GEOTECHNICAL EXPLORATION REPORT
ROSETON PLANT RESERVOIR AND BOOSTER STATION
17456 SOUTH ROSETON AVENUE
ARTESIA, CALIFORNIA

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Project No. 13492.001

May 13, 2022

May 13, 2022

Project No.13492.001

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Attention: Mr. Tom Epperson

Subject: Geotechnical Exploration Report
Roseton Plant Reservoir and Booster Pump Station
17456 South Roseton Avenue
Artesia, California

In accordance with your request and authorization, Leighton Consulting, Inc. has prepared this geotechnical exploration report for the proposed Roseton Plant Reservoir and Booster Pump Station at 17456 South Roseton Avenue in the city of Artesia, California. This report is based on the scope of services presented in our proposal dated February 21, 2022. The purpose of our geotechnical exploration was to evaluate subsurface conditions at the site and provide geotechnical recommendations to aid in design and construction of the proposed project.

Our subsurface exploration indicates that the site is underlain by a thin layer of man-made fill associated with construction of the existing improvements at the site and quaternary-age alluvial fan deposits. The fill is up to 5½ feet thick and consisted mainly of silt, silty sand, and silty clay. Below the fill, the alluvial deposits generally consisted of loose to very dense sandy silt and stiff to very stiff silty clay with layers of medium dense sand and silty sand. Groundwater was encountered during our field exploration at depths of 26 and 29 feet below the existing grade. No known active or potentially active faults are mapped to cross the site and the site is not located within an Alquist-Priolo Special Studies Zones. However, significant ground shaking should be anticipated at the site during the expected design life of the proposed structure.

Review of the *Seismic Hazard Zone Report for the Los Alamitos Quadrangle* (CGS, 1998) indicates the subject site is located within an area that has been identified as being potentially susceptible to the occurrence of liquefaction, requiring a site-specific liquefaction evaluation. Based on our analysis, soil layers between 10 to 40 feet may be susceptible to liquefaction during a strong local earthquake. Liquefaction-induced settlement was estimated to be in the range of 3 to 5 inches based on the current groundwater level and 6 to 8 inches if the groundwater rises to its historically high level of 10 feet deep. The seismic differential settlement was estimated to be on the order of two inches over 30 feet. The potential for surface manifestation of liquefaction, such as sand boils and ground fissures, may exist at the site if the historically high groundwater level is considered, due to the relatively shallow and relatively thick layers of the liquefiable soils.

Foundation for the proposed structures should be underlain by compacted fill reinforced with geogrid to provide uniform support and reduce potential for differential settlement and adverse impact from liquefaction. If the proposed structures cannot tolerate the estimated seismic settlement, ground improvement, such as stone columns, ramped aggregate piers or deep soil mixing, may be performed to mitigate and reduce the liquefaction potential of the soils.

Presented in this report are our findings and recommendations for the proposed improvements based on our geotechnical exploration of the site and the anticipated behavior of the soils during and after construction.

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 AND ASSOCIATES, INC.

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1.0 INTRODUCTION

1.1 Site and Proposed Improvements

The Roseton Plant is approximately a 200- by 135-foot rectangular parcel located at 17456 South Roseton Avenue in the city of Artesia, California. The plant currently includes two active groundwater production wells, two MCC's, one SCE transformer, Fe/Mn treatment train for one well, Fe/Mn backwash tank with decant return pumps, well pump to waste facilities, and various site equipment and appurtenances. The approximate site location is shown on Figure 1, *Site Location Map*.

We understand that the proposed improvements include a new 0.75 MG welded steel reservoir, booster pump station, backup power generator, and new entrance sliding gates. In addition, the Fe/Mn backwash tank and skid mounted recycle pump, and MCC for Well No. 2 will be relocated. The existing Well Nos. 1 and 2, three bay chemical building, Fe/Mn treatment system, and SCE transformer may remain in-place if they are appropriately sized for the proposed and existing facilities.

1.2 Purpose and Scope

The purpose of our work was to evaluate the general geotechnical conditions of the site relative to the proposed improvements and provide recommendations to aid in design and construction. Our scope of work 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 5.0.

- *Field Exploration* – We advanced two hollow-stem auger borings (LB-1 and LB-2) to a depth of 51½ feet below existing grade on April 6, 2022. The borings were logged and sampled using Standard Penetration Test (SPT) and 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 borings 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, representative bulk soil samples were also collected from the borings. 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 logs included in Appendix A. After completion of drilling, the borings were backfilled with soil cuttings and compacted by a tamper. The approximate locations of our borings are shown on Figure 2, *Boring Location Map*.

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

  - Moisture content and dry density (ASTM D 2216 and ASTM D 2937);
  - Percent passing No. 200 sieve (ASTM D 1140);
  - Atterberg Limits (ASTM D 4318);
  - Consolidation (ASTM D 2435);
  - Direct shear (ASTM D 3080);
  - R-Value (California Test Method 301); 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 were evaluated and analyzed to develop geotechnical recommendations for the proposed improvements.

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

2.0 GEOTECHNICAL FINDINGS

2.1 Subsurface Soil Conditions

The surface at the boring locations consisted of up to 4 inches of poorly graded gravel. Subsurface soils that underlie the gravel, as encountered during our field exploration, consisted of up to 5½ feet of artificial fill (Af) overlying Quaternary-aged young alluvial fan deposits (Qyf) to the maximum explored depth of 51½ feet.

The fill consisted mainly of silt, silty sand, and silty clay. Below the fill, the alluvial deposits consisted primarily of loose to very dense sandy silt and stiff to very stiff silty clay with occasional interlayers of medium dense sand and silty sand. 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 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. The test results are presented in Appendix B and summarized in Table 1.

Table 1 – Summary of Corrosivity Test Results

Test Parameter	Test Results	General Classification of Hazard
Water-soluble sulfate content	78 ppm	Negligible sulfate exposure to buried concrete (per ACI 318)
Water-soluble chloride content	120 ppm	Non-Corrosive to reinforcing steel of buried concrete (per Caltrans Specifications)
pH	8.89	Moderately alkaline, relatively passive to buried metals
Minimum resistivity (in saturated condition)	995 Ohm-cm	Corrosive to buried ferrous pipes

Based on the laboratory test results, the subsurface soils have low soluble sulfate contents. Therefore, the potential for sulfate attack on concrete is considered low.

However, the onsite soils are considered potentially corrosive to buried ferrous metal in direct contact with the soils.

2.3 Groundwater Conditions

Groundwater was encountered during our field exploration at depths of 26 and 29 feet below ground surface. Review of the *Seismic Hazard Zone Report for the Los Alamitos Quadrangle* (CGS, 1998) indicates that the historically high groundwater in the project site area was reported to be on the order of 10 feet below ground surface.

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 Faulting and Seismicity

No active faults are mapped or known to cross the site and the site is not located within an Alquist-Priolo Earthquake Fault Zone (Bryant and Hart, 2007). The principal seismic hazard at the site is ground shaking resulting from an earthquake occurring along any of several major active and potentially active faults in southern California. Known regional active faults that could produce significant ground shaking at the site include the Puente Hills Blind Thrust and Newport-Inglewood faults located approximately 5.0 mile and 6.4 miles, respectively, from the site. The San Andreas Fault is the largest fault in the region and is located approximately 41 miles 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 SEAOC/OSHPD Seismic Design Maps Tool (https://seismicmaps.org/) to obtain seismic design parameter values from the United States Geological Survey (USGS), the peak ground acceleration for the Maximum Considered Earthquake (MCE_G) adjusted for the Site Class effects (PGA_M) is 0.727g. Based on the USGS online unified hazard tool program (USGS, 2020a), the modal seismic event is Moment Magnitude (M_W) 7.3 at a distance of 6.1 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. The potential for seismic hazards at the site is discussed below.

Liquefaction Potential – Liquefaction is a seismic phenomenon in which loose, saturated, fine-grained 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) high-intensity ground motion. Effects of liquefaction on level ground can include sand boils, settlement, and bearing capacity failures below structural foundations.

Based on the *Seismic Hazard Zone Report for the Los Alamitos Quadrangle* (CGS, 1998), the site is located within an area identified by the State of California as being potentially susceptible to the occurrence of liquefaction. We performed evaluation for liquefaction potential at the site and its effects on the proposed improvements in accordance with guidelines in the CGS Special Publication 117A (CGS, 2008). The following input parameters were utilized in our evaluations:

- Historically high groundwater level of 10 feet below existing grade;
- Peak horizontal ground acceleration of 0.727g; and
- Modal Moment Magnitude of 7.3.

Our analysis, presented in Appendix D, *Liquefaction Analysis*, identifies layers of potentially liquefiable soils at depths ranging from 10 to 40 feet. The potential for surface manifestation of liquefaction (e.g., sand boils and ground fissures) may exist at the site because the potentially liquefiable layers are relatively shallow and relatively thick, should the groundwater rises to the historically high level.

The settlements of the potentially liquefiable layers were estimated to result in a cumulative settlement of 3 to 5 inches based on the current groundwater level and 6 to 8 inches based on the historically high groundwater level. The seismic differential settlement was estimated to be on the order of two inches over 30 feet.

Lateral Spreading – For lateral spreading to occur, a continuous, laterally unconstrained liquefiable zone must be free to move along gently sloping ground toward an unconfined area. The site is relatively flat, therefore, the potential for lateral spreading is considered low.

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, we consider the potential for earthquake-induced flooding of the site to be low.

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 CONCLUSIONS AND RECOMMENDATIONS

Based on our analysis, the seismically induced settlement was estimated to be on the order of 3 to 5 inches based on the current groundwater level and 6 to 8 inches considering the historically high groundwater level, with a differential settlement estimated to be on the order of two inches over 30 feet. The potential for surface manifestation of liquefaction, such as sand boils and ground fissures, may exist at the site if the groundwater rises to its historically high level.

Foundation for the proposed structures should be underlain by compacted fill reinforced with geogrid to provide a uniform support and reduce potential for differential settlement and potential adverse impact from liquefaction. The differential settlement that has been estimated due to liquefaction and the resulting angular distortion is recommended to be reviewed by the structural engineer to determine if any special detailing or other design techniques are required for structural connections to ensure the water storage tank and other structures can sufficiently withstand the estimated level of distortion without structural failure.

The intent of the above recommendations for site preparation relative to liquefaction is to maintain structural integrity but may not maintain serviceability of the facility without potentially significant repairs should liquefaction occur. If the potential for loss of the structure is not acceptable to the owner, ground improvement, such as stone columns, ramped aggregate piers or deep soil mixing, may be performed to mitigate and reduce the liquefaction potential of the soils. However, considering that the more severe consequences of liquefaction require a substantial rise in groundwater elevation, the likelihood is that such conditions will not occur at the time of the design seismic event, an earthquake that statistically has a relatively low probability of occurrence.

Presented below are the geotechnical recommendations for the proposed project. These recommendations are based upon the exhibited geotechnical engineering properties of the soils and their anticipated response both during and after construction. These recommendations are considered minimal and may be superseded by more restrictive requirements of the civil and structural engineers, Golden State Water Company, and the City of Artesia.

3.1 Site Grading

All site grading should be performed in accordance with the applicable local codes and in accordance with the project specifications that are prepared by the appropriate design professional.

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 improvements, including utility lines, should be identified prior to the start of grading and abandoned or relocated as necessary. Trenches resulted from removal of existing improvements should be excavated to competent materials and properly backfilled under the observation and testing of the geotechnical engineer.

3.1.2 Overexcavation and Recompaction

Foundation for the proposed structures should be underlain by compacted fill reinforced with geogrid to provide a uniform support and reduce potential for differential settlement and adverse impact from liquefaction. 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. The compacted fill should be reinforced with placement of three layers of geogrid starting from bottom of removal and each geogrid layer separated by 8 inches of soils (see Section 3.1.5). If ground improvement is performed, subgrade preparation may be required after completion of the ground improvement but the overexcavation and recompaction recommended above is not considered necessary.

Pavement areas, driveway, and concrete flatwork should be underlain by a minimum 1 foot of compacted fill. Local conditions may be encountered that could require additional overexcavation beyond the above noted minimum to obtain an acceptable subgrade. The actual depths and lateral extents of remedial grading will be determined by Leighton, based on subsurface conditions encountered during grading.

3.1.3 Subgrade Preparation

Prior to placing fill materials, the subgrade should be scarified to a minimum depth of 8 inches, moisture conditioned, and proof rolled. 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, construction debris or oversized material larger than 6 inches. Any imported soils should have an Expansion Index less than 50 and 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 or dried as necessary to slightly percent above moisture optimum and compacted to a minimum of 90 percent of the maximum dry density as determined by ASTM D 1557.

3.1.5 Geogrid Placement

A minimum three layers of reinforcement geogrid should be placed below the proposed structures at vertical intervals of approximately 8 inches with the lowest geogrid placed at bottom of the overexcavation. The geogrid layers should cover the entire structure footprint and extend a minimum 5 feet beyond the footprint, where feasible.

The geogrid should comply with *Standard Specifications for Public Works Construction* (Greenbook) Table 213.5.2 (D) Biaxial S2 or approved equivalent. Installation of the geogrid should be performed in accordance with the manufacturer's guidelines. In general, geogrid should be placed on smooth surfaces of compacted fill and installed by unrolling, not by dragging. The end edges of geogrid roll should be nailed with 6-inch long "U" staples and/or other approved fasteners. The geogrid should be pulled to remove any slack and compacted fill should be placed from the fastened geogrid side to unfastened geogrid side. Each geogrid should be overlapped by at least 12 inches horizontally. Construction equipment should not be contacting the geogrid directly. The geogrid should be placed continuously under the proposed foundation footprint.

In cases of damaged geogrid, the geogrid should be carefully cut and repaired by overlapping geogrid patch at least one foot on both sides of cut geogrid or reconnecting the existing geogrid. Construction sequencing of underground utilities should take the geogrid layers into considerations. The geogrid layers may be deepened to accommodate installation of shallow utility lines.

3.2 Foundation Design Parameters

Following site grading recommended in Section 3.1, the proposed structures may be supported on a mat foundation system or a conventional shallow foundation system. Design recommendations are presented in the following subsections.

3.2.1 Mat Foundation

A mat foundation bearing on properly compacted fill may be designed using a maximum allowable bearing capacity of 2,500 psf and a coefficient of vertical subgrade reaction of 40 pounds per cubic inch (pci). The bearing capacity may be increased by one-third for wind or seismic loading. The perimeter of the mat foundation should have a minimum embedment of 12 inches below the lowest adjacent grade.

Total and differential settlements of the mat foundation due to static loads are expected to be on the order of 1 inch and ½ over a distance of 30 feet, respectively. Seismic settlement due to liquefaction should also be considered in design.

3.2.2 Spread Footings

An allowable bearing capacity of 2,500 pounds per square foot (psf) may be used for footing design. The footings should have a minimum width of 12 inches and a minimum embedment of 18 inches. A one-third increase in the bearing value for short duration loading, such as wind or seismic forces may be used.

Total and differential settlements due to static loads are expected to be on the order of 1 inch and ½ over a distance of 30 feet, respectively. Seismic settlement due to liquefaction should also be considered in design.

3.2.3 Lateral Load Resistance

Lateral loads can be resisted by soil friction and by the passive resistance of the soils. A coefficient of friction of 0.40 can be used between the footings/ floor slab and the supporting soils. The passive pressure of undisturbed natural soils or engineered fill is presented in Table 4 of Section 3.7.

3.3 Ground Improvement

The soils at the site contain layers that are susceptible to liquefaction that may result in liquefaction-induced settlement and surface manifestation. Ground improvement may be performed to reduce the liquefaction potential of the subsurface soils.

In-place ground improvement techniques, such as stone columns or rammed aggregate piers, may be used to mitigate the potentially liquefiable soils and reduce the settlement potential. These techniques basically improve the strength of the soils and/or provide drainage paths for pore water pressure dissipation. The columns or piers are installed in a grid pattern with a center-to-center spacing of typically 8 to 10 feet and mainly intended to reduce the potential for liquefaction and foundation settlement. Design of the ground improvement will require consulting with a specialty contractor.

