GEOTEchnical ENGINEERING STUDY

FOR

PARTELLO 30-INCH WATER MAIN LINE PHASE II
FRED WILSON AVENUE TO HAYES AVENUE

PREPARED FOR

CDM SMITH
EL PASO, TEXAS
Project No. AEA14-018-00
November 11, 2014

Horacio Juarez, P.E.
Senior Project Manager
CDM Smith
4110 Rio Bravo, Suite 201
El Paso, Texas 79902

RE: Geotechnical Engineering Study
Partello 30-inch Water Main Line Phase II
Fred Wilson Avenue to Hayes Avenue
El Paso, Texas

Dear Mr. Juarez:

RABA KISTNER Consultants, Inc. (RKCI) is pleased to submit the report of our Geotechnical Engineering Study for the above-referenced project. This study was performed in accordance with RKCI Proposal No. PEA14-003-00, dated January 23, 2014. Authorization for this study was received by our firm on May 16, 2014. The purpose of this study was to drill borings within the proposed water main alignment, to perform laboratory testing to classify and characterize subsurface conditions, and to prepare an engineering report presenting pipeline installation and backfill recommendations.

We appreciate the opportunity to be of service to you on this project. Should you have any questions about the information presented in this report, or if we may be of additional assistance on the materials testing-quality control program during construction, please contact us.

Very truly yours,

RABA KISTNER CONSULTANTS, INC.

Ivan Avelar, P.E.      Thomas M. Vick, P.E., PMP
Project Engineer     Vice President

IA/TMV

Attachments

Copies Submitted: Above (3-letterhead, 1-electronic)
GEOTECHNICAL ENGINEERING STUDY

For

PARTELLO 30-INCH WATER MAIN LINE PHASE II
FRED WILSON AVENUE TO HAYES AVENUE

Prepared for

CDM SMITH
EL PASO, TEXAS

Prepared by

RABA KISTNER CONSULTANTS, INC.
EL PASO, TEXAS

PROJECT NO. AEA14-018-00

NOVEMBER 2014
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INTRODUCTION

RABA KISTNER Consultants, Inc. (RKCI) has completed the authorized subsurface exploration for the proposed Partello 30-inch water main line to be installed in north-central El Paso. This report briefly describes the procedures utilized during this study and presents our findings along with our recommendations for pipeline installation and backfill.

PROJECT DESCRIPTION

El Paso Water Utilities (EPWU) is planning to install a 30-inch diameter main water line that will be approximately 1820 feet in length through Fort Bliss property, extending from Fred Wilson Avenue to Hayes Avenue, in northeast El Paso, Texas. The alignment will be installed at an invert elevation of about 8 feet below the existing ground surface, and will be located in close proximity and parallel to Dyer Street right-of-way, with the majority of the pipe easement planned to be inside Fort Bliss property.

We anticipate open-trench pipe installation techniques will be used to install the pipeline within the majority of the alignment, with the exception of the northern segment crossing Fred Wilson Avenue, in which we anticipate the use of tunneling techniques may be needed. We also understand no grading changes to the existing ground surface elevation will be required to reach the finished surface grades within the planned pipeline easement.

LIMITATIONS

This engineering report has been prepared in accordance with accepted Geotechnical Engineering practices in the region of west Texas, and for the use of CDM Smith (Client) and its representatives for design purposes. This report may not contain sufficient information for purposes of other parties or other uses. This report is not intended for use in determining construction means and methods.

The recommendations submitted in this report are based on the data obtained from the borings drilled at this site, our understanding of the project information provided to us, and the assumption that site grading will result in only minor changes in the existing topography. If final grade elevations are significantly different from existing grades, our office should be informed about these changes. If needed and/or if desired, we will reexamine our analyses and make supplemental recommendations.

This report may not reflect the actual variations of the subsurface conditions across the site. The nature and extent of variations across the site may not become evident until construction commences. The construction process itself may also alter subsurface conditions. If variations appear evident at the time of construction, it may be necessary to reevaluate our recommendations after performing on-site observations and tests to establish the engineering impact of the variations.
If the project information described in this report is incorrect, is altered, or if new information is available, we should be retained to review and modify our recommendations.

The scope of our Geotechnical Engineering Study does not include an environmental assessment of the air, soil, rock, or water conditions either on or adjacent to the site. No environmental opinions are presented in this report.

**BORINGS AND LABORATORY TESTS**

We evaluated the subsurface soil conditions at the site by drilling a total of three (3) borings within the proposed pipe alignment at the locations shown on the Boring Location Map, Figure 1. The borings were each drilled to a depth of 15 feet below the existing ground surface using a truck-mounted drilling rig. The boring locations shown in Figure 1 are for illustration purposes, and as such, the locations on the boring location map should only be considered approximate.

Soil samples at conventional sampling depths were collected and transported to our laboratory for classification and testing. During drilling operations, 18 split-spoon samples (with Standard Penetration Test) and 1 grab samples from auger cuttings were collected. Each sample was visually classified in the laboratory by a member of our Geotechnical Engineering staff. The geotechnical engineering properties of the strata were evaluated by the following tests:

<table>
<thead>
<tr>
<th>Type of Test</th>
<th>Number Conducted</th>
</tr>
</thead>
<tbody>
<tr>
<td>Natural Moisture Content</td>
<td>18</td>
</tr>
<tr>
<td>Atterberg Limits (Plasticity Index)</td>
<td>8</td>
</tr>
<tr>
<td>Sieve Analysis</td>
<td>9</td>
</tr>
<tr>
<td>Corrosivity</td>
<td></td>
</tr>
<tr>
<td>pH</td>
<td>1</td>
</tr>
<tr>
<td>Sulfate Content</td>
<td>1</td>
</tr>
<tr>
<td>Chloride Content</td>
<td>1</td>
</tr>
<tr>
<td>Electrical Resistivity</td>
<td>1</td>
</tr>
</tbody>
</table>

The results of the laboratory tests are presented in graphical or numerical form on the boring logs illustrated on Figures 2 through 4. A key to classification terms and symbols used on the logs is presented on Figure 5. The results of the laboratory and field testing are also tabulated on Figure 6 for ease of reference. Results of our sieve analyses are presented on Figures 7 and 8.

Standard penetration test results are noted as “Blows per foot” on the boring logs and Figure 6, where “Blows per foot” refers to the number of blows by a falling 140-lb hammer and split-spoon assembly required for 1 foot of penetration into the soil. Where hard or dense materials were encountered, the tests were terminated at 50 blows even if one foot of penetration had not been achieved.
Samples will be retained in our laboratory for 30 days after submittal of this report. Other arrangements may be provided at the request of the Client.

