANS Geo understands that PVR will create movement that could impact the performance of structures and other site improvements. There are a number of methods which can be used to mitigate potential damage caused by the shrinking of swelling of these expansive clays for pile foundations, shallow foundations, and other site grading. These methods include over-excavation, the introduction of chemical admixtures, and the implementation of moisture control barriers. Over-excavation of the expansive clay and/or pre-wetting are some commonly used techniques to diminish the effect of these expansive clays. Recommendations for over-excavation for shallow and mat foundations have been provided in **Section 7.4** and **Section 8.3**.

7.4 Recommended Soil Parameters – Shallow Foundations

ANS Geo anticipates that shallow foundations such as concrete footings, housekeeping pads, inverter pads, or sonotubes will be used to support non-critical and lightly loaded structures. As such, we recommend the soil parameters depicted within **Table 8** be considered for such foundation designs, assuming a maximum post-construction vertical movement of one-inch. Load-bearing foundations should be installed atop properly prepared subgrade as indicated in **Section 8.3**.

Table 8: Recommended Soil Parameters for Shallow Foundations

Depth (ft)	Material	Max. Allowable Bearing Pressure (psf)	Vertical Subgrade Modulus	Soil / Concrete Friction Factor
1 to 2	Clay	600	75	0.3
2 to 4	Clay	1,000	125	0.3
4+	Clay	1,250	150	0.3

ANS Geo notes that **Table 8** includes bearing capacities for layers which may be impacted by frost. For foundations which are founded within the frost zone (as noted in **Section 7.2**), these foundations should be frost-protected by way of appropriately designed haunched edges, foam insulation, and/or free-draining structural fill extending to the frost depth. Should the maximum allowable bearing capacity be lower than required, ANS Geo recommends over-excavating below the recommended excavation depth and replacement of native material using additional structural fill placed and prepared as noted in **Section 8.3**. For each additional 12-inches of over-excavation and replacement of structural fill beyond the recommended minimum, an increase of 250 psf can be achieved.

The capacities and parameters noted in **Table 8** are based on foundation considerations and assumptions detailed in **Section 8.3** . The above recommendations in **Table 8** are based on strip footings and isolated spread footings with dimensions producing less than 100 square feet.

7.4.1 Mat Foundations

Mat foundations (100 square feet or larger, such as larger substation slabs) should be founded at a depth of at least two (2) feet or greater on at least 12 inches of properly compacted structural fill as indicated in **Section 8.3**.

Table 9: Recommended Soil Parameters for Mat Foundations

Depth (ft)	Material	Max. Allowable Bearing Pressure (psf)	Vertical Subgrade Modulus	Soil / Concrete Friction Factor
1 to 2	Clay	500	75	0.3
2 to 4	Clay	800	125	0.3
4+	Clay	1,100	150	0.3

Rigid mat foundations placed on properly compacted fill may be designed for a maximum allowable bearing capacity as shown in **Table 9** and is designed for an increased maximum settlement of two (2) inches. The mat foundation should be constructed on the compacted structural fill layer. Adequate construction joints and reinforcement should be provided to reduce the potential for cracking of the floor slab due to differential movement.

Lastly, sliding resistance of any shallow foundations will be largely provided by the friction between the concrete foundation and the underlying subgrade soils. Although the concrete foundation will be separated from the native soil by a compacted structural fill layer, we have conservatively considered direct contact on native fine-grained soils for purposes of obtaining a design value. The base friction coefficient for the foundation on native soils are provided in the above tables. The strains required to mobilize base friction are not compatible with the strains required to mobilize passive resistance. Therefore, we recommend that passive earth pressure be ignored.

7.5 Recommended Soil Parameters – Deep Foundations

If critical substation structures or transmission poles are subjected to heavy compressive and/or overturning loads, it is recommended that drilled pier foundations be used. Geotechnical design values have been created for use in Ensoft LPILE, Fad Tool's MFAD, or CAISSON software. These parameters have been provided in **Table A** immediately preceding the attachments.

