GEOTECHNICAL EXPLORATION REPORT
CITY OF ORANGE WELL NO. 28 PROJECT
235 WEST MAPLE AVENUE
CITY OF ORANGE, CALIFORNIA

Prepared for:

TETRA TECH, INC.

17885 Von Karman Avenue, Suite 500
Irvine, California 92614

Project No. 12451.001

August 23, 2019

Leighton Consulting, Inc.
A LEIGHTON GROUP COMPANY

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Tetra Tech, Inc.
17885 Von Karman Avenue, Suite 500
Irvine, California 92614

Attention: Mr. Laurence Esguerra, PE

Subject: Geotechnical Exploration Report
City of Orange Well No. 28 Project
235 West Maple Avenue
City of Orange, California

In response to your request, Leighton Consulting, Inc. has conducted a geotechnical engineering exploration as a subconsultant to Tetra Tech for the proposed City of Orange Well No. 28 Project. The proposed project consists of construction of a new well (Well No. 28) and a public mini park at 235 West Maple Avenue in the city of Orange, California. The purpose of our service was to explore the subsurface conditions at the well site in order to provide geotechnical recommendations to aid in design and construction of the project.

Based on our field exploration, the project site is underlain by a thin mantle of artificial fill overlying alluvial deposits. The fill materials generally consisted of clayey sand and sandy clay, and the alluvial deposits consisted of medium stiff to stiff sandy clay, and dense to very dense sand and gravel. Groundwater was not encountered in our borings drilled to a maximum depth of 26 feet.

Based upon the results of this geotechnical exploration, the proposed project is feasible from a geotechnical standpoint. Specific recommendations for the geotechnical aspects of the project are presented in this report.

We appreciate the opportunity to be of service to you on this project. If you have any questions or if we can be of further service, please contact us at your convenience.

Respectfully submitted,

LEIGHTON CONSULTING, INC.

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Project Engineer

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1.0 INTRODUCTION

1.1 Site Location and Proposed Project

The proposed project consists of construction of a new well (Well No. 28) and a public mini park at 235 West Maple Avenue in the city of Orange, California. The proposed well will be located at the northeast quadrant of the site and the remainder of the site will be developed as a mini park. The structure for the well includes a sound enclosure around the wellhead, a transformer, and a building containing a motor control room, chlorine storage, and electrical meters. The mini park would potentially include a historic structure re-located from another property and includes decomposed granite and concrete pathways, benches, fencing, landscaping, removable bollards, and trash receptacles.

Well No. 28 site is an asphalt paved rectangular-shaped lot of 118 by 132 feet that is vacant with no buildings or structures. It is bound by Maple Avenue to the south, Lemon Street to the west, industrial buildings to the north, and residential buildings to the east. The approximate site location is shown on Figure 1, *Site Location Map*.

1.2 Purpose and Scope of Exploration

The purpose of our geotechnical exploration was to explore the subsurface conditions at the well site in order to provide geotechnical recommendations to aid in design and construction. This geotechnical exploration was performed based on our proposal dated April 19, 2019.

The scope of this exploration included the following tasks:

- *Background Review* – A background review was performed of readily available, relevant geotechnical and geological literature pertinent to the site. References used in preparation of this report are listed in Section 6.0.

- *Pre-Field Exploration Activities* – Boring locations were marked and Underground Service Alert (USA) was notified to locate and mark existing underground utilities prior to our subsurface exploration.

- *Field Exploration* – We advanced one hollow-stem auger boring (LB-1) at the well site to a depth of 26 feet below existing grade on July 18, 2019. The boring was logged and sampled using Standard Penetration Test (SPT) and

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California Ring samplers at selected intervals. The SPT and Ring samplers were driven into the soil with a 140-pound hammer, free falling 30 inches. The number of blows was noted for every 6 inches of sampler penetration. Relatively undisturbed samples were collected from the boring using the Ring sampler. The sampling procedures generally followed ASTM D 1586 and D 3550 for SPT and split-barrel sampling of soil. In addition to driven samples, a representative bulk soil sample was also collected from the boring. Each soil sample collected was described in general conformance with the Unified Soil Classification System (USCS). The samples were sealed, packaged, and transported to our soil laboratory. The soil descriptions and depths are noted on the boring log included in Appendix A. After completion of drilling, the boring was backfilled with soil cuttings, compacted by a tamper and patched with asphalt. The approximate location of our boring is shown on Figure 2, *Site Exploration Map*.

- *Field Percolation Testing* – Two shallow borings were drilled to a depth of 4 feet and converted to a temporary percolation test wells (P-1 and P-2). The borings were pre-soaked upon completion of drilling in preparation for in-situ percolation testing. The testing was performed in general accordance with County of Orange *Technical Guidance Document (OCTGD) for the Preparation of Conceptual/Preliminary and/or Project Water Quality Management Plans (WQMPs)*, dated December 20, 2013. A 2-inch-diameter polyvinyl chloride (PVC) pipe with a perforated section (.020 slotted screen) was placed in the boreholes and the annulus was filled with clean sand (No. 3 Monterey Sand).

  After pre-soaking, the test wells were filled to a water level at least five times the boring radius above the bottom of the boring to determine the time interval for the percolation test. Once the time interval was established for each well, the percolation test was performed by measuring the drop of water level in the pipe and the time associated with the change in water level. The water drop was measured using a manual water sounder. At the end of the time interval, the well was refilled approximately to the initial water level and the procedure repeated until the tests were completed. Field data and calculated infiltration rate for each well is presented in Appendix C, *Percolation Test Results*. After the conclusion of percolation testing, the PVC pipe was removed from each test well. The test wells were backfilled with the soil cuttings and capped with cold asphalt concrete.

- *Laboratory Tests* – Laboratory tests were performed on selected soil samples obtained during our field exploration. The laboratory testing program was designed to evaluate the physical and engineering characteristics of the onsite soil. Tests performed during this exploration include:

  - In situ moisture content and dry density (ASTM D 2216 and ASTM D 2937);
  - Passing No. 200 Sieve (ASTM D 1140);
  - Consolidation (ASTM D 2435);
  - R-value (California Test Method 301);
  - Direct Shear (ASTM D 3080); and
  - Corrosivity Suite – pH, Sulfate, Chloride, and Resistivity (California Test Methods 417, 422, and 532/643).

  Test results of the in situ moisture content and dry density are presented on the boring logs in Appendix A. Other laboratory test results are presented in Appendix B, *Laboratory Test Results*.

- *Engineering Analysis* - The data obtained from our background review, field exploration, and laboratory testing program were evaluated and analyzed to develop the conclusions and recommendations presented in this report for the proposed project.

- *Report Preparation* - The results of the exploration are summarized in this report presenting our findings, conclusions, and recommendations.

2.0 GEOTECHNICAL FINDINGS

2.1 Subsurface Soil Conditions

Existing pavement penetrated at the boring locations consisted of 4 to 7 inches of asphalt concrete. Subsurface soils that underlie the pavement sections, as encountered during our field exploration, consisted of up to 3 feet of artificial fill (Af) overlying Quaternary-aged older alluvial fan deposits (Qvof) to the maximum explored depth of 26 feet.

The fill materials generally consisted of clayey sand and sandy clay. Below the fill, the alluvial deposits generally consisted of medium stiff to stiff sandy clay to a depth of 10 feet. Below the clay, we encountered dense to very dense sand and gravel. A detailed description of the subsurface soils encountered in our borings is presented in the boring logs (Appendix A).

2.2 Soil Corrosivity

In general, soil environments that are detrimental to concrete have high concentrations of soluble sulfates and/or pH values of less than 5.5. Soils with chloride content greater than 500 parts per million (ppm) per California Test 532 are considered corrosive to steel, either in the form of reinforcement protected by concrete cover or plain steel substructures, such as steel pipes. Additionally, soils with a minimum resistivity of less than 1,000 Ohm-cm are considered corrosive to ferrous metal. Based on the laboratory test results, the subsurface soils at the site generally have low soluble sulfate contents and neutral pH values. The minimum resistivity test results indicate that the soils have low corrosion potential to buried ferrous metals. The test results are included in Appendix B of this report.

