GEOTECHNICAL ENGINEERING INVESTIGATION REPORT
PROPOSED WELL 38 TREATMENT
TURLOCK, CALIFORNIA

BSK PROJECT G19-223-11F

PREPARED FOR:

PROVOST & PRITCHARD
286 W. CROMWELL AVENUE
FRESNO, CALIFORNIA 93711

DECEMBER 18, 2019

GEOTECHNICAL ENGINEERING INVESTIGATION REPORT
PROPOSED WELL 38 TREATMENT
TURLOCK, CALIFORNIA

Prepared for:

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BSK Project: G19-223-11F

December 19, 2019

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1 INTRODUCTION

1.1 General

This report presents the results of a geotechnical engineering investigation report, for a proposed Well 38 arsenic treatment project to be situated at the northwest corner of Mountain View Road and West Christofferson Parkway in Turlock, California. The geotechnical engineering investigation was conducted in accordance with BSK Proposal GF18-18545, dated June 12, 2019. The proposed improvements and exploratory borings are shown on Figure 2, Boring Location Map.

This report provides a description of the geotechnical conditions at the site and provides specific recommendations for earthwork and foundation design with respect to the planned structures. In the event that changes occur in the design of the project, this report's conclusions and recommendations will not be considered valid unless the changes are reviewed with BSK and the conclusions and recommendations are modified or verified in writing. Examples of such changes would include location, size of structures, foundation loads, etc.

1.2 Planned Description

BSK understands that this project consists of the design and construction of an arsenic treatment facility at Well Site 38. Well 38 is located at the northwest corner of Mountain View Road and West Christofferson Parkway in Turlock, California. The facility is anticipated to consist of a new filtration system, vertical pressure filters, an equalization tank, and chemical storage building. The proposed structures are anticipated to be constructed at-grade and supported on shallow or mat foundations. Additional improvements are anticipated to include underground utilities.

1.3 Purpose and Scope of Services

The objective of this geotechnical investigation was to characterize the subsurface conditions in the areas of the proposed structures, and provide geotechnical engineering recommendations for the preparation of plans and specifications. The scope of the investigation included a field exploration, laboratory testing, engineering analyses, and preparation of this report.

2 FIELD INVESTIGATION AND LABORATORY TESTING

2.1 Field Exploration

The field exploration was conducted on November 14, 2018, under the oversight of a BSK Engineer. Four (4) test borings were drilled to depths of 16.5 to 41.5 feet below existing ground surface (bgs) at the proposed improvements. Borings were drilled using a truck-mounted drill rig equipped with hollow stem augers. Details of the field exploration and the boring logs are provided in Appendix A.

The soil materials encountered in the borings were visually classified in the field, and the logs were recorded during the excavating and sampling operations. Visual classification of the materials encountered in the borings was made in general accordance with the Unified Soil Classification System (ASTM D2488). A soil classification chart is presented in Appendix A.

Boring logs are presented in Appendix A and should be consulted for more details concerning subsurface conditions. Stratification lines were approximated by the field staff based on observations made at the time of excavating, while the actual boundaries between soil types may be gradual and soil conditions may vary at other locations.

2.2 Laboratory Testing

Laboratory tests were performed on selected soil samples to evaluate moisture content, dry density, shear strength, gradation analysis, and corrosion characteristics. A description of the laboratory test methods and results are presented in Appendix B.

3 SITE CONDITIONS

The following sections address the site descriptions and surface conditions, subsurface conditions, groundwater conditions, and seismic design criteria at the site. This information is based on BSK's field exploration and published maps and reports.

3.1 Site Description and Surface Conditions

The facility is located at Well 38, located northwest of Mountain View and West Christofferson Parkway. At the time of our subsurface investigation the site contained seasonal weeds and was unpaved. The site was bound to the north by Sandstone Street, to the east by Mountain View Road, to the south by West Christofferson Parkway, and to the west by the Sierra Oaks Apartments. The existing well and maintenance facility was located at the southern portion of the site.

3.2 Subsurface Conditions

The subsurface material in the borings generally consisted of silty sand in the upper 5 feet bgs underlain by laterally discontinuous layers of sandy silt and poorly graded sand to the maximum depth of exploration (41.5 feet bgs). The relative density of the coarse-grained soils was loose to very dense. The relative consistency of the fine-grained soils was stiff to very stiff.

The Boring logs in Appendix A provide a more detailed description of the materials encountered, including the applicable Unified Soil Classification System symbols.

3.3 Groundwater Conditions

Groundwater was encountered in our soil borings on November 14, 2019 at boring B-4 at approximately 38.5 feet bgs. Based on the groundwater elevation data from the California Department of Water Resources (DWR), the regional groundwater depth at the site could be as shallow as 20 feet bgs. However, groundwater levels may fluctuate both seasonally and from year to year due to variations in rainfall, temperature, pumping from wells and possibly as the result of other factors such as irrigation, that were not evident at the time of our investigation. Groundwater is not anticipated to affect construction.

3.4 Seismic Design Criteria

There are no known active fault zones within the vicinity of the project site. In accordance with Section 1613.2.2 of the 2019 California Building Code (CBC) and Table 20.3-1 of ASCE 7-16, the site can be classified as Site Class D (stiff soil profile).

Use of the 2019 California Building Code (CBC) seismic design criteria is considered appropriate and the following parameters are considered applicable for the structural design of foundations.

Table 1: Seismic Design Parameters

Seismic Design Parameter	2019 CBC Value		Reference
MCE Mapped Spectral Acceleration (g)	$S_S = 0.676$	$S_1 = 0.267$	USGS Mapped Value
Amplification Factors (Site Class D)[1]	$F_a = 1.259$	$F_v = 2.066$	Table 1613.2.3
Site Adjusted MCE Spectral Acceleration[1] (g)	$S_{MS} = 0.851$	$S_{M1} = 0.552$	Equations 11.4-1, 2, ASCE
Design Spectral Acceleration[1] (g)	$S_{DS} = 0.568$	$S_{D1} = 0.368$	Equations 11.4-3, 4, ASCE
Geometric Mean PGA (g)	$PGA_M = 0.371$		ASCE Equations 11.8-1
Long-period transition period (seconds)	12		ASCE Figures 22-14 through 22-17

Notes:

1. F_v must only be used for calculation of T_s.
2. See requirements for site-specific ground motions in Section 11.4.8 of ASCE 7. Values provided based on use of exception, as provided in Section 11.4.8.2 to Site-Specific Ground Motion Procedures and assumes the value of the seismic response coefficient C_s is 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 $T > T_L$.

