Geotechnical Engineering Report

SS Effluent Line and Headwall Structure
South Laredo WWTP Expansion Project
Laredo, Texas
October 29, 2013
Terracon Project No. 89135033

Prepared for:
City of Laredo – Utilities Department
Laredo, Texas

Prepared by:
Terracon Consultants, Inc.
Laredo, Texas
October 29, 2013

City of Laredo – Utilities Department
5816 Daugherty Avenue
Laredo, Texas 78041

Attn: Mr. Tomas M. Rodriguez, Jr. P.E.
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Re: Geotechnical Engineering Report
SS Effluent Line and Headwall Structure
South Laredo WWTP Expansion Project
Laredo, Texas
Terracon Project No. 89135033

Dear Mr. Rodriguez:

Terracon Consultants, Inc. (Terracon) is pleased to submit this Geotechnical Engineering Report for the proposed Sanitary Sewer Effluent Line and Headwall Structure located within the South Laredo Wastewater Treatment Plant facility in Laredo, Texas. We trust that this report is responsive to your project needs. Please contact us if you have any questions or if we can be of further assistance.

We appreciate the opportunity to work with you on this project and look forward to providing Construction Materials Testing services in the future.

Sincerely,
Terracon Consultants, Inc.

Martin Reyes, E.I.T.
Senior Staff Engineer
Geotechnical Engineering Division

APR reviewed by Mike T. Ghazawi, P.E. – 89135033

Copies To: Addressee: (2) Bound & (1) Electronic
CDM Smith: Ms. Yue Sun, P.E.: suny@cdmsmith.com; (1) Electronic

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Geotechnical · Environmental · Construction Materials · Facilities
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EXECUTIVE SUMMARY

This summary should be used in conjunction with the entire report for design purposes. It should be recognized that details were not included or fully developed in this section, and the report must be read in its entirety for a comprehensive understanding of the items contained herein. The section titled GENERAL COMMENTS should be read for an understanding of the report limitations.

A geotechnical investigation has been performed for the proposed Sanitary Sewer Effluent Line and Headwall Structure located within the South Laredo Wastewater Treatment Plant facility in Laredo, Texas. As requested, a total of 2 borings were drilled to depths of approximately 25 to 50 feet below the existing grade within the proposed sewer line alignment. Initially, the 50-foot boring was proposed to be excavated to a depth of 70 feet; however, gravel stratum between 25 and 33 feet (El. 342.0 and El. 332.0 feet) was encountered, thus causing the borehole to cave in. Additionally, very hard Claystone was encountered below depth of 33.5 feet (El. 332.0 feet) and the borehole could not be advanced below 50 feet (El. 315.5 feet) without using special equipment designed to drill through rock.

Based on the information obtained from our subsurface exploration the following geotechnical considerations were identified:

- Groundwater was observed at this site between depths of about 24 and 28 feet (El. 341.5 and El. 337.5 feet) below existing grade during and upon completion of the drilling operations.

- The proposed structures may be supported on shallow or deep foundation systems bearing on the on-site soils.

- Based on the 2012 International Building Code (IBC) seismic site classification for this site is D.
1.0 INTRODUCTION

Terracon is pleased to submit this Geotechnical Engineering Report for the proposed Sanitary Sewer Effluent Line and Headwall Structure located within the South Laredo Wastewater Treatment Plant facility in Laredo, Texas. This project was authorized by Mr. Tomas M. Rodriguez, Jr. P.E. of City of Laredo – Utilities Department through signature of our client’s “Agreement for Services” on September 16, 2013.

The purpose of this report is to describe the subsurface conditions observed at the boring locations drilled for this study, analyze and evaluate the test data, and provide recommendations with respect to:

- Subsurface soil conditions
- Groundwater conditions
- Earthwork recommendations
- Foundation design and construction
- Allowable bearing capacity
- Lateral earth pressures
- Seismic site considerations
- Utility Construction Considerations

2.0 PROJECT INFORMATION

2.1 Project Description

<table>
<thead>
<tr>
<th>Item</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Site Layout</td>
<td>See Appendix A, Exhibit A-3, Bore Location Plan</td>
</tr>
<tr>
<td>Structures</td>
<td>The project will include installation of 48-inch diameter sanitary sewer effluent line and construction of its headwall structure at the discharging point. The headwall structure will be constructed adjacent to the Rio Grande River.</td>
</tr>
<tr>
<td>Construction Type</td>
<td>The new headwall structure will consist of concrete and riprap supported by shallow or deep foundation system. The 48-inch diameter sanitary sewer effluent line will be fiberglass pipe.</td>
</tr>
<tr>
<td>Finished Construction Elevations</td>
<td>The headwall structure will be at or near existing grades and pipeline invert will be at about 6 to 10 feet below existing grades.</td>
</tr>
</tbody>
</table>
2.2 Site Location and Description

<table>
<thead>
<tr>
<th>Item</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Location</td>
<td>This project site is located within South Laredo Wastewater Treatment Plant (WWTP) facility in Laredo, Texas.</td>
</tr>
<tr>
<td>Existing Improvements</td>
<td>The South Laredo WWTP is a typical facility comprised of clarifiers, trickling filters, aeration basins, contact basin, sludge wet well, chemical feed, and discharge, storage and laboratory buildings are also located within the WWTP facility.</td>
</tr>
<tr>
<td>Current Ground Cover</td>
<td>Native grass, trees, brush, reed plants, bare soils and dirt roadways.</td>
</tr>
<tr>
<td>Existing Topography</td>
<td>Based on information provided by CDM Smith the site is sloping downward from east to west with abrupt changes in elevations near the river. The difference in elevation from the proposed boring locations surface to river water level is approximately 26 feet.</td>
</tr>
</tbody>
</table>

3.0 SUBSURFACE CONDITIONS

3.1 Geology
The Geologic Atlas of Texas, Laredo Sheet, 1976, published by the University of Texas at Austin, Bureau of Economic Geology, has mapped the Fluviatile Terrace Deposits (Qt) of the Pleistocene geologic age in the vicinity of the site. The Fluviatile Terrace Deposits (Qt) consists of gravel, sand, silt and clay; composed of materials similar to those present in contiguous alluvium.