2.3 Groundwater

Groundwater was not encountered in our boring drilled to a maximum depth of 26 feet below ground surface. The groundwater contour map in the *Seismic Hazard Zone Report for the Orange 7.5-Minute Quadrangle* (California Geological Survey, 2001) indicates that the historically high groundwater table in the area is on the order of 40 feet below the existing grade. Groundwater is not expected to adversely impact the proposed construction.

Fluctuations of the groundwater level, localized zones of perched water, and an increase in soil moisture should be anticipated during and following the rainy seasons or periods of locally intense rainfall or stormwater runoff.

2.4 Primary Seismic Hazard

Our review of available in-house literature indicates that the project site is not located within an Alquist-Priolo (AP) Earthquake Fault Zone (Hart and Bryant, 2007). The principal seismic hazard that could affect the site is ground shaking resulting from an earthquake occurring along any one of several major active faults in the region. The known regional faults that could produce the most significant ground shaking at the project site include the San Joaquin Hills Blind Thrust and Whittier faults located approximately 6.9 miles and 8.8 miles, respectively, from the site.

The intensity of ground shaking at a given location depends primarily upon the earthquake magnitude, the distance from the source, and the site response characteristics. Peak horizontal ground accelerations are generally used to evaluate the intensity of ground motion. Using the United States Geological Survey (USGS) Seismic Design Maps (USGS, 2018), the peak ground acceleration for the Maximum Considered Earthquake (MCE_G) adjusted for the Site Class effects (PGA_M) is 0.51g. Based on the USGS online unified hazard tool program (USGS, 2014), the modal seismic event is Moment Magnitude (M_W) 6.9 at a distance of 7.1 miles.

2.5 Secondary Seismic Hazards

Secondary seismic hazards in the region could include soil liquefaction and the associated surface manifestation, earthquake-induced landsliding and flooding, seiches, and tsunamis. A site-specific evaluation of these potential hazards is discussed in the following sections.

2.5.1 Liquefaction

Liquefaction is a seismic phenomenon in which loose, saturated, granular soils behave similarly to a fluid when subjected to high-intensity ground shaking. Liquefaction occurs when three general conditions exist: 1) shallow groundwater; 2) low density, fine, clean sandy soils; and 3) strong ground motion. Effects of liquefaction can include sand boils, settlement, and bearing capacity failures below structural foundations.

Review of the *Seismic Hazard Zone Map for the Orange Quadrangle* (CGS, 1998) indicates that the subject site is not located within an area that has been identified by the State of California as being potentially susceptible to the occurrence of liquefaction. Additionally, due to the presence dense to very dense sandy soils and a relatively deep historically high groundwater of 40 feet below grade, the liquefaction potential at the site is very low.

2.5.2 Earthquake-Induced Settlement

Seismically induced settlement consists of dry dynamic settlement (above groundwater) and liquefaction-induced settlement (below groundwater). These settlements occur primarily within loose to medium dense sandy soil due to reduction in volume during, and shortly after, an earthquake event. We have performed analyses to calculate the potential earthquake-induced settlement at the site. The settlements of these strata were estimated to result in a cumulative settlement of less than ½ inches. Differential settlement is estimated to be approximately one-half of the total settlement.

2.5.3 Seismically Induced Landslides

No significant ground slopes exist at the site and in the vicinity. Therefore, the potential for seismically induced landslides is considered negligible.

2.5.4 Earthquake-Induced Flooding

Earthquake-induced flooding can be caused by failure of dams or other water-retaining structures as a result of earthquakes. Due to the absence of these structures near the site, the potential for earthquake-induced flooding of the site in considered low.

2.5.5 Seiches and Tsunamis

Seiches are large waves generated in enclosed bodies of water in response to ground shaking. Tsunamis are waves generated in large bodies of water by fault displacement or major ground movement. Based on the absence of an enclosed water body near the site and the inland location of the site, seiche and tsunami risks at the site are considered negligible.

3.0 DESIGN RECOMMENDATIONS

Geotechnical recommendations for the proposed improvements are presented in the following sections. Construction considerations are discussed in Section 4.0 of this report. These recommendations are based upon the exhibited geotechnical engineering properties of the soils and their anticipated response both during and after construction as well as proper field observation and testing during construction. These recommendations are considered minimal and may be superseded by more conservative requirements of the civil engineer, building code, or the City of Orange. All earthwork should be performed in accordance with the recommendations below, unless specifically revised or amended by future review of project plans.

3.1 Earthwork

3.1.1 Site Preparation

Vegetation, debris, and other deleterious materials should be removed and disposed of offsite prior to the commencement of grading operations. Existing underground utilities, including irrigation lines, should be identified prior to the start of grading and abandoned or relocated as necessary. Abandoned utility trenches should be excavated to competent materials and properly backfilled under the observation and testing of the geotechnical engineer.

3.1.2 Overexcavation and Recompaction

The foundation for the proposed structures should be underlain by compacted fill to provide a uniform support and reduce potential for differential settlement. The compacted fill should extend a minimum 3 feet below bottom of the foundation and a minimum 3 feet beyond outside edges of the foundation. Pavement areas, driveway, and concrete flatwork should be underlain by a minimum 1 foot of compacted fill. Local conditions may be encountered which may require additional removals and recompaction. The exact extent of removals can best be determined during grading by the geotechnical engineer when direct observation and evaluation of materials are possible.

3.1.3 Subgrade Preparation

Prior to placing fill materials, the subgrade should be scarified to a minimum depth of 6 inches, moisture conditioned, and proofrolled. Any soft and/or unsuitable materials encountered at the bottom of the excavations should be removed and replaced with fill material.

3.1.4 Fill Placement and Compaction

The onsite soils to be used as compacted structural fill should be free of organic material or construction debris. Imported fill soils, if any, should be approved by the geotechnical engineer prior to placement as fill. Fill soils should be placed in loose lifts not exceeding 8 inches, moisture-conditioned as necessary to at least two percent above moisture optimum and compacted to a minimum of 90 percent of the maximum dry density as determined by ASTM D 1557.

3.2 Foundation Design Parameters

Conventional shallow foundations such as continuous and/or spread footings may be used to support the loads of the proposed structures.

3.2.1 Allowable Bearing Capacity

Footings should have a minimum embedment depth of 18 inches and a minimum width of 12 inches. An allowable bearing pressure of 2,000 psf may be used based on the minimum embedment depth and width. The allowable bearing value may be increased by 200 psf per foot increase in depth or width to a maximum allowable bearing pressure of 3,000 psf. The allowable bearing pressures are for the total dead load and frequently applied live loads and may be increased by one third when considering loads of short duration, such as those imposed by wind and seismic forces. The allowable bearing pressures are net values; the weight of the footing may be neglected for design purposes. All continuous footings should be reinforced with top and bottom steel to provide structural continuity and to permit spanning of local irregularities. It is essential that a geotechnical engineer observes footing excavations before reinforcing steel is placed.

The recommended allowable bearing capacity for shallow footings is generally based on a total allowable static settlement of 1 inch. Since settlement is a function of footing size and contact bearing pressure, differential settlement can be expected between adjacent columns or walls where a large differential loading condition exists. The differential settlement should be less than approximately ½ inch, assuming no more than 50 percent variation in dead plus sustained live load between adjacent columns. These settlement estimates should be reviewed by Leighton Consulting when final foundation plans and loads for the proposed structures become available.

3.2.2 Lateral Load Resistance

Resistance to lateral loads will be provided by a combination of friction between the soils and foundation interface and passive pressure acting against the vertical portion of the foundation. A friction coefficient of 0.30 may be used at the soil-concrete interface for calculating the sliding resistance. A passive pressure based on an equivalent fluid pressure of 300 pounds per cubic foot (pcf) may be used for calculating the lateral passive resistance. The lateral passive resistance can be taken into account only if it is ensured that the soils against embedded structures will remain intact with time. The above values do not contain an appreciable factor of safety, so the structural engineer should apply the applicable factors of safety and/or load factors during design.

