GEOTECHNICAL ENGINEERING REPORT

SUBSTATION NO. 3

 Pryor, Oklahoma

Prepared for:

FINLEY ENGINEERING
P.O. Box 148
Lamar, Missouri 64759

Prepared by:

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Environmental Services
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PROJECT NUMBER: 224708

November 25, 2014
November 25, 2014

Finley Engineering  
P.O. Box 148  
Lamar, Missouri  64759

Attn: Mr. Mark Thatcher, P.E.  
Phone: 417-682-5531  
Fax: 417-682-3220  
Email: m.thatcher@fecinc.com

RE: Geotechnical Engineering Report  
Substation No. 3  
Pryor, Oklahoma  
PPI Project Number: 224708

Dear Mr. Thatcher:

Attached, please find the report summarizing the results of the geotechnical investigation conducted for the above referenced project. We appreciate this opportunity to be of service. If you have any questions, please don’t hesitate to contact this office.

PALMERTON & PARRISH, INC.  
By:

Shane M. Rader, P.E.  
Geotechnical Engineer

Submitted: One (1) Bound Copy  
One (1) Electronic .pdf Copy

SMR/BRP/BRP/jrh
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EXECUTIVE SUMMARY

A Geotechnical Investigation was performed at the site planned for construction of the new Substation No. 3 located along the west side of Gaither Road in Pryor, Oklahoma. The proposed Substation is anticipated to include, but is not limited to, a Switching Station, miscellaneous Dead End Structures, Transformer foundations and potential Control Buildings. Further, a Transmission Tower is anticipated to be constructed along the east side of Gaither Road. Foundation and floor slab loadings are anticipated to be light to moderate, with the exception of the moderate to heavy loadings (both compressive and overturning) for the new Transmission Tower. Minimal depths of cut and/or fill are anticipated to achieve finish subgrade elevations.

As requested by Finley Engineering, a total of four (4) geotechnical borings were drilled at locations predetermined by the Client. All borings were discontinued in limestone at depths ranging from 16 to 19.3 ft. below the existing ground surface. Upon completion of Boring 1, a 10 ft. copper ground rod was installed as requested by Finley Engineering.

Based upon the information obtained from the borings and subsequent laboratory testing, the site is suitable for construction of the proposed new Substation No. 3. Important geotechnical considerations for the project are summarized below. However, users of the information contained in the report must review the entire report for specific details pertinent to geotechnical design considerations.

- Shallow natural soils encountered at the project site contain little to no rock/sand content and may undergo loss of shear strength properties upon an increase in soil moisture or when disturbed by heavier construction equipment. Additional undercut depths may be required during site development, to provide a stable subgrade. Delay of site grading until dryer months should reduce subgrade preparation difficulties and associated costs;

- Limestone and/or sandstone was encountered in all borings at depths ranging from 6 to 9.5 ft. below the existing ground surface;
It is anticipated that a “balanced” site may be desired during earthwork procedures. Site Designers should refer to Sections 7.0 and 8.0 regarding soil conditions which may have a profound effect upon earthwork quantities;

Higher plasticity clays at this site have a low potential for shrink/swell behavior. Based upon laboratory swell tests, a minimum of 2 ft. of Low Volume Change (LVC) fill material should be placed below all shallow foundations and floor slabs;

Based upon the geotechnical borings drilled and the presence of relatively shallow limestone, structures exhibiting light to moderate foundation loads may be supported upon shallow foundations bearing on 2 ft. of LVC fill material or limestone bedrock. If higher bearing capacities are desired, foundation excavations may extend to limestone bedrock or deep foundations bearing in limestone may be utilized. Recommendations for shallow and deep foundations are provided in Section 8.0 of this report;

The project site classifies as a Site Class C in accordance with Section 1613 of the 2012 International Building Code (IBC); and

Palmerton & Parrish, Inc. should be retained for construction observation and construction materials testing. Close monitoring of subgrade preparation work is considered critical to achieve adequate foundation and subgrade performance.
1.0 INTRODUCTION

This is the report of the Geotechnical Investigation performed at the site planned for construction of the new Substation No. 3 located on the west side of Gaither Road in Pryor County, Oklahoma. This investigation was authorized by a letter proposal dated September 12, 2014, and signed by Mr. Philip W. Carroll, Vice President of Finley Engineering. The approximate site location is shown below.
The purpose of the Geotechnical Investigation was to provide information for foundation design and construction planning, and to aid in site development. Palmerton & Parrish Inc.’s (PPI) scope of services included field and laboratory investigation of the subsurface conditions in the vicinity of the proposed project site, engineering analysis of the collected data, development of recommendations for foundation design and construction planning, and preparation of this engineering report.