GENERAL SITE CONDITIONS

SITE DESCRIPTION

The majority of the project site is a 30-foot wide easement located at the westernmost portion of a Fort Bliss residential development, in El Paso, Texas. This Fort Bliss segment is bound by Fred Wilson Avenue to the north, Hayes Avenue to the south, Dyer Street to the west, and residential properties to the east. The soil surface of the site is exposed and is relatively clear of vegetation.

The short northern and southern segmental ends of the alignment are planned to cross street right-of-way to connect to existing water lines. The northern segmental end is planned to cross Fred Wilson Avenue, whereas the southern end is planned to cross one travel lane of Hayes Avenue.

Existing structures include buried utilities (such as sewer, gas, and fiber-optic cable) traversing the aforementioned streets and a pedestrian bridge support near the southern end of the alignment. The topography generally slopes downward toward the southeast with vertical relief of about 5 feet across the site.

GEOLOGY

Regional Geologic Setting

The project site is located on a remnant terrace of the Rio Grande River. Based on our cursory review of the *Geological Map of West Hueco Bolson; El Paso Region, Texas 2000*, the project site is situated on the Undivided Alluvium of drainage ways, young alluvial fans, and young arroyo terraces located along Rio Grande valley border and windblown sand. This geologic setting consists of sands, gravels, silts, and mud and includes undivided young deposits in relatively active settings that may be inset against older deposits. Based upon the physical location of the project site with respect to the Franklin Mountains, anticipated depth to competent bedrock would be on the magnitude of 1,000 feet or more below the existing ground surface.

Site Geology

A review of the *Geologic Map of Texas, Van Horn-El Paso Sheet* indicates the site is naturally underlain with young quaternary colluvium deposits (sloughing material from a sloped hill/mountain surface), consisting of gravels and sandy soils. The site is also naturally underlain with Old Quaternary alluvium (arroyo deposits) and colluvium, consisting of caliche and gypsite on surfaces dissected by modern drainage.
STRATIGRAPHY

The subsurface conditions encountered at the boring locations are shown on the boring logs, Figures 2 through 4. These boring logs represent our interpretation of the subsurface conditions based on the field logs, visual examination of field samples by our personnel, and laboratory test results of selected field samples. Materials encountered in the project borings were classified using the Unified Soil Classification System (USCS). Each material stratum was selected by grouping soils that possess similar physical and engineering characteristics. The lines showing the interfaces between strata on the boring logs represent approximate boundaries. Transitions between strata may be gradual.

In general, we observed soils consisting of dense to very dense sands and gravels with various amounts of clay and silt (SM, SC, GW, GP, GC per USCS, and combinations thereof) in our samples below the ground surface to the boring termination depth of 15 feet.

Soil Properties

Laboratory and field testing were performed on representative samples of the soil strata encountered during our field drilling operations. Laboratory testing primarily focused on the classification and strength characteristics of the subsurface soils.

Water Content – In general, measured moisture contents from samples within our borings ranged from about 2 to 11 percent, indicating relatively dry to slightly moist conditions.

Plasticity Index – The plasticity index (PI) is an established measure of the range of consistency and deformation characteristics of the clay portion that is present in the soil. The plasticity index (PI) is defined as the difference between the liquid limit (LL) and plastic limit (PL) values of the clay. The PL and LL values are also known as Atterberg limits. The PL is typically interpreted as the moisture content below which the soil seems to behave as a semisolid material, whereas the LL is typically interpreted as the moisture content above which the soil seems to behave as a viscous liquid. Generally, soils with a LL greater than 50, or a PI greater than about 20 are subject to significant shrink or swell movements with moisture variations. As required by the ASTM standard, the Atterberg Limits are tested only in the portion of the sampled material finer than the No. 40 sieve (0.425 mm opening size).

The LL values within the sampled soils increased slightly with depth and ranged from 19 to 29. The PI values also increased slightly with depth and ranged from 4 to 12. The measured LL and PI values indicate the clays are of low plasticity. The moisture content and PI value of the clays further revealed the clays were at a moisture content that was well below their PL values. This indicated a relatively low potential for swell and shrinkage due to moisture changes.

Relative Density – Based on SPT blow counts measured during drilling operations, the granular soils at this site were observed to be in medium dense to very dense relative density conditions.
DEPTHTO-WATER

Borings were drilled using dry-auger techniques in an attempt to measure depth-to-water level in the open boreholes. Free water was not observed in the borings either during or immediately upon completion of the drilling operations. The borings remained dry during the field exploration phase.

SOIL CONDITION CONSIDERATIONS

CORROSION POTENTIAL

Many factors can affect the corrosion potential of soil including soil moisture content, resistivity, and pH. The following subsections outline our recommendations regarding soil corrosivity.

Electrical Resistivity

Soil resistivity, which is a measure of how easily electrical current flows through soils, is generally the most influential factor defining the level of soil corrosivity. Based on published literature, the approximate relationship between soil resistivity and soil corrosiveness was developed as shown in the table below.

<table>
<thead>
<tr>
<th>Soil Resistivity (ohm-cm)</th>
<th>Classification of Soil Corrosiveness</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 to 900</td>
<td>Very Severely Corrosive</td>
</tr>
<tr>
<td>900 to 2,300</td>
<td>Severely Corrosive</td>
</tr>
<tr>
<td>2,300 to 5,000</td>
<td>Moderately Corrosive</td>
</tr>
<tr>
<td>5,000 to 10,000</td>
<td>Mildly Corrosive</td>
</tr>
<tr>
<td>10,000 to &gt;100,000</td>
<td>Very Mildly Corrosive</td>
</tr>
</tbody>
</table>

The corrosivity characteristics due to the electrical resistivity of bulk soil samples obtained from our borings from a depth of 5 to 10 feet were evaluated by the Soil-Box test procedure in general accordance with ASTM G-57. When using this method, the resistivity of the bulk soil sample is calculated as:

\[ \rho_e = \frac{RA}{a} \approx 0.92R \]

where,

\[ \rho_e = \text{electrical soil resistivity, ohm-cm}; \]
\[ R = \text{measured resistance, ohms} \]
\[ A = \text{cross-sectional area of soil box (perpendicular to current flow), cm}^2 \text{ (about 11.5 cm}^2 \text{ for standard soil boxes)} \]
\[ a = \text{inner electrode spacing, cm (about 12.5 cm for standard soil boxes)} \]
Since many factors such as moisture content and relative density are known to affect the results of our laboratory soil resistivity tests. The resistivity test results at different moisture contents are hence presented on Figure 9 of this report.