7.5.1 Deep Foundation Capacities

Design capacities can be calculated using the diameter of the shaft, depth of the shaft, installation method, and various geotechnical parameters, provided in **Table A**, that define how the soil will behave under load. A summary of the recommended ultimate skin friction and end bearing values for drilled shaft design is provided in **Table A**. Piers should extend a minimum of 1.5 pier diameters into a given soil stratum to fully develop the recommended design end bearing strengths. A minimum factor of safety of two (2) must be applied to the skin friction values and three (3) to the end bearing capacities for design purposes.

Post-construction settlement for drilled piers designed for end bearing should be limited to one (1) inch or less, based on the recommended capacities provided herein. Foundation loads and dimensions would be required to calculate an explicit anticipated settlement. ANS Geo should be consulted with the final dimensions and loading of the proposed foundations to allow calculation of the anticipated settlement of each structure and confirm the settlement remains within a serviceable limit.

7.5.2 Deep Foundation Construction

Based on the variable presence of coarse grained soils within the upper fine-grained soils, temporary casing *may* be required, depending on the design embedment depth of the piers, to maintain borehole integrity prior to placement of reinforcing steel and concrete. Contractors should be required to provide bid prices for varying sizes and lengths of temporary casing based on the design depths and diameters of deep foundation elements. Piers should be poured the same day they are drilled and must not be left open overnight. If a pier cannot be poured on the same day as drilling, they may be loosely backfilled and re-drilled the following day for installation. To the extent possible, cast-in-place concrete should be placed "in the dry"; pumps or casing may be necessary to remove or prevent infiltration of groundwater into open excavations prior to placement of concrete.

Pier holes should be inspected for verticality (plumbness), proper depth of drilling, proper bearing strata, and cleanliness of the bottom of the excavation prior to introduction of reinforcing steel or concrete. ANS Geo encourages that concrete should be placed via tremie method to avoid consolidation or segregation of the aggregates in the concrete.

8 Construction Recommendations

8.1 Excavation

Depending on proposed foundation configurations, degree of earthwork, and depth of utilities, some excavations may extend deeper than four (4) feet below grade. Temporary excavations deeper than four (4) feet should be shored or sloped and benched, in accordance with OSHA regulations, to ensure safe working conditions within the excavations. For benching purposes, overburden clays may be considered as "Type A" material and should be sloped no steeper than 3/4H:1V (horizontal to vertical). All OSHA soil classifications should be field-determined by the contractor's "competent person" prior to excavation. Any proposed shoring systems should be designed by the contractor's "competent person", be certified by a Professional Engineer licensed in the State of Kansas and should be submitted to the engineer for review.

8.2 Dewatering

At the time of our geotechnical investigation, perched water and/or groundwater was encountered within one (1) of the eight (8) borings at approximately 30 feet below ground surface existing at the time of our investigation. As such, dewatering is not anticipated for shallow excavations. Notwithstanding, the contractor should be prepared to manage groundwater, perched water, and/or infiltrated stormwater as needed using localized sump-and-pump, wellpoint, or similar techniques to allow for concrete foundation construction in-the-dry. Water discharge should be managed in compliance with applicable state and local regulations. The contractor should be sure to grade the surface as necessary to divert stormwater away from open excavation to the extent possible.

8.3 Subgrade Preparation and Compaction

Prior to the installation of shallow concrete foundations, ANS Geo recommends over-excavating the subgrade by at least 12 inches, proof-rolling the subgrade, lining the exposed material with a geotextile separation fabric, and bringing the subgrade back up to the design foundation elevation with compacted structural fill as specified within **Table 10**. If geotextile fabric is not desired, an additional two (2) inches of stone should be provided to account for some impregnation of the stone into native soil, to maintain a capillary break, and maintain drainage.

Native material beneath the separation fabric should be inspected for unsatisfactory conditions such as standing water, frozen soil, unsuitable soil, organics, protruding cobbles or boulders, or deleterious materials. Should any unsatisfactory conditions exist within the native subgrade, the excavation should be undercut an additional six (6) inches (18 total inches beneath proposed foundation depth) prior to placement of the geotextile separation fabric.