3.3 Slab-On-Grade

Concrete slabs-on-grade subjected to special loads should be designed by the structural engineer. Where conventional light floor loading conditions exist, the following minimum recommendations for conventional slabs-on-grade should be used. More stringent requirements may be required by local agencies, the structural engineer, the architect, or the CBC.

- A minimum slab thickness of 5 inches. Slab reinforcement should be designed by the structural engineer but as a minimum should consist of No. 3 rebar placed at 24 inches on center in each direction and provided with adequate concrete cover.

- A vapor barrier, 10-mil or thicker, should be placed below slabs where moisture-sensitive floor coverings or equipment is planned. The vapor barrier should be properly sealed at all joints and any penetrations.

- To reduce the potential for excessive cracking, concrete slabs-on-grade should be provided with construction or weakened plane joints at frequent intervals. Joints should be laid out to form approximately square panels.

- The subgrade soils should be wetted thoroughly prior to placing the vapor barrier, steel, or concrete.

Our experience indicates that use of reinforcement in slabs can generally reduce the potential for drying and shrinkage cracking. Some cracking should be expected as the concrete cures. Minor cracking is considered normal; however, it is often aggravated by a high water/cement ratio, high concrete temperature at the time of placement, small nominal aggregate size, and rapid moisture loss due to hot, dry, and/or windy weather conditions during placement and curing. Cracking due to temperature and moisture fluctuations can also be expected. The use of low slump concrete can reduce the potential for shrinkage cracking.

3.4 Seismic Design Parameters

The following values may be used for seismic design based on the 2016 California Building Code:

Table 1 – 2016 CBC Based Seismic Design Parameters

Categorization/Coefficient	Design Value
Site Class	D
Adjusted (5% damped) spectral response acceleration parameter at short period, S_{MS}	1.497
Adjusted (5% damped) spectral response acceleration parameter at a period of 1 sec, S_{M1}	0.819
Design (5% damped) spectral response acceleration parameter at short period, S_{DS}	0.998
Design (5% damped) spectral response acceleration parameter at a period of 1 sec, S_{D1}	0.546

3.5 Lateral Earth Pressures

The following recommendations may be used for design and construction of retaining structures at the site. We recommend that any permanent earth retaining structures be backfilled with onsite or import soil with Expansion Index (EI) of not greater than 50 (per ASTM D 4829).

Table 2 – Equivalent Fluid Pressures

Condition	Level Backfill
Active	38 pcf
At-Rest	58 pcf
Passive	300 pcf (Maximum of 3,000 psf)

The above values do not contain an appreciable factor of safety, so the structural engineer should apply the applicable factors of safety and/or load factors during design. Retaining walls should be provided with a drainage system behind the wall to prevent build-up of hydrostatic pressure.

Cantilever walls that are designed for a deflection at the top of the wall of at least 0.001H, where H is equal to the wall height, may be designed using the active earth pressure condition. Rigid walls that are not free to rotate, walls that are braced at the top, and walls that provide indirect support for foundations should be designed using the at-rest condition.

Lateral load resistance will be provided by the sliding resistance at the base of the foundation and the passive pressure developed along the front of the foundation. A frictional resistance coefficient of 0.30 may be used at the concrete and soil interface.

In addition to the above lateral forces due to retained earth, the appropriate loads due to surcharges should be considered in the design of retaining structures.

3.6 Cement Type

Based on the results of laboratory testing, concrete structures in contact with the onsite soil are expected to have negligible exposure to water-soluble sulfates in the soil. As such, the concrete may be designed for negligible sulfate exposure in accordance with ACI 318-14. Type V cement should be used if the concrete is to be exposed to reclaimed water.

3.7 Pavement Design

Driveways and parking areas can be constructed using conventional asphalt concrete (AC) over aggregate base (AB). We have designed the pavement sections using a design R-value of 18 for different Traffic Indices (TI) and the minimum pavement thickness is presented in Table 3. The pavement design was performed using the method in *Orange County Highway Design Manual*.

Table 3 – Pavement Sections

Traffic Index	Flexible Pavement (inches)	
	AC	AB
5 or less	4.0	6.0
6	5.0	7.0
7	6.0	9.0

Concrete pavement, if used, may consist of 6 inches of Portland Cement Concrete (PCC) over 6 inches of AB. The PCC pavement sections should be provided with crack-control joints spaced no more than 10 feet on-center each way, to control where cracks develop.

All pavement construction should be performed in accordance with the *Standard Specifications for Public Works Construction*. Field inspection and periodic testing, as needed during placement of the base course materials, should be undertaken to ensure that the requirements of the standard specifications are fulfilled. Prior to placement of aggregate base, the subgrade soil should be processed to a minimum depth of 8 inches, moisture-conditioned, as necessary, and recompacted to a minimum of 90 percent relative compaction. Localized areas of loose soils may be encountered that require deeper removal and recompaction. The actual extent of the removal depth will be best determined during construction when direct observation of the subgrade soils can be made.

Aggregate base should be moisture conditioned, as necessary, and compacted to a minimum of 95 percent relative compaction.

Aggregate base and asphalt materials should conform to Sections 200-2 and 203, respectively, of the *Standard Specifications for Public Works Construction*.

PCC should conform to Section 201 of the *Standard Specifications for Public Works Construction*.

3.8 Infiltration Rates

The percolation test was performed using the falling-head method, which records the drop of water level inside the test well over the specified time interval and repeated several times until consistent measurements are achieved. The field ("observed") infiltration rate was calculated based on the Porchet method provided in the OCTGD (2013). The field percolation test data and infiltration rate calculation is provided in Appendix C. Results of the field percolation testing are summarized in Table 4.

The field ("observed") infiltration rates must be reduced by applying an appropriate factor of safety to determine design infiltration rate that will represent long-term performance of the proposed infiltration BMP device. Based on the OCTGD, the safety factor consists of two categories of reduction factors, Suitability Assessment (Category A) and Design (Category B). The safety for Category B will be determined by the BMP devices designer. The recommended reduction factor at the test location for the Suitability Assessment Category is included in Table 4.

Table 4 – Field Percolation Test Summary

Percolation Test Well No.	Screen Interval Depth	Field Infiltration Rate ("Observed") inch/hour	Suitability Assessment Safety Factor (Worksheet H Factor Category A)
P-1	1 to 4	0.6	2.0
P-2	1 to 4	1.4	2.0

The following recommendation should be considered minimal from a geotechnical viewpoint as there may be more restrictive requirements of the governing agencies. As a minimum, we recommend the following setbacks of the stormwater infiltration system.

Table 5 – Stormwater infiltration System Setbacks

Setback from	Distance
Public right-of-way limits	10 feet
Any foundation	10 feet or a 1:1 plane drawn up from the bottom of foundation, whichever is greater
Water wells used for drinking water	100 feet

In general, a vast majority of geotechnical distress issues are related to improper drainage. Distress in the form of foundation movement could occur. Soil saturation could lead to a loss of soil support of foundations and pavements, settlement or collapse, internal erosion (piping) and expansion. Offsite properties could be affected and those improvements may become subjected to seeps, springs, foundation movement or other geotechnical issues related to infiltration and water migration. Additionally, infiltration water can migrate along pipe backfill (typically sand or gravel bedding), thereby impacting improvements away from the point of infiltration. Any proposed infiltration system should not be located near existing or proposed improvements in order to reduce the geotechnical distress issues related to infiltration. Where sufficient distance from improvements cannot be achieved, additional recommendations may need to be provided.

As with all systems that are designed to concentrate surface flow and direct water into the subsurface soils, some type of nuisance water and other geotechnical water related issues should be anticipated. We recommend sufficient distances between infiltration devices and sensitive improvements be maintained. Routine maintenance should be required of any infiltration system.

3.9 Additional Geotechnical Services

Geotechnical observation and testing should be provided during the following activities:

- Upon completion of site clearing, where applicable;
- During site earthwork;
- Compaction of all fill materials;
- Utility trench backfilling and compaction;

- During installation of temporary shoring, wherever needed;
- Pavement subgrade and base preparation;
- After foundation excavations and prior to placement of concrete;
- Placement of asphalt concrete; and
- When any unusual conditions are encountered.