### 2.0 PROJECT DESCRIPTION

<table>
<thead>
<tr>
<th>Item</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Site Layout</td>
<td>See Figure 1: Boring Location Plan</td>
</tr>
<tr>
<td>Pryor Substation No. 3</td>
<td>The Substation is anticipated to consist of, but is not limited to:</td>
</tr>
<tr>
<td></td>
<td>• Switching Station structures;</td>
</tr>
<tr>
<td></td>
<td>• Dead End structures;</td>
</tr>
<tr>
<td></td>
<td>• Transformer foundations; and</td>
</tr>
<tr>
<td></td>
<td>• Control Buildings.</td>
</tr>
<tr>
<td>Transmission Tower</td>
<td>The Transmission Tower is anticipated to be constructed on the east side of Gaither Road.</td>
</tr>
<tr>
<td>Substation No. 3 Foundation Loadings</td>
<td>Anticipated to be light to moderate.</td>
</tr>
<tr>
<td>Transmission Tower Foundation Loadings</td>
<td>Anticipated to be moderate to heavy, both compressive and overturning.</td>
</tr>
<tr>
<td>Anticipated Grading</td>
<td>Minimal grade changes to achieve finish subgrade elevation across the site.</td>
</tr>
</tbody>
</table>

### 3.0 SITE DESCRIPTION

<table>
<thead>
<tr>
<th>Item</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Township/Range/Section</td>
<td>T21N/R19E/S8</td>
</tr>
<tr>
<td>Latitude/Longitude (± Center of Project Site)</td>
<td>36.309187°N/-95.292951°W</td>
</tr>
<tr>
<td>Current Ground Cover</td>
<td>Grass covered open field.</td>
</tr>
<tr>
<td>Existing Topography</td>
<td>Gently sloping topography from the northeast to the southwest with elevations decreasing from 673 to 666.</td>
</tr>
<tr>
<td>Drainage Characteristics</td>
<td>Poor to fair.</td>
</tr>
</tbody>
</table>

### 4.0 SUBSURFACE INVESTIGATION

Subsurface conditions were investigated through completion of subsurface borings and subsequent laboratory testing.
4.1 Subsurface Borings

As requested by Finley Engineering, subsurface conditions at this site were investigated by drilling a total of four (4) sample borings located within the proposed development. All borings were selected by Finley Engineering and staked in the field by PPI using a site plan provided by Client. Borings were staked using a standard measuring wheel and right angles from known features identified on the site plan provided by the Client. Approximate boring locations are shown on Figure 1: Boring Location Plan.

All borings were discontinued in limestone bedrock at depths ranging from 16 to 19.3 ft. below the existing ground surface. Upon boring completion, a 10 ft. cooper clad ground rod was installed in Boring 1 per the Client’s request. The Oklahoma One-Call System was notified prior to the investigation to assist in locating buried public utilities. Logs of the borings showing descriptions of soil and rock units encountered, as well as results of field and laboratory tests are presented in Appendix I. Approximate surface elevation of each boring location was obtained using the contours provided on the site plan and are noted on each log form in Appendix I.

Borings were drilled November 4 through 6, 2014 using 4.5-inch O.D. continuous flight augers powered by a BK-51 ATV-mounted drill-rig. Soil samples were collected at 2.5 to 5-ft. centers during drilling. Soil sample types included split spoon samples collected while performing the Standard Penetration Test (SPT) in general accordance with ASTM D1586 and thin walled Shelby tubes pushed hydraulically in advance of drilling in accordance with ASTM D1587.

Upon encountering bedrock within all borings, rock coring procedures were implemented using an NQ₂ core barrel and a Series 10 diamond impregnated core bit. Please refer to Appendix III for general notes regarding boring logs and additional soil sampling information.