3.2 Typical Subsurface Profile
Specific conditions encountered at the boring locations are indicated on the individual boring logs. Stratification boundaries on the boring logs represent the approximate location of changes in soil types; in-situ, the transition between materials may be gradual. Detailed boring logs, presenting the stratum descriptions, types of sampling used, and additional field data, are included in Appendix A. Based on the results of the borings, subsurface conditions on the project site can be generalized as follows:

<table>
<thead>
<tr>
<th>Approximate Depth of Stratum, feet</th>
<th>Material Encountered</th>
<th>Consistency/Density</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 to 4.5</td>
<td>SILTY CLAYEY SAND (^1); light brown</td>
<td>Loose</td>
</tr>
<tr>
<td>0 to 25</td>
<td>LEAN CLAY (^2); light brown</td>
<td>Stiff to hard</td>
</tr>
<tr>
<td>4.5 to 23.5</td>
<td>SILTY SAND (^2); light brown</td>
<td>Loose to medium dense</td>
</tr>
<tr>
<td>23.5 to 33.5</td>
<td>SILTY SAND WITH GRAVEL (^1); light brown</td>
<td>Medium dense</td>
</tr>
<tr>
<td>33.5 to 50</td>
<td>CLAYSTONE (^3); bluish gray</td>
<td>Hard</td>
</tr>
</tbody>
</table>
Approximate Depth of Stratum, feet | Material Encountered | Consistency/Density
--- | --- | ---
1. | SILTY CLAYEY SAND (SC-SM), SAND (SM) and SILTY SAND WITH GRAVEL (SM) materials could undergo low volumetric changes (shrink/swell) should they experience changes in their in-place moisture content. These materials are considered volumetrically stable with regards to change in moisture content due to their granular nature may transmit water easily during rainfall seasons.

2. | The LEAN CLAY (CL) materials could undergo low volumetric changes (shrink/swell) should they experience changes in their in-place moisture content.

3. | CLAYSTONE is expected to undergo low to moderate volumetric changes with fluctuations in its moisture content. However, when being excavated or drilled this material typically behaves more like a rock than soil thereby requiring construction equipment and procedures typically used for rock.

### 3.3 Groundwater

The boreholes were observed while drilling and after completion for the presence and level of groundwater. The water levels observed in the boreholes are noted on the attached boring logs, and are summarized below:

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>Depth to groundwater while drilling, ft. (^1) (El.)</th>
<th>Depth to groundwater after 15 minutes while drilling, ft. (^1) (El.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1</td>
<td>28 (337.5)</td>
<td>24 (341.5)</td>
</tr>
<tr>
<td>B-2</td>
<td>---</td>
<td>---</td>
</tr>
</tbody>
</table>

\(^1\) Depths measured from existing ground surface at time of measurement to intervals of 15 minutes. Groundwater levels have been rounded to the nearest 1/2 foot.

Groundwater was not observed in boring B-2 while drilling, or for the short duration that the boring was allowed to remain open. However, this does not necessarily mean this boring terminated above groundwater, or that the water levels summarized above are stable groundwater levels. Long term observations in piezometers or observation wells sealed from the influence of groundwater are often required to define groundwater levels.

Groundwater level fluctuations occur due to seasonal variations in the amount of rainfall, runoff and other factors not evident at the time the borings were performed. Therefore, groundwater levels during construction or at other times in the life of the structure may be higher or lower than the levels indicated on the boring logs. The possibility of groundwater level fluctuations should be considered when developing the design and construction plans for the project.
4.0 RECOMMENDATIONS FOR DESIGN AND CONSTRUCTION.

The following recommendations are based upon the data obtained from our field and laboratory programs, project information provided to us and on our experience with similar subsurface and site conditions.

4.1 Geotechnical Considerations
The near surface soils within portions of this site exhibited an increased silt/sand content and low plasticity. This moisture sensitive soils tend to lose significant strength with increases in there in-situ moisture contents. Therefore, these surficial soils may pose construction difficulties, especially during and after periods of wet weather conditions.

Based on our findings, the subsurface soils at this site generally exhibit a low expansion potential. Based on the information developed from our field and laboratory programs and on method TEX-124-E in the Texas Department of Transportation (TxDOT) Manual of Testing Procedures, we estimate that the subgrade soils at this site exhibit a Potential Vertical Rise (PVR) of about 1 inch in their present condition. The actual movements could be greater if inadequate drainage, ponded water, and/or other sources of moisture are allowed to infiltrate beneath the structures after construction.

4.2 Earthwork
The following presents recommendations for site preparation and placement of engineered fills on the project. The recommendations presented for design and construction of earth supported elements including foundations are contingent upon following the recommendations outlined in this section.

Earthwork on the project should be observed and evaluated by Terracon. The evaluation of earthwork should include observation and testing of engineered fill if any, foundation bearing soils, and other geotechnical conditions exposed during the construction of the project.

4.2.1 General Site Preparation
Construction operations may encounter difficulties due to the wet or soft surface soils becoming a general hindrance to equipment due to rutting and pumping of the soil surface, especially during and soon after periods of wet weather. If the subgrade cannot be adequately compacted to minimum densities as described in the “Compaction Requirements” section of this report, one of the following measures may be required:

- Removal and replacement with select fill;
- Drying by natural means if the schedule allows; and
- Chemical treatment.

Prior to placing any fill, all loose material and any otherwise unsuitable materials should be removed from the construction area. Wet or dry material should either be removed or moisture conditioned and re-compacted. After stripping and grubbing, the subgrade should be proof-
rolled where possible to aid in locating loose or soft areas. Proof-rolling can be performed with a 15-ton roller or fully loaded dump truck. Soft, dry and low-density soil should be removed or compacted in place prior to placing fill.

4.3 Underground Utility Design Recommendations and Construction Considerations

The recommendations and criteria presented in the following subsections can be used to aid in the design and analysis of buried pipes and utilities at this site.

4.3.1 Trench Bearing Pressures.

The subsurface soils have sufficient bearing capacity to support buried pipes. A net allowable bearing pressure of 1,500 pounds per square foot (psf) may be used to support the buried pipes. This bearing pressure includes a factor of safety of 3. The bearing pressure also assumes that the bearing surface will be relatively free and clean of any soft or moist material and loose debris.

4.3.2 Modulus of Soil Reaction.

A modulus of soil reaction for the in-situ soil, $E_s$ or $E_{in}$, of at least 600 psi may be used in the design of the flexible pipe. Additionally, the modulus of soil reaction, $E_b$ or sometimes referred to as $E'$, of the backfill material supporting the sides of the pipe is also used in the design of the flexible piping. This value is a function of several variables that include:

- Soil type that comprises the backfill material supporting the pipe sides.
- Degree of compaction of the backfill material supporting the pipe sides.
- Lift thickness of the backfill material supporting the pipe sides.