As shown above, the short period design spectral response acceleration coefficient, S_{DS}, is greater than 0.5, therefore the Site lies in Seismic Design Category D as specified in Section 1613.2.5 of the 2019 CBC. The long period design spectral response acceleration coefficient, S_{D1}, is greater than 0.2, therefore the

Site lies in Seismic Design Category D as specified in Section 1613.2.5 of the 2019 CBC. In accordance with the 2019 CBC, each structure shall be assigned to the more severe seismic design category in accordance with Table 1613.2.5(1) or 1613.2.5(2), irrespective of the fundamental period of vibration of the structure.

3.4.1 Liquefaction

Liquefaction describes a condition in which a saturated, cohesionless soil loses shear strength during earthquake shocks. Ground motion from an earthquake may induce cyclic reversals of shearing strains of large amplitude. Lateral and vertical movements of the soil mass, combined with loss of bearing strength, usually result from this phenomenon. Historically, liquefaction of soils has caused severe damage to structures, berms, levees and roads. Seed and Idriss (1971) demonstrated that liquefaction potential depends on soil type, void ratio, depth to groundwater, duration of shaking and confining pressures over the potentially liquefiable soil mass. Fine, well-sorted, loose sand, shallow groundwater, severe seismic ground motion and particularly long durations of ground shaking are conditions conducive for liquefaction. Based on anticipated ground shaking at the site, approximately 1 inch of seismically induced settlement was calculated. Calculations are provided in Appendix C.

4. CONCLUSIONS AND RECOMMENDATIONS

Based upon the data collected during this investigation, and from a geotechnical engineering standpoint, it is our opinion that the soil conditions would not preclude the construction of the proposed improvements. The proposed improvements may be supported on shallow foundations or mat foundations provided the recommendations presented herein are incorporated into the design and construction of the project.

4.1 Soil Corrosivity

Soil samples were tested to evaluate the potential for concrete deterioration or steel corrosion due to attack by soluble salts in the soils at the proposed tank sites. Results are presented in Appendix B. Based on the test results, near-surface soils have minimal soluble sulfate and chloride contents, a low minimum resistivity and are alkaline. Thus, on-site soils are considered to have a low corrosion potential with respect to buried concrete and highly corrosive potential to unprotected metal. We recommend that Type II cement be used in the formulation of concrete, and that buried reinforcing steel protection be provided with a minimum concrete cover required by the American Concrete Institute (ACI) Building Code for Structural Concrete, ACI 318, Chapter 20. Buried metal conduits must have protective coatings in accordance with the manufacturer’s specifications. If detailed recommendations for corrosion protection are desired, a corrosion specialist should be consulted.

4.2 Site Preparation Recommendations

The following procedures must be implemented during site preparation for the proposed site improvements. References to maximum dry density, optimum moisture content, and relative compaction are based on ASTM D1557 (latest test revision) laboratory test procedures.

1. The proposed areas supporting foundations must be over-excavated to a minimum of one foot below the bottom of the proposed foundations or site grade, whichever is deeper. The over-excavation should extend a minimum of 5 feet from edge of the foundation or areas to receive fill. The bottom of the over-excavation must be scarified 8 inches, brought to at or above optimum moisture content and compacted to 90 percent of ASTM D1557.
2. Following the required stripping and over excavation, the exposed ground surface must be inspected by the Geotechnical Engineer to evaluate if loose or soft zones are present that will require over excavation.
3. Imported soil or native, non-expansive, excavated soils, free of organic materials or deleterious substances, may be placed as compacted engineered fill. The material must be free of oversized fragments greater than 3-inches in greatest dimension. Engineered fill must be placed in uniform layers not exceeding 8-inches in loose thickness, moisture conditioned at or above optimum moisture content, and compacted to at least 90 percent relative compaction.
4. BSK must be called to the site to verify the import material properties through laboratory testing.
5. If possible, earthwork operations should be scheduled during a dry, warm period of the year. Should these operations be performed during or shortly following periods of inclement weather, unstable soil conditions may result in the soils exhibiting a "pumping" condition. This condition is caused by excess moisture in combination with moving construction equipment, resulting in saturation and zero air voids in the soils. If this condition occurs, the adverse soils will need to be over-excavated to the depth at which stable soils are encountered, and replaced with suitable soils compacted as engineered fill. Alternatively, the Contractor may proceed with grading operations after utilizing a method to stabilize the soil subgrade, which should be subject to review and approval by BSK prior to implementation.
6. Import fill materials must be free from organic materials or deleterious substances. The project specifications must require the contractor to contact BSK to review the proposed import fill materials for conformance with these recommendations at least one week prior to importing to the Site, whether from on-site or off-site borrow areas. Imported fill soils must be non-hazardous and derived from a single, consistent soil type source conforming to the following criteria:

Plasticity Index:	< 12
Expansion Index:	< 20 (Very Low Expansion Potential)
Maximum Particle Size:	3 inches
Percent Passing #4 Sieve:	65 - 100
Percent Passing #200 Sieve:	20 - 45
Low Corrosion Potential:	Soluble Sulfates < 1,500 ppm
	Soluble Chlorides < 150 ppm
	Minimum Resistivity > 2,000 ohm-cm

4.3 Foundations

Provided the recommendations contained in this report are implemented during design and construction, it is our opinion that the structures can be supported on mat or shallow foundations. A structural engineer should evaluate reinforcement, embedment depth based on the requirements for the structural loadings, shrinkage and temperature stresses.

4.3.1 Shallow Foundations

The proposed structures may be supported on reinforced concrete spread footings bearing on engineered fill. Footing design must follow the criteria listed below:

The allowable bearing pressure applies to the dead load plus live load (DL + LL) condition. Footing design must follow the criteria listed below:

Table 2: Allowable Bearing Pressure				
Footing Embedment(2) (inches)	Minimum Footing Width (inches)		Allowable Bearing Capacity(1) (psf)	
	Continuous Footing	Isolated Spread Footing	Continuous Footing	Isolated Spread Footing
12	12	24	1,700	3,100

Note (1) – The bearing pressure can be increased one-third for transient loading such as wind or seismic.
(2) – Measure with respect to the lowest adjacent subgrade surface.

The estimated total and differential settlement for the recommended spread footings is shown below:

Table 3: Estimated Settlement			
Footing Type	Post-Construction Settlement (inches)	Differential Settlement (inches)	Angular Distortion
Continuous	1.0	--	0.001
Isolated	1.0	0.5	--

Isolated footing differential settlement is based on adjacent similarly loaded footings spaced at 30-feet. The settlement values given above are applicable to the maximum loading conditions. For loads, other than the design maximum loads, the settlements can be decreased proportionally.