4.2 Laboratory Testing

Collected samples were sealed and transported to the laboratory for further evaluation and visual examination. Laboratory soil testing included the following:
• Moisture Content (ASTM D2216);
• Unconfined Compressive Strength (ASTM D2166);
• Atterberg Limits (ASTM D4318);
• Swell Tests (ASTM D4546); and
• Pocket Penetrometers.

Laboratory test results are shown on each boring log in Appendix I and are summarized in the following table.

<table>
<thead>
<tr>
<th>Boring</th>
<th>Depth (ft.)</th>
<th>Liquid Limit (LL)</th>
<th>Plastic Limit (PL)</th>
<th>Plasticity Index (PI)</th>
<th>Moisture Content (%)</th>
<th>USCS Symbol</th>
<th>Cohesion (psf)</th>
<th>Dry Unit Wt. (pcf)</th>
<th>Percent Swell (%)</th>
<th>Swell Pressure (tsf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0 to 2</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>21.0</td>
<td>CL-CH</td>
<td>1003</td>
<td>101.4</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>1</td>
<td>3 to 4.5</td>
<td>57</td>
<td>17</td>
<td>40</td>
<td>19.7</td>
<td>CH</td>
<td>2943</td>
<td>101.9</td>
<td>0.72</td>
<td>0.57</td>
</tr>
<tr>
<td>2</td>
<td>1 to 3</td>
<td>49</td>
<td>16</td>
<td>33</td>
<td>20.6</td>
<td>CL-CH</td>
<td>1412</td>
<td>103.5</td>
<td>1.18</td>
<td>0.6</td>
</tr>
<tr>
<td>3</td>
<td>0 to 2</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>20.3</td>
<td>CL</td>
<td>1137</td>
<td>100.7</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

5.0 SITE GEOLOGY

The general site area is underlain at depth with Mississippian Age deposits. These rock units are cyclic in nature containing alternating beds of limestone, shale and sandstone. The upper surface of this limestone is often irregular due to the effects of differential vertical weathering and solution activities. Overburden soils at the site are typically residual having developed through chemical and physical weathering of the underlying parent bedrock. The boundary between overburden soils and comparatively unweathered limestone is usually abrupt, while the boundary between overburden soils and sandstone/shale is generally a more gradual transition.

6.0 GENERAL SITE & SUBSURFACE CONDITIONS

Based upon subsurface conditions encountered within the borings drilled at the project site, generalized subsurface conditions are summarized in the table below. Soil stratification lines on the boring logs indicate approximate boundary lines between
different types of soil and rock units based upon observations made during drilling. In-situ transitions between soil and some rock types are typically gradual.

<table>
<thead>
<tr>
<th>Description</th>
<th>Borings</th>
<th>Approx. Depth to Bottom of Stratum</th>
<th>Material Encountered</th>
<th>Moisture</th>
<th>Consistency/Density</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stratum 1</td>
<td>All</td>
<td>0.5 ft.</td>
<td>Topsoil</td>
<td>Moist to Very Moist</td>
<td>Soft</td>
</tr>
<tr>
<td>Stratum 2</td>
<td>All</td>
<td>1.5 ft.</td>
<td>Lean Clay</td>
<td>Moist</td>
<td>Medium Stiff to Stiff</td>
</tr>
<tr>
<td>Stratum 3</td>
<td>All</td>
<td>Top of Limestone/Sandstone</td>
<td>Lean to Fat Clay or Fat Clay</td>
<td>Moist</td>
<td>Medium Stiff to Very Stiff</td>
</tr>
<tr>
<td>Stratum 4</td>
<td>All</td>
<td>Boring Completion</td>
<td>Limestone/Sandstone</td>
<td>-</td>
<td>Medium Hard</td>
</tr>
</tbody>
</table>

6.1 Bedrock

Limestone and/or sandstone was encountered within all borings at depths ranging from 6 to 9.5 ft. below the existing ground surface. The sandstone was logged as tan brown fine grained and slightly weathered, while the limestone was logged as light gray to tan, slightly weathered, fine crystalline and medium hard. Percent recoveries ranging from 80 to 100 percent and RQD values ranging from 0 to 100 percent (but primarily 42 to 100 percent) were recorded within the limestone indicating very poor to excellent (but primarily poor to excellent) rock quality. Refer to the following table for additional information regarding the limestone bedrock obtained during coring procedures.