4.0 CONSTRUCTION CONSIDERATIONS

4.1 Trench Backfill

Utility trenches can be backfilled with the onsite material, provided it is free of debris, organic material and oversized material (greater than 6 inches in diameter). All backfill should be placed in thin lifts (appropriate for the type of compaction equipment), moisture conditioned above optimum, and mechanically compacted to at least 90 percent relative compaction, relative to the ASTM D 1557 laboratory maximum density.

Prior to backfilling the trench, pipes should be bedded in and covered with sand that exhibits a Sand Equivalent (SE) of 30 or greater. The pipe bedding should extend at least 4 inches below the pipeline invert and at least 12 inches over the top of the pipeline. The bedding and shading sand is recommended to be densified in place by vibratory, lightweight compaction equipment and not by water jetting. Jetting or hydro-consolidation is not suitable for this project site and will result in unstable saturated subgrade.

Where utility trenches cross underneath building footing, the trenches should be plugged by a minimum of 2 feet of impermeable clayey soils or sand/cement slurry to reduce the potential for water intrusion underneath the slab.

4.2 Temporary Excavation and Shoring Design

All temporary excavations should be performed in accordance with project plans, specifications, and all OSHA requirements. Excavations 5 feet or deeper should be laid back or shored in accordance with OSHA requirements before personnel are allowed to enter.

Typical cantilever shoring should be designed using an active earth pressure presented in Table 2. If excavations are braced at the top and at specific design intervals, the active pressure may then be approximated by a rectangular soil pressure distribution with the pressure per foot of width equal to 25H, where H is equal to the depth of the excavation being shored. These lateral earth pressures are for a drained condition. For an undrained condition, hydrostatic pressure should be included.

During construction, the soil conditions should be regularly evaluated to verify that conditions are as anticipated. The contractor should be responsible for providing the “competent person” required by OSHA, standards to evaluate soil conditions. Close coordination between the competent person and the geotechnical engineer should be maintained to facilitate construction while providing safe excavations.

5.0 LIMITATIONS

This report was based solely on data obtained from a limited number of geotechnical exploration, and soil samples and tests. Such information is, by necessity, incomplete. The nature of many sites is such that differing soil or geologic conditions can be present within small distances and under varying climatic conditions. Changes in subsurface conditions can and do occur over time. Therefore, the findings, conclusions, and recommendations presented in this report are only valid if Leighton Consulting has the opportunity to observe subsurface conditions during grading and construction, to confirm that our preliminary data are representative for the site. Leighton Consulting should also review the construction plans and project specifications, when available, to comment on the geotechnical aspects.

It should be noted that the recommendations in this report are subject to the limitations presented in this section. An information sheet prepared by GBC (Geotechnical Business Council) is also included at the rear of the text. We recommend that all individuals using this report read the limitations along with the attached information sheet.

Our professional services were performed in accordance with the prevailing standard of professional care as practiced by other geotechnical engineers in the area. We do not make any warranty, either expressed or implied. The report may not be used by others or for other projects without the expressed written consent of our client and our firm.

6.0 REFERENCES

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Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, clients can benefit from a lowered exposure to the subsurface problems that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed below, contact your GBA-member geotechnical engineer. Active involvement in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Geotechnical-Engineering 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 given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. *Those who rely on a geotechnical-engineering report prepared for a different client can be seriously misled.* No one except authorized client representatives should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one – not even you – should apply this report for any purpose or project except the one originally contemplated.*

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read it *in its entirety*. Do not rely on an executive summary. Do not read selected elements only. *Read this report in full.*

You Need to Inform Your Geotechnical Engineer about Change

Your geotechnical engineer considered unique, project-specific factors when designing the study behind this report and developing the confirmation-dependent recommendations the report conveys. A few typical factors include:

- the client's goals, objectives, budget, schedule, and risk-management preferences;
- the general nature of the structure involved, its size, configuration, and performance criteria;
- the structure's location and orientation on the site; and
- other planned or existing site improvements, such as retaining walls, access roads, parking lots, and underground utilities.

Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- 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;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the 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. *The geotechnical engineer who prepared this report cannot accept responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

This Report May Not Be Reliable

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, that it could be unwise to rely on a geotechnical-engineering report whose reliability may have been affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If your geotechnical engineer has not indicated an "apply-by" date on the report, ask what it should be*, and, in general, *if you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying it. A minor amount of additional testing or analysis – if any is required at all – could prevent major problems.

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface through various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing were performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgment to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team from project start to project finish, so the individual can provide informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, *they are not final*, because the geotechnical engineer who developed them relied heavily on judgment and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* revealed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a full-time member of the design team, to:

- confer with other design-team members,
- help develop specifications,
- review pertinent elements of other design professionals' plans and specifications, and
- be on hand quickly whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction observation.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for informational purposes only*. To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report, but they may rely on the factual data relative to the specific times, locations, and depths/elevations referenced. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include 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 personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures*. If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. As a general rule, *do not rely on an environmental report prepared for a different client, site, or project, or that is more than six months old.*

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, none of the engineer's services were designed, conducted, or intended to prevent uncontrolled migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration.*** Confront the risk of moisture infiltration by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

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Telephone: 301/565-2733
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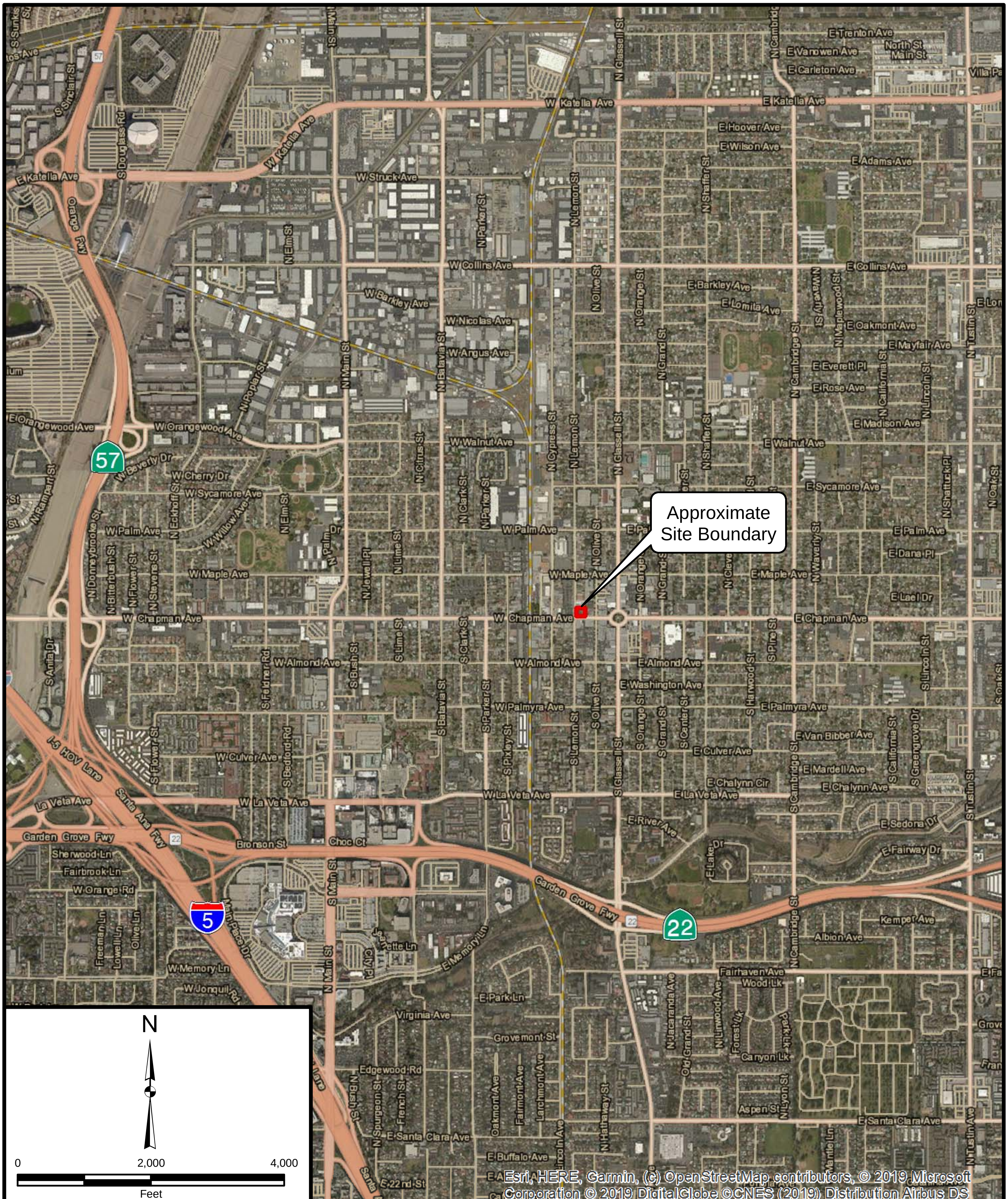