4.3.2 Mat Foundations

Considering the anticipated base dimension of the proposed equalization tank, 24 feet, the allowable bearing capacity exceeds 10,000 psf. Based on analysis, the proposed mat foundations will have approximately up to 1.0 inch of total and approximately 0.5 inch of differential settlement for a design load of 2,000 psf. The mat foundations may be designed for a maximum allowable bearing pressure of 2,000 pounds per square foot (psf) due to dead plus live loads. This value may be increased by one-third for transient loads such as seismic or wind. If settlement analysis for other conditions is desired, we can provide settlement based on geometry and loading.

4.4 Concrete Slabs-on-Grade

Non-structural concrete slab-on-grade must be a minimum of 4-inches thick and must be supported on a compacted subgrade prepared in accordance with the “Site Preparation and Earthwork Construction” section of this report. Existing onsite surface soils are considered to have a low expansion potential for design purposes. In order to regulate cracking of the slabs, construction joints and/or saw-cut control joints must be provided in each direction at a maximum spacing of 10 feet on centers along with steel reinforcement as recommended by the project’s Structural Engineer. Control joints must have a minimum depth of one-quarter of the slab thickness. It is recommended that steel reinforcement used in concrete slabs-on-grade consist of steel rebar. Structural concrete slabs-on-grade may be designed using an unadjusted long-term Modulus of Subgrade Reaction (Ks) of 200 pounds per cubic inch (pci) constructed on a properly compacted subgrade or engineered fill. This value is based on the correlations to soil strength using one foot by one foot plate-load tests and should therefore be scaled (adjusted) to the actual slab width. For sandy soils, such as those found at this site, the adjusted Ks value can be obtained by multiplying the value provided above by $[(B+B_1)/(2B)]^2$, where B is the slab width in feet and B_1 is 1 foot (width of a one foot by one foot plate-load test apparatus).

4.5 Lateral Earth Pressures and Frictional Resistance

Provided the site is prepared as recommended above, the following earth pressure parameters for footings may be used for design purposes. The parameters shown in the table below are for drained conditions of select engineered fill or properly compacted and moisture conditioned native soil.

Table 4: Recommended Static Lateral Earth Pressures for Footings

Lateral Pressure Condition	Equivalent Fluid Density (pcf) Drained Condition
Active Pressure	40
At Rest Pressure	60
Passive Pressure	350

The lateral earth pressures listed herein are obtained by the conventional equation for active, at rest, and passive conditions assuming level backfill and a bulk unit weight of 120 pcf for the site soils. A coefficient of friction of 0.62 may be used between soil sub-grade and the bottom of footings. The coefficient of friction and passive earth pressure values given above represent ultimate soil strength values.

BSK recommends that a safety factor consistent with the design conditions be included in their usage in accordance with Sections 1806.3.1 through 1806.3.3 of the 2019 CBC. For stability against lateral sliding that is resisted solely by the passive earth pressure against footings or friction along the bottom of footings, a minimum safety factor of 1.5 is recommended. For stability against lateral sliding that is resisted by combined passive pressure and frictional resistance, a minimum safety factor of 2.0 is recommended. For lateral stability against seismic loading conditions, a minimum safety factor of 1.2 is recommended.

4.6 Excavation Stability

Soils encountered within the depth explored are generally classified as Type C soils in accordance with OSHA (Occupational Safety and Health Administration). The slopes surrounding or along temporary excavations should be no steeper than 1.5H:1V for excavations that are deeper than five feet, up to a maximum depth of 10 feet. Zones of poorly graded sand were encountered in the borings. If loose sands are encountered, the slopes should be laid back flatter. Certified trench shields or boxes may also be used to protect workers during construction in excavations that have vertical sidewalls and are greater than 5 feet deep. Temporary excavations for the project construction should be left open for as short a time as possible and should be protected from water runoff. In addition, equipment and/or soil stockpiles must be maintained at least H feet away from the top of the excavations, where H is the depth of the excavation. Because of variability in soils, BSK must be afforded the opportunity to observe and document sloping and shoring conditions at the time of construction. Slope height, slope inclination, and excavation depths (including utility trench excavations) must in no case exceed those specified in local, state, or federal safety regulations, (e.g., OSHA Health and Safety Standards for Excavations, 29 CFR Part 1926, or successor regulations).

4.7 Trench Backfill and Compaction

Processed, non-expansive, on-site soils, which are free of organic material, are suitable for use as general trench backfill above the pipe envelope. Non-expansive native soil with particles less than three inches in the greatest dimension may be incorporated into the backfill and compacted as specified above, provided they are properly mixed into a matrix of friable soils. The backfill must be placed in thin layers not exceeding 12 inches in loose thickness, be well-blended and consistent texture, moisture conditioned to at least optimum moisture content, and compacted to at least 90 percent of the maximum dry density as determined by the ASTM D1557.

We recommend that trench backfill be tested for compliance with the recommended Relative Compaction and moisture conditions. Field density testing should conform to ASTM Test Methods D1556 or D6938.

We recommend that field density tests be performed in the utility trench bedding, envelope and backfill for every vertical lift, at an approximate longitudinal spacing of not greater than 150 feet. Backfill that does not conform to the criteria specified in this section should be removed or reworked, as applicable over the trench length represented by the failing test so as to conform to BSK recommendations.

4.8 Drainage Considerations

The control surface drainage in the project areas is an important design consideration. BSK recommends that final grading around shallow foundations must provide for positive and enduring drainage away from the structures, and ponding of water must not be allowed around, or near the shallow foundations. Ground surface profiles next to the shallow foundations must have at least a 0.5 percent gradient away from the structures.

5. PLANS AND SPECIFICATIONS REVIEW

BSK recommends that it be retained to review the draft plans and specifications for the project, with regard to foundations and earthwork, prior to their being finalized and issued for construction bidding.

6. CONSTRUCTION TESTING AND OBSERVATIONS

Geotechnical testing and observation during construction is a vital extension of this geotechnical investigation. BSK recommends that it be retained for those services. Field review during Site preparation and grading allows for evaluation of the exposed soil conditions and confirmation or revision of the assumptions and extrapolations made in formulating the design parameters and recommendations. BSK's observations must be supplemented with periodic compaction tests to establish substantial conformance with these recommendations. BSK must also be called to the Site to observe foundation excavations, prior to placement of reinforcing steel or concrete, in order to assess whether the actual bearing conditions are compatible with the conditions anticipated during the preparation of this report. BSK must also be called to the Site to observe placement of foundation and slab concrete.

If a firm other than BSK is retained for these services during construction, then that firm must notify the owner, project designers, governmental building officials, and BSK that the firm has assumed the responsibility for all phases (i.e., both design and construction) of the project within the purview of the geotechnical engineer. Notification must indicate that the firm has reviewed this report and any subsequent addenda, and that it either agrees with BSK's conclusions and recommendations, or that it will provide independent recommendations.