<table>
<thead>
<tr>
<th>Boring</th>
<th>Approximate Surface Elevation</th>
<th>Approximate Depth to Top of Bedrock (ft.)</th>
<th>Correlating Elevation of Top of Bedrock</th>
<th>REC (%)</th>
<th>RQD (5)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>669.0</td>
<td>6.0</td>
<td>663.0</td>
<td>100/90/100</td>
<td>0/73/90</td>
</tr>
<tr>
<td>2</td>
<td>667.5</td>
<td>8.3</td>
<td>659.2</td>
<td>100/100/80</td>
<td>98/95/80</td>
</tr>
<tr>
<td>3</td>
<td>666.0</td>
<td>9.5</td>
<td>656.5</td>
<td>100/100/100</td>
<td>42/92/100</td>
</tr>
<tr>
<td>4</td>
<td>669.5</td>
<td>8.5</td>
<td>661.0</td>
<td>100/100/100</td>
<td>95/95/77</td>
</tr>
</tbody>
</table>

Photographs of the rock core are provided in Appendix II.
6.2 Groundwater

Shallow groundwater was not observed within the borings on the date drilled. Groundwater levels should be expected to fluctuate with changes in site grading, precipitation, and regional groundwater levels. Groundwater may be encountered at shallower depths during wetter periods.

7.0 EARTHWORK

Although site specific grading plans for this project have not been reviewed, it is anticipated that minimal depths of cut and/or fill will be required to provide finish subgrade elevations across the project site. It is also anticipated that finish subgrade elevation will be selected to “balance” cut and fill volumes. The initial phase of site preparation should include the following:

- Clearing and grubbing of all vegetative matter and organic topsoil, if any;

- Topsoil, if any should be stockpiled outside of areas to receive controlled fill and may be reused in lawn and landscape areas only;

- Due to the low potential for shrink/swell of the shallow clays, LVC material is recommended below floor slabs and shallow foundations. Grading contractors should refer to Section 8.0 and 10.0 of this report; and

- Proof-rolling of areas scheduled to receive controlled fill in accordance with the following section of this report.

Proof-rolling consists essentially of rolling the ground surface with a loaded tandem axle dump truck or similar heavy rubber tired construction equipment and noting any areas which rut or deflect during rolling. All soft subgrade areas identified during proof-rolling should be undercut and replaced with compacted fill as outlined below. Proof-rolling, undercutting and replacement should be monitored by a qualified representative of PPI. The depth and areal extent of undercutting will be largely dependent upon the time of year and related soil moisture conditions. If construction is initiated during wetter months, the requirement for undercutting soft surficial soils below normal topsoil stripping and/or use of “bridge” lift procedures should be
anticipated and reflected in contract documents. This is discussed further in Section 7.4.

In lieu of, or in addition to removal and replacement, use of “bridge” lift procedures may also be considered. “Bridge” lift procedures usually consist of placing an initial 24 to 30 inch thick lift of select soil or large size stone full thickness in front of all construction equipment. The surface is then track walked using a crawler tractor to provide a stable surface for placement and compaction of subsequent lifts. Use of earth “bridge” lifts is not recommended within foundation areas. Bridge lifts may also incorporate geogrid to improve stability and/or reduce “bridge” lift thickness.

Chemical stabilization may also be considered for subgrade improvement. Chemical agents which may be considered include Type C Flyash, Code “L” kiln dust and hydrated lime. Our office will be happy to provide more detailed recommendations for chemical stabilization, if selected, and after final grading plans are prepared.
### 7.1 Fill Material Types

<table>
<thead>
<tr>
<th>Fill Type</th>
<th>USCS Classification</th>
<th>Acceptable Location for Placement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low Volume Change Engineered Fill²</td>
<td>CL, GC, or SC (LL&lt;50)</td>
<td>All locations and elevations</td>
</tr>
<tr>
<td>On-Site Natural Soils</td>
<td>CL, CL-CH &amp; CH³</td>
<td>All locations and elevations</td>
</tr>
<tr>
<td>Rock Fill³</td>
<td>GW</td>
<td>All locations and elevations</td>
</tr>
</tbody>
</table>

1. Controlled, compacted fill should consist of approved materials that are free of organic matter and debris and contain a maximum rock size of 4 to 6 in. Frozen material should not be used and fill should not be placed on a frozen subgrade. A sample of each material type should be submitted to the Geotechnical Engineer for evaluation prior to its use.