Table 10: Recommended Specification of Structural Fill

Sieve Size	Percent Passing
2-inch	100
1 ½-inch	60 – 100
No. 4	30 – 60
No. 200	0 – 10
Max. Liquid Limit	**Max. Plasticity Index**
30	10

Should structural fill material not be available, in accordance with the specifications highlighted in **Table 10**, ANS Geo should be contacted to evaluate alternate materials. Structural fill should be placed in loose lifts not exceeding 12-inches if using large equipment, or 8-inches if using hand-operated tools such as jumping jacks, tamping plates, or similar equipment. Structural fill should be placed within two (2) percent of its optimum moisture content and be compacted to at least 95 percent of its Modified Proctor Density (ASTM D1557). The subgrade preparation (over-excavation, fabric, and structural fill) should horizontally extend at least two (2) times the compacted vertical structural fill thickness beyond each edge of the foundation. For example, a 12-inch over-excavation and compacted structural fill thickness should extend at least 24 inches laterally beyond each foundation edge.

As indicated in **Section 7.3**, potentially expansive soils were observed at the site, with the potential for movement of up to 1 inch, which is our assumed maximum tolerance for foundations. Shallow foundations are typically designed to accommodate a total movement of up to one inch, however on occasion a thickened slab with increased reinforcement (stiffened slab) may be able to accommodate up to two inches of movement and reduce differential movements. If shallow foundations are desired, ANS Geo recommends over-excavating soil beneath concrete slabs and shallow foundations to a depth of 12 inches and replacing the material with compacted structural fill, which would reduce the potential vertical rise (PVR). Structural fill should follow the material as indicated in **Section 8.3**. In addition, we recommend that any shallow-founded slabs be reinforced with either traditional reinforcing steel, or post-tensioned reinforcing cables.

Should the recommendations within this Report be followed and the applied bearing pressures provided in **Table 8** be followed, we expect that the vertical movement will be minimized to one-inch or less.

8.4 Backfilling and Compaction

8.4.1 Re-Use of Native Soils

ANS Geo notes that any native soils with considerable fine-grained content (more than 20 percent) may be difficult to handle, place, and compact without proper moisture conditioning and protection. ANS Geo recommends the following measures be considered to reduce the adverse impacts of moisture-sensitive soils:

- Positive measures should be implemented and maintained to intercept and direct surface water away from moisture-sensitive subgrade surfaces.
- Subgrade surfaces should be sloped and, as appropriate, seal-rolled to facilitate proper drainage. Surfaces should be properly prepared in anticipation of inclement weather. Moisture should not be allowed to collect on subgrade surfaces.
- To the extent practical, the limits of exposed subgrade soils should be minimized.
- Construction traffic should be limited to properly constructed haul roads.
- Disturbed soils should be removed and replaced with compacted controlled fill material.
- In place moisture contents should be maintained with two percent wet/dry of the optimum moisture content as determined by the Modified Proctor Test (ASTM D1557).

These soils may be re-used across the project area for fill in landscaped areas; however, it should not be used under or above foundations or load-bearing structures where typically imported structural fill or general backfill are used, respectively. Native material used as backfill for cable trenches should be handled and placed at a moisture content at or above its optimum value to ensure representative thermal properties are maintained. Native soils may also be used in required "fill" areas within the PV array footprint(s), provided that the material is placed and compacted consistent with the "general backfill" recommendations described herein.

8.4.2 General Backfill

In areas around and above installed foundations, large utilities, and other buried site features, ANS Geo recommends well-graded granular soils with less than 20 percent fine-grained content may be used as general backfill. Native soils meeting these criteria, if and where present, may also be used. General backfill material should be screened of any cobbles, boulders, and any particles larger than 3 inches in diameter, and should not be used beneath any load-bearing structures. General backfill should be placed in loose lift thicknesses not exceeding 12 inches and be compacted to at least 95 percent of its Modified Proctor Density (ASTM D1557). Soil used as backfill should not be handled when frozen and should be free of excessive moisture, organics, and deleterious material.

In fill areas beneath foundations and load-bearing structures, ANS Geo recommends structural fill as described in **Section 8.3** and **Table 10**.

8.4.3 Compaction Testing

Compaction testing should be performed at each discrete equipment foundation location for each compacted lift at a minimum of one test per 2,500 square feet. For linear sections such as trenches, the contractor and/or the owner's representative should perform a visual trench bottom inspection along the length of the trench to confirm no angular, sharp, deleterious, frozen, trash, organic material, or standing water exists at the bottom of trench. For backfilling and compaction of trenches, a minimum of one compaction test per 500 linear feet and minimum one per lift, should be performed. In all cases, the subgrade should be maintained, covered, or protected if concrete is not immediately placed. Excessively wet or dry material should be removed or improved prior to the placement of foundations.