Values for $E_b$ vary, depending on the pipe backfill and bedding materials. Fine-grained soils consisting of primarily silt should not be used for bedding materials and backfill around the pipe. More specific information regarding this design parameter is included in ASTM D2321 entitled “Standard Practice for Underground Installation of Thermoplastic Pipe for Sewers and Other Gravity Flow Applications.” The following table presents typical modulus of soil reaction values, $E_b$, for various backfill materials at different compaction ranges.
### Table: Modulus of Soil Reaction, $E_b$, psi, For Degrees of Compaction

<table>
<thead>
<tr>
<th>Type of Material</th>
<th>Dumped (no compaction)</th>
<th>Slight &lt;85%</th>
<th>Moderate 85% to 95%</th>
<th>High &gt;95%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fine Grained Soil (LL&lt;50): CL, ML</td>
<td>NR</td>
<td>NR</td>
<td>NR</td>
<td>NR</td>
</tr>
<tr>
<td>Fine Grained Soil (LL&lt;50) with &gt;25% Coarse-Grained Material: CL, ML</td>
<td>NR</td>
<td>NR</td>
<td>1000</td>
<td>2000</td>
</tr>
<tr>
<td>or Coarse-Grained Soil with fines: GM, GC, SM, SC</td>
<td>NR</td>
<td>1000</td>
<td>2000</td>
<td>3000</td>
</tr>
<tr>
<td>Coarse-Grained Soil with &lt;12% fines: GW, GP, SW, SP</td>
<td>1000</td>
<td>3000</td>
<td>3000</td>
<td>3000</td>
</tr>
<tr>
<td>Crushed Rock</td>
<td>1000</td>
<td>3000</td>
<td>3000</td>
<td>3000</td>
</tr>
</tbody>
</table>

- These values do not include a factor of safety. A factor of safety may be needed for design purposes. These values are for use in predicting the initial deflections only. If a high degree of compaction is not achieved in the backfill adjacent to the sides of the pipe, an approximate deflection lag factor should be applied for long-term deflection estimates. It should be noted that LL refers to the Liquid Limit, and NR means that the use of these materials is not recommended by ASTM D2321 for the backfill envelope.

### 4.3.3 Excavations.

Various excavations are planned for site improvements. The actual excavation depths were not provided to us at the time of this report submittal. However, shoring, bracing, sloping, benching or a combination of each will be required during excavation or trenching of the surrounding soils during construction operations. Excavations and trenches should follow Occupational Safety and Health Administration (OSHA) Safety and Health Standards (29 CFR Part 1926 Revised, 1989), state and federal standards and guidelines.

### 4.3.4 Trench Backfill.

Appropriate trench backfill is generally determined by several factors including the bearing capacity of the soil supporting the pipe, requirements of the pipe manufacturer regarding support of the pipe, and the proposed improvements at the ground surface along the trench. Pipe manufacturers generally require a specified bedding and granular material around the pipe.

Typically, the bedding and embedment material around buried utilities is designed to support and protect the piping. The material above this material (which we call backfill) also helps to protect the piping and to support any overlying structure, roadway, or other improvement. Inadequate compaction of this material can lead to excessive settlement of the backfill, stress in the pipe, and premature distress to any overlying improvement. Therefore, we recommend that the embedment and backfill material be properly placed, moisture conditioned, and compacted in accordance with the appropriate project documents or those requirements established by applicable city standard specifications for public works construction.
Backfill beneath roadways (if any) should attempt to match the soil type exposed in the excavation sidewalls. As a compaction guideline, we recommend that all trench backfill be placed in loose lifts not to exceed 8 inches, moisture conditioned between -2 and plus +3 percentage points of the optimum moisture content, and compacted to at least 95 percent of the maximum dry density as evaluated by ASTM D 698.

Flowable fill can be used as an alternative to soil backfill, particularly beneath roadways or inaccessible areas. Flowable fill typically consists of a mixture of sand, portland cement, fly ash, and water and is readily available from ready-mixed concrete suppliers. This very low strength cementitious fill is placed in a slurry form and readily takes the shape of the excavation. Properly designed and placed, it can be trenched through by a backhoe for future repairs or modifications as required.

Embedment backfill along the sides to the top of the pipe and possibly 12 to 24 inches above the pipe should consist of materials that are acceptable to the project civil engineer or materials meeting those requirements established by any applicable city standard specifications for public works construction. To avoid potential damage to the pipe, the embedment material should not contain materials exceeding 3 inches in maximum dimension. Onsite soils should be suitable as backfill above the embedment material provided that the soils do not contain deleterious material or particles exceeding 3 inches in maximum dimension.

Construction equipment with wheel or gross loads exceeding the pipe’s design strength should not be driven over or close to the pipeline. Additional cover placed on top of the pipe or an alternate route should be provided for machinery producing excessive loads.

4.4 Spread Footings
Spread footings may be used to support the headwall loads of the proposed discharge alignment. The spread footings should be at 3 feet below bottom of the wall. Spread footings may be designed for an allowable bearing pressure of 1,500 psf, based on dead load condition or 2,250 psf based on total load, whichever results in a larger bearing surface. The above dead load bearing pressures include a factor of safety of 3. The total load bearing pressures include a factor of safety of 2.

The spread footings can provide some uplift resistance for those structures subjected to wind, soil pressures or other induced structural loading. The uplift resistance of a spread footing may be computed using the effective weight of the soil above the spread footing along with the weight of the spread footing and structure. A soil unit weight of 110 pcf may be assumed for the on-site soils placed above the footing, provided the fill is properly compacted.

4.4.1 Construction Considerations
Footing foundations should preferably be neat excavated. Excavation should be accomplished with a smooth-mouthed bucket. If a toothed bucket is used, excavation with this bucket should be stopped 6 inches above the final excavation surface and the excavation completed with a
smooth-mouthed bucket or by hand labor. Due to the presence of sand, caving of footing excavations may occur. Therefore, the contractor should be prepared to use forms.

If the footing foundations are overexcavated and formed, the backfill around the foundation sides should be achieved with compacted select fill, lean concrete, compacted cement stabilized sand (two (2) sacks cement to one (1) cubic yard of sand) or flowable fill. Compaction of select fill should be as described later in this section of the report.