2. Low plasticity cohesive soil with a liquid limit below 45% or granular soil having at least 15% low plasticity fines.

3. CL-CH or CH Clays with a Liquid Limit equal to or greater than 45 is considered suitable for use as controlled fill, only if the percentage of sand/rock content exceeds 35% or if placed at least 2 ft. below shallow foundations and floor slabs.

4. Rock fill should consist of 4 to 6 inch top size rock fill with no particles greater than 8 inches. Rock material should be placed in horizontal layers having a thickness of approximately the maximum size of the large rock comprising the lift and compacted with a minimum of three (3) passes with a heavy self-propelled vibratory roller. Rock fill should not be dumped into place, but should be distributed in horizontal lifts by blading and dozing in such a manner as to ensure proper placement into final position. Finer material including rock fines and limited soil fines should be worked into the rock voids during this blading operation. Excessive soil and rock fine particles preventing interlock of cobble and boulder sized rock should be prohibited. The testing of rock fill quality should include the requirements that a representative of the Geotechnical Engineer be present daily, but not necessarily continuous during the placement of the fill to observe the placement of rock fill in order to determine fill quality and to observe that the contractors work sequence is in compliance with this specification. Progress reports indicative of the quality of the fill should be made at regular intervals to the Owner. If improper placement procedures are observed during the placement of the fill the Geotechnical Engineer should inform the Contractor, and no additional fill should be permitted on the affected area until the condition causing the low densities has been corrected and the fill has been reworked to obtain sufficient density.

### 7.2 Compaction Requirements

<table>
<thead>
<tr>
<th>Item</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Subgrade Scarification Depth</td>
<td>At least 8 inches</td>
</tr>
<tr>
<td>Fill Lift Thickness</td>
<td>8-inches (loose)</td>
</tr>
<tr>
<td>Compaction Requirements¹</td>
<td>95% Standard Proctor Density (ASTM D-698)</td>
</tr>
<tr>
<td>Moisture Content</td>
<td>±2% optimum moisture for CL, GC or SC soil types; and 0 to 4% above optimum for CL-CH &amp; CH soil types</td>
</tr>
</tbody>
</table>

1. We recommend that engineered fill (including scarified compacted subgrade) be tested for moisture content and compaction during placement. Should the results of the in-place density tests indicate the specified moisture or compaction limits have not been met, the area represented by the test should be reworked and retested as required until the specified moisture and compaction requirements are achieved. One (1) field density test for each 2500 and 5000 sq. ft. of fill lift, but no less than 3 tests per lift, is recommended in structure and pad/roadway areas, respectively.
7.3 Earthwork Construction Considerations

Once grading and filling operations have been completed, the moisture within the subgrade should be maintained and soils not be allowed to dry and desiccate prior to construction of floor slabs and footings. Grading of the site should be performed in such a manner so that ponding of surface water on prepared subgrade or in excavations is avoided. During construction, if the prepared subgrade should become frozen, desiccated, saturated, or disturbed, the affected material should be scarified or removed, moisture conditioned and recompacted prior to construction.

7.4 Inclement Weather

If construction is initiated during wetter months, the requirement for undercutting soft surficial soils below normal site stripping should be anticipated and reflected in contract documents. Undercut depths on the order of 1.5 or more ft. are considered possible within the proposed substation footprint. Based upon past experience of this firm, the lean clay subgrade at the site is known to significantly lose strength when saturated and disturbed by construction equipment. Further, material removed from undercuts may not be suitable for use as compacted fill due to high soil moisture if poor drying conditions (cool temperatures and/or frequent precipitation) occur during site grading. If the construction schedule will not permit delay for better drying conditions, the project budget should include an allowance for subgrade undercut and replacement soil material containing appreciable quantities of sand and gravel from an off-site borrow area that meet the requirements above. As an alternate to select fill, rock fill subbase (4 to 8 inch top size stone) may be placed to improve subgrade stability.

7.5 Excavations

Based upon the subsurface conditions encountered during this investigation, the on-site soils typically classify as Type B in accordance with OSHA regulations. Temporary excavations in soils classifying as Type B with a total height of less than 20 ft. should be cut no steeper than 1H:1V in accordance with OSHA guidelines. Limestone or sandstone may be cut to near vertical walls. Confirmation of soil
classification during construction, as well as construction safety (including shoring, if required), is the responsibility of the contractor.