As shown on Figure 9 of the Attachments of this report, soil resistivity results ranged from 110,000 ohm-centimeters (in the sampled moisture condition) to 3,800 ohm-centimeters (in a saturated condition). In our opinion, based on the resistivity results and the resistivity correlations presented, it appears that the corrosion potential to buried metallic improvements may be characterized as moderately to very mildly corrosive. Nevertheless, we recommend that a sacrificial thickness be added to the design thickness required for the buried metallic improvements, which corresponds to the mass presumed to be affected by corrosion at the end of the required service life.

**Chemical Corrosivity**

Chloride and sulfate ion concentration, and pH appear to play secondary roles in affecting corrosion potential. High chloride levels tend to reduce soil resistivity and buried metallic improvements of reinforced concrete structures. Sulfate ions in the soil can be highly aggressive to Portland cement concrete by combining chemically with certain constituents of hydrated cement, principally monosulfoalluminate to form ettringite during the long-term hardening stage of concrete. Ettringite expands upon formation and eventually causes the breakdown of the hydrated cement matrix. Published literature provides requirements for concrete exposed to sulfate containing solutions as summarized in the table below.

<table>
<thead>
<tr>
<th>Water-Soluble Sulfate (SO₄) in soil, ppm</th>
<th>Sulfate Exposure</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 to 1,000</td>
<td>Negligible</td>
</tr>
<tr>
<td>1,000 to 2,000</td>
<td>Moderate*</td>
</tr>
<tr>
<td>2,000 to 20,000</td>
<td>Severe</td>
</tr>
<tr>
<td>Over 20,000</td>
<td>Very Severe</td>
</tr>
</tbody>
</table>

Content levels found in seawater

The chemical corrosivity characteristics of the subsurface soils were evaluated by measuring pH, chloride content and sulfate content. These tests were conducted on bulk soil specimens obtained from our borings between a depth of 5 and 10 feet. The chemical corrosivity test results are summarized on Figure 9 of the Attachments of this Report.

The measured water-soluble sulfates concentration were measured to be 50 ppm for the soil sample tested, indicating negligible exposure. Therefore, Portland Cement Concrete planned to be placed directly in contact with the native soil may be designed for negligible sulfate exposure conditions. Published literature suggests that Type I or II cement may be used.

Acidity is an important factor of soil corrosivity. The lower the pH (the more acidic the environment), the higher the soil corrosivity with respect to buried metallic structures. As soil pH increases above 7 (the
neutral value), the soil is increasingly more alkaline and less corrosive to buried steel structures due to protective surface films which form on steel in high pH environments. A pH between 7 and 8.5 is generally considered relatively passive from a corrosion standpoint. Based on the results of our acidity tests, which yielded a pH value of 8.6, we do not anticipate the need for additional measures to protect buried utilities or structural elements against observed soil pH.

**BURIED UTILITY PIPE RECOMMENDATIONS**

The following sections provide our recommendations with respect to buried pipe design including considerations regarding loads imposed on buried pipe and our recommendations for bedding and backfill. In addition, installation considerations and guidelines are also provided with respect to trench safety, excavation dewatering and equipment. Our recommendations are based, in part, on the assumption that open-cut techniques will be used for installation and that invert depths of no more than 5 feet below the existing ground surface is planned for the alignment.

We have not been provided specific project data with respect to the utility installation (i.e., plans, profiles, pipe types, sizes, etc.). As such, the following recommendations are general in nature and should be reviewed once detailed project information becomes available. The following recommendations are intended to apply to both gravity (or passive) systems and pressurized (force main) systems unless specifically noted.

**GENERAL**

Loads on buried pipes result from a combination of material properties of the pipe and surrounding soils, the methods and techniques used during the installation process (i.e. material used for the haunch, the amount of compactive effort in the backfill materials, etc.), live loads such as roadway traffic, and internal forces due to the transmission of fluids within the pipe. As such, care should be taken to assure design assumptions are validated by review of project specifications prior to construction and appropriate quality control/quality assurance monitoring during construction.

**EARTH LOADS**

**Vertical Soil Pressures**

The weight of the soil over the top of a buried pipe is dependant upon the installation method, the backfill materials, and the degree of compaction achieved during construction. The soil prism method is a common way to describe the weight of the soil directly over the top of a buried pipe. Methods to estimate the weight of the soil prism as defined by accepted industry practices or as recommended by guidelines set forth by the American Society of Civil Engineers (ASCE) or the American Water Works Association (AWWA) may be used for this project. Alternatively, the soil prism load per foot of alignment may be defined as:
\[ W_{sp} = \gamma_s (H + 0.11B_c)B_c \]

where:

- \( W_{sp} \) = soil prism load, lbs/ft
- \( B_c \) = outside diameter of the pipe, ft
- \( H \) = depth of fill over the pipe, ft
- \( \gamma_s \) = total unit weight, pcf (no less than 120 pcf)

**Lateral Soil Pressures**

Equivalent fluid density values for computation of lateral soil pressures acting on pipeline and manhole walls were evaluated for various types of backfill materials in which the manhole structure may be installed. These values, as well as corresponding lateral earth pressure coefficients and estimated unit weights, are presented below. Since the pipeline and manhole walls are expected to act as rigid, non-yielding retaining structures, the “At Rest Condition” should be used.

<table>
<thead>
<tr>
<th>Back Fill Type</th>
<th>Estimated Total Unit Weight (pcf)</th>
<th>At Rest Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Earth Pressure Coefficient, ( k_o )</td>
</tr>
<tr>
<td>Gravels</td>
<td>135</td>
<td>0.45</td>
</tr>
<tr>
<td>Sands</td>
<td>120</td>
<td>0.5</td>
</tr>
<tr>
<td>Clayey Gravels or Sands</td>
<td>135</td>
<td>0.48</td>
</tr>
<tr>
<td>Clays</td>
<td>120</td>
<td>0.74</td>
</tr>
</tbody>
</table>

The values presented above assume the surface of the backfill materials to be level. Sloping the surface of the backfill materials will increase the surcharge load acting on the structures. The above values also do not include the effect of surcharge loads such as construction equipment, vehicular loads, or future storage near the structures. Nor do the values account for possible hydrostatic pressures resulting from groundwater seepage entering and ponding within the backfill materials. However, these surcharge loads and groundwater pressures should be considered in designing any structures subjected to lateral earth pressures.