The bearing surface should be excavated with a slight slope to create an internal sump for runoff water collection and removal. If surface runoff water in excess of 2 inches accumulates at the bottom of the excavation, it should be pumped out prior to concrete placement. Under no circumstances should water be allowed to adversely affect the quality of the bearing surface.

If the spread footing is buried, backfill above the foundation maybe the excavated on-site soils or select fill soils. Backfill soils should be compacted to at least 95 percent of the maximum dry density as determined by the standard moisture/density relationship test (ASTM D 698). Moisture contents for on-site soils ranging from -2 to +3 and imported select fill soils should range from -2 to +3 percentage points of the optimum moisture content. The backfill should be placed in thin, loose lifts not to exceed 8 inches, with compacted thickness not to exceed 6 inches.

### 4.5 Drilled Pier Foundation

We understand that a drilled pier foundation system may support the loads of the proposed headwall structure. Due to the presence of sand/gravel, cemented soils and groundwater, underreamed piers may be difficult to construct at this site. Therefore, only recommendations for straight-sided (non-underreamed) piers are provided in this report.

#### 4.5.1 Straight-Sided Piers

Loads for the new structure may be supported on straight-sided piers bearing at least 10 feet below the bottom of the headwall structure as required to develop the required design load. Depending on the type of structure being supported, the straight-sided shafts should be sized for the following soil parameters:

<table>
<thead>
<tr>
<th>Bearing Elevation, ft</th>
<th>Net Allowable End Bearing Pressure, psf</th>
<th>Net Allowable Side Friction, psf</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Total Load</td>
<td>Dead Load</td>
</tr>
<tr>
<td>365 – 355</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>355 – 345</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>345 – 330</td>
<td>4,500</td>
<td>3,000</td>
</tr>
<tr>
<td>330 – 340</td>
<td>15,000</td>
<td>10,000</td>
</tr>
<tr>
<td>340 – 320</td>
<td>22,500</td>
<td>15,000</td>
</tr>
</tbody>
</table>
These bearing pressures include factors of safety against a bearing capacity failure of approximately 2 and 3, respectively. Piers should not extend deeper than El. 317.0 feet without contacting our office. The above bearing pressures assume that the bearing surface will be free and clean of loose debris.

The allowable side shear values in the above table have a factor of safety of at least 2.

Other design considerations:

- The side shear should be neglected for the upper 3 feet below Final Grade Elevation (FGE) in contact with the pier shaft.

- Clearance of at least 3 diameters of the drilled pier, center to center, should be provided between the drilled piers to develop the recommended bearing pressures and to control settlements. If a clearance of 3 diameters cannot be maintained in every case, the above bearing capacities should be reduced by 20 percent. Drilled piers closer than a clearance of 2 diameters, center to center, are not recommended.

Axial tension force due to the swelling soils along the pier shaft is not of concern at this site. However, the cross-sectional area of the reinforcing steel should not be less than one-half (½) percent of the gross cross-sectional area of the drilled pier shaft. The reinforcing steel should extend from the top to the bottom of the shaft to resist this potential uplift force.

Settlements – For piers, total settlements, based on the indicated bearing pressures, should be about 1 inch for properly designed and constructed drilled piers. Settlement beneath individual piers will be primarily elastic with most of the settlement occurring during construction. Differential settlement may also occur between adjacent piers. The amount of differential settlement could approach 50 to 75 percent of the total pier settlement. For properly designed and constructed piers, differential settlement between adjacent piers is estimated to be less than 1 inch. Settlement response of drilled piers is impacted more by the quality of construction than by soil-structure interaction.

Improper pier installation could result in differential settlements significantly greater than we have estimated. In addition, larger magnitudes of settlement should be expected if the soil is subjected to bearing pressures higher than the allowable values presented in this report.

4.5.2 Foundation Installation

Groundwater was encountered in boring B-1 between depths of 24 and 28 feet below the existing grade (El. 341.5 and El. 337.5 feet). Groundwater levels are influenced by seasonal and climatic conditions, which result in fluctuations in groundwater elevations. Additionally, it is common for water to be present after periods of significant rainfall. Due to the presence of sand and very silty soils, caving of the pier wall excavation should be anticipated. Therefore, the contractor should be prepared to utilize casing techniques to control groundwater influx and pier wall caving during excavation if it occurs. Prior to any excavation, the contractor should verify
the groundwater levels. The contractor should consider performing a “test” pier excavation to determine the constructability of a drilled pier with the dry auger process. High-torque, high-powered (rock) drilling machinery equipped to cut and excavate rock will be required at this site. The casing and slurry methods of foundation installation are discussed in the following paragraphs.

**Casing Method** - Casing will provide stability of the excavation walls but may not completely eliminate groundwater influx potential or stability of the pier excavation bottom unless the casing penetrates below any pervious soils. Casing that terminates in pervious soils may generate “boils” due to the head differential between the inside and outside of the casing and require that the casing be extended until the excess seepage or boils are eliminated. The actual casing depth should be chosen by the drilling subcontractor. If this operation is not successful or to the satisfaction of the engineer, the pier excavation should be flooded with fresh water to offset the differential water pressure caused by the unbalanced water levels inside and outside of the casing. When the pier excavation depth is achieved and the bearing area has been cleaned, steel and concrete should then be placed immediately in the excavation. If more than 6 inches of water is present in the excavation, water should be removed by pumping or the concrete should be tremied completely to the bottom of the excavation with a closed-end tremie.

Removal of casing should be performed with extreme care and under proper supervision to minimize mixing of the surrounding soil and water with the fresh concrete. Rapid withdrawal of casing or the auger may develop suction that could cause the soil to intrude into the excavation. An insufficient head of concrete in the casing during its withdrawal could also allow the soils to intrude into the wet concrete. Both of these conditions may induce “necking”, a section of reduced diameter, in the pier.

**Slurry Method** - As an alternate to the use of casing to install the pier foundations, water or a weighted drilling fluid may be considered. Slurry displacement drilling can only prevent sloughing and water influx but cannot control sloughing once it has occurred. Therefore, slurry displacement drilling techniques must begin at the ground surface, not after sloughing materials are encountered.