Project: 12451.001	Eng/Geol: DJC	**SITE LOCATION MAP** City of Orange Well No. 28 235 West Maple Avenue Orange, California	Figure 1
Scale:1 " = 2,000 '	Date: August 2019		Leighton

Base Map: ESRI ArcGIS Online 2019
Thematic Information: Leighton
Author: Leighton Geomatics (btran)

Map Saved as V:\Drafting\12451\001\Maps\12451-001_F01_SLM_2019-08-20.mxd on 8/20/2019 4:16:56 PM

Legend

- Approximate Location of Geotechnical Boring Shown with Total Depth (T.D.) in Feet. Groundwater Not Encountered.
- Approximate Location of Percolation Test Well Total Depth = 4 feet
- Approximate Site Boundary

Project: 12451.001	Eng/Geol: DJC
Scale:1 " = 30 '	Date: August 2019

Base Map: ESRI ArcGIS Online 2019
Thematic Information: Leighton
Author: Leighton Geomatics (btran)

SITE EXPLORATION MAP

City of Orange Well No. 28
235 West Maple Avenue
Orange, California

Figure 2

Leighton

Map Saved as V:\Drafting\12451\001\Maps\12451-001_F02_SEM_2019-08-20.mxd on 8/20/2019 4:58:37 PM

APPENDIX A

BORING LOGS

GEOTECHNICAL BORING LOG LB-1

Project No.	12451.001	**Date Drilled**	7-18-19
Project	City of Orange Well No. 28	**Logged By**	SG
Drilling Co.	2R Drilling, Inc.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	'
Location	See Figure 2 - *Site Exploration Map*	**Sampled By**	SG

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	0			B-1				SC	@Surface: 4 inches of asphalt concrete. No base. **Artificial fill (Af):** @1': Clayey SAND; dark gray; moist; trace gravel, subangular	RV, CR
								CL	**Quaternary-aged older alluvial fan deposits (Qvof):** @3': Sandy CLAY; reddish brown; moist; low plasticity	
	5			R-1	4 4 7	111	12			DS, CN
				S-2	3 5 10		13		@7.5': Trace coarse gravel, subangular	
	10			R-3	16 27 48	126	4	SP	@10': Poorly-graded SAND; dense; dark brown; slightly moist; fine to medium sand; trace gravel	
	15			S-4	13 13 38		4		@15': Very dense; fine to coarse sand	
								GW-GM	@16': Well-graded GRAVEL with Silt and Sand; olive brown; slightly moist; cobble, subangular, flat	
	20			R-5	39 42 36	119	3		@20': Very dense; fine to coarse gravel, round to subangular	-200
	25			S-6	28 50/5"			SP	@25': Poorly-graded SAND; very dense; yellow brown; slightly moist; medium to coarse sand; trace gravel, subround	
									Total Depth of Boring: 25.9 feet **No groundwater encountered.** **Backfilled with soil cuttings and capped with cold asphalt.**	
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

* * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * *

GEOTECHNICAL BORING LOG P-1

Project No.	12451.001	**Date Drilled**	7-18-19
Project	City of Orange Well No. 28	**Logged By**	SG
Drilling Co.	2R Drilling, Inc.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger	**Ground Elevation**	'
Location	See Figure 2 - *Site Exploration Map*	**Sampled By**	SG

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
									This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.	
	0							CL	@Surface: 4 inches of asphalt concrete. No base. @0.3': Sandy CLAY; reddish brown; moist; fine sand; low plasticity	
				B-1						-200
	5								**Total Depth of Boring: 4 feet** **Boring converted into a percolation test well.** **Percolation testing performed on 7/19/2019.** **Percolation test well removed and boring backfilled with soil cuttings and capped with cold asphalt.**	
	10									
	15									
	20									
	25									
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
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- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

*** * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * ***

GEOTECHNICAL BORING LOG P-2

Project No.	12451.001	**Date Drilled**	7-18-19
Project	City of Orange Well No. 28	**Logged By**	SG
Drilling Co.	2R Drilling, Inc.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger	**Ground Elevation**	'
Location	See Figure 2 - *Site Exploration Map*	**Sampled By**	SG

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0								@Surface: 7 inches of asphalt concrete. No base.	
								CL	@0.6': Sandy CLAY; reddish brown; moist; fine sand; low plasticity	
				B-1						-200
	5								**Total Depth of Boring: 4 feet** **Boring converted into a percolation test well.** **Percolation testing performed on 7/19/2019.** **Percolation test well removed and boring backfilled with soil cuttings and capped with cold asphalt.**	
	10									
	15									
	20									
	25									
	30									