7. LIMITATIONS

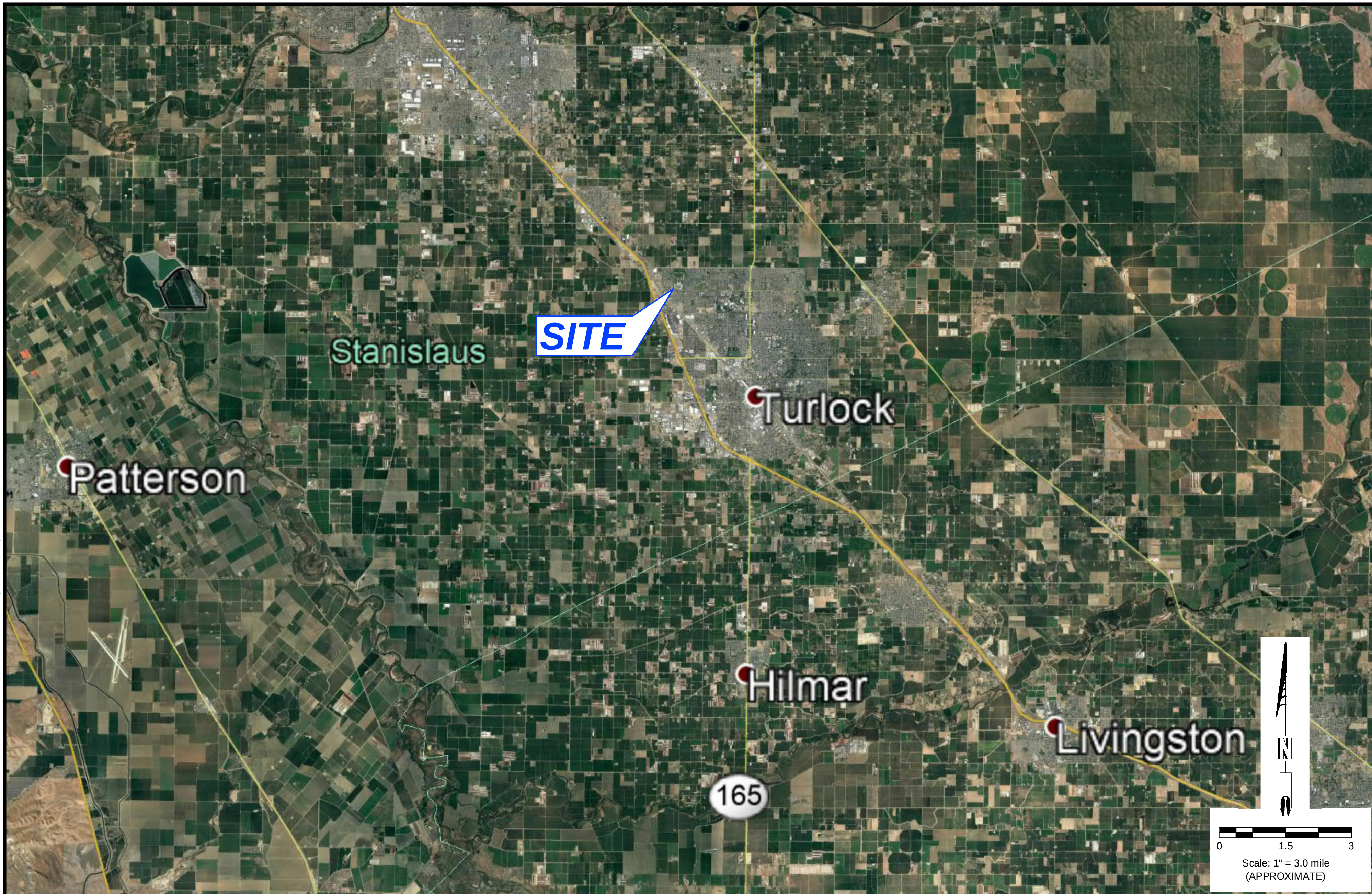
The analyses and recommendations submitted in this report are based upon the data obtained from the borings performed at the locations shown on the Boring Location Map, Figure 2. The report does not reflect variations which may occur between or beyond the borings. The nature and extent of such variations may not become evident until construction is initiated. If variations then appear, a reevaluation of the recommendations of this report will be necessary after performing on-site observations during the excavation period and noting the characteristics of the variations.

The validity of the recommendations contained in this report is also dependent upon an adequate testing and observation program during the construction phase. BSK assumes no responsibility for construction compliance with the design concepts or recommendations unless it has been retained to perform the testing and observation services during construction as described above.

The findings of this report are valid as of the present. However, changes in the conditions of the Site can occur with the passage of time, whether caused by natural processes or the work of man, on this property or adjacent property. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation, governmental policy or the broadening of knowledge.

BSK has prepared this report for the exclusive use of the Client and members of the project design team. The report has been prepared in accordance with generally accepted geotechnical engineering practices which existed in Stanislaus County at the time the report was written. No other warranties either expressed or implied are made as to the professional advice provided under the terms of BSK's agreement with Client and included in this report.

FIGURES



REFERENCE IMAGE: Google Earth

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SITE VICINITY MAP

Turlock Well 38 Arsenic Treatment
Turlock, California

FIGURE 1

JOB NO.	G19-223-11F
DATE	December 5, 2019
DR. BY	TG
CH. BY	NP
SCALE AS SHOWN	
SHEET NO.	1
OF	1 SHEETS

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LEGEND:

B-4 APPROXIMATE BORING LOCATIONS

REFERENCE IMAGE: Google Earth

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BORING LOCATION MAP

Turlock Well 38 Arsenic Treatment
Turlock, California

FIGURE 2

JOB NO. G19-223-11F
DATE December 5, 2019
DR. BY TG
CH. BY NP
SCALE AS SHOWN
SHEET NO. 1
OF 1 SHEETS

APPENDIX A

FIELD EXPLORATION

APPENDIX A
FIELD EXPLORATION

The field exploration was conducted on November 14, 2019, under the oversight of a BSK Staff Engineer. Four (4) test borings were drilled to depths of 16.5 to 41.5 feet below existing ground surface (bgs) at the proposed site. Borings were drilled using a truck-mounted drill rig equipped with hollow stem augers. The approximate locations of the test borings are indicated on Figure 2, Boring Location Map.

The soil materials encountered in the test borings were visually classified in the field, and logs were recorded by the staff engineer during the drilling and sampling operations. Visual classification of the materials encountered in the test borings were made in general accordance with the Unified Soil Classification System (ASTM: D2487). A soil classification chart is presented herein. Boring logs are presented herein and should be consulted for more details concerning subsurface conditions. Stratification lines were approximated by the field staff on the basis of observations made at the time of drilling while the actual boundaries between different soil types may be gradual and soil conditions may vary at other locations.