The target mitigation goal (design criteria) is to reduce the seismic settlement and surface manifestation of liquefaction to an acceptable level to support the proposed structures on a shallow foundation system upon implementation of the mitigation measures for liquefaction. Based on our liquefaction analysis (Appendix D), the depth of the soils to be treated is recommended to be at least 35 feet below the existing grade. Upon implementation of ground improvement, the seismic settlement is estimated to be on the order of 1½ inches or less, with a differential settlement estimated to be on the order of ½ inch over 30 feet.

A site-specific supplemental geotechnical exploration is recommended to include cone penetration test (CPT) soundings prior to and after ground improvement is implemented. The CPTs provide a continuous record of the subsurface stratigraphy of the subsoil and is a cost-effective method to evaluate ground improvement. The geotechnical engineer should constantly monitor the effectiveness of any testing/evaluation program and modify the program if necessary.

3.4 Slab-On-Grade

From a geotechnical standpoint, we recommend slab-on-grade floor slab be a minimum 5 inches thick with No. 3 rebar placed at the center of the slab at 18 inches on center in each direction. The structural engineer should design the actual thickness and reinforcement based on anticipated loading conditions. Where moisture-sensitive floor coverings or equipment is planned, the slabs should be protected by a minimum 10-mil thick vapor barrier between the slab and subgrade.

Exterior concrete slabs that are not subject to vehicular loading, such as patio slabs and sidewalks, should be at least 4 inches thick. We suggest that the exterior concrete slabs be reinforced using No. 3 rebar, 18 inches on center in both directions, placed at mid-thickness.

Minor cracking of concrete after curing due to drying and shrinkage is normal and should be expected; however, concrete is often aggravated by a high water/cement ration, 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 or low water/cement ratios can reduce the potential for shrinkage cracking. Additionally, our experience indicates that the use of reinforcement in slabs and foundations can generally reduce the potential for concrete cracking.

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.

3.5 Seismic Design Parameters

Moderate to strong ground shaking due to seismic activity is expected at the site during the life span of the project. The potentially liquefiable layer is 10 feet thick or less based on the current groundwater level of 26 to 29 feet deep. As such, Site Class D is being used for seismic design

The 2019 CBC code-based seismic design parameters are summarized in Table 2.

Table 2 – Code-Based 2019 CBC Seismic Design Parameters

Categorization/Coefficients	Design Value
Site Latitude	33.78184
Site Longitude	-118.08683
Mapped Spectral Response Acceleration at 0.2s Period, S_S	1.549g
Mapped Spectral Response Acceleration at 1s Period, S_1	0.554g
Short Period Site Coefficient at 0.2s Period, F_a	1.0
Long Period Site Coefficient at 1s Period, F_v	1.75
Adjusted Spectral Response Acceleration at 0.2s Period, S_{MS}	1.549g
Adjusted Spectral Response Acceleration at 1s Period, S_{M1}	0.967g[1]
Design Spectral Response Acceleration at 0.2s Period, S_{DS}	1.033g
Design Spectral Response Acceleration at 1s Period, S_{D1}	0.645g[1]
Design Peak Ground Acceleration, PGA_M	0.727g

[1]Per Exception 2 in Section 11.4.8 of ASCE 7-16, seismic response coefficient C_S to be determined by Eq. 12.8-2 for values of $T \leq 1.5T_s$ and taken as equal to 1.5 times the value computed in accordance with either Eq. 12.8-3 for $T_L \geq T > 1.5T_s$ or Eq. 12.8-4 for $T > T_L$

3.6 <u>Response Spectra</u>

Site-specific response spectra were developed for the site based on a uniform-hazard approach in accordance with ASCE 7-16 and the 2019 California Building Code. The uniform-hazard approach assumes that the same level of hazard is uniformly applied to the entire response spectra. The spectral values were developed for a seismic event associated with the Maximum Considered Earthquake (MCEG) with a return period of 2,475 years (2 percent chance of exceedance in 50 year). Response spectral values were calculated for 5 percent damping and modified for other damping ratios (0.5 and 2 percent) using damping/spectral amplification factors (Rezaeian et al., 2012). Recommended site-specific response spectra are presented on Figure 3 as tripartite plots and the digitized values are presented in Table 3.

Table 3 – Spectral Accelerations

Period (second)	Spectral Acceleration (g)		
	0.5% Damping	2% Damping	5% Damping
0.01	0.71	0.71	0.71
0.05	1.02	0.93	0.88
0.10	2.14	1.61	1.25
0.20	2.99	2.15	1.59
0.30	3.03	2.18	1.61
0.40	2.85	2.06	1.54
0.50	2.62	1.91	1.43
0.60	2.35	1.73	1.32
0.70	2.16	1.60	1.23
0.80	1.97	1.47	1.14
0.90	1.78	1.34	1.05
1.00	1.63	1.24	0.97
2.00	0.84	0.65	0.52
3.00	0.53	0.41	0.33
4.00	0.38	0.30	0.24
5.00	0.31	0.24	0.20
6.00	0.25	0.20	0.16
7.00	0.22	0.17	0.14
8.00	0.19	0.15	0.12
9.00	0.16	0.13	0.10
10.00	0.14	0.11	0.09

3.7 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 4 – Equivalent Fluid Pressures

Condition	Level Backfill
Active	37 pcf
At-Rest	57 pcf
Passive	360 pcf (Maximum of 3,600 psf)

Retaining walls retaining more than 6 feet of soil should consider a seismic earth pressure increment with an inverted triangular distribution of 20 psf/foot in addition to the active earth pressure provided above. 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 0.40 may be used at the concrete and soil interface. The lateral passive resistance can be taken into account only if it is ensured that the soil against embedded structures will remain intact with time.

3.8 Cement Type and Corrosion Protection

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. Common Type II cement may be used for concrete construction onsite and the concrete should be designed in accordance with CBC requirements. However, if the concrete is expected to be in contact with recycled water, Type V cement should be used.

Based on the available laboratory test results, the onsite soil is considered severely corrosive to ferrous metals. Ferrous pipe should be avoided by using high-density polyethylene (HDPE), polyvinyl chloride (PVC) or other non-ferrous pipe when possible. Ferrous pipe, if used, should be protected by polyethylene bags, tape or coatings, di-electric fittings or other means to separate the pipe from onsite soils. The corrosion information presented in this report should be provided to your underground utility subcontractors.

3.9 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 the R-value of 50 for different Traffic Indices (TI) and the minimum pavement thickness is presented in the following table. R-value of the tested near-surface soil sample was 63. The pavement design was performed using the method in the Caltrans *Highway Design Manual*.

Table 5 – Asphalt Concrete Pavement Sections

Traffic Index (TI)	Asphalt Concrete (inches)	Base Course (inches)
5 or less	3.0	4.5
6	3.5	5.0
7	4.5	5.0
8	5.0	5.5

Portland cement concrete (PCC) pavement may also be considered. The PCC pavement sections should be a minimum 6 inches thick, underlain by 4 inches of aggregate base and provided with crack-control joints spaced no more than 8 feet on-center each way to control where cracks develop. As a minimum, we suggest concrete pavement be reinforced using No. 3 rebar, 18 inches on center in both directions, placed at mid-thickness. Concrete reinforcement should be designed by the structural engineer for appropriate loading conditions.

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.

3.10 Temporary Excavation and Shoring

All temporary excavations, including utility trenches and footing 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.

No surcharge loads should be permitted within a horizontal distance equal to the height of cut or 5 feet, whichever is greater from the top of the cut, unless the cut is shored appropriately. Excavations that extend below an imaginary plane inclined at 45 degrees below the edge of any adjacent existing site foundation should be properly shored to maintain support of the adjacent structure.

Typical cantilever shoring should be designed based on the active fluid pressure presented for retaining walls in Section 3.7. 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 24H, where H is equal to the depth of the excavation being shored.

During construction, the soil conditions should be regularly evaluated to verify that conditions are as anticipated. The contractor shall 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.

3.11 Trench Backfill

Utility trenches can be backfilled with the onsite material, provided it is free of debris, organic material and oversized material (greater than 8 inches in diameter). 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 be densified in-place using mechanical compaction equipment with care to not damage the pipe. The native backfill should be placed in lifts, moisture conditioned as necessary to achieve moisture content slightly above optimum, and mechanically compacted using a minimum standard of 90 percent relative compaction. The maximum lift thickness should also be determined based on the compaction equipment used in accordance with the latest edition of the *Standard Specifications for Public Works Construction*. Where utility trenches cross underneath building footing, the trenches should be plugged by a minimum of 2 feet of onsite soil or sand/cement slurry to reduce the potential for water intrusion underneath the slab.

3.12 Surface Drainage

Surface drainage should be designed to direct water away from foundations and toward approved drainage devices. Irrigation of landscaping should be controlled to maintain, as much as possible, consistent moisture content sufficient to provide healthy plant growth without overwatering.

3.13 Additional Geotechnical Services

The geotechnical recommendations presented in this report are based on subsurface conditions as interpreted from limited available data. Leighton Consulting should also review the grading and foundation plans, when available, to comment on the geotechnical aspects. Our recommendations should be revised, as necessary, based on future plans and incorporated into the final design plans and specifications

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

- Grading and excavation of the site;
- Subgrade preparation;
- Compaction of all fill materials;
- Utility trench backfilling and compaction;
- Foundation excavation and slab-on-grade preparation;
- During ground improvement operations, if performed;
- Pavement subgrade and base preparation;
- Placement of asphalt concrete and/or concrete; and
- When any unusual conditions are encountered.

4.0 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.

5.0 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, you can benefit from a lowered exposure to problems associated with subsurface conditions at project sites and development of them 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 herein, contact your GBA-member geotechnical engineer. Active engagement in GBA exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Understand the Geotechnical-Engineering Services Provided for this Report

Geotechnical-engineering services typically include the planning, collection, interpretation, and analysis of exploratory data from widely spaced borings and/or test pits. Field data are combined with results from laboratory tests of soil and rock samples obtained from field exploration (if applicable), observations made during site reconnaissance, and historical information to form one or more models of the expected subsurface conditions beneath the site. Local geology and alterations of the site surface and subsurface by previous and proposed construction are also important considerations. Geotechnical engineers apply their engineering training, experience, and judgment to adapt the requirements of the prospective project to the subsurface model(s). Estimates are made of the subsurface conditions that will likely be exposed during construction as well as the expected performance of foundations and other structures being planned and/or affected by construction activities.

The culmination of these geotechnical-engineering services is typically a geotechnical-engineering report providing the data obtained, a discussion of the subsurface model(s), the engineering and geologic engineering assessments and analyses made, and the recommendations developed to satisfy the given requirements of the project. These reports may be titled investigations, explorations, studies, assessments, or evaluations. Regardless of the title used, the geotechnical-engineering report is an engineering interpretation of the subsurface conditions within the context of the project and does not represent a close examination, systematic inquiry, or thorough investigation of all site and subsurface conditions.

Geotechnical-Engineering Services are Performed for Specific Purposes, Persons, and Projects, and At Specific Times

Geotechnical engineers structure their services to meet the specific needs, goals, and risk management preferences 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.

Likewise, geotechnical-engineering services are performed for a specific project and purpose. For example, it is unlikely that a geotechnical-engineering study for a refrigerated warehouse will be the same as one prepared for a parking garage; and a few borings drilled during a preliminary study to evaluate site feasibility will not be adequate to develop geotechnical design recommendations for the project.

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

- for a different client;
- for a different project or purpose;
- 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, the reliability of a geotechnical-engineering report can be 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 you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying the recommendations in it. A minor amount of additional testing or analysis after the passage of time – if any is required at all – could prevent major problems.

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read the report in its entirety. Do not rely on an executive summary. Do not read selective elements only. *Read and refer to the report in full.*

You Need to Inform Your Geotechnical Engineer About Change

Your geotechnical engineer considered unique, project-specific factors when developing the scope of study behind this report and developing the confirmation-dependent recommendations the report conveys. Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the elevation, configuration, location, orientation, function or weight of the proposed structure and the desired performance criteria;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project or site 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.*

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

Before construction begins, geotechnical engineers explore a site's subsurface using various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing is performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgement 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 through project completion to obtain 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 <u>not</u> final, because the geotechnical engineer who developed them relied heavily on judgement and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* exposed 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 continuing 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 available 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-phase observations.

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 information 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. 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. This happens in part because soil and rock on project sites are typically heterogeneous and not manufactured materials with well-defined engineering properties like steel and concrete. 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 <u>Not</u> 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 provide 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 obtained your own environmental information about the project site, ask your geotechnical consultant for a recommendation on how to find environmental risk-management guidance.

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, the engineer's services were not designed, conducted, or intended to prevent 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 <u>not</u> 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 <u>not</u> building-envelope or mold specialists.***

GEOPROFESSIONAL BUSINESS ASSOCIATION

Telephone: 301/565-2733
e-mail: info@geoprofessional.org www.geoprofessional.org

Project: 13492.001	Eng/Geol: DJC
Scale: 1 " = 2,000 '	Date: April 2022

Reference: Copyright:© 2013 National Geographic Society, i-cubed

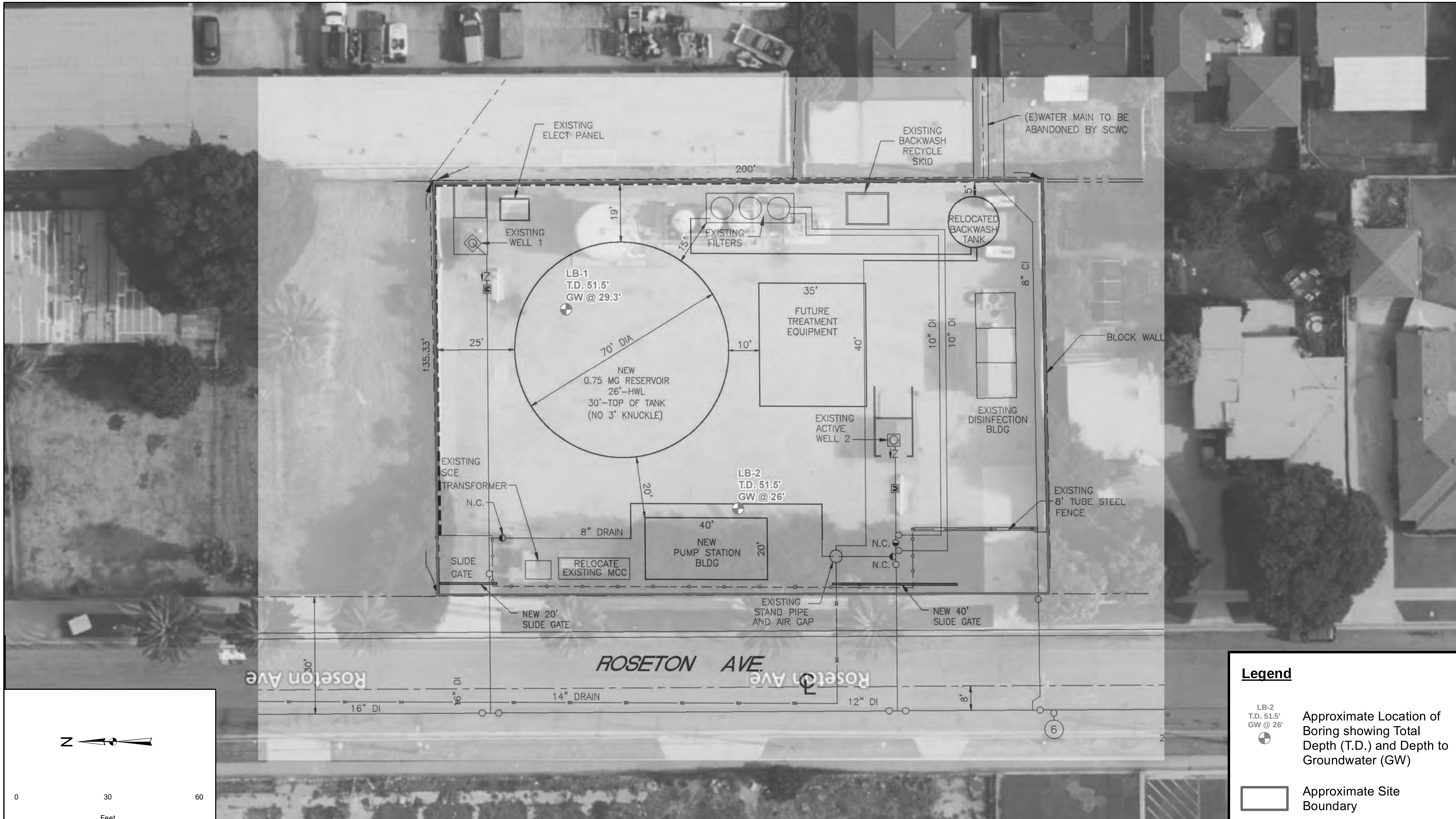
SITE LOCATION MAP

Roseton Plant Reservoir and Booster Pump Station
17456 South Roseton Avenue
Artesia, CA