**VEHICULAR TRAFFIC LOADS**

The Project Civil Engineer should review anticipated traffic loading and frequencies to appropriately account for traffic loading and frequency for buried pipes crossing underneath roadways or driveways (parking lots, drop-off zones or access lanes). We recommend using the simplified load distribution
method suggested in the AASHTO Standard Specifications for Highway Bridges\textsuperscript{1}. That is, AASHTO assumes the stress induced by traffic at the ground surface is uniformly distributed to an area with sides equal to 1\(\frac{3}{4}\) times the depth of fill above the buried pipe. As an example, the single dual wheel HS-20 case (or 16,000-pound load) can be modeled as follows:

\[
\text{LiveLoad} = \frac{P}{(0.83 + 1.75H)(1.67 + 1.75H)}
\]

where:

\[
\begin{align*}
P & = \text{applied load (lbs)} \\
H & = \text{depth of fill above the buried pipe (ft)}
\end{align*}
\]

When the planned pipe crosses an existing or planned roadways (parking lots, drop-off zones or access lanes), we recommend the crossing be designed in accordance with applicable industry standards and construction practices.

If the pipe is planned to be of PVC construction, and when the planned pipeline crosses an existing or planned City roadways, we recommend the crossing be designed in accordance with the \textit{City of El Paso Design Standards for Construction (June 2008)} Standard Detail – Street Paving Cut for Flexible Pipe (Sheet 3-45).

\textbf{PRESSURES ON PRIMARY AND PERMANENT TUNNEL LINERS}

The pipe should be sufficiently strong to withstand anticipated long-term ground loads and not be subject to deterioration by substances either in the ground or in the waterline. In many cases, factors other than soil loading will control the final pipe design. These factors could include loading from pipe jacking, live loads, or minimum structural code thickness. The earth and live loads may be calculated as shown above for buried pipe.

\textbf{PIPING SYSTEM THRUST RESTRAINT}

Changes in fluid direction within pressurized pipelines generate an increase in horizontal stress due to thrust forces along the pipe. This increase in horizontal stress translates into unbalanced forces that need to be resisted by the soil mass, typically by means of a concrete thrust block. Thrust blocks are often installed at pipe bends and or at a number of different pipe fittings such as tees, wyes, reducers, valves, offsets, etc., where unbalanced thrust forces are expected to be significant. The main purpose of the thrust blocks is to transfer the pipe thrust force to the soil structure. Thrust blocks allow for an increase of the area of contact between the soil and the pipe system, distributing the thrust load in a way that will not cause separation of unrestrained joints.

A convenient method to dimension a thrust block is based on the bearing capacity of the soils where the reaction is being generated. We recommend a net allowable bearing capacity of 2,000 psf be used for blocks bearing on natural soils and providing that the integrity of the bearing soils is verified prior to construction of the blocks. This verification should be conducted by the Geotechnical Engineer-of-Record or his qualified representative. If the thrust block is to be installed against disturbed soils, we recommend placing granular materials compacted to a minimum 95 percent of maximum standard Proctor. Additional details in reference to fill materials and placement are presented in the *Bedding and Backfill* section of this report.

**BEDDING AND BACKFILL**

Bedding and backfill recommendations for the proposed pipelines should be in accordance with applicable industry standards and construction practices. Alternatively, trench preparation and pipeline installation methods as recommended by guidelines set forth by ASCE or AWWA may be used for this project.

**Trench Bottom**

The bottoms of trench excavations should expose strong competent soils and should be dry and free of loose, soft, or disturbed soil. If fill soils are encountered at the base of trench excavations, their competency should be verified through probing and density testing. Soft, wet, weak, or deleterious materials should be over-excavated to expose strong competent soils.

At locations where soft or weak soils extend for some depth, overexcavation to stronger soils may prove infeasible and/or uneconomical. In the event of these areas are encountered, we recommend that the bottom of the trench excavation be over-excavated by 1 to 2 feet, and replaced with an open-graded aggregate that will allow for drainage of water, as well as provide a stable working platform. A non-woven geotextile fabric should be placed along the bottom of the overexcavated area before backfilling.

**Bedding**

Bedding is the material used along the bottom of the trench that provides uniform support for the buried pipe. Bedding may be compacted or uncompacted, depending on the recommendations of the design engineer. Bedding that is uncompacted allows the pipe to sink into the bedding soil allowing for a more uniform distribution of stress on the bottom of the pipe. When rock or other unyielding foundation material is encountered, a more compressible material should be used to bed the pipe.

Under installed conditions, the vertical load on a pipe is distributed over its width and the reaction is distributed in accordance with the type of bedding. When the pipe strength used in design has been determined by controlled laboratory testing, a factor must be applied that relates the in-place supporting strength to that obtained in the lab. We recommend the pipe designer use a bedding factor to account for the width of the soil reaction at the bottom of the pipe.
The bedding materials should be selected in accordance to Sections 3-43 or 3-44 of the City of El Paso Design Standards for Construction (June 2008) or applicable industry standards to provide as much of a uniform contact between the pipe and the material as possible. Granular soils such as bank run sand, concrete sand, gem sand, pea gravel, crushed limestone, or cement treated sand may be used as the bedding material. It is essential that bedding materials are placed in loose lift thicknesses of no more than 8 inches and compacted to a minimum of 95 percent of maximum dry density determined by ASTM D 1557. The moisture content of the subgrade should be maintained at or above optimum moisture content until permanently covered.

Soils classified as CH, CL, MH, ML, OH, OL, PT (or any combination thereof) under the Unified Soil Classification System (USCS) shall not be allowed for use as bedding materials.

**Backfill**

We recommend backfill materials and placement be in accordance with applicable industry standards and construction practices. In addition, backfill for trenches should not be started until the pipeline is properly bedded in accordance with the above recommendations. Materials removed from the trench excavations will generally be suitable as backfill above the bedding, provided they are not saturated and do not contain organic matter, debris, or other deleterious material.

Material for backfilling within the pipe zone (typically, defined as the depth extending from the bedding materials to 6 or 12 inches above the pipe) shall be equivalent to those used for bedding. Trench excavation materials may be re-used as backfill above the pipe zone provided they are free of deleterious material, organic matter, cobbles or boulders over ¾ inches in nominal size.

**Soil Backfill**

Backfill materials shall have a maximum liquid limit not exceeding 40, a plasticity index less than 15, and a maximum particle size not exceeding 4 inches or one-half the loose lift thickness, whichever is smaller. In addition, grain size analyses and Atterberg Limits must be performed during placement at a rate of one test each per 5,000 cubic yards of material. Soils classified as GW, GP, GM, GC, SW, SP, SM, SC, CL, ML (or combinations thereof) under the Unified Soil Classification System (USCS) are considered to be acceptable for use as backfill. The soils encountered at this site are considered suitable for use as backfill.