8.0 FOUNDATIONS

As previously mentioned, relatively shallow limestone was encountered within all boring locations at depths ranging from 6 to 9.3 ft. below the existing ground surface. Further, it is anticipated that the planned new Substation No. 3 will consist of, but not be limited to miscellaneous Support Stands, Transformer Pads, Switching Substations, and a Control Building which may exhibit light to moderate foundation loads. Based upon this information, as well as the low potential for shrink/swell of the shallow natural clays, it is recommended that these structures be supported upon shallow foundations on a minimum of 2 ft. of LVC fill material. In lieu of placing LVC material below the shallow foundations, the Design Team can elect to support these structures upon shallow foundations bearing on bedrock or deep foundations.

Subgrade preparation should be in accordance with Section 8.0 of this report. Refer to the following section for shallow foundation recommendations.
### Description

<table>
<thead>
<tr>
<th>Description</th>
<th>Mat (Spread Footing)</th>
<th>Continuous Footing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Net allowable bearing pressure(^1)</td>
<td>2,500 psf (controlled fill)</td>
<td>2,000 psf (controlled fill)</td>
</tr>
<tr>
<td></td>
<td>6,000 (^2) psf (bedrock)</td>
<td>6,000 (^2) psf (bedrock)</td>
</tr>
<tr>
<td>Minimum dimensions(^3)</td>
<td>2.5 ft.</td>
<td>1.5 ft.</td>
</tr>
<tr>
<td>Recommended bearing depth(^4)</td>
<td>2.0 ft.</td>
<td>2.0 ft.</td>
</tr>
<tr>
<td>Maximum footing width, if full allowable bearing pressure is utilized</td>
<td>8.0 ft.</td>
<td>8.0 ft.</td>
</tr>
<tr>
<td>Minimum embedment below finished grade for frost protection and variation in soil moisture (footings on soil)(^5)</td>
<td>2.5 ft.</td>
<td>2.5 ft.</td>
</tr>
<tr>
<td>Minimum footing bearing depth below compacted fill surface</td>
<td>1 ft.</td>
<td>1 ft.</td>
</tr>
<tr>
<td>Allowable passive pressure(^6)</td>
<td>500 psf (controlled fill)</td>
<td>500 psf (controlled fill)</td>
</tr>
<tr>
<td></td>
<td>1,500 (^7) psf (bedrock)</td>
<td>1,500 (^7) psf (bedrock)</td>
</tr>
<tr>
<td>Coefficient of sliding friction(^8)</td>
<td>0.4 (controlled fill)</td>
<td>0.4 (controlled fill)</td>
</tr>
<tr>
<td></td>
<td>0.7 (bedrock)</td>
<td>0.7 (bedrock)</td>
</tr>
</tbody>
</table>

1. The recommended net allowable bearing pressure is the pressure in excess of the minimum surrounding overburden pressure at the footing base elevation. The recommended pressure considers all unsuitable and/or soft or loose soils, if encountered, are undercut and replaced with tested and approved new engineered fill. Footing excavations should be free of loose and disturbed material, debris, and water when concrete is placed.
2. In lieu of bearing the shallow foundations directly upon bedrock, the contractor may excavate the foundation to bedrock and place lean concrete to desired footing bottom.
3. Footing widths should be designed as narrow as possible (but permissible by local building codes) to increase bearing pressure upon the shallow clays, if foundations are designed to bear upon LVC fill material.
4. PPI should be retained to observe footing bottoms prior to placement of reinforcing steel.
5. For perimeter footings and footings beneath unheated areas.
6. Allowable passive pressure value considers a Factor of Safety of about 2. Passive pressure value applies to undisturbed native clay or properly compacted fill. If formed footings are constructed, the space between the formed side of a footing and excavation sidewall should be cleaned of all loose material, debris, and water and backfilled with tested and approved fill compacted to at least 95% of the material’s Standard Proctor dry density. Passive resistance should be neglected for the upper 2.5 ft. of the soil below the final adjacent grade due to strength loss from freeze/thaw and shrink/swell.
7. Allowable passive pressure value considers a Factor of Safety of about 2. Passive pressure values apply to clean rock surface free of all loose material, debris and water.
8. Coefficient of friction value is an ultimate value and does not contain a Factor of Safety.