FIGURE 1

Leighton

Map Saved as J:\Drafting\13492\001\Maps\13492.001_F01_SLM_2022-03-30.mxd on 4/21/2022 11:08:20 AM Author: KVM (kmanchikanti)



Legend

LB-2 T.D. 51.5' GW @ 26' — Approximate Location of Boring showing Total Depth (T.D.) and Depth to Groundwater (GW)

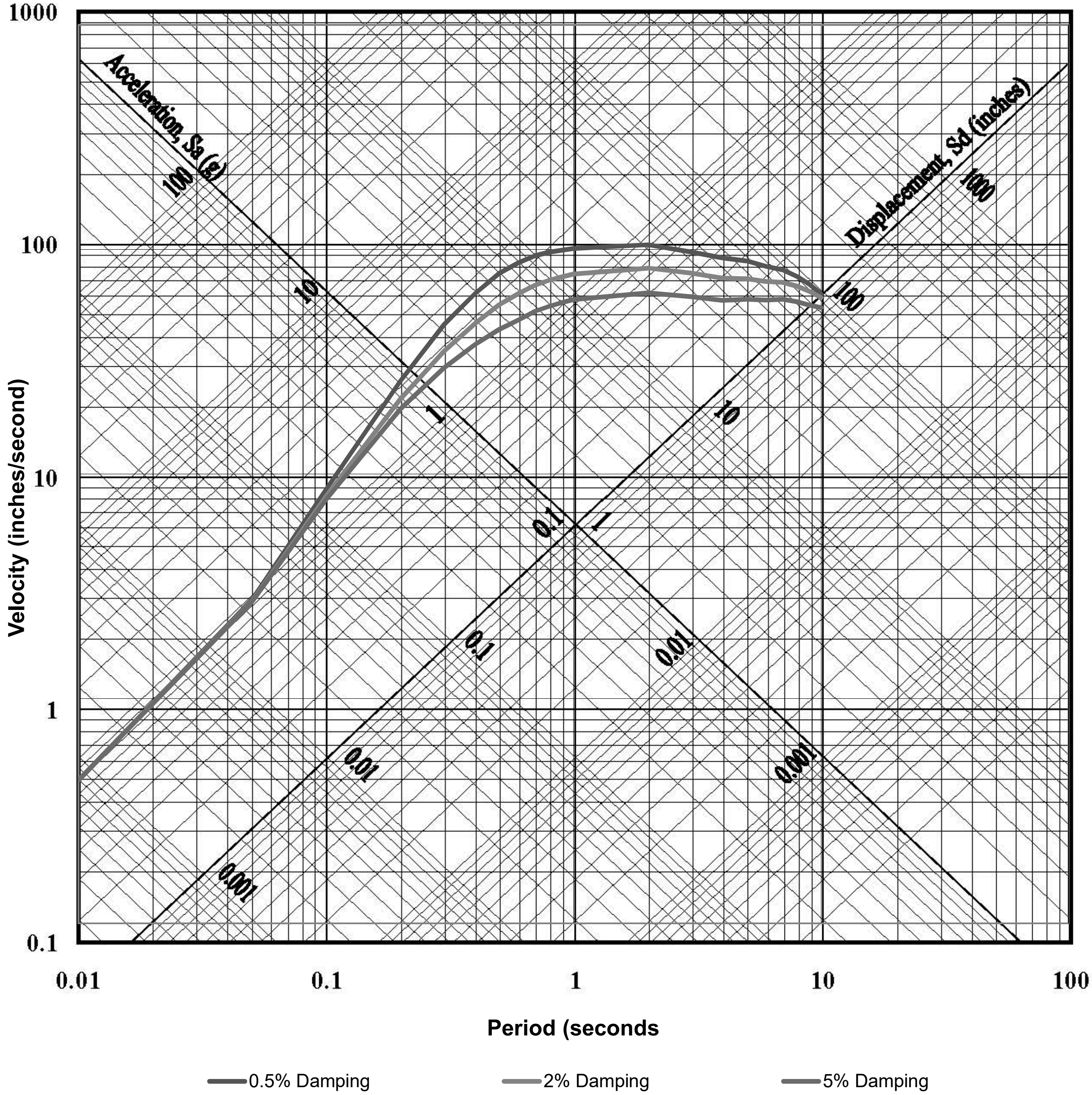
Approximate Site Boundary

Project: 13492.001	Eng/Geol: DJC	**EXPLORATION LOCATION MAP** Roseton Plant Reservoir and Booster Pump Station 17456 South Roseton Avenue Artesia, CA	**FIGURE 2**
Scale: 1 " = 30 '	Date: May 2022		Leighton
Reference: © 2022 Microsoft Corporation © 2022 Maxar ©CNES (2022) Distribution Airbus DS © 2022 TomTom			

Map Saved as J:\Drafting\13492\001\Maps\13492.001_F02_ELM_2022-03-23.mxd on 5/4/2022 9:33:01 PM Author: KVM (kmanchikanti)

SITE-SPECIFIC PROBABILISTIC RESPONSE SPECTRA

RISK TARGETED MAXIMUM CONSIDERED EARTHQUAKE



RESPONSE SPECTRA TRIPARTITE Roseton Plant Reservoir and Booster Pump Station City of Artesia, California	**PROJECT NAME:** Roseton Plant Improvements **PROJECT NUMBER:** 13492.001 **DESIGNED/CHECKED BY:** EDB/CD **DATE:** 5/1/2022	Leighton **FIGURE 3**

APPENDIX A

GEOTECHNICAL BORING LOGS

GEOTECHNICAL BORING LOG LB-1

Project No.	3492.	**Date Dri ed**	4-6-22
Project	Roseton Plant Reservoir an Booster Pump Station	**Logged By**	EDB
Drilling Co.	2R Drilling, Inc.	**Ho e Diameter**	"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground E evation**	56'
Location	*See Figure ri g Locati Map*	**Samp ed By**	EDB

Elevation eet	Dept eet	Graphic Log (N S)	Attitudes	Sample No.	B ows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil C ass. U.S.C.S.)	**SOIL DESCRIPTION** *This S il Descripti applies ly t a l cati of the expl rati at the time of sampli g. Subsurface c diti s may differ at ther l cati s and may change ith time. The descriptio is a simplificatio f the actual c diti s e countered. Tra sit io s bet een s il types may be gradual.*	Type of Tests
	0			B-				GP ML	**Artificia ill Af** ': Poorly Gra e GRAVEL; gray, slightly moist, coarse gravel, few fine to coarse san @ .33': SILT; ark gray, moist, non-plastic, mica	CR
55				R-	2			CL-ML	**uaternary-Aged A uvial an Deposits yf** @2': SILTY CLAY; very stiff, olive brown an brown, moist, low plasticity, Fe stains, slightly micaceous	
	5			R-2	5			ML	@5': Stiff @5.5': SANDY SILT; ark olive brown an grayish brown, moist, low plasticity, fine san , micaceous	CN, DS
50				R-3	5			CL-ML	@7.5': SILTY CLAY; very stiff, olive brown an brown, moist, low plasticity, Fe stains, 2-inch thick interbe of SILT; gray, moist, low plasticity, fine san , micaceous	
	10			R-4	3 4 4		27	SP-SM CL-ML	@ ': Poorly Graded SAND with SILT; loose, gray, moist, fine san , Fe stains @ .5': SANDY SILTY CLAY; me ium stiff, brown, moist, low plasticity, fine san , Fe stains	
45										
	15			S-5	2 5		27	SM	@ 5': SANDY SILT; me ium dense, olive brown an gray, moist, non-plastic, fine san , Fe stains, micaceous	AL
40										
	20			R-	3	4	20	ML	@2 ': Blueish gray an gray, very moist, non-plastic, fine san , very micaceous	
35										
	25			S-7	2 3 5		30	CL-ML	@25': SANDY SILTY CLAY; stiff, olive brown an grayish brown, wet, low plasticity, fine san , slightly micaceous @26': Groundwater encountere	AL
30										
	30									

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

TYPE O TESTS:
- -200 % INES 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 E UIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

Leighton

GEOTECHNICAL BORING LOG LB-1

Project No.	3492.	**Date Dri ed**	4-6-22
Project	Roseton Plant Reservoir an Booster Pump Station	**Logged By**	EDB
Drilling Co.	2R Drilling, Inc.	**Ho e Diameter**	"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground E evation**	56'
Location	*See Figure ri g Locati Map*	**Samp ed By**	EDB

Elevation eet	Dept eet	Graphic Log (N S)	Attitudes	Sample No.	B ows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil C ass. (U.S.C.S.)	SOIL DESCRIPTION *This S il Descripti applies ly t a l cati of the expl rati at the time of sampli g. Subsurface c diti s may differ at ther l cati s and may change ith time. The descriptio is a simplificatio f the actual c diti s e countered. Tra sit io s bet een s il types may be gradual.*	Type of Tests
25	30			R-	4 5 7	7	28	SW ML	@3 ': Well Gra e SAND; me ium dense, ark yellowish brown an gray, wet, fine to coarse san @3 .5': SANDY SILT; me ium ense, blueish gray, wet, non-plastic, micaceous, fine sand	AL
20	35			S- A S- B	4 2 4			SP-SM CL-ML	@35': Poorly Gra e SAND with SILT; ense, yellowish brown an grayish brown, wet, fine san @36': SANDY SILTY CLAY; har , yellowish brown an olive brown, wet, low plasticity, fine san	
15	40			S- A S- B	4			SP-SM ML	@4 ': Poorly Gra e SAND with SILT; ense, yellowish brown an grayish brown, wet, fine san , gra ing to SANDY SILT @4 .2': SILT; blueish gray, very moist, low plasticity, oxi ize	
10	45			S- A S- B	7 26			SP-SM	@45': SANDY SILT; very dense, olive an blueish gray, wet, non-plastic, fine sand, micaceous, trace clay @45.75': Poorly Gra e SAND with SILT; very ense, blueish gray, wet, fine san , Fe stains, slighlty micaceous	
5	50			S- 2	2			CL-ML	@5 ':SANDY SILTY CLAY; har , light brown gray an olive gray, wet, low plasticity, fine san , slightly micaceous	
									Tota Dept = 51.5 feet **Groundwater encountered at 26 feet** **Backfilled wit soil cuttings**	
0	55									
	60									

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

TYPE O TESTS:

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

Leighton

***** This lo is of a e or by Lei ht on nd should no be used s st nd- l one document. *****

GEOTECHNICAL BORING LOG LB-2

Project No.	3492.	**Date Dri ed**	4-6-22
Project	Roseton Plant Reservoir an Booster Pump Station	**Logged By**	EDB
Drilling Co.	2R Drilling, Inc.	**Ho e Diameter**	"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground E evation**	56'
Location	*See Figure ri g Locati Map*	**Samp ed By**	EDB

Elevation eet	Dept eet	Graphic Log (N S)	Attitudes	Sample No.	B ows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil C ass. (U.S.C.S.)	SOIL DESCRIPTION *This S il Descripti applies ly t a l cati of the expl rati at the time of sampli g. Subsurface c diti s may differ at ther l cati s and may change ith time. The descriptio is a simplificatio f the actual c diti s e countered. Tra sit io s bet een s il types may be gradual.*	Type of Tests
	0			B-				GP CL-ML	**Artificia ill Af** ': Poorly Gra e GRAVEL; gray, slightly moist, coarse gravel, few fine to coarse san @ .33': SANDY SILTY CLAY; very stiff, brown, moist, low plasticity, fine san , trace rootlets	RV
55										
								ML	@4': SANDY SILT; yellowish brown, moist, non-plastic, fine san	
	5			R-	4 7			SM CL-ML	@5': SILTY SAND; me ium ense, mottle dark brown an brown, moist, fine san , trace rootlets	
50									**uaternary-Aged A uvial an Deposits yf** @5.5': SANDY SILTY CLAY; very stiff, olive brown an brown, moist, low plasticity, fine san	
				R-2	5	25	3	ML	@7.5': SANDY SILT; me ium ense, olive brown an brown, moist, non-plastic, fine sand, micaceous, trace clay	
	10			S-3	2		28	SP-SM CL	@ ': Poorly Graded SAND with SILT; loose, yellowish gray, moist, fine sand, slightly micaceous @ .33': Lean CLAY with SILT; me ium stiff, brown, moist, me ium plasticity, slightly micaceous	AL
45										
	15			R-4	4 5	5	23	SM	@ 5': SILTY SAND; me ium dense, yellowish brown an gray, moist, non-plastic, fine san , Fe stains, very micaceous, /1 " laminations of Lean CLAY; ark brown, moist, low plasticity	-2 , CN
40										
	20			S-5	5 2		23	ML	@2 ': SANDY SILT; ense, olive gray an gray, very moist, non-plastic, fine san , very micaceous	
35										
	25			S-	3 4 5			CL-ML	@25': SANDY SILTY CLAY; stiff, olive brown an grayish brown, wet, low plasticity, fine san , slightly micaceous	
30										
	30								@29': Groundwater encountere	

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

TYPE O TESTS:

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

Leighton

GEOTECHNICAL BORING LOG LB-2

Project No.	3492.	**Date Dri ed**	4-6-22
Project	Roseton Plant Reservoir an Booster Pump Station	**Logged By**	EDB
Drilling Co.	2R Drilling, Inc.	**Ho e Diameter**	"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground E evation**	56'
Location	*See Figure ri g Locati Map*	**Samp ed By**	EDB

Elevation eet	Dept eet	Graphic Log (N S)	Attitudes	Sample No.	B ows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil C ass. (U.S.C.S.)	SOIL DESCRIPTION *This S il Descripti applies ly t a l cati of the expl rati at the time of sampli g. Subsurface c diti s may differ at ther l cati s and may change ith time. The descriptio is a simplificatio f the actual c diti s e countered. Tra sit io s bet een s il types may be gradual.*	Type of Tests
25	30			S-7	2		28	ML	@3 ': SILT; loose, ark gray, wet, non-plastic, very micaceous, sporadic -inch interbeds of Lean CLAY; very ark gray, moist, me ium plasticity	
20	35			S-	3 5				@35': SANDY SILT; me ium dense, ark blueish gray, wet, non-plastic, very micaceous, fine san @36': Olive brown an brown, Fe stains, trace clay	
15	40			S-					@40': Dense @4 .33': SILT; ense, light blueish gray, very moist, low plasticity, micaceous	
10	45			S-	5 21			SM	@45': SILTY SAND; very ense, blueish gray an ark gray, wet, non-plastic, very micaceous	
5	50			S- A S- B	7 2 4			 ML	@5 ': Dense, ark gray, micaceous @5 . ': SANDY SILT; gray, wet, non-plastic, fine san , micaceous	
									Tota Dept = 51.5 feet **Groundwater encountered at 29 feet** **Backfi ed with soi cuttings**	
0	55									
	60									

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

TYPE O TESTS:
- -200 % INES 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 E UIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

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***** This lo is of a e or by Lei ht on nd should no be used s st nd- l one document. *****

APPENDIX B

LABORATORY TEST RESULTS

Boring No.	LB-2								
Sample No.	R-4								
Depth (ft.	15.0								
Sample Type	Ring								
Soil Identification	Olive silty sand (SM)								
Moisture Correction									
Wet Weight of Soil + Container (g)	0.00								
Dry Weight of Soil + Container (g	0.00								
Weight of Container (g)	1.00								
Moisture Content (%)	0.00								
Sample Dry Weight Determination									
Weight of Sample + Container (g	402.94								
Weight of Container (g)	107.22								
Weight of Dry Sample (g)	295.72								
Container No.:									
After Wash									
Method (A or B)	A								
Dry Weight of Sample + Cont. (g	336.73								
Weight of Container (g)	107.22								
Dry Weight of Sample (g)	229.51								
% Passing No. 200 Sieve	**22.4**								
% Retained No. 200 Sieve	77.6								