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

APPENDIX B

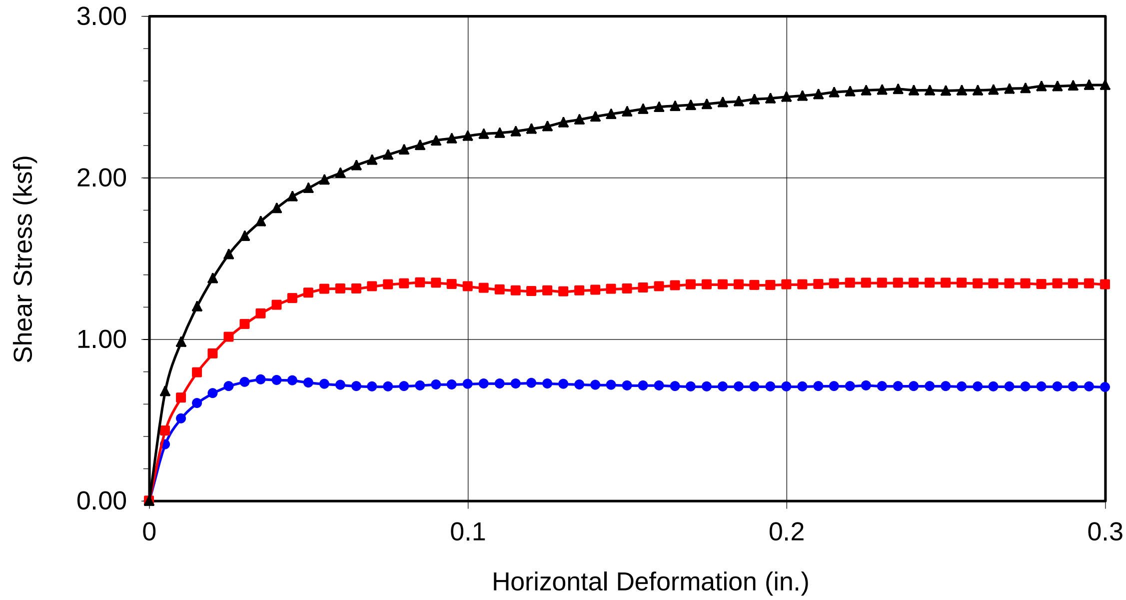
LABORATORY TEST RESULTS

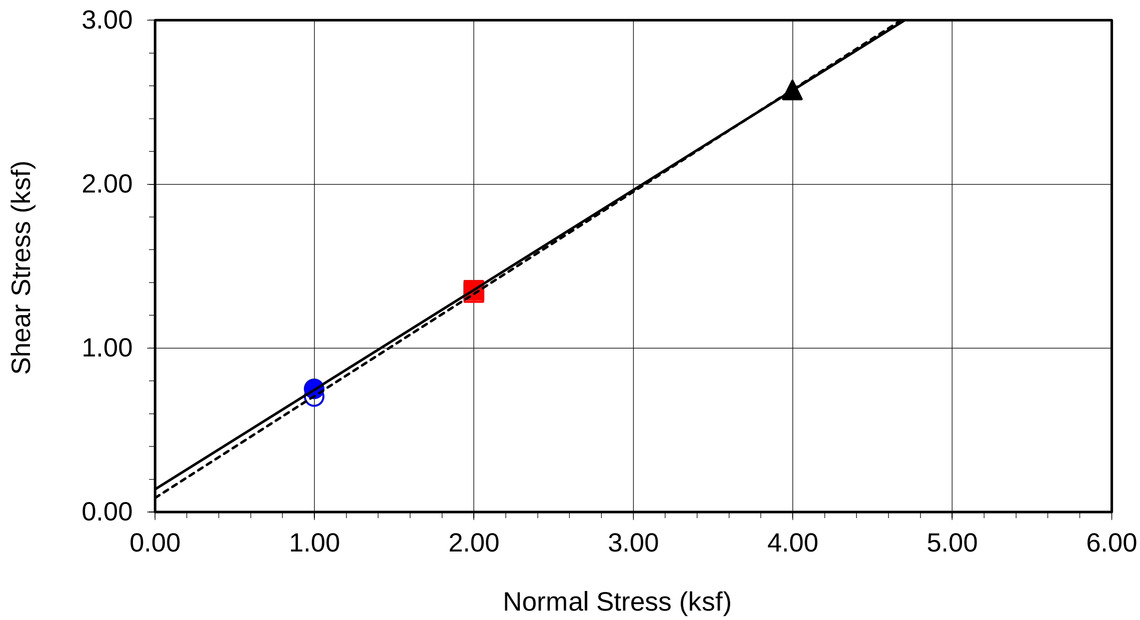
Boring No.	LB-1	P-1	P-2						
Sample No.	R-5	B-1	B-1						
Depth (ft.)	20.0	3.0	3.0						
Sample Type	Ring	SPT	SPT						
Soil Identification	Olive gray well-graded gravel with silt and sand (GW-GM)s	Brown silty clay with sand (CL-ML)s	Brown sandy silty clay s(CL-ML)						
Moisture Correction									
Wet Weight of Soil + Container (g)	0.00	0.00	0.00						
Dry Weight of Soil + Container (g)	0.00	0.00	0.00						
Weight of Container (g)	1.00	1.00	1.00						
Moisture Content (%)	0.00	0.00	0.00						
Sample Dry Weight Determination									
Weight of Sample + Container (g)	1100.60	574.81	577.31						
Weight of Container (g)	244.01	248.15	219.50						
Weight of Dry Sample (g)	856.59	326.66	357.81						
Container No.:									
After Wash									
Method (A or B)	A	B	B						
Dry Weight of Sample + Cont. (g)	1050.10	343.00	338.00						
Weight of Container (g)	244.01	248.15	219.50						
Dry Weight of Sample (g)	806.09	94.85	118.50						
% Passing No. 200 Sieve	**5.9**	**71.0**	**66.9**						
% Retained No. 200 Sieve	94.1	29.0	33.1						

Leighton

PERCENT PASSING No. 200 SIEVE ASTM D 1140

Project Name:	Tetra Tech Orange Well #28
Project No.:	12451.001
Client Name:	
Tested By:	G. Bathala
Date:	07/30/19





Boring No.	**LB-1**
Sample No.	**R-1**
Depth (ft)	**5**
Sample Type:	Ring

Soil Identification:
Brown silty clay (CL-ML)

Strength Parameters

	C (psf)	ϕ (°)
Peak	140	31
Ultimate	86	32

Normal Stress (kip/ft²)	1.000	2.000	4.000
Peak Shear Stress (kip/ft²)	● 0.751	■ 1.352	▲ 2.575
Shear Stress @ End of Test (ksf)	○ 0.704	□ 1.339	△ 2.575
Deformation Rate (in./min.)	0.0017	0.0017	0.0017
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	11.61	11.61	11.61
Dry Density (pcf)	109.7	109.9	112.4
Saturation (%)	58.4	58.8	62.7
Soil Height Before Shearing (in.)	0.9913	0.9858	0.9814
Final Moisture Content (%)	17.0	16.7	15.0

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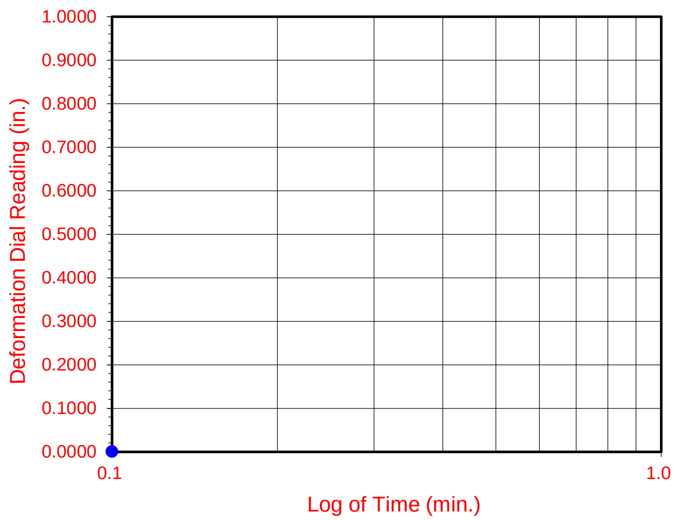
DIRECT SHEAR TEST RESULTS
Consolidated Drained - ASTM D 3080