### 8.1.1 Uplift Capacity

Resistance of shallow spread footings to uplift (\(U_p\)) may be based upon the dead weight of the concrete footing structure (\(W_C\)) and the weight of soil backfill contained in an inverted cone or pyramid directly above the footings (\(W_S\)). The following parameters may be used in design:
The base of the cone or pyramid should be the top of the footing and the pyramid or cone sides should form an angle of 30 degrees with the vertical. Allowable uplift capacity \( U_p \) should be computed as the lesser of the two (2) equations listed below:

\[
U_p = \left( \frac{W_s}{2.0} \right) + \left( \frac{W_c}{1.25} \right)
\]

\[
U_p = \frac{W_s + W_c}{1.5}
\]

### 8.1.2 Construction Considerations

Footing/mat bottoms should not be allowed to become frozen, desiccated, saturated or disturbed prior to concrete placement to help reduce the potential for shrink/swell behavior. All affected materials should be removed from excavations. Footings/mats should be clean and free of standing water, debris and loose soil at the time of concrete placement. Footing/mat excavations should be inspected by a representative of PPI to confirm soil subgrade conditions are consistent with the design bearing pressure.

### 8.2 Drilled Piers

In lieu of, or in addition to supporting structures upon the bedrock utilizing shallow foundations, drilled piers may also be considered. It is anticipated that the Transmission Tower which will be constructed on the east side of Gaither Road will exhibit moderate to heavy compressive and overturning loadings requiring support by drilled piers. If drilled piers are selected for the substation structures or Transmission Tower, all drilled shafts should have relatively straight shafts, plumb pier bottoms and should be founded at least 1 ft. into competent limestone/sandstone. The piering contractor should be capable of removing a few to several feet of limestone to provide a level pier bottom with adequate rock socket. It is recommended that pier shafts for this project be observed by a representative of PPI to assure a relatively flat pier bottom, plumb pier shaft and competent bearing.
rock consistent with the recommended bearing pressure and removal of essentially all groundwater prior to concrete placement. With pier installation as outlined above, an allow end bearing pressure of 12 ksf may be used in pier design. It is recommended that pier shafts have a minimum shaft diameter of 18-inches. For frost protection, it is recommended that exterior grade beams, if any, be embedded at least 2.5 ft. below finish grade.

The underlying bedrock is capable of supporting a much higher bearing pressure. If a higher bearing pressure is desired, the bedrock in each pier bottom should be evaluated by drilling a 2-inch diameter probe boring to a depth of 1.5 times the pier diameter or a minimum of 5 ft. below pier bottom. Probe borings should be scraped with a right angle chisel point (scratch tested) by a representative of PPI to verify the bearing rock is free of clay seams and/or voids. Pier shafts should also be observed prior to concrete placement to verify relatively level pier bottom and plumb pier shaft. With proof-testing measures implemented in the field, an allowable end bearing pressure of 40 ksf may be used in pier design.

Although perched groundwater was not encountered in the borings, pier drilling contractors should be prepared to provide casing capable of being screwed into the limestone and sealing out soft wet soils below the groundwater table and limiting groundwater inflow into pier shafts. Pier shafts should be dewatered to a maximum water depth of 2 to 3 inches. prior to concreting or a tremie used for concrete placement. Some suggested items for inclusion in the drilled shaft section of project specifications are included with this report as Appendix V.

8.2.1 Uplift Resistance for Drilled Shafts

For resistance to uplift, side friction may also be applied to the sidewalls, with the exception of the top 1 times the diameter of the drilled shaft or 2.5 ft. whichever is shallower. An allowable side friction of 400 psf may be used for the overburden soils which typically extend to depths ranging from 6 to 9 ft. below existing ground surface. An allowable side friction of 1.0 ksf may be used for shaft length embedded into bedrock. However, due to the top of the limestone often being
weathered, the upper 1 ft. of the rock socket should be ignored. Side friction within the bedrock may also be used to resist compressive loads.

### 8.2.2 Lateral Loadings For Drilled Shafts

It is anticipated that resistance of the foundations to lateral loading and the associated lateral deflection will be evaluated using finite difference computer models based on the horizontal modulus of subgrade reaction ($K_h$). The following values may be used in the analysis for this site.