Typical drilling fluids include those which contain polymers or bentonite. If a polymer is used with “hard” mixing water, a water softening agent may be required to achieve intimate mixing and the appropriate viscosity. The polymer manufacturer should be consulted concerning proper use of the polymer. If bentonite slurry is used, the bentonite should be mixed with water several hours before placing in the pier excavation. Prior mixing gives the bentonite sufficient time to hydrate properly. The drilling fluid should only be of sufficient viscosity to
control sloughing of the excavation walls and groundwater flow into the excavation. Care should be exercised while extracting the auger so that suction does not develop and cause disturbance or create “necking” in the excavation walls as described above. Casing should not be employed in conjunction with the slurry drilling technique due to possible trapping of loose soils and slurry between the concrete and natural soil.

The use of weighted drilling fluid when installing drilled pier foundations requires extra effort to ensure an adequate bearing surface is obtained. A clean-out bucket should be used just prior to pier completion in order to remove any cuttings and loose soils which may have accumulated in the bottom of the excavation. Reinforcing steel and concrete should be placed in the excavation immediately after pier completion. A closed-end tremie should be used to place the concrete completely to the bottom of the excavation in a controlled manner to effectively displace the slurry during concrete placement.

When the pier excavation depth is achieved and the bearing area has been cleaned, steel and concrete should then be placed immediately in the excavation. The concrete should be placed completely to the bottom of the excavation with a closed-end tremie in the pier excavation if more than 6 inches of water is ponded on the bearing surface or the slurry drilling technique is used. A short tremie may be used if the excavation has less than 6 inches of ponded water or if the water is pumped out prior to concrete placement. The fluid concrete should not be allowed to strike the pier reinforcement, temporary casing (if required) or excavation sidewalls during concrete placement.

The foundation excavations should be augered and constructed in a continuous manner. The reinforcing steel and concrete should be placed in the excavations immediately following drilling and evaluation for proper bearing stratum, embedment, and cleanliness. Under no circumstances should the foundation excavations remain open overnight.

All aspects of concrete design and placement should comply with the American Concrete Institute (ACI) 318-08 Code Building Code Requirements for Structural Concrete; ACI 336.1-01 entitled Reference Specification for the Construction of Drilled Piers, and ACI 336.3R-93 (Reapproved 2006) entitled Design and Construction of Drilled Piers. Concrete should be designed to achieve the specified 28-day strength when placed at a 7 inch slump with a ±1 inch tolerance. Adding water to a mix that has been designed for a lower slump does not meet the intent of this recommendation. If a high range water reducer is used to achieve this slump, the span of slump retention for the specific admixture under consideration should be thoroughly investigated. Compatibility with other concrete admixtures should also be considered. A technical representative of the admixture supplier should be consulted on these matters.
Concrete aggregates in the area could have a history of problems associated with Alkali Silica Reactivity (ASR). If aggregates are known to have a history of ASR, then one of the following should be incorporated in the concrete used for the foundations:

Option 1: Replace 20% to 35% of the cement with Class C or Class F fly ash. However, if sulfate resistant concrete is required, do not use a Class C fly ash and do not use Type I Portland cement.

Option 2: Use a lithium nitrate admixture at a minimum dosage of 0.55 gallons of 30% lithium nitrate solution per pound of alkalies present in the portland cement. Coordinate with admixture supplier.

Option 3: When using portland cement only, ensure that the total alkali contribution from the cement in the concrete does not exceed 4.00 lb. per cubic yard of concrete when calculated as follows:

Pounds of alkali per cu. yd. = (pounds of cement per cu. yd.) x (%Na₂O equivalent in cement)/100.

In the above calculation, use the maximum cement alkali content reported on the cement mill certificate.

Option 4: Test both coarse and fine aggregate separately, in accordance with ASTM C 1260, using 440g of the proposed cementitious material in the same proportions of portland cement to supplementary cementing material to be used in the mix. Before use of the mix, provide the certified test report, signed and sealed by a licensed professional engineer, demonstrating that the ASTM C 1260 test result for each aggregate does not exceed 0.10% expansion.

Successful installation of drilled piers is a coordinated effort involving the general contractor, design consultants, subcontractors and suppliers. Each must be properly equipped and prepared to provide their services in a timely fashion. Several key items of major concern are:

- Proper drilling rig with proper equipment (including casing and augers); High torque (rock) drilling equipment will be required;
- Reinforcing steel cages tied to meet project specifications;
- Proper scheduling and ordering of concrete for the piers; and
- Monitoring of the installation by design professionals.
Pier construction should be carefully monitored to assure compliance of construction activities with the appropriate specifications. A number of items of concern for foundation installation include those listed below.

- Pier locations
- Vertical alignment
- Competent bearing
- Steel placement
- Concrete properties and placement
- Proper casing seal for subsurface water control
- Casing removal
- Slurry viscosity (if required)

If the contractor has to deviate from the recommended foundations, Terracon should be notified immediately so additional engineering recommendations can be provided for an appropriate foundation type.

### 4.6 Lateral Earth Pressures

The lateral earth pressure recommendations given in the following paragraphs are applicable to the design of rigid retaining walls subject to slight rotation, such as cantilever, or gravity type concrete walls. These recommendations are not applicable to the design of modular block - geogrid reinforced backfill (MSE) walls.

Earth pressures will be influenced by structural design of the walls, conditions of wall restraint, methods of construction and/or compaction and the strength of the materials being restrained. Two wall restraint conditions are shown. Active earth pressure is commonly used for design of free-standing cantilever retaining walls and assumes wall movement. The "at-rest" condition assumes no wall movement. The recommended design lateral earth pressures do not include a factor of safety and do not provide for possible hydrostatic pressure on the walls.
### Earth Pressure Coefficients

<table>
<thead>
<tr>
<th>Earth Pressure Conditions</th>
<th>Coefficient for Backfill Type</th>
<th>Equivalent Fluid Density, pcf</th>
<th>Surcharge Pressure, psf (p₁)</th>
<th>Earth Pressure, psf (p₂)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Active, Ka</td>
<td>Granular – 0.33</td>
<td>40</td>
<td>(0.33)S</td>
<td>(40)H</td>
</tr>
<tr>
<td></td>
<td>On-site Soil – 0.49</td>
<td>59</td>
<td>(0.49)S</td>
<td>(59)H</td>
</tr>
<tr>
<td>At-Rest, Ko</td>
<td>Granular – 0.46</td>
<td>55</td>
<td>(0.46)S</td>
<td>(55)H</td>
</tr>
<tr>
<td></td>
<td>On-site Soil – 0.66</td>
<td>79</td>
<td>(0.66)S</td>
<td>(79)H</td>
</tr>
<tr>
<td>Passive, Kp</td>
<td>Granular – 3.4</td>
<td>425</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td></td>
<td>On-site Soil – 2.0</td>
<td>245</td>
<td>---</td>
<td>---</td>
</tr>
</tbody>
</table>