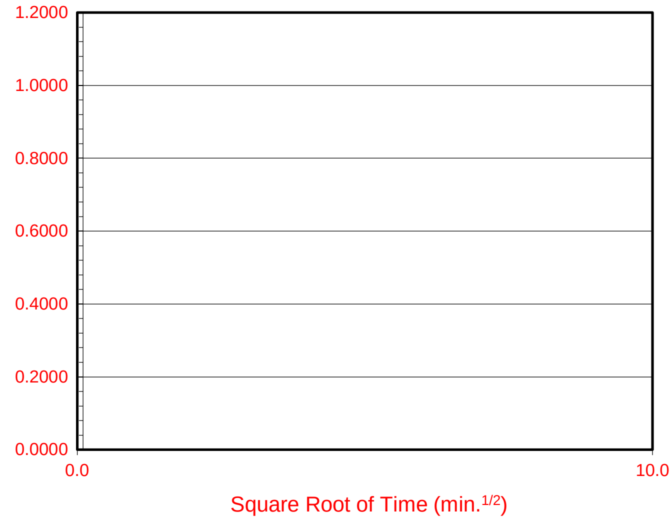
Project No.: 12451.001

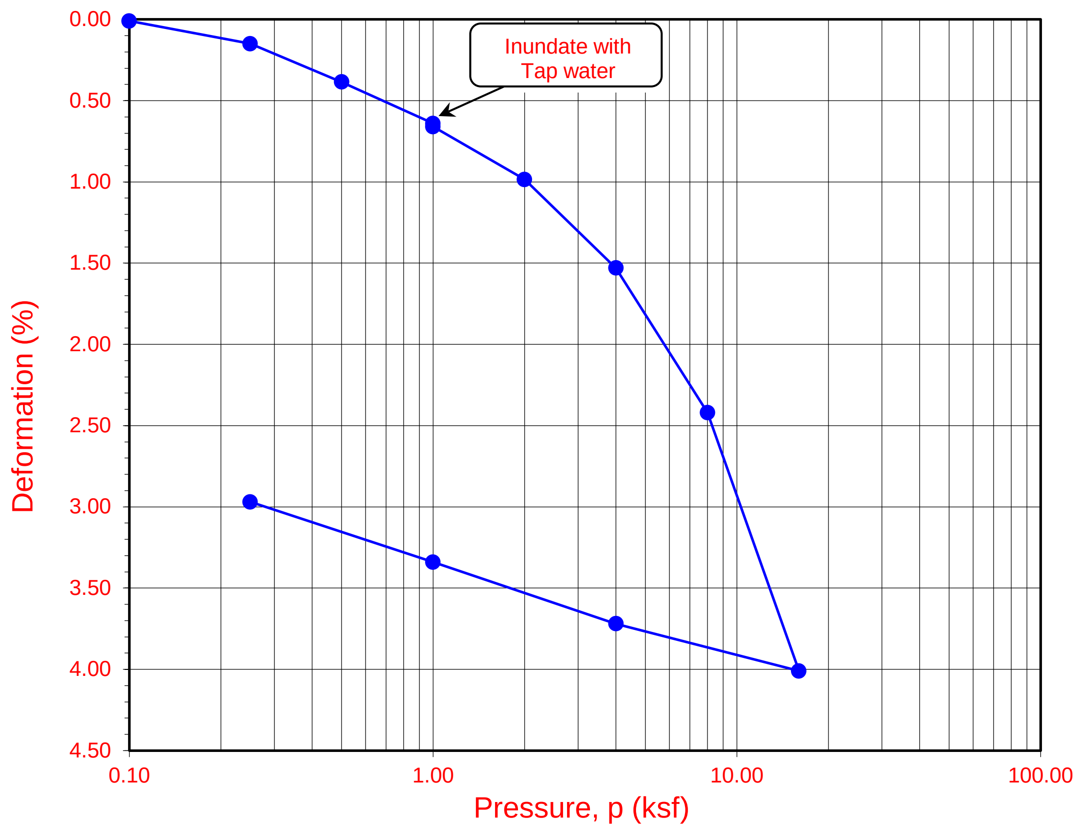
Tetra Tech Orange Well #28

08-19

Time Readings







Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
LB-1	**R-1**	**5.0**	**11.6**	**16.9**	**110.1**	**111.4**	**0.530**	**0.485**	**59**	**89**

Soil Identification: Brown silty clay (CL-ML)

Leighton

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 12451.001

Tetra Tech Orange Well #28

SOIL RESISTIVITY TEST

DOT CA TEST 643

Leighton

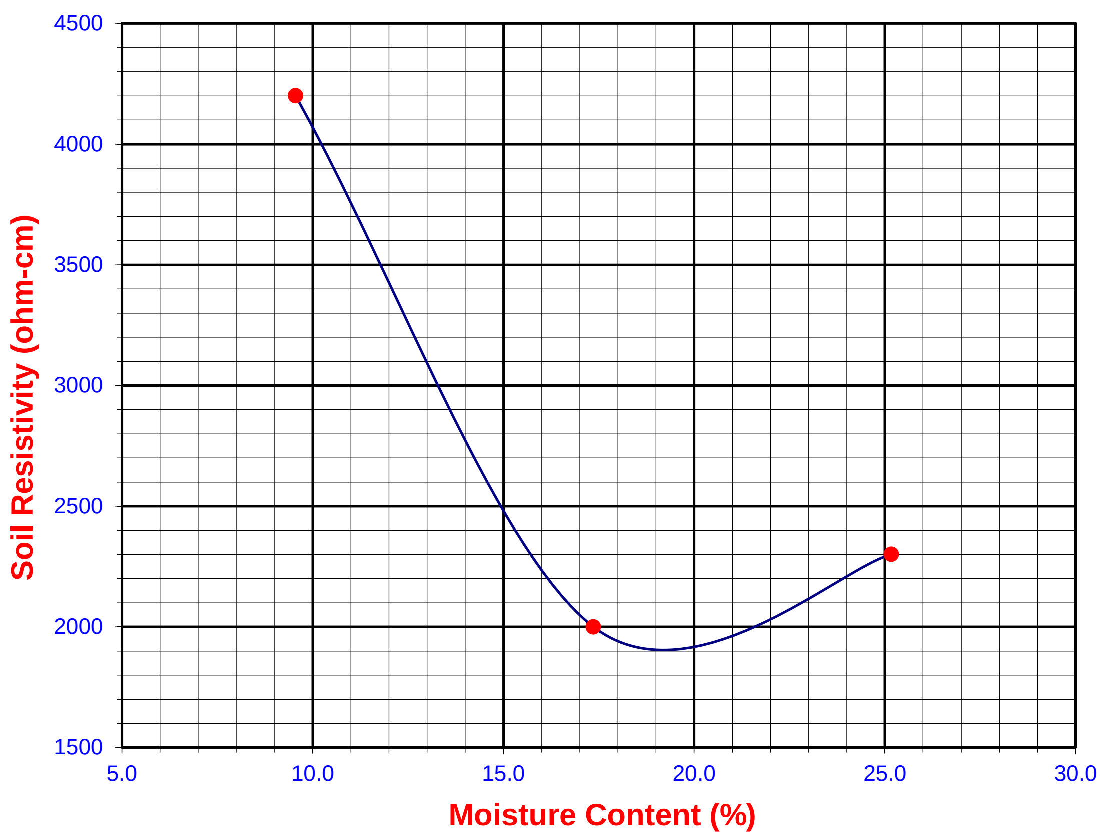
Project Name:	Tetra Tech Orange Well #28	Tested By :	O. Figueroa Date: 07/29/19
Project No. :	12451.001	Input By:	A. Santos Date: 08/06/19
Boring No.:	LB-1	Depth (ft.) :	1.0
Sample No. :	B-1		
Soil Identification:*	Brown (SC)g		

*California Test 643 requires soil specimens to consist only of portions of samples passing through the No. 8 US Standard Sieve before resistivity testing. Therefore, this test method may not be representative for coarser materials.

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	10	9.55	4200	4200
2	20	17.36	2000	2000
3	30	25.17	2300	2300
4				
5				

Moisture Content (%) (MCi)	1.74
Wet Wt. of Soil + Cont. (g)	206.98
Dry Wt. of Soil + Cont. (g)	204.61
Wt. of Container (g)	68.49
Container No.	
Initial Soil Wt. (g) (Wt)	130.27
Box Constant	1.000
MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 643	
1900	**19.3**	**117**	**61**	**6.88**	**20.1**



Leighton

R-VALUE TEST RESULTS

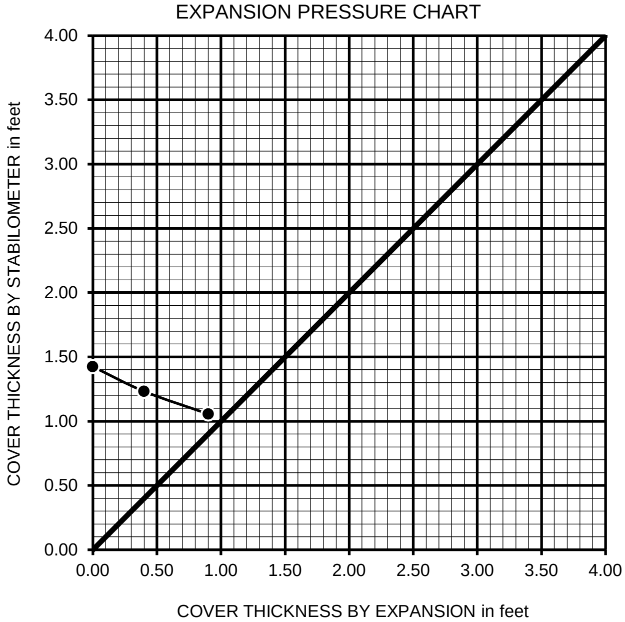
DOT CA Test 301

PROJECT NAME:	Tetra Tech Orange Well	PROJECT NUMBER:	12451.001
BORING NUMBER:	LB-1	DEPTH (FT.):	1.0'
SAMPLE NUMBER:	B-1	TECHNICIAN:	R. Manning
SAMPLE DESCRIPTION:	Brown clayey sand with gravel (SC)g	DATE COMPLETED:	8/1/2019