<table>
<thead>
<tr>
<th>Pier Depth</th>
<th>Unit Weight (pcf)</th>
<th>Static $K_h$ (pci)</th>
<th>Cyclic $K_h$ (pci)</th>
<th>Cohesion $S_u$ (ksf)</th>
<th>$\Sigma_{s0}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>*0-1 Pier Diameter</td>
<td>Ignore</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>*1 Pier Diameter to Bedrock</td>
<td>110</td>
<td>500</td>
<td>200</td>
<td>1.5</td>
<td>0.005</td>
</tr>
<tr>
<td>Upper 1 ft. of Bedrock</td>
<td>135</td>
<td>1000</td>
<td>400</td>
<td>4.0</td>
<td>0.004</td>
</tr>
<tr>
<td>Bedrock Unit Below Upper 1 ft. of Bedrock</td>
<td>135</td>
<td>&gt;2000</td>
<td>&gt;800</td>
<td>72.0</td>
<td>&lt;0.004</td>
</tr>
</tbody>
</table>

*Lateral parameters for the upper 1 pier diameter, or 2.5 ft., whichever is shallower, should be ignored.

The above values were measured or based upon published correlations with anticipated soil strength and classification tests.

### 9.0 SEISMIC CONSIDERATIONS

<table>
<thead>
<tr>
<th>Code Used</th>
<th>Site Classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>2012 International Building Code (IBC)¹</td>
<td>C</td>
</tr>
</tbody>
</table>

1. In general accordance with the *2012 International Building Code*, Section 1613.

### 10.0 FLOOR SLABS

A slab-on-fill type construction is considered appropriate within portions of this project site provided subgrade preparation and fill placement is performed in accordance with this report. Due to the low potential for shrink/swell based upon laboratory swell test, a minimum of 2 ft. of LVC fill material is recommended below floor slabs. Placement of 4 or more inches of compacted free-draining granular base course below slabs is recommended to limit moisture rise through slabs and to improve slab support,
particularly at joints. It is recommended that a 6-mil impervious moisture barrier or equivalent be provided below slabs as required by the 2012 IBC Code. If slab areas are particularly sensitive to moisture due to intended use, a 10-mil impervious barrier or thicker should be used.

11.0 CONSTRUCTION OBSERVATION & TESTING

The construction process is an integral design component with respect to the geotechnical aspects of a project. Since geotechnical engineering is influenced by variable depositional and weathering processes and because we sample only a small portion of the soils affecting the performance of the proposed structures, unanticipated or changed conditions can be disclosed during grading. Proper geotechnical observation and testing during construction is imperative to allow the Geotechnical Engineer the opportunity to evaluate assumptions made during the design process. Therefore, we recommend that PPI be kept apprised of design modifications and construction schedule of the proposed project to observe compliance with the design concepts and geotechnical recommendations, and to allow design changes in the event that subsurface conditions or methods of construction differ from those assumed while completing this study. We recommend that during construction all earthwork be monitored by a representative of PPI, including site preparation, placement of all engineered fill and trench backfill, and all foundation excavations as outlined below.

- An experienced Geotechnical Engineer or Engineering Technician of PPI should observe the subgrade throughout the proposed project site immediately following stripping to evaluate the native clay, identify areas requiring additional undercutting, and evaluate the suitability of the exposed surface for fill placement;

- An experienced Engineering Technician of PPI should monitor and test all fill placed within the structure areas to determine whether the type of material, moisture content, and degree of compaction are within recommended limits;
• An experienced Technician or Engineer of PPI should observe and test all footing excavations. Where unsuitable bearing conditions are observed, remedial procedures can be established in the field to avoid construction delays; and

• The condition of the subgrade should be evaluated immediately prior to construction of the building floor slabs to determine whether the moisture content and relative density of the subgrade soils are as recommended.

12.0 REPORT LIMITATIONS

This report has been prepared in accordance with generally accepted practices of other consultants undertaking similar studies at the same time and in the same geographical area. Palmerton & Parrish, Inc. observed that degree of care and skill generally exercised by other consultants under similar circumstances and conditions. Palmerton & Parrish’s findings and conclusions must be considered not as scientific certainties, but as opinions based on our professional judgment concerning the significance of the data gathered during the course of this investigation. Other than this, no warranty is implied or intended.
This boring is to take place within the right-of-way of the existing 69 KV high voltage line.