Alternative, pit-run or imported backfill materials shall have a maximum liquid limit not exceeding 40, a plasticity index between 7 and 20, and a maximum particle size not exceeding 4 inches or one-half the loose lift thickness, whichever is smaller. In addition, if these materials are utilized, grain size analyses and Atterberg Limits must be performed during placement at a rate of one test each per 5,000 cubic yards of material due to the high degree of variability associated with pit-run materials.

It should be noted that when using fill materials that are classified as CL, or ML under the USCS, difficulties may be experienced with respect to moisture control and mixture homogeneity during and subsequent to
fill placement, as well as with erosion, particularly when exposed to inclement weather. This may result in pumping of the fill materials.

To reduce potential settlements of the ground surface resulting from consolidation of the trench backfill, we recommend that trench backfill be placed in 8-inch thick loose lifts and compacted to at least 95 percent of the maximum dry density as determined by ASTM D698. The moisture content of the fill should be maintained within the range of 3 percentage points below to 3 percentage points above the optimum moisture content until final compaction.

Soil Cement

In segments of the alignment that will be installed using open trench techniques across city right-of-way, backfill material may be selected and placed in accordance with Sections 3-43 or 3-44 of the City of El Paso Design Standards for Construction (June 2008).

ROADWAY CROSSINGS

When a planned flexible pipe crosses an existing or planned city right-of-way, we recommend the crossing be designed in accordance with the City of El Paso Design Standards for Construction (June 2008) Standard Detail – Street Paving Cut for Flexible Pipe (Sheet 3-45), otherwise, we recommend applicable industry standards be used.

BURIED PIPE CONSTRUCTION CONSIDERATIONS

TRENCH SAFETY

As previously mentioned, if utility trenches extend to or below a depth of 5 feet below construction grade, the contractor or others shall be required to develop a trench safety plan to protect personnel entering the trench or trench vicinity. The collection of specific geotechnical data and the development of such a plan, which could include designs for sloping and benching or various types of temporary shoring is beyond the scope of this study. Any such designs and safety plans shall be developed in accordance with current OSHA guidelines and other applicable industry standards.

In sections of the utility alignment where it may be feasible or desired to slope the sides of the trench excavation, the sloping of the excavation should follow our recommendations in the Excavation Sloping and Safety section of this report.

Shielded Trench Guidelines

If the utility installation contractor chooses to use shielded trenching methods, care should be taken so that the loads subjected to the shields do not exceed the shield Manufacturer’s recommended values. We
recommend a safety factor of at least 2 be used when determining the size (or design strength) of the required shields.

Shields should be installed in a manner that resists lateral displacements and/or other hazardous movements of the shield in the event of sudden changes in lateral loads, such as sidewall collapse, or impact from excavation equipment or any other potential force.

Employees shall not be allowed in shielded trenches when shields are being installed, removed, or moved vertically or horizontally. Employees should not be permitted in trenches, which show possible loss of soil from behind or below the bottom of the shield. Excavations of earth material to a level not greater than 2 feet below the bottom of a shield shall be permitted. Shields should extend to a minimum of 18 inches above the top of the vertical side or crest of the excavation.

The trench box system should be used in accordance with the Manufacturer’s recommendations in accordance with the requirements of a trench safety plan and relevant OSHA regulations. Under no circumstances should excavations be permitted to remain open overnight.

**Bracing Pressures**

In order to properly design the supports for shielding workers within an excavation, the type of shoring planned for use, as well as the geometry (i.e., the vertical spacing of struts) must be known. We recommend the Structural Engineer use a method similar to the Department of the Navy’s method\(^2\) for determining the pressure distribution for internally braced flexible walls.

Alternatively, lateral soil pressures acting on braced trench walls may be determined as a uniform lateral load as follows:

\[
\sigma_h = k_a \gamma H
\]

where:
- \(\sigma_h\) = applied lateral soil pressure (psf)
- \(k_a\) = active-condition pressure coefficient as shown in the table below
- \(\gamma\) = total unit weight (pcf) as shown in the table below
- \(H\) = depth of trench (ft)

<table>
<thead>
<tr>
<th>Back Fill Type</th>
<th>Estimated Total Unit Weight (pcf)</th>
<th>Active Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gravel</td>
<td>135</td>
<td>0.29</td>
</tr>
<tr>
<td>Sands</td>
<td>120</td>
<td>0.31</td>
</tr>
</tbody>
</table>

The values presented above are unfactored values and therefore should be increased with a minimum factor of safety of 1.2 for the design of bracing structures. In sands, the factored lateral pressure may be reduced with a factor of 0.65 (by 35 percent) in accordance with accepted industry practice.

The values do not include the effect of surcharge loads such as construction equipment, vehicular loads, or future storage near the structures, nor do the values account for possible hydrostatic pressures resulting from groundwater seepage entering and ponding within the cut soils. However, these surcharge loads and groundwater pressures should be considered, if applicable, in designing any structures subjected to lateral earth pressures.

EXCAVATION EQUIPMENT

Our exploratory boring logs are not intended for use in determining construction means and methods and may therefore be misleading if used for that purpose. We recommend that earth-work and utility contractors interested in bidding on the work perform their own tests to determine the quantities of the different materials to be excavated, as well as the preferred excavation methods and equipment for this site.

CONSTRUCTION RELATED SERVICES

CONSTRUCTION MATERIALS TESTING AND OBSERVATION SERVICES

As presented in the attachment to this report, Important Information About Your Geotechnical Engineering Report, subsurface conditions can vary across a project site. The conditions described in this report are based on interpolations derived from a limited number of data points. Variations will be encountered during construction, and only the geotechnical design engineer will be able to determine if these conditions are different than those assumed for design.

Construction problems resulting from variations or anomalies in subsurface conditions are among the most prevalent on construction projects and often lead to delays, changes, cost overruns, and disputes. These variations and anomalies can best be addressed if the geotechnical engineer of record, RABA KISTNER, is retained to perform construction observation and testing services during the construction of the project. This is because:

- RKCI has an intimate understanding of the geotechnical engineering report’s findings and recommendations. RKCI understands how the report should be interpreted and can provide such interpretations on site, on the client’s behalf.
- RKCI knows what subsurface conditions are anticipated at the site.
- RKCI is familiar with the goals of the owner and project design professionals, having worked with them in the development of the geotechnical scope of work. This enables RKCI to suggest remedial measures (when needed) which help meet the owner’s and the design teams’ requirements.
RKCI has a vested interest in client satisfaction, and thus assigns qualified personnel whose principal concern is client satisfaction. This concern is exhibited by the manner in which contractors’ work is tested, evaluated and reported, and in selection of alternative approaches when such may become necessary.