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PERCENT PASSING
No. 200 SIEVE
ASTM D 1140

Project Name: GSWC Roseton Plant Improvements
Project No.: 13492.001
Tested By: G. Bathala
Date: 04/14/22

ATTERBERG LIMITS
ASTM D 4318

Project Name:	GSWC Roseton Plant Improvements	Tested By:	A. Santos	Date:	04/15/22
Project No. :	13492.001	Input By:	G. Bathala	Date:	04/18/22
Boring No.:	LB-2	Checked By:	G. Bathala		
Sample No.:	S-3	Depth (ft.)	10.0		
Soil Identification:	Olive lean clay (CL)				

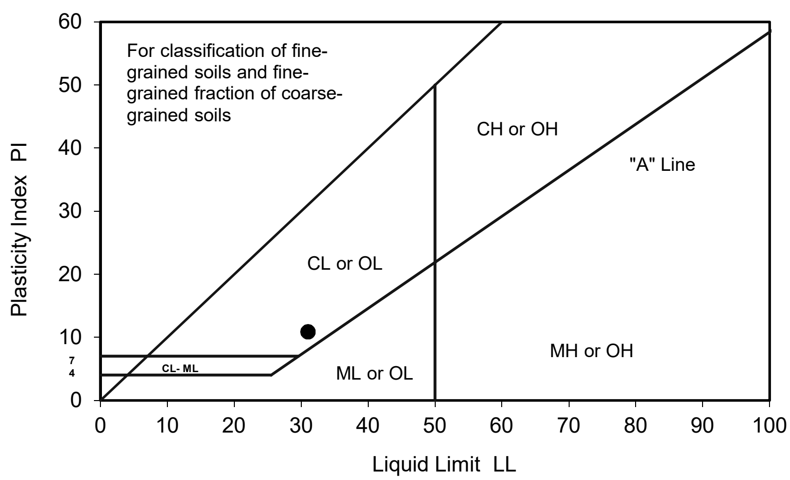
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			33	26	20	
Wet Wt. of Soil + Cont. (g)	11.06	10.40	18.59	19.74	19.22	
Dry Wt. of Soil + Cont. (g)	9.39	8.83	14.50	15.33	14.88	
Wt. of Container (g)	1.07	1.05	1.11	1.10	1.12	
Moisture Content (%) [Wn]	20.07	20.18	30.55	30.99	31.54	

Liquid Limit	**31**
Plastic Limit	**20**
Plasticity Index	**11**
Classification	**CL**

PI at "A" - Line = 0.73(LL-20) | 8.03

One - Point Liquid Limit Calculation

$LL = Wn(N/25)^{0.12}$



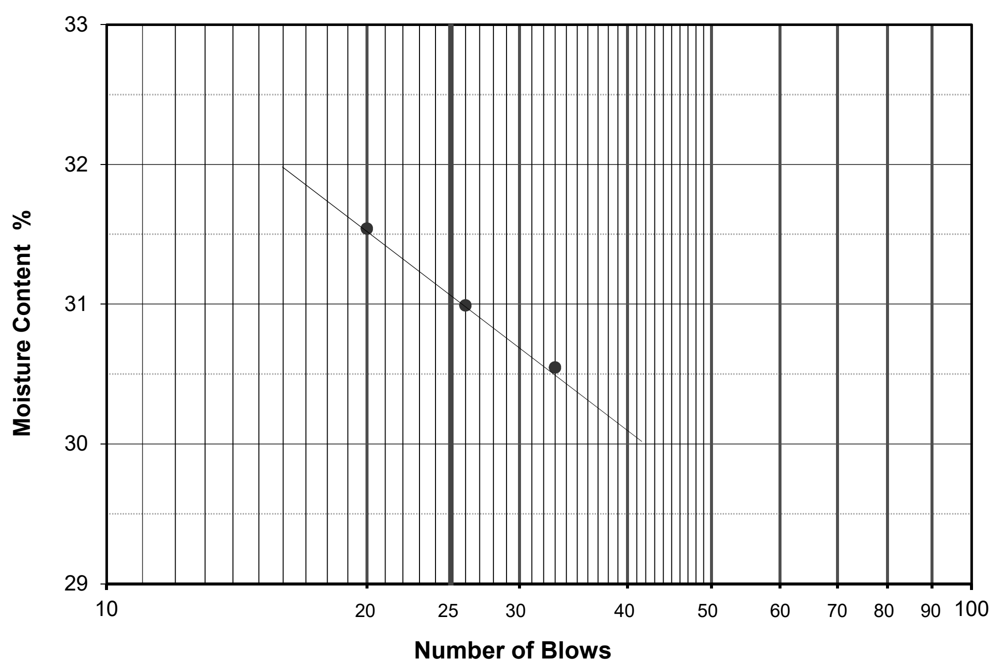
PROCEDURES USED

☐ Wet Preparation
Multipoint - Wet

☒ Dry Preparation
Multipoint - Dry

☒ Procedure A
Multipoint Test

☐ Procedure B
One-point Test



ATTERBERG LIMITS

ASTM D 4318

Project Name:	GSWC Roseton Plant Improvements	Tested By:	A. Santos	Date:	04/15/22
Project No. :	13492.001	Input By:	G. Bathala	Date:	04/18/22
Boring No.:	LB-1	Checked By:	G. Bathala		
Sample No.:	S-5	Depth (ft.)	15.0		
Soil Identification:	Olive silty sand (SM)				

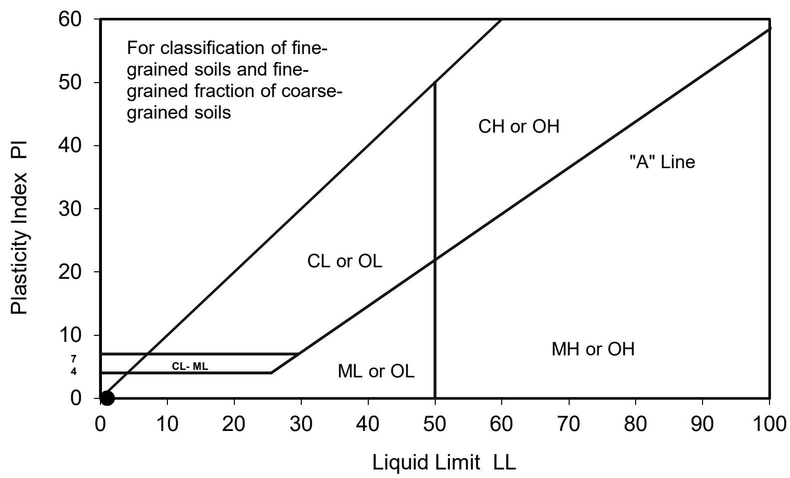
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			5			
Wet Wt. of Soil + Cont. (g)	**Cannot be rolled:**		20.33	**Cannot get more than 6 blows:**		
Dry Wt. of Soil + Cont. (g)	**NonPlastic**		15.64	**NonPlastic**		
Wt. of Container (g)			1.03			
Moisture Content (%) [Wn]			32.10			

Liquid Limit	**NP**
Plastic Limit	**NP**
Plasticity Index	**NP**
Classification	**NP**

PI at "A" - Line = 0.73(LL-20) =

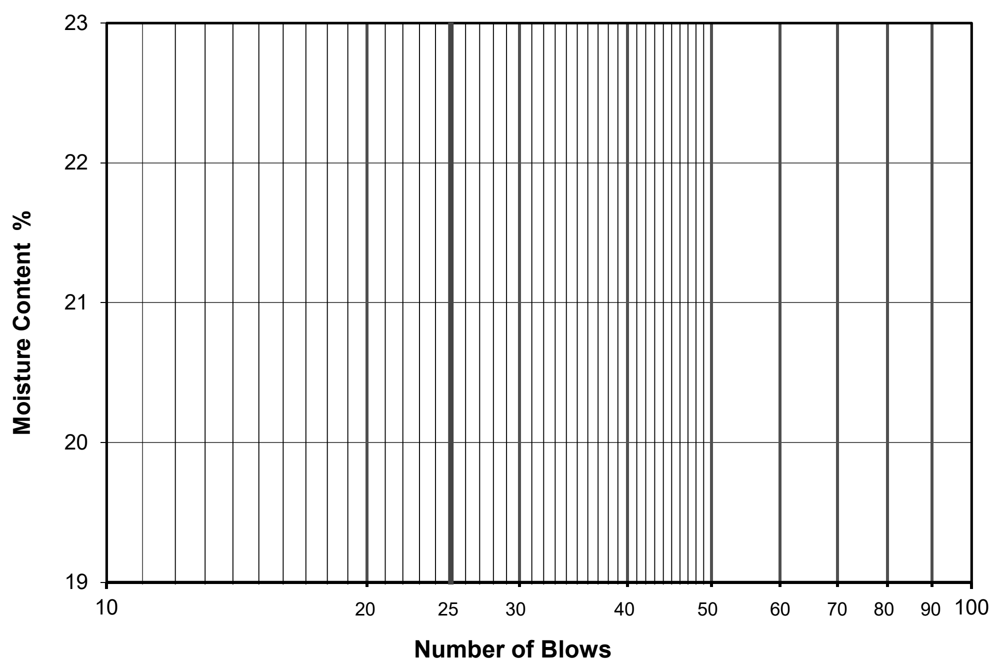
One - Point Liquid Limit Calculation

$LL = Wn(N/25)^{0.12}$



PROCEDURES USED

- [ ] Wet Preparation
  Multipoint - Wet
- [X] Dry Preparation
  Multipoint - Dry
- [X] Procedure A
  Multipoint Test
- [ ] Procedure B
  One-point Test



ATTERBERG LIMITS
ASTM D 4318

Project Name:	GSWC Roseton Plant Improvements	Tested By:	A. Santos	Date:	04/15/22
Project No. :	13492.001	Input By:	G. Bathala	Date:	04/18/22
Boring No.:	LB-1	Checked By:	G. Bathala		
Sample No.:	S-7	Depth (ft.)	25.0		
Soil Identification:	Olive silty clay with sand (CL-ML)s				

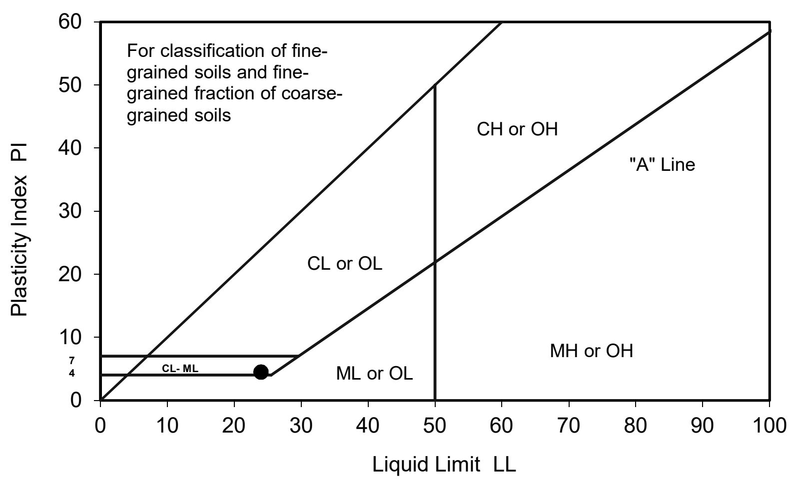
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			31	23	16	
Wet Wt. of Soil + Cont. (g)	9.27	9.10	16.52	19.82	17.11	
Dry Wt. of Soil + Cont. (g)	7.93	7.80	13.58	16.11	13.84	
Wt. of Container (g)	1.10	1.09	1.06	1.06	1.02	
Moisture Content (%) [Wn]	19.62	19.37	23.48	24.65	25.51	

Liquid Limit	**24**
Plastic Limit	**19**
Plasticity Index	**5**
Classification	**CL-ML**

PI at "A" - Line = 0.73(LL-20) 2.92

One - Point Liquid Limit Calculation

$LL = Wn(N/25)^{0.12}$



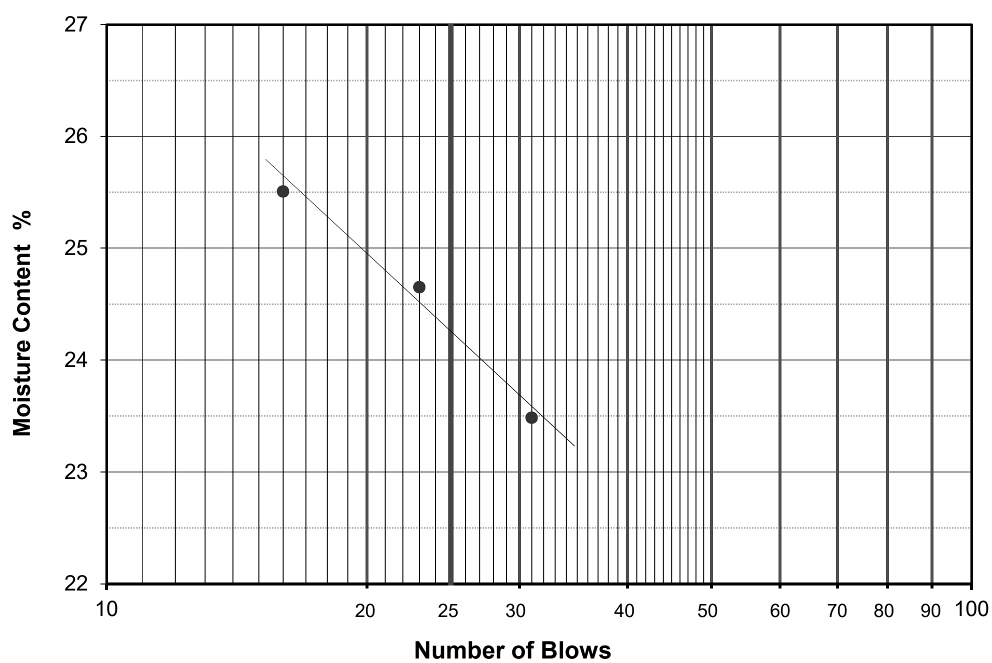
PROCEDURES USED

☐ Wet Preparation
Multipoint - Wet

☒ Dry Preparation
Multipoint - Dry

☒ Procedure A
Multipoint Test

☐ Procedure B
One-point Test



ATTERBERG LIMITS

ASTM D 4318

Project Name:	GSWC Roseton Plant Improvements	Tested By:	A. Santos	Date:	04/15/22
Project No. :	13492.001	Input By:	G. Bathala	Date:	04/18/22
Boring No.:	LB-1	Checked By:	G. Bathala		
Sample No.:	R-8	Depth (ft.)	30.0		
Soil Identification:	Dark olive gray silty sand (SM)				

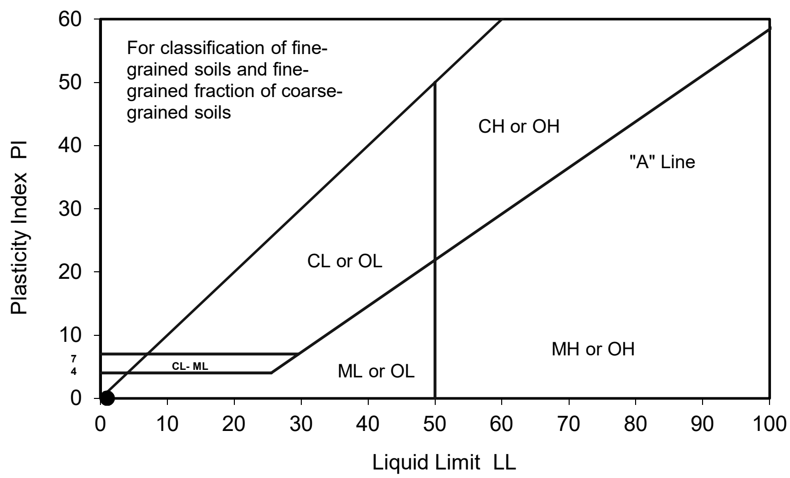
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			5			
Wet Wt. of Soil + Cont. (g)	**Cannot be rolled:**		18.40	**Cannot get more than 6 blows:**		
Dry Wt. of Soil + Cont. (g)	**NonPlastic**		14.55	**NonPlastic**		
Wt. of Container (g)			1.05			
Moisture Content (%) [Wn]			28.52			