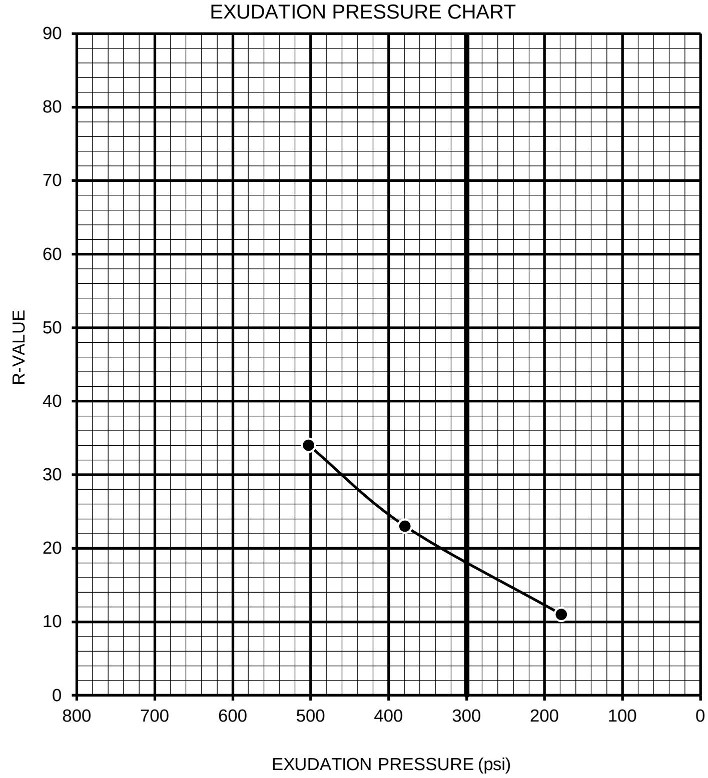
TEST SPECIMEN	a	b	c
MOISTURE AT COMPACTION %	10.8	11.4	12.5
HEIGHT OF SAMPLE, Inches	2.40	2.45	2.50
DRY DENSITY, pcf	123.4	120.9	118.5
COMPACTOR PRESSURE, psi	150	125	100
EXUDATION PRESSURE, psi	502	379	178
EXPANSION, Inches x 10exp-4	27	12	0
STABILITY Ph 2,000 lbs (160 psi)	85	110	130
TURNS DISPLACEMENT	4.00	3.84	4.67
R-VALUE UNCORRECTED	36	23	11
R-VALUE CORRECTED	34	23	11

DESIGN CALCULATION DATA	a	b	c
GRAVEL EQUIVALENT FACTOR	1.0	1.0	1.0
TRAFFIC INDEX	5.0	5.0	5.0
STABILOMETER THICKNESS, ft.	1.06	1.23	1.42
EXPANSION PRESSURE THICKNESS, ft.	0.90	0.40	0.00

EXPANSION PRESSURE CHART



EXUDATION PRESSURE CHART



R-VALUE BY EXPANSION:	38
R-VALUE BY EXUDATION:	18
EQUILIBRIUM R-VALUE:	18

APPENDIX C

PERCOLATION TEST RESULTS

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Boring Percolation Test Data Sheet

Project Number:	12451.001	**Test Hole Number:**	P-1
Project Name:	Orange Well No. 28	**Date Excavated:**	7/18/2019
USCS Soil Type:	Sandy Clay	**Date Tested:**	7/19/2019
Liquid Description:	Tap water	**Depth of boring (ft):**	4
Tested By:	SG	**Radius of boring (in):**	4
Time Interval Standard		**Radius of casing (in):**	1
Start Time for Pre-Soak:	7/18/2019 10:00	**Length of slotted of casing (ft):**	4
Start Time for Standard:	7/19/2019 11:00		
Standard Time Interval Between Readings, mins:	30	**Note:**	

Percolation Data

Reading	Time	Time Interval, Δt (min.)	Initial/Final Depth to Water (ft.)	Initial/Final Water Height, H_0/H_f (in.)	Total Water Drop, Δd (in.)	Percolation Rate (min./in.)	Infiltration Rate (in./hr.)
1	11:50	30	1.63	28.4	5.2	5.81	0.74
	12:20		2.06	23.3			
2	12:20	30	1.64	28.3	5.0	5.95	0.73
	12:50		2.06	23.3			
3	12:50	30	1.63	28.4	5.0	5.95	0.72
	1:20		2.05	23.4			
4	1:20	30	1.65	28.2	4.8	6.25	0.69
	1:50		2.05	23.4			
5	1:50	30	1.65	28.2	4.6	6.58	0.65
	2:20		2.03	23.6			
6	2:20	30	1.63	28.4	4.4	6.76	0.63
	2:50		2.00	24.0			
7	2:50	30	1.63	28.4	4.2	7.14	0.59
	3:20		1.98	24.2			
8	3:20	30	1.65	28.2	4.1	7.35	0.58
	3:50		1.99	24.1			
9	3:50	30	1.64	28.3	4.1	7.35	0.58
	4:20		1.98	24.2			
10	4:20	30	1.63	28.4	4.2	7.14	0.59
	4:50		1.98	24.2			
11	4:50	30	1.65	28.2	4.2	7.14	0.60
	5:20		2.00	24.0			
12	5:20	30	1.63	28.4	4.0	7.58	0.56
	5:50		1.96	24.5			

Observed Infiltration Rate, I (Last Reading)

$I_t = \Delta H*(60r) / \Delta t(r+2H_{avg})$ = 0.6 in./hr.

Suitability Assessment Safety Factor = 2.00

(Worksheet H Factor Category A)

Reference: Technical Guidance Document (TGD) for the Preparation of Conceptual/Preliminary and/or Project Water Quality Management Plans, Appendix VII, December 2013.

Boring Percolation Test Data Sheet

Project Number:	12451.001	**Test Hole Number:**	P-2
Project Name:	Orange Well No. 28	**Date Excavated:**	7/18/2019
USCS Soil Type:	Sandy Clay	**Date Tested:**	7/19/2019
Liquid Description:	Tap water	**Depth of boring (ft):**	4
Tested By:	SG	**Radius of boring (in):**	4
Time Interval Standard		**Radius of casing (in):**	1
Start Time for Pre-Soak:	7/18/2019 10:30	**Length of slotted of casing (ft):**	4
Start Time for Standard:	7/19/2019 11:10		
Standard Time Interval Between Readings, mins:	30	**Note:**	

Percolation Data

Reading	Time	Time Interval, Δt (min.)	Initial/Final Depth to Water (ft.)	Initial/Final Water Height, H_0/H_f (in.)	Total Water Drop, Δd (in.)	Percolation Rate (min./in.)	Infiltration Rate (in./hr.)
1	12:00 / 12:30	30	1.25 / 2.29	33.0 / 20.5	12.5	2.40	1.74
2	12:30 / 1:00	30	1.30 / 2.32	32.4 / 20.2	12.2	2.45	1.73
3	1:00 / 1:30	30	1.25 / 2.18	33.0 / 21.8	11.2	2.69	1.52
4	1:30 / 2:00	30	1.25 / 2.15	33.0 / 22.2	10.8	2.78	1.46
5	2:00 / 2:30	30	1.30 / 2.18	32.4 / 21.8	10.6	2.84	1.45
6	2:30 / 3:00	30	1.25 / 2.15	33.0 / 22.2	10.8	2.78	1.46
7	3:00 / 3:30	30	1.25 / 2.25	33.0 / 21.0	12.0	2.50	1.66
8	3:30 / 4:00	30	1.30 / 2.15	32.4 / 22.2	10.2	2.94	1.39
9	4:00 / 4:30	30	1.25 / 2.11	33.0 / 22.7	10.3	2.91	1.38
10	4:30 / 5:00	30	1.25 / 2.10	33.0 / 22.8	10.2	2.94	1.36
11	5:00 / 5:30	30	1.28 / 2.14	32.6 / 22.3	10.3	2.91	1.40
12	5:30 / 6:00	30	1.25 / 2.10	33.0 / 22.8	10.2	2.94	1.36

Observed Infiltration Rate, I (Last Reading)

$I_t = \Delta H^*(60r) / \Delta t(r+2H_{avg})$ = 1.4 in./hr.

Suitability Assessment Safety Factor = 2.00

(Worksheet H Factor Category A)

Reference: Technical Guidance Document (TGD) for the Preparation of Conceptual/Preliminary and/or Project Water Quality Management Plans, Appendix VII, December 2013.