RKCI cannot be held accountable for problems which result due to misinterpretation of our findings or recommendations when we are not on hand to provide the interpretation which is required.

BUDGETING FOR CONSTRUCTION TESTING

Appropriate budgets need to be developed for the required construction testing and observation activities. At the appropriate time before construction, we advise that RKCI and the project designers meet and jointly develop the testing budgets, as well as review the testing specifications as it pertains to this project.

Once the construction testing budget and scope of work are finalized, we encourage a preconstruction meeting with the selected contractor to review the scope of work to make sure it is consistent with the construction means and methods proposed by the contractor. RKCI looks forward to the opportunity to provide continued support on this project, and would welcome the opportunity to meet with the Project Team to develop both a scope and budget for these services.

* * * * * * * * * *

The following figures are attached and complete this report:

- Figure 1: Boring Location Map
- Figures 2 through 4: Logs of Borings
- Figure 5: Key to Terms and Symbols
- Figure 6: Results of Soil Sample Analyses
- Figures 7 and 8: Grain Size Analyses
- Figure 9: Soil Corrosivity Testing Results
- A.S.F.E. Insert: Important Information About Your Geotechnical Engineering Report
ATTACHMENTS
**LOG OF BORING NO. B-1**

Partello 30-inch Water Main Phase II
Fred Wilson Avenue to Hayes Avenue
El Paso, Texas

**LOCATION:** N 10679792.00; E 399740.00

**DRILLING METHOD:** CME 75, 8" (O.D.) H.S. Auger

**SURFACE ELEVATION:** 3940 ft

---

### DESCRIPTION OF MATERIAL

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Symbol</th>
<th>Samples</th>
<th>Description of Material</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td></td>
<td></td>
<td>SAND, Silty-Clayey with Gravel (SM), Very Dense, Light Brown, Slightly Moist</td>
</tr>
<tr>
<td>5.0</td>
<td></td>
<td></td>
<td>GRAVEL, Well-Graded (GW-GC), Very Dense, Light Brown, Slightly Moist, with clay and sand</td>
</tr>
<tr>
<td>10.0</td>
<td></td>
<td></td>
<td>GRAVEL, Clayey (GC), Very Dense, Light Brown, Dry, with sand</td>
</tr>
<tr>
<td>15.0</td>
<td></td>
<td></td>
<td>Boring Terminated</td>
</tr>
</tbody>
</table>

---

### BLOW COUNTS

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Ref/ 3&quot;</th>
<th>Ref/ 6&quot;</th>
<th>Ref/ 9&quot;</th>
<th>Ref/ 12&quot;</th>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>15.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

---

### NOTES:

1. Free water was not observed during or immediately after our drilling operations.
2. Backfilled with soil cuttings.
3. Boring location coordinates were estimated with the use of the software program Corpscon 6.0.1 published by the US Corps of Engineers; format is NAD 83 Texas State Plane Coordinate System, Central Texas Zone 4203 (US Survey feet).
4. Surface elevations are only approximate and were estimated using available topographical maps published by the US Geological Survey.

---

**DEPTH DRILLED:** 15.0 ft
**DATE DRILLED:** 8/22/2014
**DEPTH TO WATER:** Dry
**DATE MEASURED:** 8/22/2014
**PROJ. No.:** AEA14-018-00
**FIGURE:** 2

---

**SHEAR STRENGTH, TONS/FT**

<table>
<thead>
<tr>
<th>Plastic Limit</th>
<th>Water Content</th>
<th>Liquid Limit</th>
<th>Plastic Index</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5</td>
<td>1.0</td>
<td>1.5</td>
<td>2.0</td>
</tr>
</tbody>
</table>

**UNIT DRY WEIGHT,pcf**

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>10</th>
<th>20</th>
<th>30</th>
<th>40</th>
<th>50</th>
<th>60</th>
<th>70</th>
<th>80</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
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<td>15.0</td>
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<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

---

**NOTE:** THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT.
**LOG OF BORING NO. B-2**

Partello 30-inch Water Main Phase II
Fred Wilson Avenue to Hayes Avenue
El Paso, Texas

**DRILLING METHOD:** CME 75, 8" (O.D.) H.S. Auger

**LOCATION:** N 10679151.00; E 399440.00

**DESCRIPTION OF MATERIAL**

**SURFACE ELEVATION:** 3940 ft

**GRAVEL,** Poorly Graded with Silt (GP-GM), Dense, Dark Brown, Slightly Moist

**GRAVEL,** Clayey (GC), Very Dense, Tan, Slightly Moist

**SAND,** Silty-Clayey (SC-SM), Very Dense, Grayish-Brown, Dry, with gravel

**GRAVEL,** Well Graded (GW), Very Dense, Grayish-Brown, Dry, with sand

**Boring Terminated**

**NOTES:**
1. Free water was not observed during or immediately after our drilling operations.
2. Backfilled with soil cuttings.
3. Boring location coordinates were estimated with the use of the software program Corpscon 6.0.1 published by the US Corps of Engineers; format is NAD 83 Texas State Plane Coordinate System, Central Texas Zone 4203 (US Survey feet).
4. Surface elevations are only approximate and were estimated using available topographical maps published by the US Geological Survey.

**SHEAR STRENGTH, TONS/FT**

<table>
<thead>
<tr>
<th>DEPTH FT</th>
<th>SYMBOL</th>
<th>SAMPLES</th>
<th>UNIT DRY WEIGHT, pcf</th>
<th>PLASTICITY INDEX</th>
<th>PLASTIC LIMIT</th>
<th>LIQUID LIMIT</th>
<th>WATER CONTENT</th>
<th>BLOWS PER FT</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5</td>
<td>-200</td>
<td></td>
<td>10</td>
<td>36</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.0</td>
<td>-200</td>
<td></td>
<td>20</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.5</td>
<td>-200</td>
<td></td>
<td>30</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.0</td>
<td>-200</td>
<td></td>
<td>40</td>
<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td>2.5</td>
<td>-200</td>
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<td>50</td>
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<td></td>
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<td></td>
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<tr>
<td>3.0</td>
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<td>60</td>
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<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>3.5</td>
<td>-200</td>
<td></td>
<td>70</td>
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<td></td>
<td></td>
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</tr>
<tr>
<td>4.0</td>
<td>-200</td>
<td></td>
<td>80</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**SYMBOL**