Liquid Limit	**NP**
Plastic Limit	**NP**
Plasticity Index	**NP**
Classification	**NP**

PI at "A" - Line = 0.73(LL-20) =

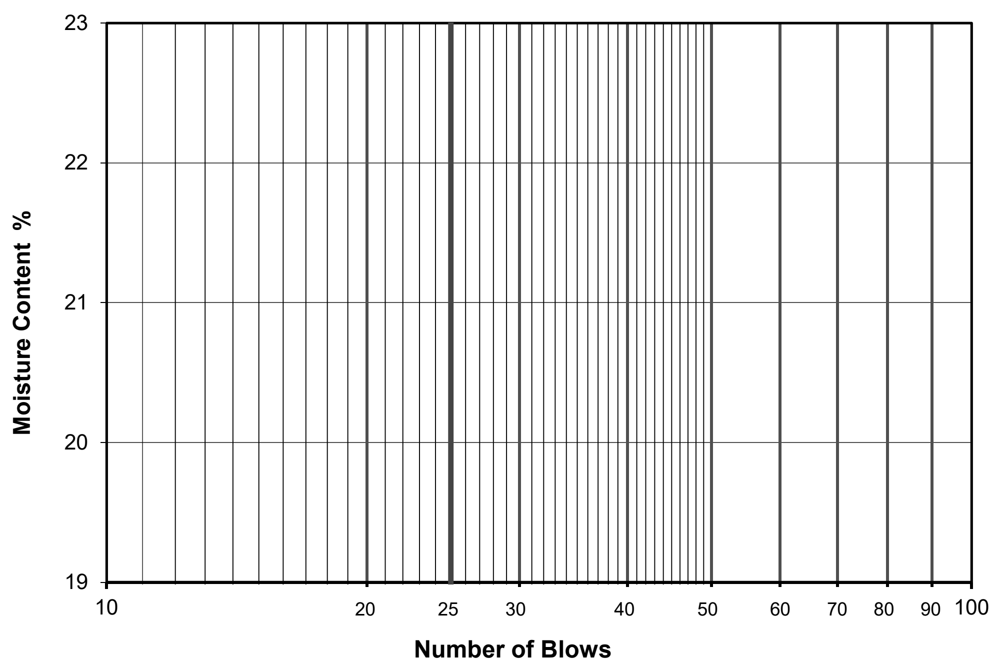
One - Point Liquid Limit Calculation

$LL = Wn(N/25)^{0.12}$

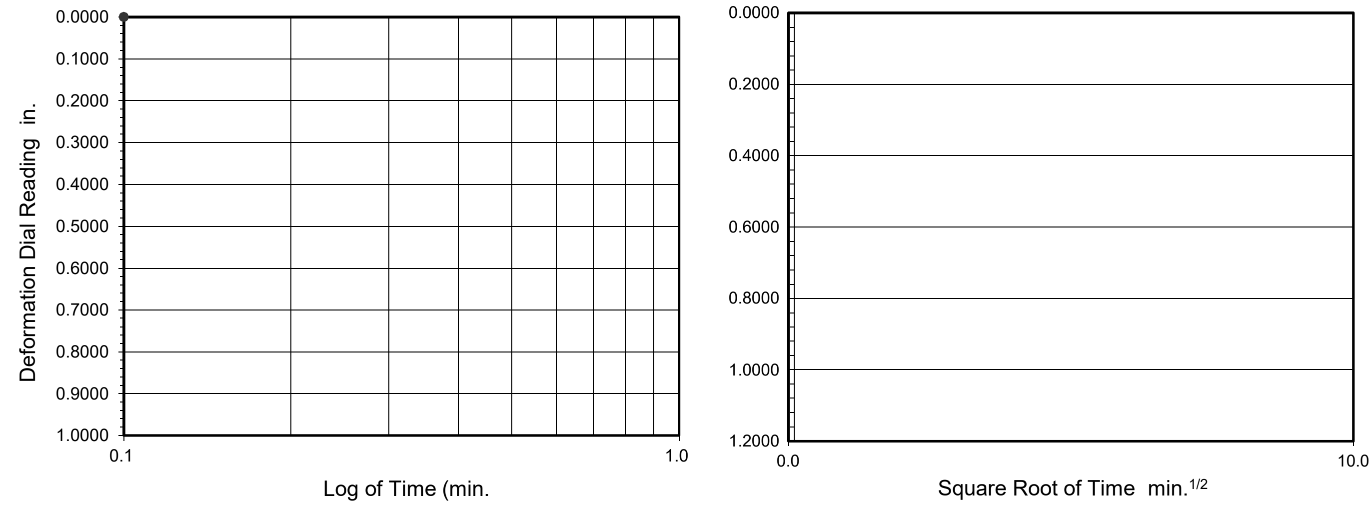


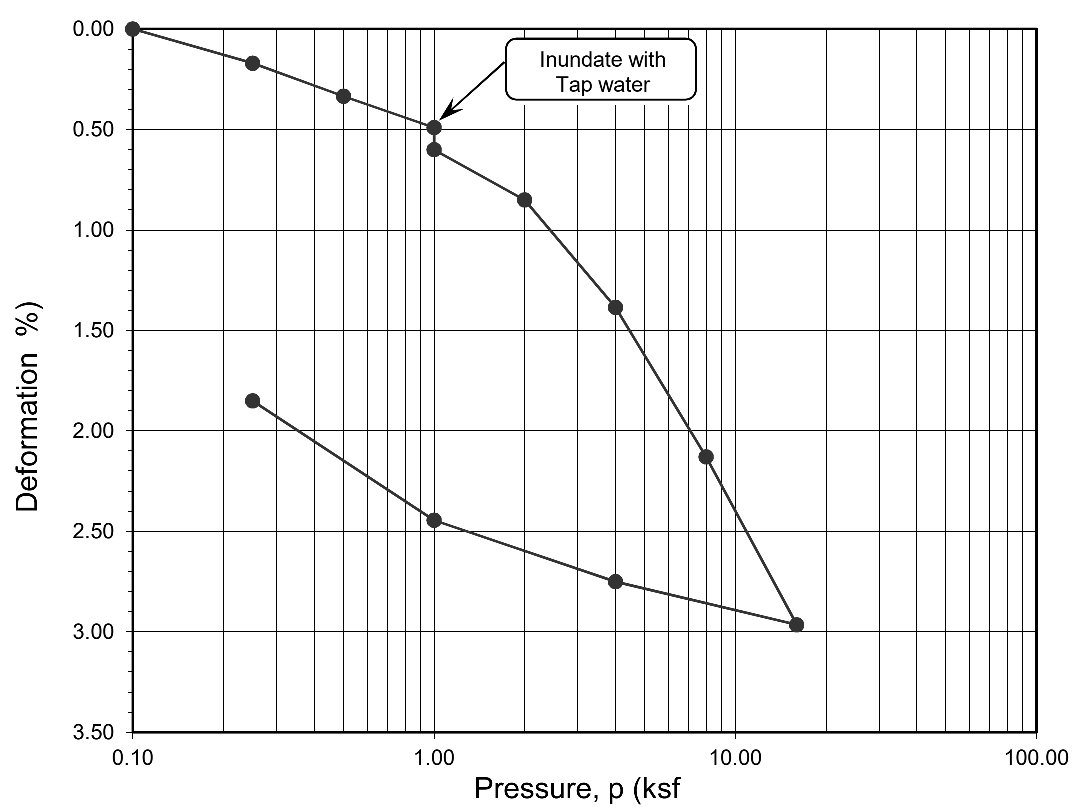
PROCEDURES USED

- [ ] Wet Preparation Multipoint - Wet
- [X] Dry Preparation Multipoint - Dry
- [X] Procedure A Multipoint Test
- [ ] Procedure B One-point Test



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-2**	**5**	**8.2**	**23.1**	**97.8**	**99.4**	**0.730**	**0.698**	**30**	**89**

Soil Identification: Olive silt with sand (ML)s

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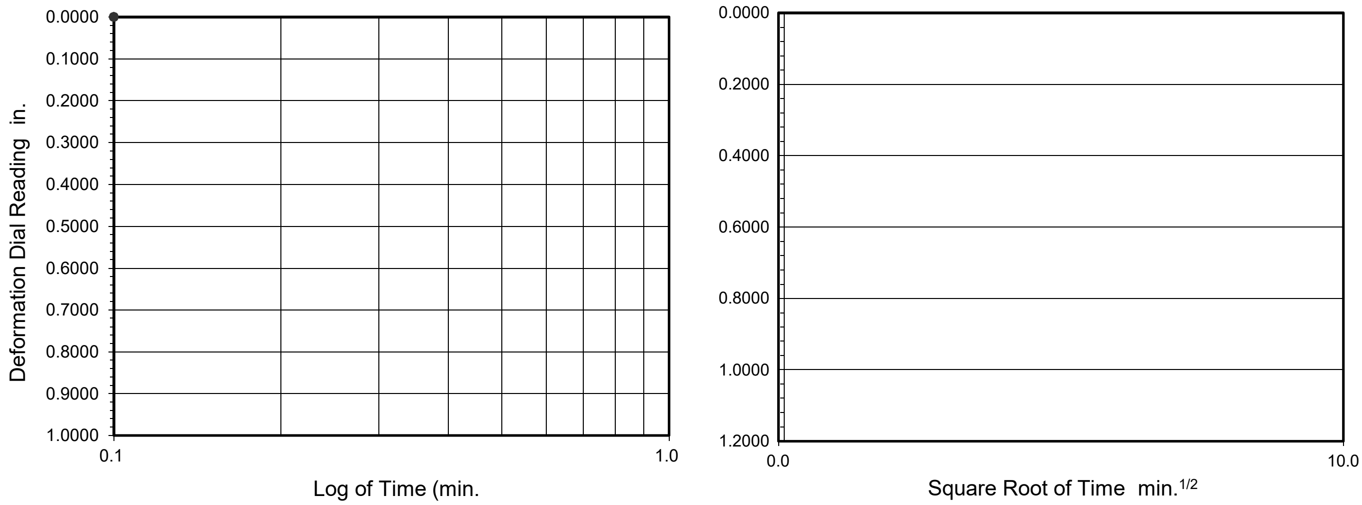
ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

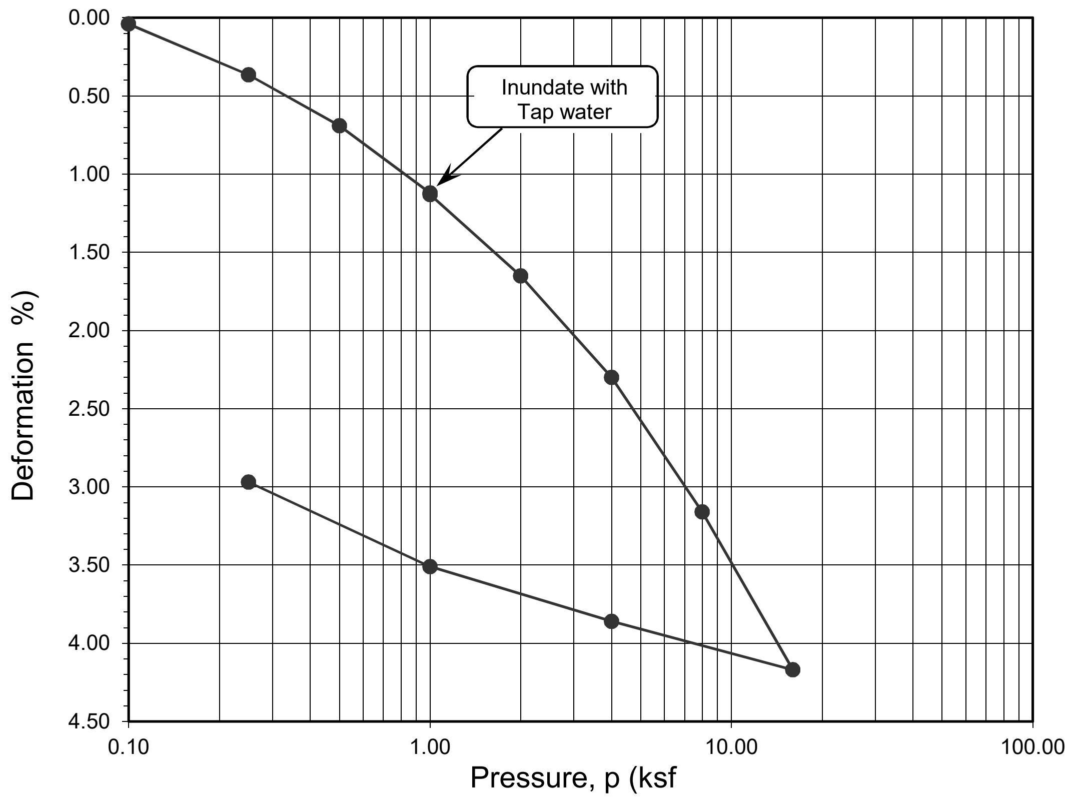
Project No.: 13492.001

GSWC Roseton Plant Improvements

04-22

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-2	**R-4**	**15**	**23.4**	**25.0**	**95.4**	**96.8**	**0.773**	**0.721**	**82**	**91**

Soil Identification: Olive silty sand (SM)

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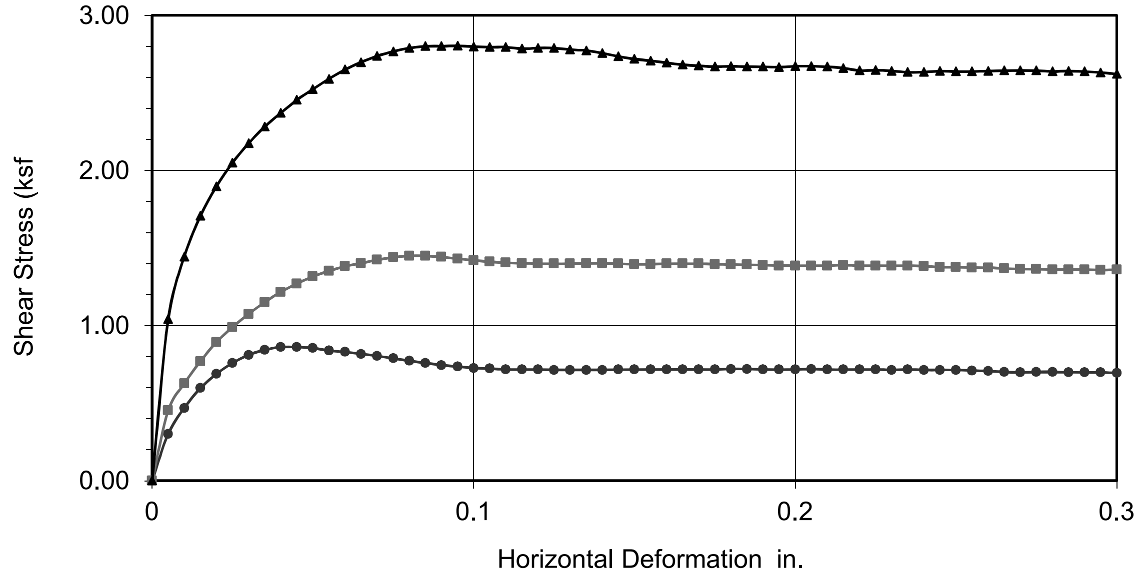
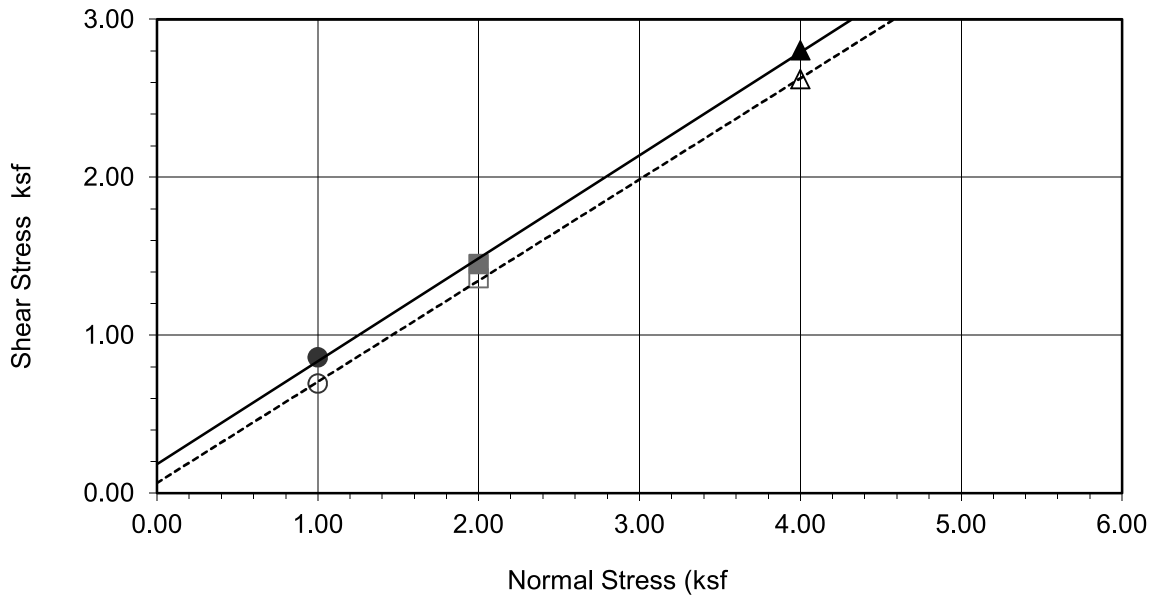
ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 13492.001