Subsurface samples were obtained at the successive depths shown on the boring logs by driving samplers which consisted of a 2.5-inch inside diameter (I.D.) California Sampler and a 1.4-inch I.D. Standard Penetration Test (SPT) Sampler. The samplers were driven 18-inches using a 140-pound hammer dropped from a height of 30-inches by means of either an automatic hammer or a down-hole “safety hammer”. The number of blows required to drive the last 12-inches was recorded as the blow count (blows/foot) on the boring logs. The relatively undisturbed soil core samples were capped at both ends to preserve the samples at their natural moisture content. Soil samples were also obtained using the SPT Sampler (marked X in logs) lined with metal tubes or unlined in which case the samples were placed and sealed in polyethylene bags. At the completion of the field exploration, the test borings were backfilled with the excavated soil cuttings.

It should be noted that the use of terms such as “loose”, “medium dense”, “dense” or “very dense” to describe the consistency of a soil is based on sampler blow count and is not necessarily reflective of the in-place density or unit weight of the soils being sampled. The relationship between sampler blow count and consistency is provided in the following Tables A-1 and A-2 for coarse-grained (sandy and gravelly) soils and fine grained (silty and clayey) soils, respectively.

Table A-1: Density of Coarse-Grained Soil versus Sampler Blow Count		
Consistency	SPT Blow Count Blows / Foot)	2.5” I.D. Cal. Sampler (Blows / Foot)
Very Loose	<4	<6
Loose	4 – 10	6 – 15
Medium Dense	10 – 30	15 – 45
Dense	30 – 50	45 – 80
Very Dense	>50	>80

Table A-2: Consistency of Fine-Grained Soil versus Sampler Blow Count		
Consistency	SPT Blow Count (Blows / Foot)	2.5” I.D. Cal. Sampler (Blows / Foot)
Very Soft	<2	<3
Soft	2 – 4	3 – 6
Medium Stiff	4 – 8	6 – 12
Stiff	8 – 15	12 – 24
Very Stiff	15 – 30	24 – 45
Hard	>30	>45

MAJOR DIVISIONS				TYPICAL NAMES
COARSE GRAINED SOILS More than Half > #200 sieve	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW	WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES
			GP	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES
		GRAVELS WITH OVER 15% FINES	GM	SILTY GRAVELS, POORLY GRADED GRAVEL-SAND-SILT MIXTURES
			GC	CLAYEY GRAVELS, POORLY GRADED GRAVEL-SAND-CLAY MIXTURES
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS WITH LITTLE OR NO FINES	SW	WELL GRADED SANDS, GRAVELLY SANDS
			SP	POORLY GRADED SANDS, GRAVELLY SANDS
		SANDS WITH OVER 15% FINES	SM	SILTY SANDS, POOORLY GRADED SAND-SILT MIXTURES
			SC	CLAYEY SANDS, POORLY GRADED SAND-CLAY MIXTURES
FINE GRAINED SOILS More than Half < #200 sieve	SILTS AND CLAYS LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS, OR CLAYEY SILTS WITH SLIGHT PLASTICITY
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	ORGANIC CLAYS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACIOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS			Pt	PEAT AND OTHER HIGHLY ORGANIC SOILS

	Modified California	RV	R-Value
	Standard Penetration Test (SPT)	SA	Sieve Analysis
	Split Spoon	SW	Swell Test
	Pushed Shelby Tube	TC	Cyclic Triaxial
	Auger Cuttings	TX	Unconsolidated Undrained Triaxial
	Grab Sample	TV	Torvane Shear
	Sample Attempt with No Recovery	UC	Unconfined Compression
CA	Chemical Analysis	(1.2)	(Shear Strength, ksf)
CN	Consolidation	WA	Wash Analysis
CP	Compaction	(20)	(with % Passing No. 200 Sieve)
DS	Direct Shear	▽	Water Level at Time of Drilling
PM	Permeability	▼	Water Level after Drilling(with date measured)
PP	Pocket Penetrometer		

SOIL CLASSIFICATION CHART AND LOG KEY

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Project: Turlock Well 38 Arsenic Treatment

Location: NWC Mountain View and West Christofferson Parkway

Project No.: G19-223-11F

Logged By: F. Gomez

Checked By: N. Popenoe

Boring: **B-1**

Depth (Feet)	Samples	Bulk Samples	Penetration Blows / Foot	In-Situ Dry Density (pcf)	In-Situ Moisture Content (%)	% Passing No. 200 Sieve	Graphic Log	USCS	MATERIAL DESCRIPTION	REMARKS
								SM	**Silty SAND** - light brown, moist, very dense, fine to medium grained	
1										
2										
3										
4										
5										
6			50/ 6"	112.4	11.1					
7										
8										
9										
10										
11			50/ 6"							
12										
13										
14										
15										
16			45					ML	**Sandy SILT** - grayish brown, moist, dense, trace clay	
17										
18										
19								SM	**Silty SAND** - reddish brown, moist, medium dense, fine to medium grained	
20										
21			15							
22									Boring terminated at approximately 21.5 feet bgs. Borehole backfilled with soil cuttings. No groundwater encountered.	
23										
24										

Drilling Contractor: Baja Exploration
Drilling Method: Hollow Stem Auger
Drilling Equipment: Mobile B-61
Date Started: 11/14/19
Date Completed: 11/14/19

Surface Elevation:
Sample Method: 2.5" I.D. Cal Mod & 1.5" I.D. SPT Split Spoon
Groundwater Depth: Not Encountered
Completion Depth: 21.5 Feet
Borehole Diameter: 8"

* See key sheet for symbols and abbreviations used above.

BSK ASSOCIATES
BSK Associates
550 W. Locust Ave.
Fresno
Telephone: 559-497-2880
Fax: 559-497-2864

Project: Turlock Well 38 Arsenic Treatment

Location: NWC Mountain View and West Christofferson Parkway
Project No.: G19-223-11F
Logged By: F. Gomez
Checked By: N. Popenoe

Boring: **B-2**

Depth (Feet)	Samples	Bulk Samples	Penetration Blows / Foot	In-Situ Dry Density (pcf)	In-Situ Moisture Content (%)	% Passing No. 200 Sieve	Graphic Log	USCS	MATERIAL DESCRIPTION	REMARKS
0								SM	**Silty SAND** - brown, moist, medium dense, fine to medium grained	
2–3.5	■		15	120.6	5.0	30				
5									... light brown	
5–6.5	■		17	113.5	5.7					
10								ML	**Sandy SILT** - light brown, moist, medium dense, fine to medium grained sand	
10–11.5	■		35	113.9	3.9					
15								SM	**Silty SAND** - light brown, moist, medium dense, fine to medium grained	
15–16.5	⧗		16							
20–21.5	■		22							
21.5									Boring terminated at approximately 21.5 feet bgs. Borehole backfilled with soil cuttings. No groundwater encountered.	