- **Ref/5"**
- **Ref/4"**
- **Ref/3"**
- **50/2"**

**DEPTCH DRILLED:** 15.0 ft

**DATE DRILLED:** 8/22/2014

**DEPTH TO WATER:** Dry

**DATE MEASURED:** 8/22/2014

**PROJ. No.:** AEA14-018-00

**FIGURE:** 3

**NOTE:** THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT
LOG OF BORING NO. B-3
Partello 30-inch Water Main Phase II
Fred Wilson Avenue to Hayes Avenue
El Paso, Texas

DRILLING METHOD: CME 75, 8" (O.D.) H.S. Auger

LOCATION: N 10678507.00; E 399481.00

DESCRIPTION OF MATERIAL

<table>
<thead>
<tr>
<th>SYMBOL</th>
<th>SAMPLES</th>
<th>BLOWSDRFTFT</th>
<th>PLASTICITY INDEX</th>
<th>SURFACE ELEVATION: 3940 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>24</td>
<td>-200</td>
<td>SAND, Silty with Gravel (SM), Medium Dense, Light Brown, Slightly Moist</td>
</tr>
<tr>
<td></td>
<td></td>
<td>41</td>
<td>4 13</td>
<td>GRAVEL, Silty-Clayey (GC), Dense to Very Dense, Light Brown, Slightly Moist, with sand</td>
</tr>
<tr>
<td></td>
<td></td>
<td>50/3&quot;</td>
<td>4 14</td>
<td>- with trace calcareous material</td>
</tr>
<tr>
<td></td>
<td>Ref/5&quot;</td>
<td></td>
<td></td>
<td>Boring Terminated</td>
</tr>
<tr>
<td></td>
<td>Ref/6&quot;</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

NOTES:
1. Free water was not observed during or immediately after our drilling operations.
2. Backfilled with soil cuttings.
3. Boring location coordinates were estimated with the use of the software program Corpscon 6.0.1 published by the US Corps of Engineers; format is NAD 83 Texas State Plane Coordinate System, Central Texas Zone 4203 (US Survey feet).
4. Surface elevations are only approximate and were estimated using available topographical maps published by the US Geological Survey.

DEPTH DRILLED: 15.0 ft  DEPTH TO WATER: Dry  PROJ. No.: AEA14-018-00
DATE DRILLED: 8/22/2014  DATE MEASURED: 8/22/2014  FIGURE: 4
### Key to Terms and Symbols

#### Material Types

**Soil Terms**
- Calcareous
- Caliche
- Clay
- Clayey
- Gravel
- Gravelly

**Rock Terms**
- Chalk
- Claystone
- Clay-shale
- Conglomerate
- Dolomite
- Igneous
- Limestone
- Metamorphic
- Sandstone
- Shale
- Siltstone

**Other**
- Asphalt
- Base
- Concrete/Cement
- Bricks/Pavers
- Waste
- No Information

#### Well Construction and Plugging Materials

- Blank Pipe
- Bentonite
- Cuttings
- Concrete/Cement
- Gravel
- Sand
- Volclay

#### Sample Types

- Air Rotary
- Mud Rotary
- No Recovery
- Nx Core
- Geoprobe Sampler
- Rotosonic Damaged
- Rotosonic Intact
- Shelby Tube
- Split Barrel
- Split Spoon
- Texas Cone Penetrometer
- Disturbed

#### Strength Test Types

- Pocket Penetrometer
- Torvane
- Unconfined Compression
- Triaxial Compression Consolidated-Undrained
- Triaxial Compression Unconsolidated-Undrained

**Note:** Values symbolized on boring logs represent shear strengths unless otherwise noted.

---

**Project No. AEA14-018-00**

**Figure 5a**

**Revised 08/2011**
TERMINOLOGY

Terms used in this report to describe soils with regard to their consistency or conditions are in general accordance with the discussion presented in Article 45 of SOILS MECHANICS IN ENGINEERING PRACTICE, Terzaghi and Peck, John Wiley & Sons, Inc., 1967, using the most reliable information available from the field and laboratory investigations. Terms used for describing soils according to their texture or grain size distribution are in accordance with the UNIFIED SOIL CLASSIFICATION SYSTEM, as described in American Society for Testing and Materials D2487-06 and D2488-00, Volume 04.08, Soil and Rock; Dimension Stone; Geosynthetics; 2005.

The depths shown on the boring logs are not exact, and have been estimated to the nearest half-foot. Depth measurements may be presented in a manner that implies greater precision in depth measurement, i.e 6.71 meters. The reader should understand and interpret this information only within the stated half-foot tolerance on depth measurements.

<table>
<thead>
<tr>
<th>Penetration Resistance Blows per ft</th>
<th>Relative Density</th>
<th>Resistance Blows per ft</th>
<th>Consistency</th>
<th>Cohesion TSF</th>
<th>Plasticity Index</th>
<th>Degree of Plasticity</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 4</td>
<td>Very Loose</td>
<td>0 - 2</td>
<td>Very Soft</td>
<td>0.125</td>
<td>0 - 5</td>
<td>None</td>
</tr>
<tr>
<td>4 - 10</td>
<td>Loose</td>
<td>2 - 4</td>
<td>Soft</td>
<td>0.125 - 0.25</td>
<td>5 - 10</td>
<td>Low</td>
</tr>
<tr>
<td>10 - 30</td>
<td>Medium Dense</td>
<td>4 - 8</td>
<td>Firm</td>
<td>0.25 - 0.5</td>
<td>10 - 20</td>
<td>Moderate</td>
</tr>
<tr>
<td>30 - 50</td>
<td>Dense</td>
<td>8 - 15</td>
<td>Stiff</td>
<td>0.5 - 1.0</td>
<td>20 - 40</td>
<td>Plastic</td>
</tr>
<tr>
<td>&gt; 50</td>
<td>Very Dense</td>
<td>15 - 30</td>
<td>Very Stiff</td>
<td>1.0 - 2.0</td>
<td>&gt; 40</td>
<td>Highly Plastic</td>
</tr>
<tr>
<td></td>
<td></td>
<td>&gt; 30</td>
<td>Hard</td>
<td>&gt; 2.0</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

ABBREVIATIONS

B = Benzene    Qam, Qas, Qal = Quaternary Alluvium    Kef = Eagle Ford Shale
T = Toluene    Qat = Low Terrace Deposits    Kbu = Buda Limestone
E = Ethylbenzene    Qbc = Beaumont Formation    Kdr = Del Rio Clay
X = Total Xylenes    Qt = Fluviatile Terrace Deposits    Kft = Fort Terrett Member
BTEX = Total BTEX    Qao = Seymour Formation    Kgt = Georgetown Formation
TPH = Total Petroleum Hydrocarbons    Qle = Leona Formation    Kep = Person Formation
ND = Not Detected    Q:Tu = Uvalde Gravel    Kek = Kainer Formation
NA = Not Analyzed    Ewi = Wilcox Formation    Kes = Escondido Formation
NR = Not Recorded/No Recovery    Emi = Midway Group    Kew = Walnut Formation
OVA = Organic Vapor Analyzer    Mc = Catahoula Formation    Kgr = Glen Rose Formation
ppm = Parts Per Million    EI = Laredo Formation    Kgrl = Lower Glen Rose Formation
Kknm = Navarro Group and Marlbrook Marl    Kgru = Upper Glen Rose Formation
Kpg = Pecan Gap Chalk    Kh = Hensell Sand
Kau = Austin Chalk
KEY TO TERMS AND SYMBOLS (CONT'D)