GSWC Roseton Plant Improvements

04-22





Boring No.	**LB-1**
Sample No.	**R-2**
Depth (ft	**5**
Sample Type:	Ring
Soil Identification: Olive silt with sand (ML)s	

Strength Parameters

	C (psf	ϕ °
Peak	184	33
Ultimate	64	33

Normal Stress (kip/ft²)	1.000	2.000	4.000
Peak Shear Stress (kip/ft²)	● 0.861	■ 1.449	▲ 2.804
Shear Stress @ End of Test (ksf	○ 0.695	□ 1.361	△ 2.622
Deformation Rate (in./min.)	0.0025	0.0025	0.0025
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.	2.415	2.415	2.415
Initial Moisture Content (%)	8.20	8.20	8.20
Dry Density (pcf)	98.5	99.5	100.2
Saturation (%)	31.1	31.9	32.4
Soil Height Before Shearing (in.)	0.9940	0.9857	0.9844
Final Moisture Content (%)	22.6	22.1	20.7

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DIRECT SHEAR TEST RESULTS
Consolidated Drained - ASTM D 3080

Project No.: 13492.001

GSWC Roseton Plant Improvements

04-22

SOIL RESISTIVITY TEST

DOT CA TEST 643

Project Name:	GSWC Roseton Plant Improvements	Tested By :	J. Domingo Date: 04/29/22
Project No. :	13492.001	Checked By:	A. Santos Date: 05/01/22
Boring No.:	LB-1	Depth (ft.) :	0-5
Sample No. :	B-1		

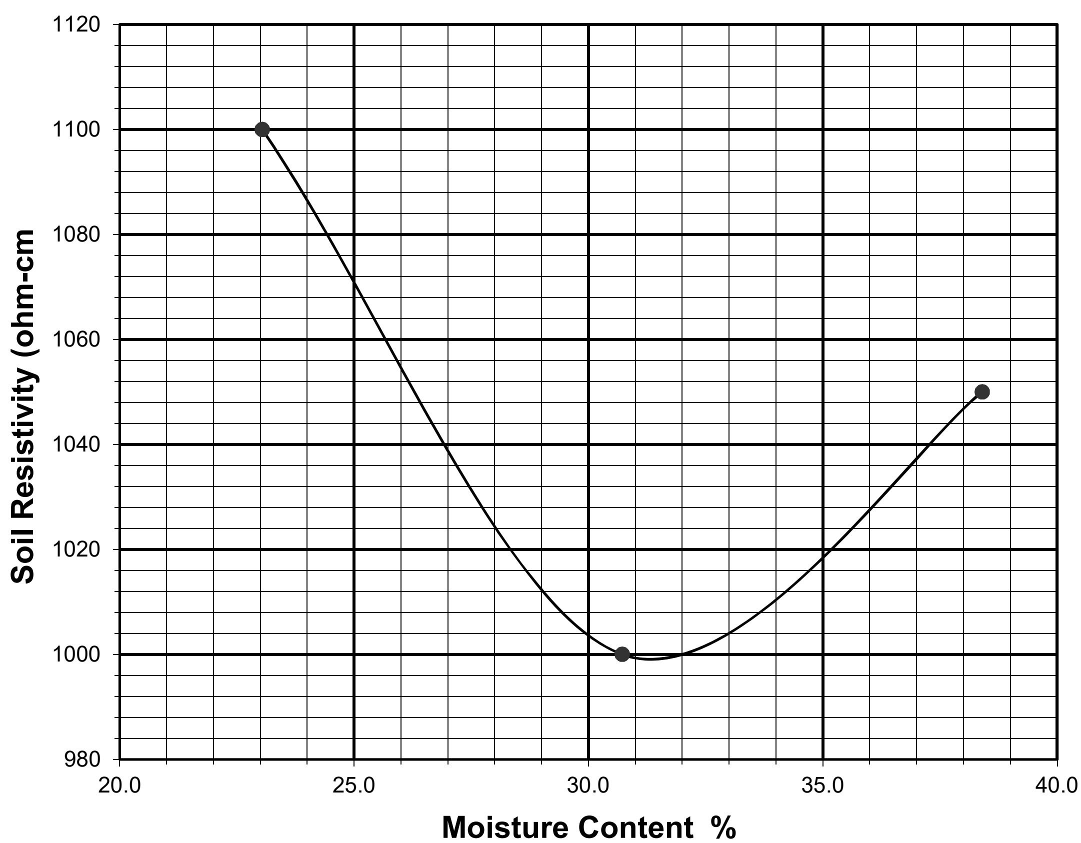
Soil Identification: Dark brown (CL-ML)

*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	30	23.04	1100	1100
2	40	30.72	1000	1000
3	50	38.40	1050	1050
4				
5				

Moisture Content (%) (MCi)	0.00
Wet Wt. of Soil + Cont. (g)	0.00
Dry Wt. of Soil + Cont. (g)	0.00
Wt. of Container (g)	1.00
Container No.	
Initial Soil Wt. (g) (Wt)	130.20
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	
995	31.5	78	120	8.89	20.2



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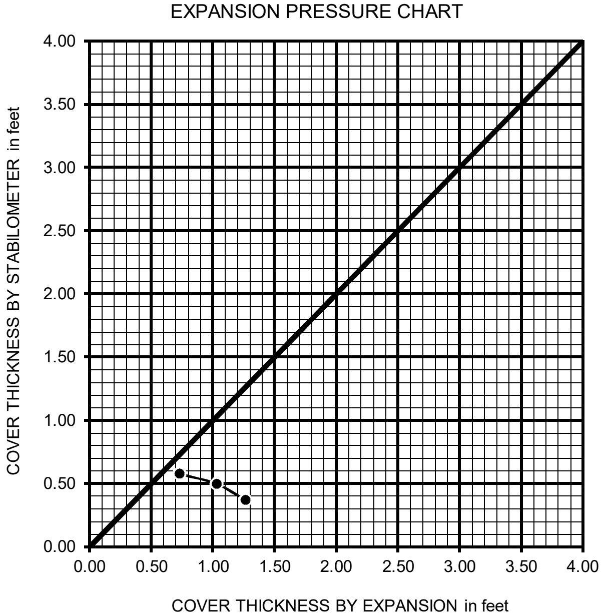
R-VALUE TEST RESULTS
DOT CA Test 301

PROJECT NAME:	Roseton Plant Improvements	PROJECT NUMBER:	13492.001
BORING NUMBER:	LB-2	DEPTH (FT. :	0-5
SAMPLE NUMBER:	B-1	TECHNICIAN:	O. Figueroa
SAMPLE DESCRIPTION:	Olive brown silt with sand (ML s	DATE COMPLETED:	4/18/2022

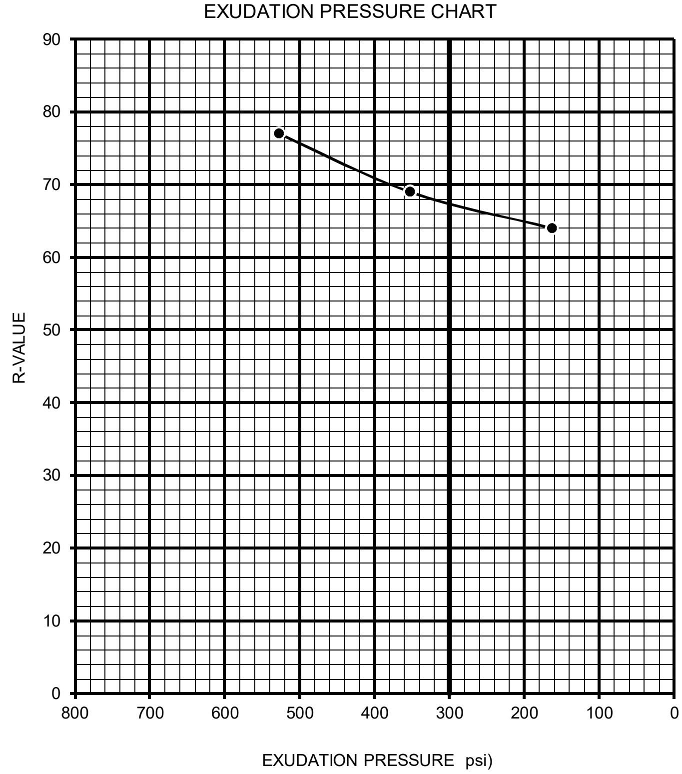
TEST SPECIMEN	a	b	c
MOISTURE AT COMPACTION %	12.1	12.5	13.0
HEIGHT OF SAMPLE, Inches	2.52	2.45	2.52
DRY DENSITY, pcf	116.8	116.0	115.5
COMPACTOR PRESSURE, psi	325	300	250
EXUDATION PRESSURE, psi	527	352	163
EXPANSION, Inches x 10exp-4	38	31	22
STABILITY Ph 2,000 lbs (160 psi	21	27	32
TURNS DISPLACEMENT	5.00	5.42	5.70
R-VALUE UNCORRECTED	77	69	64
R-VALUE CORRECTED	77	69	64

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.	0.37	0.50	0.58
EXPANSION PRESSURE THICKNESS, ft.	1.27	1.03	0.73

EXPANSION PRESSURE CHART



EXUDATION PRESSURE CHART



R-VALUE BY EXPANSION:	63
R-VALUE BY EXUDATION:	67
EQUILIBRIUM R-VALUE:	63

APPENDIX C

LIQUEFACTION ANALYSIS

Leighton Consulting, Inc.
Geotechnical Engineering Consultants
Irvine, California

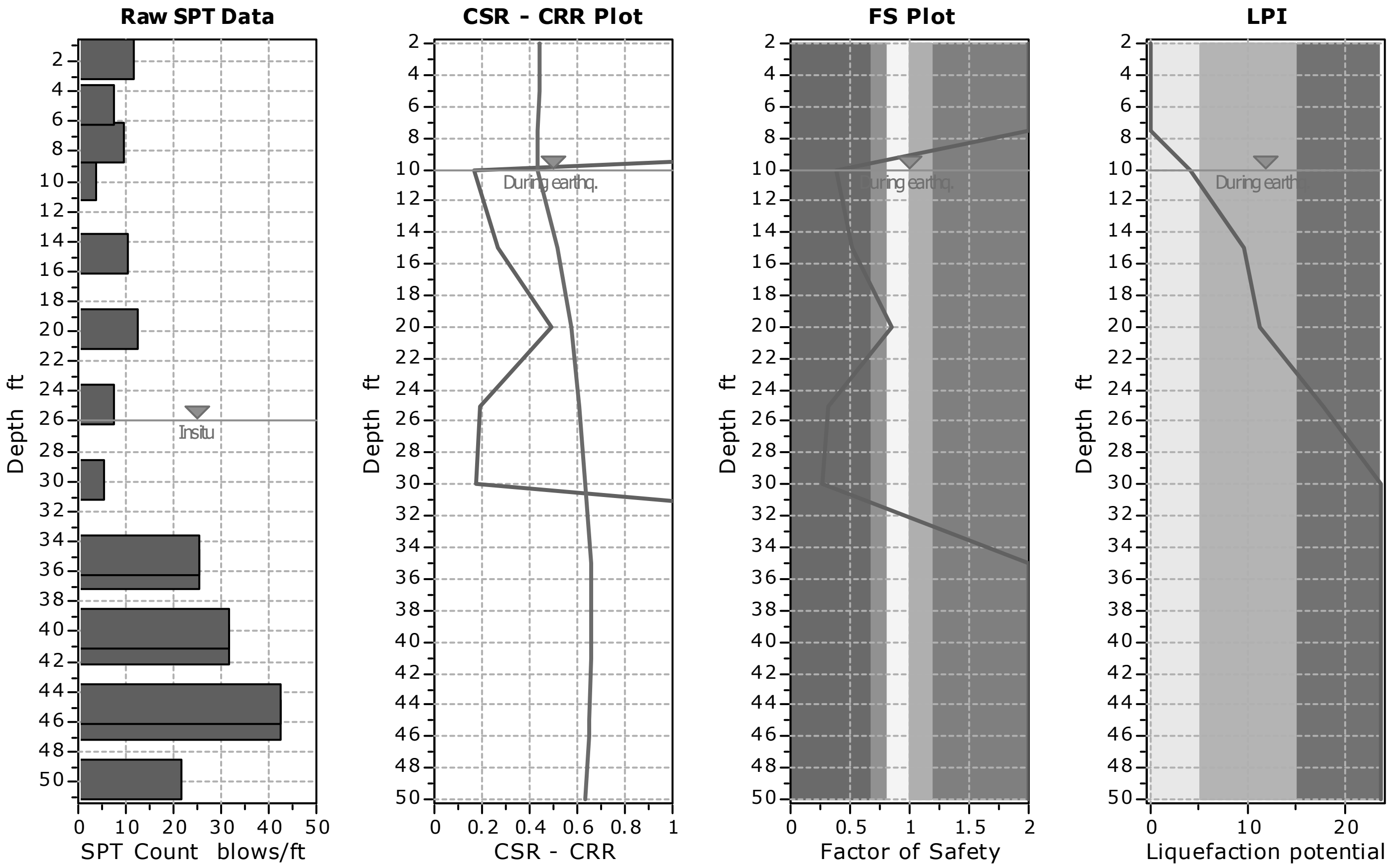
SPT BASED LIQUEFACTION ANALYSIS REPORT

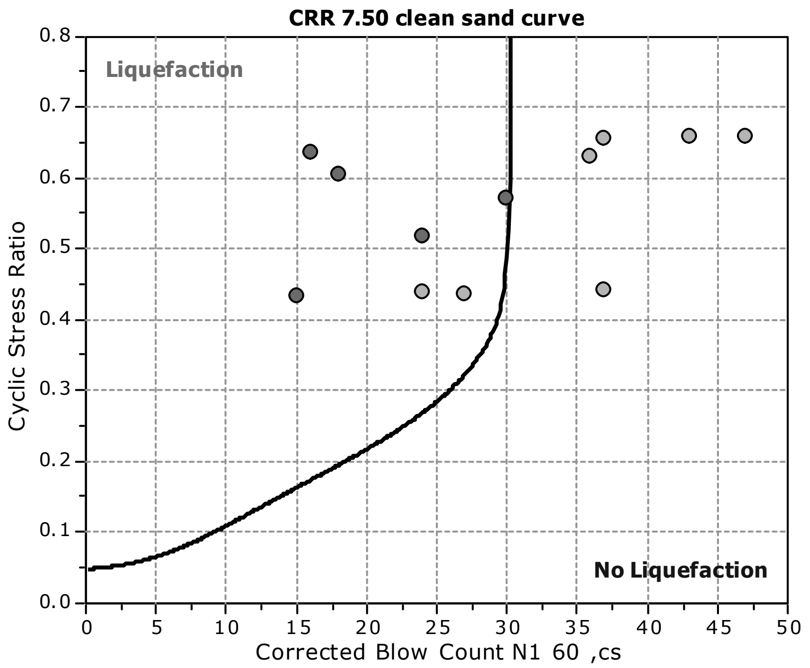
Project title : Roseton Plant Reservoir and Booster Pump Station **SPT Name: LB-1**