**Applicable conditions to the above include:**

- For active earth pressure, wall must rotate about base, with top lateral movements of about 0.002 $H$ to 0.004 $H$, where $H$ is wall height.
- For passive earth pressure to develop, wall must move horizontally to mobilize resistance.
- Uniform surcharge, where $S$ is surcharge pressure.
- In-situ soil backfill weight a maximum of 110 pcf.
- Horizontal backfill compacted at least 95 percent of standard Proctor maximum dry density.
- Loading from heavy compaction equipment not included.
- No hydrostatic pressures acting on wall.
- No dynamic loading.
- No safety factor included in soil parameters.
- Ignore passive pressure in frost zone.

Backfill placed against structures should consist of granular soils or low plasticity cohesive soils. On-site soils are suitable for use as backfill behind walls. For the granular values to be valid, the granular backfill must extend out from the base of the wall at an angle of at least 45 and 60 degrees from vertical for the active and passive cases, respectively. To calculate the resistance to sliding, a value of 0.35 should be used as the ultimate coefficient of friction between the footing and the underlying soil. A bearing pressure of 1,500 psf can be used for the wall footings. The wall footing should be beared at least 3 feet below bottom of the wall.

To control the water level behind the wall, we recommend a perimeter drain be installed at the foundation level as shown on the adjacent figure and described in the following notes.
Granular backfill in this case consists of ASTM No. 57 stone or equivalent.

Perforated pipe should be rigid PVC, sized to transport the expected water.

Exterior ground surface should consist of a 24 inch clay cap sloped to drain from building.

The clay cap can be replaced by a pavement section.

Weep holes can be considered in lieu of perimeter drains for retaining walls if the water seepage will not impact adjacent structures.

If adequate drainage is not possible, then combined hydrostatic and lateral earth pressures should be calculated for clayey soils backfill using an equivalent fluid weighing 90 and 100 pcf for active and at-rest conditions, respectively. For granular backfill, an equivalent fluid weighing 85 and 90 pcf should be used for active and at-rest, respectively. These pressures do not include the influence of surcharge, equipment or floor loading, which should be added. Heavy equipment should not operate within a distance closer than the exposed height of retaining walls to prevent lateral pressures more than those provided.

4.7 Seismic Considerations

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>2012 International Building Code Site Classification (IBC) (^1)</td>
<td>D (^2)</td>
</tr>
<tr>
<td>Maximum Considered Earthquake 0.2 second Spectral Acceleration (S(_S)) (^3)</td>
<td>0.068 g</td>
</tr>
<tr>
<td>Maximum Considered Earthquake 1.0 second Spectral Acceleration (S(_I)) (^3)</td>
<td>0.017 g</td>
</tr>
</tbody>
</table>

\(^1\) The site class definition was determined using SPT N-values in conjunction with section 1613.3.2 in the 2012 IBC and Table 20.3-1, Chapter 20 of the 2010 ASCE-7.

\(^2\) Section 20.1 in the 2010 ASCE-7 requires a site soil profile determination extending to a depth of 100 feet for seismic site classification. The current scope does not include the required 100 foot soil profile determination. Borings extended to a maximum depth of 50 feet, and this seismic site class definition considers that hard soil continues below the maximum depth of the subsurface exploration. Additional exploration to deeper depths would be needed to confirm the conditions below the current depth of exploration.

\(^3\) The Spectral Acceleration values were determined using publicly available information provided on the United States Geological Survey (USGS) website. The spectral acceleration values can be used to determine the site coefficients using Tables 1613.3.3 (1) and 1613.3.3 (2) in the 2012 IBC.
4.8 Sulfate Considerations
Sulfate tests were performed on selected samples collected from the borings to check for a possible adverse reaction with concrete structures. Test locations and depths were chosen to provide a range of test locations regards to depth and across the site. Tests were not performed in all borings nor at all depths. Test results are as follows:

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>Approximate Depth, feet</th>
<th>Sulfate Content, ppm</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1</td>
<td>6.5 - 8</td>
<td>116</td>
</tr>
<tr>
<td>B-2</td>
<td>4.5 - 6</td>
<td>246</td>
</tr>
</tbody>
</table>

The test results indicate sulfate values in the range of 116 ppm to 246 ppm. The sulfate effect at this site is considered to be negligible to moderate. Using the criteria from ACI 201.2R, the test results classify as Class 0 and 1 exposure, respectively.

The test results indicate that the sulfate concentrations in the soils are below levels deemed to be of a high risk for adverse reactions when mixed with a calcium-based additive TxDOT (>3,000 ppm), the National Lime Association (>3,000 ppm) and AASHTO (>5,000 ppm).

The American Concrete Institute (ACI) and the Texas Department of Transportation (TxDOT) provide guidance and specifications regarding sulfates in soil and groundwater. Based on these references and our experience, we recommend the following for concrete that will be in contact with the subsurface soils at this site:

- Maximum water/cement ratio of 0.50; and
- ASTM C150 Type I or II portland cement or equivalent.

5.0 GENERAL COMMENTS

Terracon should be retained to review the final design plans and specifications so comments can be made regarding interpretation and implementation of our geotechnical recommendations in the design and specifications. Terracon also should be retained to provide observation and testing services during grading, excavation, foundation construction and other earth-related construction phases of the project.

The analysis and recommendations presented in this report are based upon the data obtained from the borings performed at the indicated locations and from other information discussed in this report. This report does not reflect variations that may occur away from our boring, across the site, or due to the modifying effects of weather. The nature and extent of such variations may not become evident until during or after construction. If variations appear, we should be immediately notified so that further evaluation and supplemental recommendations can be provided.
The scope of services for this project does not include either specifically or by implication any environmental or biological (e.g., mold, fungi, and bacteria) assessment of the site or identification or prevention of pollutants, hazardous materials or conditions. If the owner is concerned about the potential for such contamination or pollution, other studies should be undertaken.