Drilling Contractor: Baja Exploration
Drilling Method: Hollow Stem Auger
Drilling Equipment: Mobile B-61
Date Started: 11/14/19
Date Completed: 11/14/19

Surface Elevation:
Sample Method: 2.5" I.D. Cal Mod & 1.5" I.D. SPT Split Spoon
Groundwater Depth: Not Encountered
Completion Depth: 21.5 Feet
Borehole Diameter: 8"

* See key sheet for symbols and abbreviations used above.

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Project: Turlock Well 38 Arsenic Treatment

Location: NWC Mountain View and West Christofferson Parkway

Project No.: G19-223-11F

Logged By: F. Gomez

Checked By: N. Popenoe

Boring: **B-3**

Depth (Feet)	Samples	Bulk Samples	Penetration Blows / Foot	In-Situ Dry Density (pcf)	In-Situ Moisture Content (%)	% Passing No. 200 Sieve	Graphic Log	USCS	MATERIAL DESCRIPTION	REMARKS
0								SM	**Silty SAND** - light brown, moist, medium dense, fine to medium grained	
3			24	124.9	5.6					
6			15	112.6	8.5					
10								SP	**Poorly Graded SAND** - brown, moist, medium dense, fine to medium grained	
11			17							
14								CL	**Sandy CLAY** - brown, moist, hard, fine grained sand	
16			50/ 6"							
15.5									Boring terminated at approximately 21.5 feet bgs. Borehole backfilled with soil cuttings. No groundwater encountered.	

Drilling Contractor: Baja Exploration
Drilling Method: Hollow Stem Auger
Drilling Equipment: Mobile B-61
Date Started: 11/14/19
Date Completed: 11/14/19

Surface Elevation:
Sample Method: 2.5" I.D. Cal Mod & 1.5" I.D. SPT Split Spoon
Groundwater Depth: Not Encountered
Completion Depth: 15.5 Feet
Borehole Diameter: 8"

* See key sheet for symbols and abbreviations used above.

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Project: Turlock Well 38 Arsenic Treatment

Location: NWC Mountain View and West Christofferson Parkway

Project No.: G19-223-11F

Logged By: F. Gomez

Checked By: N. Popenoe

Boring: **B-4**

Depth (Feet)	Samples	Bulk Samples	Penetration Blows / Foot	In-Situ Dry Density (pcf)	In-Situ Moisture Content (%)	% Passing No. 200 Sieve	Graphic Log	USCS	MATERIAL DESCRIPTION	REMARKS
0								SM	**Silty SAND** grayish brown, moist, medium dense, fine to medium grained	
3			21	110.5	4.2					
6			25	108.6	3.5					
10								ML	**Sandy SILT** - light brown, moist, very stiff, fine to medium grained sand	
11			42	112.2	10.6					
15									... hard	
16			48	119.5	7.6					
20								SM	**Silty SAND** - reddish brown, moist, medium dense, fine to medium grained	
21			17							

Drilling Contractor: Baja Exploration
Drilling Method: Hollow Stem Auger
Drilling Equipment: Mobile B-61
Date Started: 11/14/19
Date Completed: 11/14/19

Surface Elevation:
Sample Method: 2.5" I.D. Cal Mod & 1.5" I.D. SPT Split Spoon
Groundwater Depth: 38.48 Feet
Completion Depth: 41.5 Feet
Borehole Diameter: 8"

* See key sheet for symbols and abbreviations used above.

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Project: Turlock Well 38 Arsenic Treatment
Location: NWC Mountain View and West Christofferson Parkway
Project No.: G19-223-11F
Logged By: F. Gomez
Checked By: N. Popenoe

Boring: **B-4**

Depth (Feet)	Samples	Bulk Samples	Penetration Blows / Foot	In-Situ Dry Density (pcf)	In-Situ Moisture Content (%)	% Passing No. 200 Sieve	Graphic Log	USCS	MATERIAL DESCRIPTION	REMARKS
26			50/ 4.5"						**Silty SAND** - reddish brown, moist, medium dense, fine to medium grained *(continued)* ... very dense, increase sand content	
27										
28										
29										
30									... dense	
31			49							
32										
33										
34										
35								SP	**Poorly Graded SAND** - brown, wet, dense, fine to medium grained, trace silt	
36			32							
37										
38										∇
39										
40										
41			30							
42									Boring terminated at approximately 41.5 feet bgs. Borehole backfilled with soil cuttings. Groundwater encountered at 38.5 feet bgs.	
43										
44										
45										
46										
47										
48										
49										

Drilling Contractor: Baja Exploration
Drilling Method: Hollow Stem Auger
Drilling Equipment: Mobile B-61
Date Started: 11/14/19
Date Completed: 11/14/19

Surface Elevation:
Sample Method: 2.5" I.D. Cal Mod & 1.5" I.D. SPT Split Spoon
Groundwater Depth: 38.48 Feet
Completion Depth: 41.5 Feet
Borehole Diameter: 8"

* See key sheet for symbols and abbreviations used above.

APPENDIX B

LABORATORY TESTING RESULTS

APPENDIX B
LABORATORY TESTING RESULTS

Moisture-Density Tests

The field moisture content, as a percentage of dry weight of the soils, was determined by weighing the samples before and after oven drying in accordance with ASTM D2216 test procedures. Dry densities, in pounds per cubic foot, were also determined for undisturbed core samples in general accordance with ASTM D2937 test procedures. Test results are presented on the boring logs in Appendix A.

Sieve Analysis Test

One (1) Sieve Analysis Test was performed on selected soil samples in the area of planned construction. The test was performed in general accordance with Test Method ASTM D422. The results of the test are presented on Figure B-1.

Direct Shear Test

A direct shear test was performed on test specimens trimmed from a selected soil sample. The three-point shear test was performed in general accordance with ASTM Test Method D3080, Direct Shear Test for Soil under Consolidated Drained Conditions. The test specimens, each 2.42 inches in diameter and 1 inch in height, were subjected to shear along a plane at mid-height after allowing for pore pressure dissipation. The results of these tests are presented on Figure B-2.

Soil Corrosivity

The results of chemical analyses performed on a bulk soil sample using California Test Method (CT) 643 (for minimum resistivity and pH) and CT 417 and CT 422 (for soluble sulfate and chlorides, respectively) are presented below.