8.5 Access Roads

ANS Geo understands that, as part of the work, access roads will be constructed to provide access for heavy equipment such as a main power transformer, poles, and other ancillary structures, as well as long-term access for site maintenance purposes. It is expected that new, unpaved paths will be constructed of aggregate material placed on native, compacted and proof-rolled subgrade stripped of topsoil and other organic material.

During construction, the delivery and movement of heavier loads such as transformers, inverters, delivery of steel and concrete, and transportation of cabling is expected. Construction loads and vehicles are larger and heavier than the expected vehicles during long-term operation; however, the duration of these activities will be much shorter considering the access road life. Designing for short-duration, construction-phase access road would require increased thickness of aggregate, the use of geogrid, or other soil improvement, but these increased roads would be over-designed for long-term operation including routine light-duty trucks, maintenance vehicles, and infrequent accessibility to emergency personnel including fire-fighting rigs. Therefore, it is typical for access road design to be completed considering the thickness of road base required for long-term use since it is expected that the site subcontractor will be able to maintain serviceable access roads throughout construction and at turn-over of the facility by backfilling ruts greater than two-inches, back-blading and re-compacting loose and rutted areas, re-shaping roads to promote drainage and safe passage of traffic, and other improvements.

Considering the above, ANS Geo has performed an evaluation of the required access road thickness based on infrequent emergency access for firefighting vehicles as well as occasional light vehicular traffic. Our preliminary road evaluation for a post-construction access road assumed the following:

Table 11: Access Road Design Considerations

Design Consideration	Design Assumption
Equivalent Single-Axle Loads (ESALs)	10,000
Allowable Rut Depth	2 inches
Subgrade Soil	Proof-rolled stiff clay
Assumed Min. Design Subgrade CBR	1.5% (following proof-roll and compaction)

ANS Geo recommends that road base material (flexible base) consists of clean, crushed stone or roadbase material with particle size distribution as presented in **Table 12.**

Table 12: Recommended Gradation of Crushed Stone (KDOT Type AB-1 Base Course)

Sieve Size	Percent Passing
2"	100
1 – ½"	90 – 100
¾"	60 – 95
No. 4	25 – 65
No. 8	15 – 56
No. 40	5 – 22
No. 200	2 – 10

ANS Geo has provided a number of access road configurations in **Table 13** based on the assumptions in **Table 11**. The use of a geotextile fabric (such as Mirafi HP270) is recommended and presented within our evaluation. In addition, it is possible and likely that certain areas will require stabilization or additional access stone thickness where weaker soils are present. The overall cross-sectional thickness may be reduced by the use of a Class II geogrid (such as Tensar BX1200 or TX7). Cement or lime stabilization can also be utilized to reduce the access road thickness, as long as the stabilized base has a soaked CBR of greater than 35 percent . This access road thickness can also be reduced if a greater rut depth is allowed to minimize the access road thickness as long as maintenance is performed to restore the roadway to a serviceable condition as damage occurs. A comparison of various options and configurations has been provided in **Table 13.**

Table 13: Recommended Aggregate Thickness for Permanent Site Access Roads

Aggregate Construction Option	Access Road Cross Section
Aggregate on prepared native soil	14 inches of Crushed Stone
Aggregate with geotextile fabric	12 inches of Crushed Stone over geotextile
Aggregate with Class II geogrid and geotextile fabric	8 inches of Crushed Stone over Class II geogrid atop geotextile
Aggregate over Chemically Stabilized Subgrade	12-inch treatment depth, 7-9% cement by weight + 4 inches Crushed Stone for wearing surface

ANS Geo understands that chemically-stabilized subgrades have been shown to deteriorate over time, and the performance of the treated subbase may require some additional maintenance during the design life of the project. Should chemically-stabilized roads be used, ANS Geo recommends that some maintenance (such as adding additional stone, where needed) be considered; or, an additional two inches of crushed stone can be added to the wear course to accommodate the potential deterioration of the chemically-stabilized subgrade.