Location : Artesia, California

:: Input parameters and analysis properties ::

Analysis method:	NCEER 1998	G.W.T. in-situ :	26.00 ft
Fines correction method:	NCEER 1998	G.W.T. earthq. :	10.00 ft
Sampling method:	Sampler wo liners	Earthquake magnitude M_w:	7.30
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.73 g
Rod length:	3.30 ft	Eq. external load:	0.00 tsf
Hammer energy ratio:	1.50		





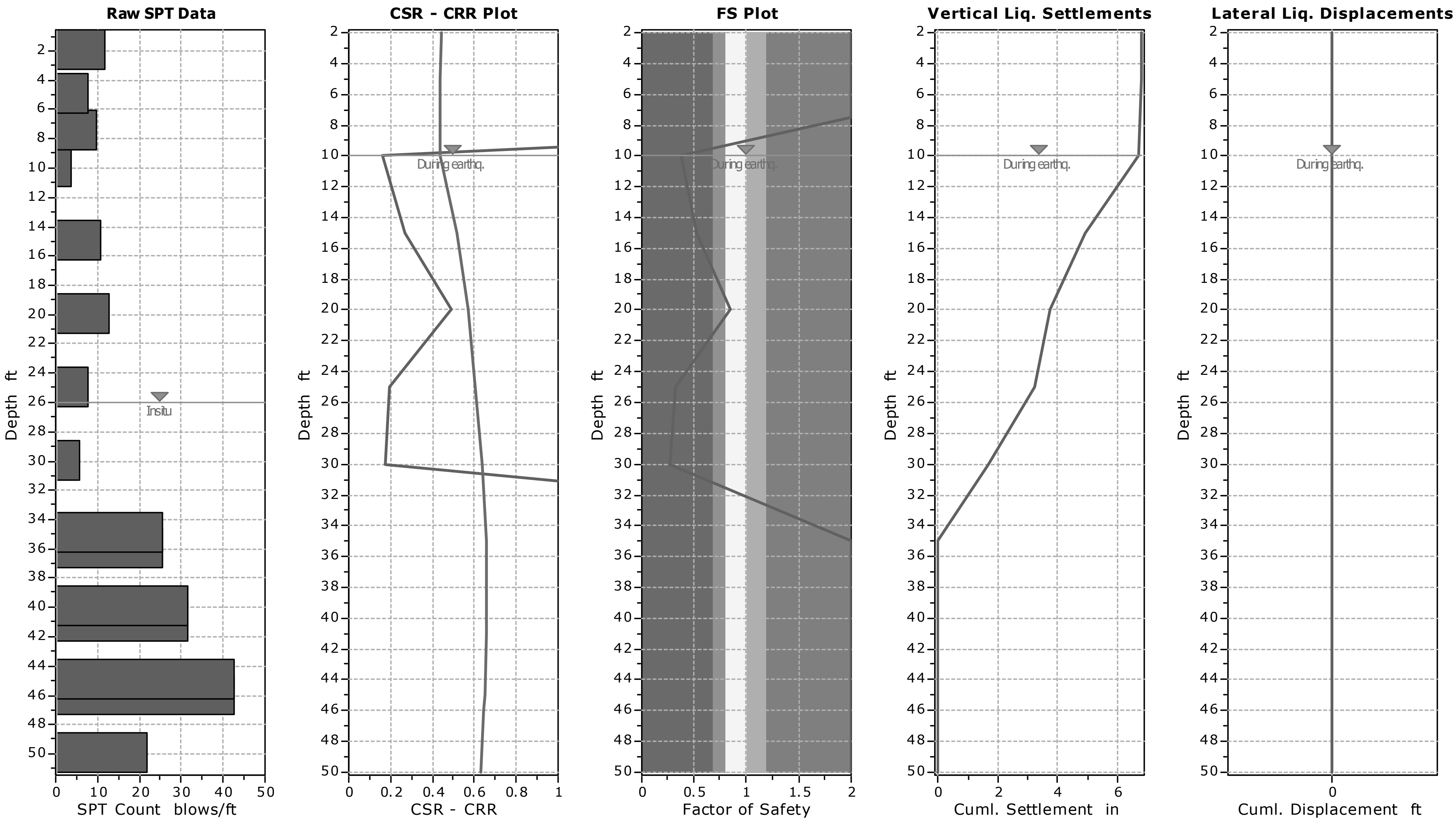
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

:: Overall Liquefaction Assessment Analysis Plots ::



Project File: \\Ds-irv\project\INFOCUS PROJECTS\13001-13500\13492 Tt Roseton Reservoir\001\Analyses\13492.001 Liquefaction Lab - edb.lsvs

:: Field input data ::

Test Depth ft)	SPT Field Value blows)	Fines Content %)	Unit Weight pcf)	Infl. Thickness ft)	Can Liquefy
2.00	12	70.00	120.00	1.00	Yes
5.00	8	60.00	120.00	2.50	Yes
7.50	10	70.00	120.00	2.50	Yes
10.00	4	60.00	120.00	5.00	Yes
15.00	11	24.00	120.00	5.00	Yes
20.00	13	60.00	120.00	5.00	Yes
25.00	8	60.00	120.00	5.00	Yes
30.00	6	60.00	120.00	5.00	Yes
35.00	26	10.00	120.00	1.00	Yes
36.00	26	60.00	120.00	4.00	Yes
40.00	32	10.00	120.00	1.00	Yes
41.00	32	60.00	120.00	4.00	Yes
45.00	43	60.00	120.00	1.00	Yes
46.00	43	10.00	120.00	4.00	Yes
50.00	22	60.00	120.00	1.50	Yes

Abbreviations

Depth:	Depth at which test was performed (ft)
SPT Field Value:	Number of blows per foot
Fines Content:	Fines content at test depth (%)
Unit Weight:	Unit weight at test depth (pcf)
Infl. Thickness:	Thickness of the soil layer to be considered in settlements analysis (ft)
Can Liquefy:	User defined switch for excluding/including test depth from the analysis procedure

:: Cyclic Resistance Ratio (CRR) calculation data ::

Depth ft)	SPT Field Value	Unit Weight pcf)	σ_v (tsf	u_o (tsf	σ'_{vo} (tsf	C_N	C_E	C_B	C_R	C_S	$N_{1\ 60}$	Fines Content %)	α	β	$N_{1\ 60cs}$	$CRR_{7.5}$
2.00	12	120.00	0.12	0.00	0.12	1.68	1.50	1.00	0.75	1.20	27	70.00	5.00	1.20	37	4.000
5.00	8	120.00	0.30	0.00	0.30	1.48	1.50	1.00	0.75	1.20	16	60.00	5.00	1.20	24	4.000
7.50	10	120.00	0.45	0.00	0.45	1.35	1.50	1.00	0.75	1.20	18	70.00	5.00	1.20	27	4.000
10.00	4	120.00	0.60	0.00	0.60	1.25	1.50	1.00	0.85	1.20	8	60.00	5.00	1.20	15	0.163
15.00	11	120.00	0.90	0.00	0.90	1.07	1.50	1.00	0.85	1.20	18	24.00	4.18	1.11	24	0.269
20.00	13	120.00	1.20	0.00	1.20	0.94	1.50	1.00	0.95	1.20	21	60.00	5.00	1.20	30	0.488
25.00	8	120.00	1.50	0.00	1.50	0.84	1.50	1.00	0.95	1.20	11	60.00	5.00	1.20	18	0.196
30.00	6	120.00	1.80	0.12	1.68	0.79	1.50	1.00	1.00	1.20	9	60.00	5.00	1.20	16	0.174
35.00	26	120.00	2.10	0.28	1.82	0.75	1.50	1.00	1.00	1.20	35	10.00	0.87	1.02	37	4.000
36.00	26	120.00	2.16	0.31	1.85	0.75	1.50	1.00	1.00	1.20	35	60.00	5.00	1.20	47	4.000
40.00	32	120.00	2.40	0.44	1.96	0.72	1.50	1.00	1.00	1.20	41	10.00	0.87	1.02	43	4.000
41.00	32	120.00	2.46	0.47	1.99	0.71	1.50	1.00	1.00	1.20	41	60.00	5.00	1.20	54	4.000
45.00	43	120.00	2.70	0.59	2.11	0.69	1.50	1.00	1.00	1.20	53	60.00	5.00	1.20	69	4.000
46.00	43	120.00	2.76	0.62	2.14	0.68	1.50	1.00	1.00	1.20	53	10.00	0.87	1.02	55	4.000
50.00	22	120.00	3.00	0.75	2.25	0.66	1.50	1.00	1.00	1.20	26	60.00	5.00	1.20	36	4.000

:: Cyclic Resistance Ratio (CRR) calculation data ::

Depth ft)	SPT Field Value	Unit Weight pcf)	σ_v (tsf	u_o (tsf	σ'_{vo} (tsf	C_N	C_E	C_B	C_R	C_S	$N_{1\ 60}$	Fines Content %)	α	β	$N_{1\ 60cs}$	$CRR_{7.5}$

Abbreviations

σ_v: Total stress during SPT test tsf
u_o: Water pore pressure during SPT test tsf
σ'_{vo}: Effective overburden pressure during SPT test (tsf
C_N: Overburden corretion factor
C_E: Energy correction factor
C_B: Borehole diameter correction factor
C_R: Rod length correction factor
C_S: Liner correction factor
$N_{1(60}$: Corrected N_{SPT} to a 60% energy ratio
α, β: Clean sand equivalent clean sand formula coefficients
$N_{1(60\ cs}$: Corected $N_{1(60}$ value for fines content
$CRR_{7.5}$: Cydic resistance ratio for M 7.5

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Depth ft)	Unit Weight pcf)	$\sigma_{v,eq}$ (tsf	$u_{o,eq}$ (tsf	$\sigma'_{vo,eq}$ (tsf	r_d	α	CSR	MSF	$CSR_{eq,M\ 7.5}$	K_{sigma}	CSR	FS	

Abbreviations

$\sigma_{v,eq}$: Total overburden pressure at test point, during earthquake tsf
$u_{o,eq}$: Water pressure at test point, during earthquake (tsf
$\sigma'_{vo,eq}$: Effective overburden pressure, during earthquake tsf
r_d : Nonlinear shear mass factor
α: Improvement factor due to stone columns
CSR : Cydic Stress Ratio adjusted for improvement
MSF : Magnitude Scaling Factor
$CSR_{eq,M\ 7.5}$: CSR adjusted for M 7.5
K_{sigma}: Effective overburden stress factor
CSR : CSR fully adjusted user FS applied
FS: Calculated factor of safety against soil liquefaction

User FS: 1.00

Depth ft)	FS	F	wz	Thickness ft)	I_L
2.00	2.000	0.00	9.70	3.00	0.00
5.00	2.000	0.00	9.24	3.00	0.00
7.50	2.000	0.00	8.86	2.50	0.00

:: Liquefaction potential according to Iwasaki ::

Depth ft)	FS	F	wz	Thickness ft)	I_L
10.00	0.377	0.62	8.48	2.50	4.03
15.00	0.519	0.48	7.71	5.00	5.66
20.00	0.852	0.15	6.95	5.00	1.57
25.00	0.322	0.68	6.19	5.00	6.39
30.00	0.273	0.73	5.43	5.00	6.01
35.00	2.000	0.00	4.67	5.00	0.00
36.00	2.000	0.00	4.51	1.00	0.00
40.00	2.000	0.00	3.90	4.00	0.00
41.00	2.000	0.00	3.75	1.00	0.00
45.00	2.000	0.00	3.14	4.00	0.00
46.00	2.000	0.00	2.99	1.00	0.00
50.00	2.000	0.00	2.38	4.00	0.00

Overall potential I_L : 23.65

I_L = 0.00 - No liquefaction
I_L between 0.00 and 5 - Liquefaction not probable
I_L between 5 and 15 - Liquefaction probable
I_L > 15 - Liquefaction certain

:: Vertical settlements estimation for dry sands ::

Depth ft)	$N_{1\ 60}$	τ_{av}	p	G_{max} (tsf	α	b	γ	ε_{15}	N_c	ε_{Nc} %	Δh ft)	ΔS in)
2.00	27	0.06	0.08	422.35	0.13	22838.69	0.00	0.00	13.34	0.02	1.00	0.005
5.00	16	0.14	0.20	578.06	0.14	13179.75	0.00	0.00	13.34	0.07	2.50	0.043
7.50	18	0.21	0.30	736.33	0.14	10333.62	0.00	0.00	13.34	0.06	2.50	0.037

Cumulative settlemetns: 0.085

Abbreviations

τ_{av}: Average cyclic shear stress
p: Average stress
G_{max}: Maximum shear modulus tsf
α, b: Shear strain formula variables
γ: Average shear strain
ε_{15}: Volumetric strain after 15 cycles
N_c: Number of cycles
ε_{Nc}: Volumetric strain for number of cycles N_c (%)
Δh: Thickness of soil layer in
ΔS: Settlement of soil layer in

:: Vertical settlements estimation for saturated sands ::

Depth ft)	D_{50} (in	q_c/N	e_v %	Δh ft)	s in)
10.00	0.00	5.00	2.96	5.00	1.775
15.00	0.00	5.00	2.01	5.00	1.207
20.00	0.00	5.00	0.86	5.00	0.516
25.00	0.00	5.00	2.55	5.00	1.529
30.00	0.00	5.00	2.81	5.00	1.684
35.00	0.00	5.00	0.00	1.00	0.000
36.00	0.00	5.00	0.00	4.00	0.000
40.00	0.00	5.00	0.00	1.00	0.000

:: Vertical settlements estimation for saturated sands ::					
Depth ft)	D_{50} (in	q_c/N	e_v %	Δh ft)	s in)
41.00	0.00	5.00	0.00	4.00	0.000
45.00	0.00	5.00	0.00	1.00	0.000
46.00	0.00	5.00	0.00	4.00	0.000
50.00	0.00	5.00	0.00	1.50	0.000

Cumulative settlements: 6.711

Abbreviations

D_{50}: Median grain size (in)
q_c/N: Ratio of cone resistance to SPT
e_v: Post liquefaction volumetric strain (%)
Δh: Thickness of soil layer to be considered (ft)
s: Estimated settlement (in)

Leighton Consulting, Inc.
Geotechnical Engineering Consultants
Irvine, California