Sample Location	pH	Sulfate (mg/kg)	Chloride (mg/kg)	Minimum Resistivity (ohms-cm)
B-1 at 0 - 3’	7.7	92	23	1,440

FIGURE B-1

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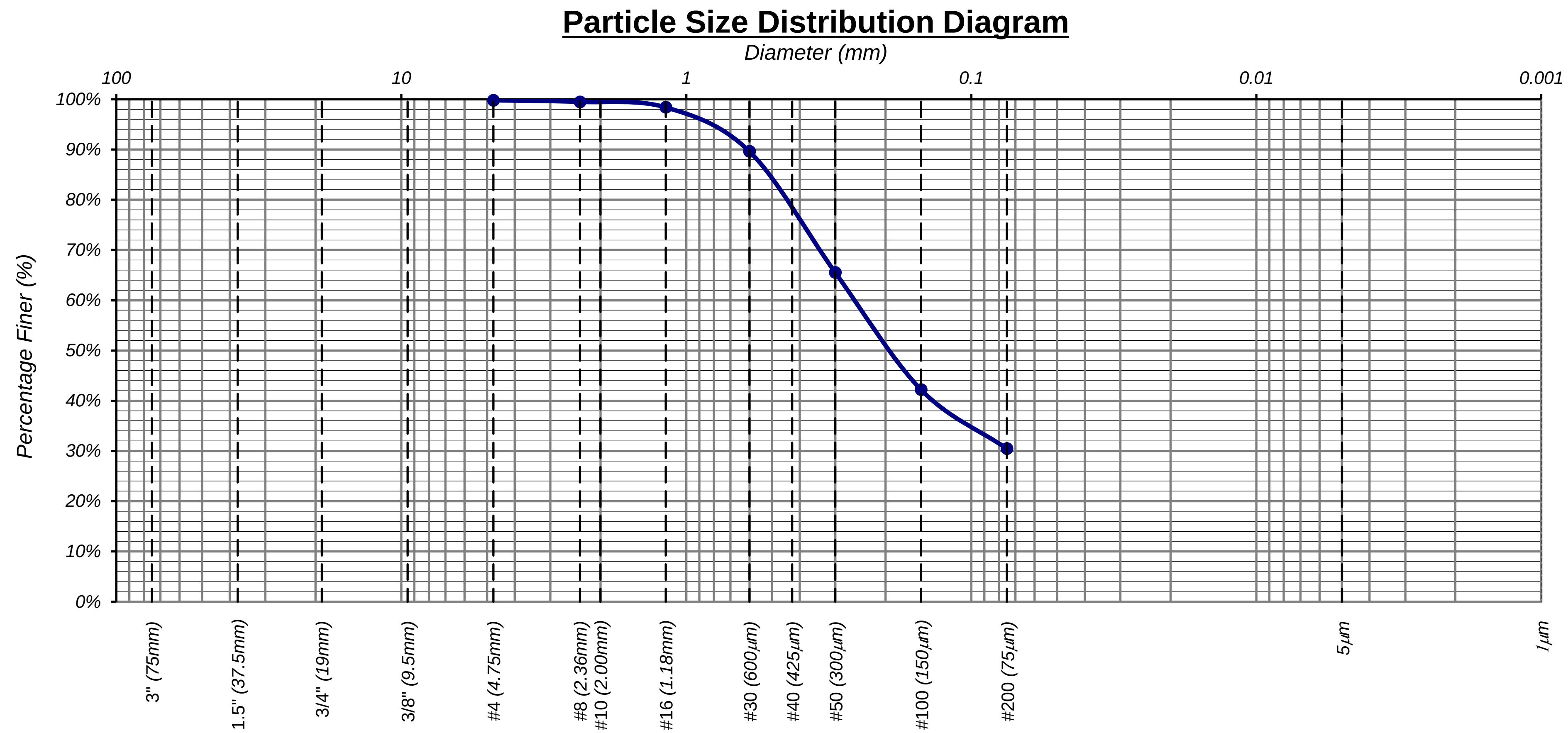
Fax: (559) 497-2886

Gradation Analysis Report
ASTM D-422 / ASTM C-136

Project Name:	Turlock Well 38	**Project Number:**	G19 - 223 - 10F	**Report Date:**	12/9/2019
Sample Location:	B - 2 @ 0 - 5'	**Sample Lab ID:**		**Sample Date:**	11/14/2019
Sample Description:	Silty SAND (SM) brown, moist & fine - med. Grained			**Test Date:**	11/16/2019



Clear Square Openings (ASTM C-136)			US Standard Series (ASTM D-422)			Hydrometer Readings (ASTM D-422)
Cobble	Gravel		Sand			Silt (Non-Plastic) to Clay (Plastic)
	Coarse	Fine	Coarse	Medium	Fine	

% Gravel = 0% **% Sand = 69%** **% Fines = 30%**

ASSOCIATES

Direct Shear Test

ASTM D-3080

FIGURE B-2
550 W. Locust
Fresno, CA 93650
Ph: (559) 497-2880
Fax: (559) 497-2886

Project Name: Turlock Well 38 Arsenic Treatment

Sampled By: F.G.

Sample Date: 11/14/2019

Tested By: D.M.