This report has been prepared for the exclusive use of our client for specific application to the project discussed and has been prepared in accordance with generally accepted geotechnical engineering practices. No warranties, express or implied, are intended or made. Site safety, excavation support, and dewatering requirements are the responsibility of others. In the event that changes in the nature, design, or location of the project as outlined in this report are planned, the conclusions and recommendations contained in this report shall not be considered valid unless Terracon reviews the changes and either verifies or modifies the conclusions of this report in writing.
APPENDIX A

FIELD EXPLORATION
FIELD EXPLORATION DESCRIPTION

Terracon personnel used the site plan provided by the client to establish the bore locations in the field. A copy of the Bore Location Plan indicating the approximate boring locations is included in Appendix A. The location of the borings should be considered accurate only to the degree implied by the means and methods used to define them.

A truck-mounted, rotary drill rig equipped with continuous flight augers was used to advance the boreholes. Soil samples were obtained by the split-barrel sampling procedure. In the split-barrel sampling procedure, a standard 2-inch O.D. split-barrel sampling spoon is driven into the ground with a 140-pound hammer falling a height of 30 inches. The number of blows required to advance the sampling spoon the last 12 inches of a normal 18-inch penetration is recorded as the standard penetration resistance value. These values are indicated on the boring logs at the depth of occurrence. The samples were sealed and transported to our laboratory for testing and classification.

Our field representative prepared the field logs as part of the drilling operations. The field logs included visual classifications of the materials encountered during drilling and our field representative interpretation of the subsurface conditions between samples. The boring logs included with this report represent the engineer's interpretation of the field logs and include modifications based on visual observations and testing of the samples in the laboratory.

The scope of services for our geotechnical engineering services does not include addressing any environmental issues pertinent to the site.
SS Effluent Line and Headwall Structure
South Laredo WWTP Expansion Project
Laredo, Texas

SITE LOCATION PLAN

Project Site

Scale: N.T.S.

File No.: 89135033

Drawn By: LC

Checked By: MR

Approved By: MR

Date: 10.08.2013

815 GALE STREET, BUILDING B
LAREDO, TX 78041

Ph: (956) 729-1100
FAX: (956) 791-1071

Consulting Engineers and Scientists

MR
Silty Clayey Sand (SC-SM), light brown, loose

Silty Sand (SM), light brown, loose to medium dense

- Clayey Sand (SC) at 18.5 feet

Silty Sand with Gravel (SM), light brown, medium dense

Claystone, bluish gray, hard, strong cementation

Boring Terminated at 50 Feet

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Notes:
Example: N=ref/2". Sampler could only be driven 2 inches of the 6-inch seating penetration before the 50-blow limit was reached.
**BORING LOG NO. B-2**

**PROJECT:** SS Effluent Line and Headwall Structure  
**CLIENT:** City of Laredo  
**SITE:** South Laredo WWTP Expansion Project, Laredo, Texas

### GRAPHIC LOG

- **LOCATION:** See Exhibit A-3
- **North:** 17052224.91  
  **East:** 661880.9
- **Surface Elev.:** 366.71 (FL)

### WATER LEVEL OBSERVATIONS

- **Surface Elev.:** 366.71 (Ft.)
- **Water Level Observations:**
  - **5:**
    - **24-12-17:** N=29
    - **11-12-9:** N=21
    - **6-5-6:** N=11
    - **6-7-6:** N=13
    - **4-4-8:** N=12
    - **11-15-22:** N=37
    - **16-21-23:** N=44
  - **25.0:** - with sand layers at 23.5 feet

### FIELD TEST RESULTS

- **Sample Type:**
- **UNCONFINED COMPRESSION (tsf):**
- **DRY UNIT WEIGHT (pcf):**
- **PERCENT FINES:**
- **ATTERBERG LIMITS**

### WATER LEVEL OBSERVATIONS

- **Depth (Ft.):**
- **Water Level Observations:**
  - **5:**
  - **10:**
  - **15:**
  - **20:**
  - **25:**

### Notes:

- **Advancement Method:** Dry Augered from 0 to 25 feet.  
- **Abandonment Method:** Borings backfilled with soil cuttings upon completion.  
- **Hammer Type:** Automatic

---

**Stratification lines are approximate. In-situ, the transition may be gradual.**

---

**This boring log is not valid if separated from original report. GeoSmart Log No. Well 89135033.GPJ**

---

**WATER LEVEL OBSERVATIONS**

- Groundwater was **not** encountered
APPENDIX B

LABORATORY TESTING
LABORATORY TESTING

Samples retrieved during the field exploration were taken to the laboratory for further observation by the project geotechnical engineer and were classified in accordance with the Unified Soil Classification System (USCS) described in this Appendix. At that time, the field descriptions were confirmed or modified as necessary and an applicable laboratory testing program was formulated to determine engineering properties of the subsurface materials.

Laboratory tests were conducted on selected soil samples and the test results are presented in this appendix. The laboratory test results were used for the geotechnical engineering analyses, and the development of foundation and earthwork recommendations. Laboratory tests were performed in general accordance with the applicable ASTM, local or other accepted standards.

Selected soil samples obtained from the site were tested for the following engineering properties:

- In-situ Water Content
- Atterberg Limits
- Amount of Material In-Soil Finer than the No. 200 Mesh (75-µm) Sieve
- Sulfate concentration (colorimetric method)

Sample Disposal
All samples were returned to our laboratory. The samples not tested in the laboratory will be stored for a period of 30 days subsequent to submittal of this report and will be discarded after this period, unless other arrangements are made prior to the disposal period.
### Descriptive Soil Classification

Soil classification is based on the Unified Soil Classification System. Coarse Grained Soils have more than 50% of their dry weight retained on a #200 sieve; their principal descriptors are: boulders, cobbles, gravel, or sand. Fine Grained Soils have less than 50% of their dry weight retained on a #200 sieve; they are principally described as clays if they are plastic, and silts if they are slightly plastic or non-plastic. Major constituents may be added as modifiers and minor constituents may be added according to the relative proportions based on grain size. In addition to gradation, coarse-grained soils are defined on the basis of their in-place relative density and fine-grained soils on the basis of their consistency.

### Location and Elevation Notes

Unless otherwise noted, Latitude and Longitude are approximately determined using a hand-held GPS device. The accuracy of such devices is variable. Surface elevation data annotated with +/- indicates that no actual topographical survey was conducted to confirm the surface elevation. Instead, the surface elevation was approximately determined from topographic maps of the area.