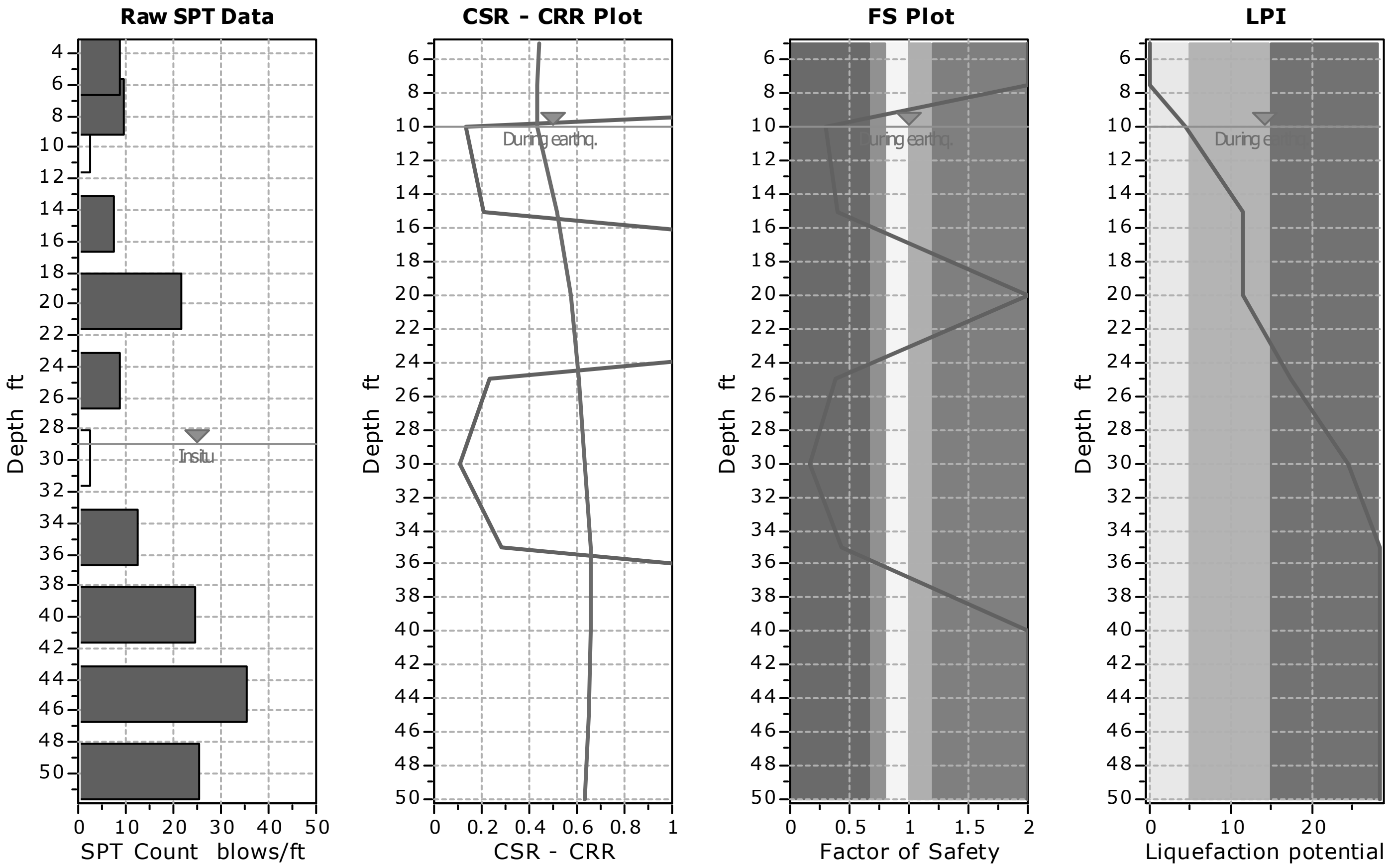
SPT BASED LIQUEFACTION ANALYSIS REPORT

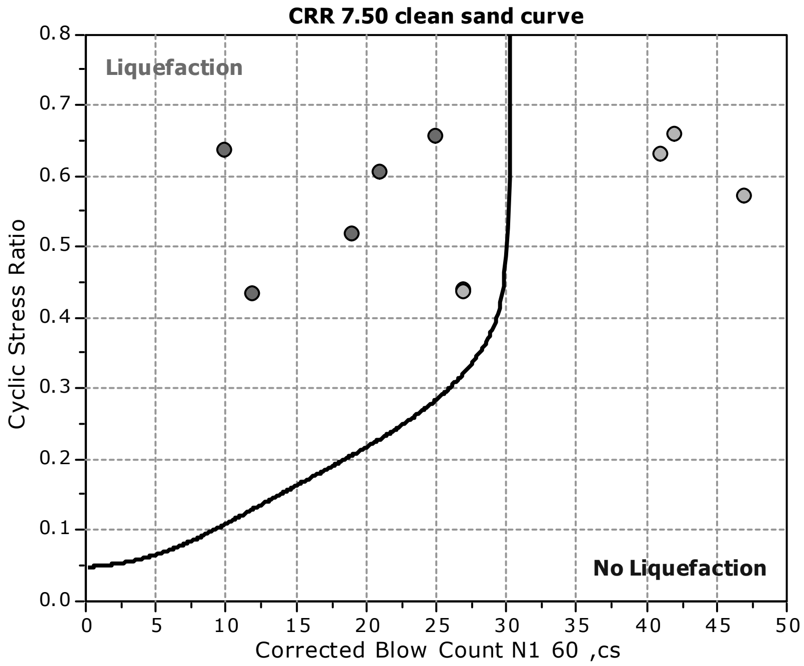
Project title : Roseton Plant Reservoir and Booster Pump Station **SPT Name: LB-2**

Location : Artesia, California

:: Input parameters and analysis properties ::

Analysis method:	NCEER 1998	G.W.T. in-situ :	29.00 ft
Fines correction method:	NCEER 1998	G.W.T. earthq. :	10.00 ft
Sampling method:	Sampler wo liners	Earthquake magnitude M_w:	7.30
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.73 g
Rod length:	3.30 ft	Eq. external load:	0.00 tsf
Hammer energy ratio:	1.50		





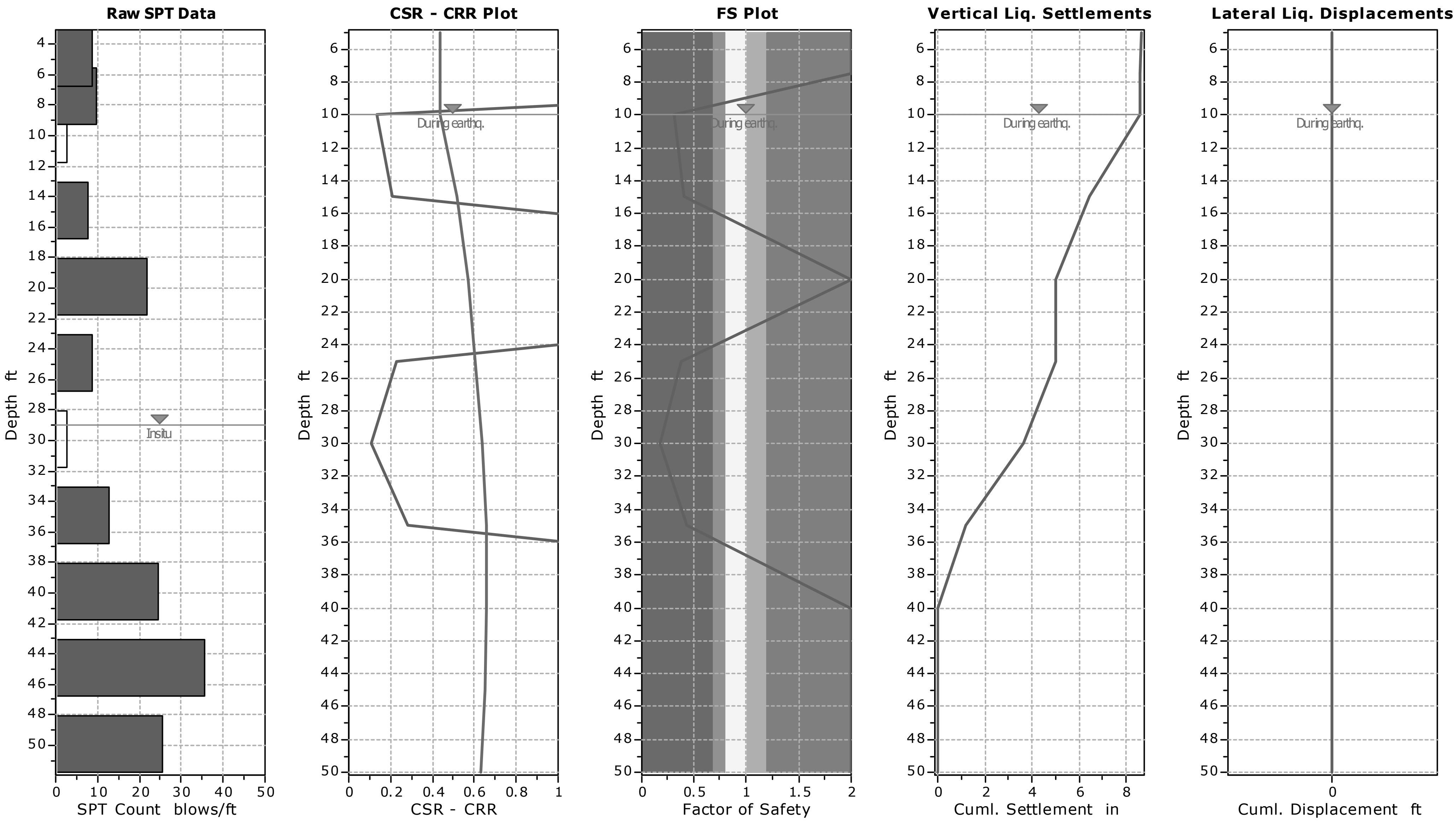
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

:: Overall Liquefaction Assessment Analysis Plots ::



:: Field input data ::

Test Depth ft)	SPT Field Value blows)	Fines Content %)	Unit Weight pcf)	Infl. Thickness ft)	Can Liquefy
5.00	9	60.00	120.00	2.50	Yes
7.50	10	60.00	120.00	2.50	Yes
10.00	3	70.00	120.00	5.00	Yes
15.00	8	24.00	120.00	5.00	Yes
20.00	22	60.00	120.00	5.00	Yes
25.00	9	70.00	120.00	5.00	Yes
30.00	3	60.00	120.00	5.00	Yes
35.00	13	60.00	120.00	5.00	Yes
40.00	25	60.00	120.00	5.00	Yes
45.00	36	24.00	120.00	5.00	Yes
50.00	26	60.00	120.00	1.50	Yes

Abbreviations

Depth:	Depth at which test was performed (ft)
SPT Field Value:	Number of blows per foot
Fines Content:	Fines content at test depth (%)
Unit Weight:	Unit weight at test depth (pcf)
Infl. Thickness:	Thickness of the soil layer to be considered in settlements analysis (ft)
Can Liquefy:	User defined switch for excluding/including test depth from the analysis procedure

:: Cyclic Resistance Ratio (CRR) calculation data ::

Depth ft)	SPT Field Value	Unit Weight pcf)	σ_v (tsf	u_o (tsf	σ'_{vo} (tsf	C_N	C_E	C_B	C_R	C_S	$N_{1\ 60}$	Fines Content %)	α	β	$N_{1\ 60cs}$	$CRR_{7.5}$
5.00	9	120.00	0.30	0.00	0.30	1.48	1.50	1.00	0.75	1.20	18	60.00	5.00	1.20	27	4.000
7.50	10	120.00	0.45	0.00	0.45	1.35	1.50	1.00	0.75	1.20	18	60.00	5.00	1.20	27	4.000
10.00	3	120.00	0.60	0.00	0.60	1.25	1.50	1.00	0.85	1.20	6	70.00	5.00	1.20	12	0.131
15.00	8	120.00	0.90	0.00	0.90	1.07	1.50	1.00	0.85	1.20	13	24.00	4.18	1.11	19	0.206
20.00	22	120.00	1.20	0.00	1.20	0.94	1.50	1.00	0.95	1.20	35	60.00	5.00	1.20	47	4.000
25.00	9	120.00	1.50	0.00	1.50	0.84	1.50	1.00	0.95	1.20	13	70.00	5.00	1.20	21	0.229
30.00	3	120.00	1.80	0.03	1.77	0.77	1.50	1.00	1.00	1.20	4	60.00	5.00	1.20	10	0.110
35.00	13	120.00	2.10	0.19	1.91	0.73	1.50	1.00	1.00	1.20	17	60.00	5.00	1.20	25	0.285
40.00	25	120.00	2.40	0.34	2.06	0.70	1.50	1.00	1.00	1.20	31	60.00	5.00	1.20	42	4.000
45.00	36	120.00	2.70	0.50	2.20	0.67	1.50	1.00	1.00	1.20	43	24.00	4.18	1.11	52	4.000
50.00	26	120.00	3.00	0.66	2.34	0.64	1.50	1.00	1.00	1.20	30	60.00	5.00	1.20	41	4.000

Abbreviations

σ_v:	Total stress during SPT test tsf
u_o:	Water pore pressure during SPT test tsf
σ'_{vo}:	Effective overburden pressure during SPT test (tsf
C_N:	Overburden corretion factor
C_E:	Energy correction factor
C_B:	Borehole diameter correction factor
C_R:	Rod length correction factor
C_S:	Liner correction factor
$N_{1(60}$:	Corrected N_{SPT} to a 60% energy ratio
α, β:	Clean sand equivalent clean sand formula coefficients
$N_{1(60\ cs}$:	Corected $N_{1(60}$ value for fines content
$CRR_{7.5}$:	Cydic resistance ratio for M 7.5

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Depth ft)	Unit Weight pcf)	$\sigma_{v,eq}$ (tsf	$u_{o,eq}$ (tsf	$\sigma'_{vo,eq}$ (tsf	r_d	α	CSR	MSF	$CSR_{eq,M\ 7.5}$	K_{sigma}	CSR	FS

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Depth ft)	Unit Weight pcf)	$\sigma_{v,eq}$ (tsf	$u_{o,eq}$ (tsf	$\sigma'_{vo,eq}$ (tsf	r_d	α	CSR	MSF	$CSR_{eq,M\ 7.5}$	K_{sigma}	CSR	FS
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Abbreviations

$\sigma_{v,eq}$: Total overburden pressure at test point, during earthquake tsf
$u_{o,eq}$: Water pressure at test point, during earthquake (tsf
$\sigma'_{vo,eq}$: Effective overburden pressure, during earthquake tsf
r_d : Nonlinear shear mass factor
α: Improvement factor due to stone columns
CSR : Cyclic Stress Ratio adjusted for improvement
MSF : Magnitude Scaling Factor
$CSR_{eq,M\ 7.5}$: CSR adjusted for M 7.5
K_{sigma}: Effective overburden stress factor
CSR : CSR fully adjusted user FS applied
FS: Calculated factor of safety against soil liquefaction

User FS: 1.00

:: Liquefaction potential according to Iwasaki ::

Depth ft)	FS	F	wz	Thickness ft)	I_L

Overall potential I_L: 28.32

I_L = 0.00 - No liquefaction
I_L between 0.00 and 5 - Liquefaction not probable
I_L between 5 and 15 - Liquefaction probable
I_L > 15 - Liquefaction certain

Depth ft)	$N_{1\ 60}$	τ_{av}	p	G_{max} (tsf	α	b	γ	ε_{15}	N_c	ε_{Nc} %	Δh ft)	ΔS in)
5.00	18	0.14	0.20	601.21	0.14	13179.75	0.00	0.00	13.34	0.05	2.50	0.033

:: Vertical settlements estimation for dry sands ::

Depth ft)	$N_{1\ 60}$	τ_{av}	p	G_{max} (tsf	α	b	γ	ε_{15}	N_c	ε_{Nc} %	Δh ft)	ΔS (in)
7.50	18	0.21	0.30	736.33	0.14	10333.62	0.00	0.00	13.34	0.06	2.50	0.037

Cumulative settlemetns: 0.069

Abbreviations

τ_{av}: Average cyclic shear stress
p: Average stress
G_{max}: Maximum shear modulus tsf
α, b: Shear strain formula variables
γ: Average shear strain
ε_{15}: Volumetric strain after 15 cycles
N_c: Number of cycles
ε_{Nc}: Volumetric strain for number of cycles N_c (%)
Δh: Thickness of soil layer in
ΔS: Settlement of soil layer in

:: Vertical settlements estimation for saturated sands ::

Depth ft)	D_{50} (in	q_c/N	e_v %	Δh ft)	s in)
10.00	0.00	5.00	3.55	5.00	2.131
15.00	0.00	5.00	2.44	5.00	1.462
20.00	0.00	5.00	0.00	5.00	0.000
25.00	0.00	5.00	2.25	5.00	1.347
30.00	0.00	5.00	4.13	5.00	2.475
35.00	0.00	5.00	1.95	5.00	1.168
40.00	0.00	5.00	0.00	5.00	0.000
45.00	0.00	5.00	0.00	5.00	0.000
50.00	0.00	5.00	0.00	1.50	0.000

Cumulative settlements: 8.583

Abbreviations

D_{50}: Median grain size (in)
q_c/N: Ratio of cone resistance to SPT
e_v: Post liquefaction volumetric strain (%)
Δh: Thickness of soil layer to be considered (ft)
s: Estimated settlement (in)

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