TERMINOLOGY

SOIL STRUCTURE

Slickensided  Having planes of weakness that appear slick and glossy.
Fissured  Containing shrinkage or relief cracks, often filled with fine sand or silt; usually more or less vertical.
Pocket  Inclusion of material of different texture that is smaller than the diameter of the sample.
Parting  Inclusion less than 1/8 inch thick extending through the sample.
Seam  Inclusion 1/8 inch to 3 inches thick extending through the sample.
Layer  Inclusion greater than 3 inches thick extending through the sample.
Laminated  Soil sample composed of alternating partings or seams of different soil type.
Interlayered  Soil sample composed of alternating layers of different soil type.
Intermixed  Soil sample composed of pockets of different soil type and layered or laminated structure is not evident.
Calcareous  Having appreciable quantities of carbonate.
Carbonate  Having more than 50% carbonate content.

SAMPLING METHODS

RELATIVELY UNDISTURBED SAMPLING

Cohesive soil samples are to be collected using three-inch thin-walled tubes in general accordance with the Standard Practice for Thin-Walled Tube Sampling of Soils (ASTM D1587) and granular soil samples are to be collected using two-inch split-barrel samplers in general accordance with the Standard Method for Penetration Test and Split-Barrel Sampling of Soils (ASTM D1586). Cohesive soil samples may be extruded on-site when appropriate handling and storage techniques maintain sample integrity and moisture content.

STANDARD PENETRATION TEST (SPT)

A 2-in.-OD, 1-3/8-in.-ID split spoon sampler is driven 1.5 ft into undisturbed soil with a 140-pound hammer free falling 30 in. After the sampler is seated 6 in. into undisturbed soil, the number of blows required to drive the sampler the last 12 in. is the Standard Penetration Resistance or "N" value, which is recorded as blows per foot as described below.

<table>
<thead>
<tr>
<th>Blows Per Foot</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>25</td>
<td>25 blows drove sampler 12 inches, after initial 6 inches of seating.</td>
</tr>
<tr>
<td>50/7&quot;</td>
<td>50 blows drove sampler 7 inches, after initial 6 inches of seating.</td>
</tr>
<tr>
<td>Ref/3&quot;</td>
<td>50 blows drove sampler 3 inches during initial 6-inch seating interval.</td>
</tr>
</tbody>
</table>

**NOTE:** To avoid damage to sampling tools, driving is limited to 50 blows during or after seating interval.
## RESULTS OF SOIL SAMPLE ANALYSES

**PROJECT NAME:** Partello 30-inch Water Main Phase II  
Fred Wilson Avenue to Hayes Avenue  
El Paso, Texas

**FILE NAME:** AEA14-018-00 PARTELLO 30-INCH WATER MAIN PHASE II.GPJ  
9/26/2014

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>Sample Depth (ft)</th>
<th>Blows per ft</th>
<th>Water Content (%)</th>
<th>Liquid Limit</th>
<th>Plastic Limit</th>
<th>Plasticity Index</th>
<th>USCS</th>
<th>Dry Unit Weight (pcf)</th>
<th>% -200 Sieve</th>
<th>Shear Strength (tsf)</th>
<th>Strength Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1</td>
<td>0.0 to 1.5</td>
<td>Ref/ 3&quot;</td>
<td>9</td>
<td>21</td>
<td>15</td>
<td>6</td>
<td>SC-SM</td>
<td></td>
<td>13</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>2.5 to 4.0</td>
<td>50/ 6&quot;</td>
<td>2</td>
<td>2</td>
<td>16</td>
<td>8</td>
<td>GW-GC</td>
<td></td>
<td>11</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>5.0 to 6.5</td>
<td>60</td>
<td>2</td>
<td>24</td>
<td>16</td>
<td>8</td>
<td>GW-GC</td>
<td></td>
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**PP** = Pocket Penetrometer  
**TV** = Torvane  
**UC** = Unconfined Compression  
**FV** = Field Vane  
**UU** = Unconsolidated Undrained Triaxial  
**CU** = Consolidated Undrained Triaxial  
**PROJECT NO.** AEA14-018-00  
**FIGURE 6**
Work Order No.: 1413304  
Client: RKCI  
Project No.: AEA 14-018-00  
Project Name: Partello 30" Water Main  
Report Date: October 27, 2014

Laboratory Test(s) Results Summary

The subject soil sample was processed with the U.S. Standard No. 10 Sieve and tested for pH (ASTM G 51-95 2005), Soil Resistivity (ASTM G 57-06), Sulfate Ion Content (ASTM D 516-07) and Chloride Ion Content (ASTM D 512-10). The test results follow:

<table>
<thead>
<tr>
<th>Sample Identification</th>
<th>pH</th>
<th>As Rec'd Resistivity (ohm-cm)</th>
<th>Saturated Resistivity (ohm-cm)</th>
<th>Sulfate Content (mg/L)</th>
<th>Chloride Content (mg/L)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1 thru B-3 Composite @ 5-10'</td>
<td>8.6</td>
<td>110,000</td>
<td>3,800</td>
<td>50</td>
<td>ND</td>
</tr>
</tbody>
</table>

*ND=No Detection

We appreciate the opportunity to serve you. Please do not hesitate to contact us with any questions or clarifications regarding these results or procedures.

Ahmet K. Kaya, Laboratory Manager
Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

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

Read the Full Report

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

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

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

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

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

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

As a general rule, always inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.

Subsurface Conditions Can Change

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

Most Geotechnical Findings Are Professional Opinions

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

A Report’s Recommendations Are Not Final

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

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members’ misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team’s plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer’s Logs

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

Give Contractors a Complete Report and Guidance

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

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled “limitations” many of these provisions indicate where geotechnical engineers’ responsibilities begin and end, to help others recognize their own responsibilities and risks. Read these provisions closely. Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

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

Obtain Professional Assistance To Deal with Mold

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

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

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you ASFE-member geotechnical engineer for more information.
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