When using geogrid, it is recommended that a geotextile fabric be placed between the clay subgrade and the geogrid to provide separation and avoid the stone aggregate to be blinded with fines. If geotextile fabric is not desired, an additional two (2) inches of stone should be provided to account for some impregnation of the stone into native soil. When geogrid is used, it should be placed in accordance with manufacturer's recommendations such as three (3) foot overlap, fastening overlapping areas, and material storage and handling.

Prior to roadway construction, the subgrade should be stripped of vegetation and topsoil, and should be confirmed to maintain a minimum CBR value assumed in **Table 11** and compaction to 95 percent of its Modified Proctor Density (ASTM D1557) to be in conformance with ANS Geo's above recommendations. Should the desired CBR and/or target compaction not be achieved, ANS Geo first recommends that the upper 12-inches be scarified, moisture-conditioned (dried or wetted to within +/- 2% of optimum moisture content), and re-placed and re-compacted. Should this not produce the desired minimum CBR and subgrade performance, soil improvement such as additional stone, and/or additional stabilization may be required to meet ANS Geo's minimum design recommendations. Crushed stone should be placed in loose lifts not exceeding eight (8) inches in height, and be compacted to ensure a minimum CBR of 35 percent is achieved.

Field conditions should be verified at the time of construction. Subgrade conditions could vary based on excavation depths, weather, drainage, and construction practices that disturb the subgrade. Dynamic cone penetrometer (DCP) testing should be completed on the prepared subgrade per ASTM D6951 and in a consistent manner by trained personnel to obtain useful and reliable data. ANS Geo recommends that, at minimum, DCP testing should be completed at a frequency of one test per each 500-linear feet of access roadway. Should conditions vary, this frequency may be increased or decreased based on observations from the site, or at the discretion of the Geotechnical Engineer of Record, Civil/Structural Engineer of Record or Owners Engineer. The tests should be staggered across the width of the road at outer wheel-tracks (left and right) and the centerline. However, the variability of the road subgrade strength will only become fully apparent when the tests have been carried out. In order to ensure statistical reliability, at least ten tests should be taken in each uniform section. The use of DCP testing may also be used to decrease the thickness of access road stone, if the prepared subgrade is stiffer (is confirmed to have a higher CBR) than ANS Geo's design assumption and no visible surface water or pumping is observed in the section of roadway being tested. ANS Geo can be contacted to provide a table of access road stone thickness compared to field-confirmed CBR.

ANS Geo notes that the presence of standing water may exist across the site around times of precipitation, during construction and development. The presence of water may make the native soil subgrade softer, and it may require additional site preparation to allow vehicles and equipment to pass. The Contractor should take these conditions into consideration, including the need for additional access stone and/or cement or lime for the stabilization of these conditions.

If chemical stabilization is performed, the contractor should perform any necessary due diligence to confirm their design, means, and methods. The subgrade should be verified below the treatment depth to evaluate the CBR value of the subgrade prior to treatment. In addition, the recommended chemical stabilization application rate should be taken as an assumed average. The actual application rate should be determined by the contractor and may vary based on the tested and desired subgrade CBR along the proposed roadway, the treatment depth required, and the moisture content. The application rate and treatment depth should be evaluated by performing several test strips at the project site prior to the start of construction and testing the test strips in the field using a dynamic cone penetrometer or plate load test to confirm the CBR. Then, once the application rate and depth are evaluated, verification and calibration testing should be performed using the dynamic cone penetrometer at intervals of no less than 500-linear feet along the access roadway.

9 Limitations

ANS Geo notes that the findings and recommendations presented within this Geotechnical Report are based on our investigation programs conducted during November 2024, and our engineering judgment. In addition, the current level of investigation does not represent the level of investigation to support a final design, and it is expected that a final, detailed-level geotechnical investigation will be completed at the site prior to final design and start of construction by an EPC to confirm and further define the recommendations provided herein. If ANS Geo's limited and preliminary investigation is used for final design, our recommendations shall only be valid for the exact and specific locations at which field investigations or laboratory testing was completed. All other areas and regions of the site which are not investigated under a final investigation to confirm if our preliminary and limited investigation is valid for the entire project site will be at the risk of the individual or entity using this Report.

If actual site subsurface conditions differ from the inferred conditions on which ANS Geo has based our confirmation-dependent recommendations, ANS Geo will need to modify our confirmation-dependent recommendations to develop final recommendations.