### Descriptive Soil Classification Table

<table>
<thead>
<tr>
<th>Descriptive Term (Density)</th>
<th>Standard Penetration or N-Value Blows/Ft.</th>
<th>Consistency of Fine-Grained Soils</th>
<th>Bedrock</th>
</tr>
</thead>
<tbody>
<tr>
<td>(More than 50% retained on No. 200 sieve.) Density determined by Standard Penetration Resistance Includes gravels, sands and silts.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>STRENGTH TERMS</strong></td>
<td><strong>Descriptive Term (Consistency)</strong></td>
<td><strong>Unconfined Compressive Strength, Qu, tsf</strong></td>
<td><strong>Standard Penetration or N-Value Blows/Ft.</strong></td>
</tr>
<tr>
<td>Very Loose</td>
<td>0 - 3</td>
<td>Very Soft</td>
<td>less than 0.25</td>
</tr>
<tr>
<td>Loose</td>
<td>4 - 9</td>
<td>Soft</td>
<td>0.25 to 0.50</td>
</tr>
<tr>
<td>Medium Dense</td>
<td>10 - 29</td>
<td>Medium-Stiff</td>
<td>0.50 to 1.00</td>
</tr>
<tr>
<td>Dense</td>
<td>30 - 50</td>
<td>Stiff</td>
<td>1.00 to 2.00</td>
</tr>
<tr>
<td>Very Dense</td>
<td>&gt; 50</td>
<td>Very Stiff</td>
<td>2.00 to 4.00</td>
</tr>
<tr>
<td>Hard</td>
<td>&gt; 4.00</td>
<td>&gt; 30</td>
<td>&gt; 42</td>
</tr>
</tbody>
</table>

### Relative Proportions of Sand and Gravel

<table>
<thead>
<tr>
<th>Descriptive Term(s) of other constituents</th>
<th>Percent of Dry Weight</th>
<th>Major Component of Sample</th>
<th>Particle Size</th>
</tr>
</thead>
<tbody>
<tr>
<td>Trace</td>
<td>&lt; 15</td>
<td>Boulders</td>
<td>Over 12 in. (300 mm)</td>
</tr>
<tr>
<td>With</td>
<td>15 - 29</td>
<td>Cobbles</td>
<td>12 in. to 3 in. (300mm to 75mm)</td>
</tr>
<tr>
<td>Modifier</td>
<td>&gt; 30</td>
<td>Gravel</td>
<td>3 in. to #4 sieve (75mm to 4.75 mm)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Sand</td>
<td>#4 to #200 sieve (4.75mm to 0.075mm)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Silt or Clay</td>
<td>Passing #200 sieve (0.075mm)</td>
</tr>
</tbody>
</table>

### Plasticity Description

<table>
<thead>
<tr>
<th>Descriptive Term(s) of other constituents</th>
<th>Percent of Dry Weight</th>
<th>Term</th>
<th>Plasticity Index</th>
</tr>
</thead>
<tbody>
<tr>
<td>Trace</td>
<td>&lt; 5</td>
<td>Non-plastic</td>
<td>0</td>
</tr>
<tr>
<td>With</td>
<td>5 - 12</td>
<td>Low</td>
<td>1 - 10</td>
</tr>
<tr>
<td>Modifier</td>
<td>&gt; 12</td>
<td>Medium</td>
<td>11 - 30</td>
</tr>
<tr>
<td></td>
<td></td>
<td>High</td>
<td>&gt; 30</td>
</tr>
</tbody>
</table>
### UNIFIED SOIL CLASSIFICATION SYSTEM

#### Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests

<table>
<thead>
<tr>
<th>Soil Classification</th>
<th>Group Symbol</th>
<th>Group Name</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coarse Grained Soils: More than 50% retained on No. 200 sieve</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gravels: More than 50% of coarse fraction retained on No. 4 sieve</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Clean Gravels: Less than 5% fines</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gravels with Fines: More than 12% fines</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sands: 50% or more of coarse fraction passes No. 4 sieve</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Clean Sands: Less than 5% fines</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sands with Fines: More than 12% fines</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gravels with 5 to 12% fines require dual symbols: GW-GM well-graded gravel with silt, GW-GC well-graded gravel with clay, GP-GM poorly graded gravel with silt, GP-GC poorly graded gravel with clay.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sands with 5 to 12% fines require dual symbols: SW-SM well-graded sand with silt, SW-SC well-graded sand with clay, SP-SM poorly graded sand with silt, SP-SC poorly graded sand with clay.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Inorganic: PI &gt; 7 and plots on or above &quot;A&quot; line</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Organic: PI &lt; 4 or plots below &quot;A&quot; line</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Silts and Clays: Liquid limit less than 50</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Inorganic: PI plots on or above &quot;A&quot; line</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Organic: PI plots below &quot;A&quot; line</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Silts and Clays: Liquid limit 50 or more</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Inorganic: PI plots on or above &quot;A&quot; line</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Organic: PI plots below &quot;A&quot; line</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Highly organic soils: Primarily organic matter, dark in color, and organic odor</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

- **A** Based on the material passing the 3-inch (75-mm) sieve
- **B** If field sample contained cobbles or boulders, or both, add “with cobbles or boulders, or both” to group name.
- **C** Gravels with 5 to 12% fines require dual symbols: GW-GM well-graded gravel with silt, GW-GC well-graded gravel with clay, GP-GM poorly graded gravel with silt, GP-GC poorly graded gravel with clay.
- **D** Sands with 5 to 12% fines require dual symbols: SW-SM well-graded sand with silt, SW-SC well-graded sand with clay, SP-SM poorly graded sand with silt, SP-SC poorly graded sand with clay.
- **E** $Cu = \frac{(D_{60})^2}{D_{10} \times D_{60}}$
- **F** If soil contains ≥ 15% sand, add “with sand” to group name.
- **G** If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.

#### Graphs

- **For classification of fine-grained soils and fine-grained fraction of coarse-grained soils**
  - Equation of "A" line
    - Horizontal at PL=4 to LL=25.5, then PL=0.73 (LL-20)
    - Vertical at LL=16 to PI=7, then PI=0.9 (LL-6)
  - Equation of "U" line
    - "U" line
    - "CL or OL"
    - "CH or OH"
    - "CL - ML"
    - "ML or OL"
    - "MH or OH"
  - PI plots on or above "A" line.
  - PI plots below "A" line.
  - PI plots on or above "A" line.
  - PI plots below "A" line.

**Exhibit C-2**