Test Date: 11/19/2019

Project Number: G19 - 223 - 10F

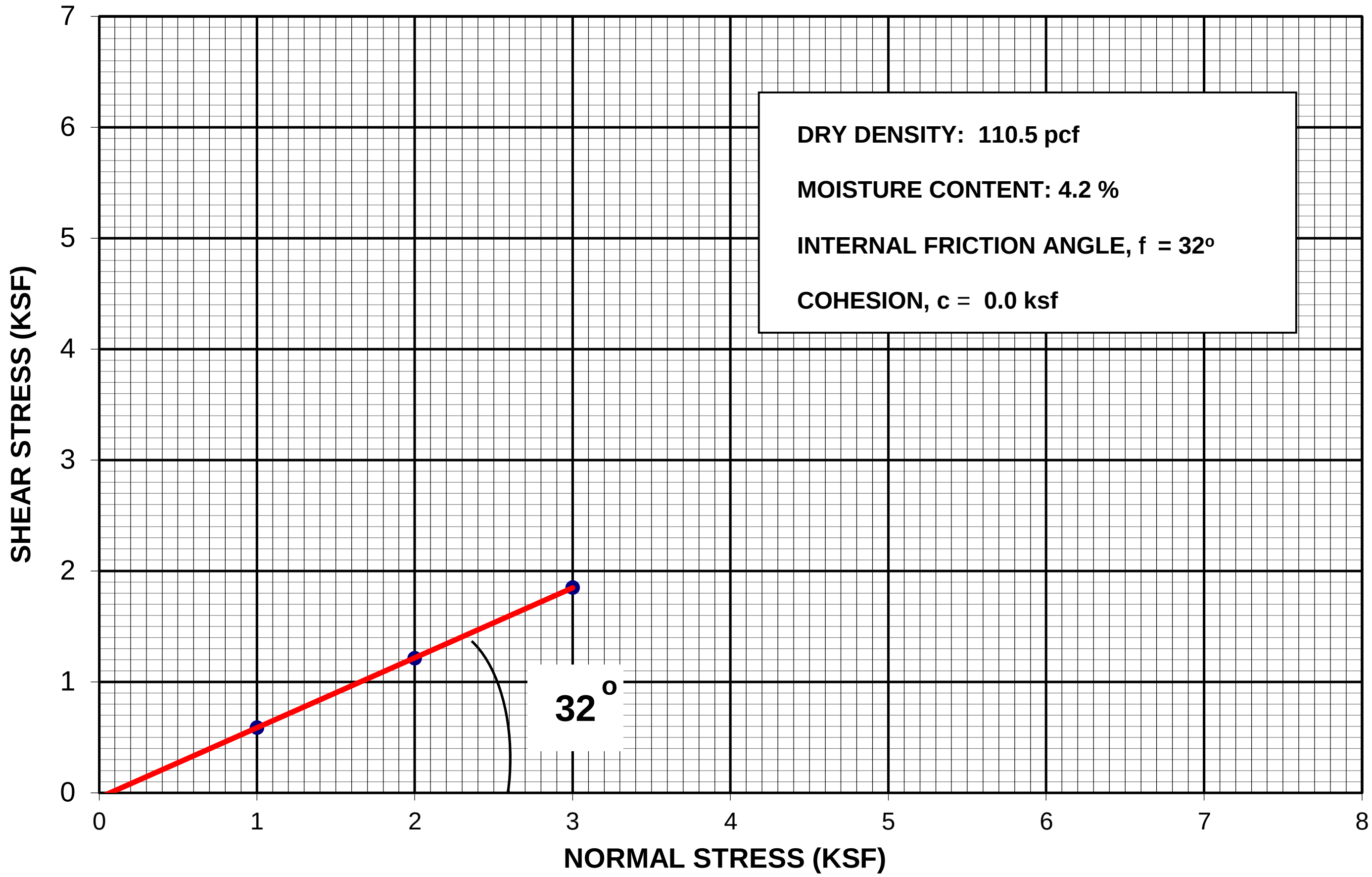
Lab Tracking ID: N/A

Report Date: 11/23/2019

Sample Location: B - 4 @ 2' - 3.5'

Sample Description: Silty SAND (SM) grayish brown, moist, fine to medium grained

SHEAR STRENGTH DIAGRAM



APPENDIX C

LIQUEFACTION CALCULATIONS

LIQUEFACTION ANALYSIS

Turlock Well 38 Arsenic Treatment

Hole No.=B-4   Water Depth=10 ft

Magnitude=5.5
Acceleration=0.371g

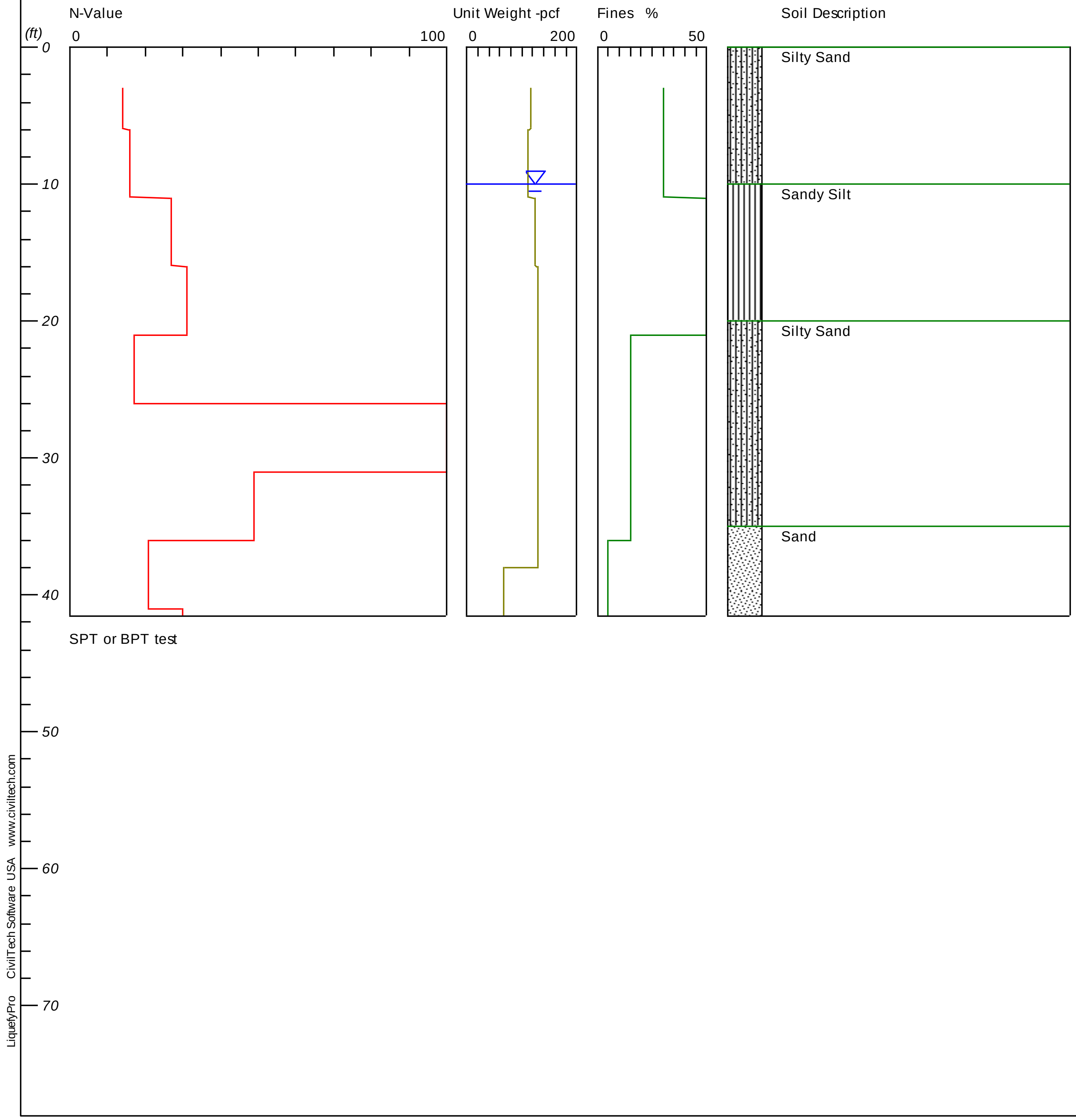


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Figure 3

LIQUEFACTION ANALYSIS

Turlock Well 38 Arsenic Treatment

Hole No.=B-4 Water Depth=10 ft

Magnitude=5.5
Acceleration=0.371g

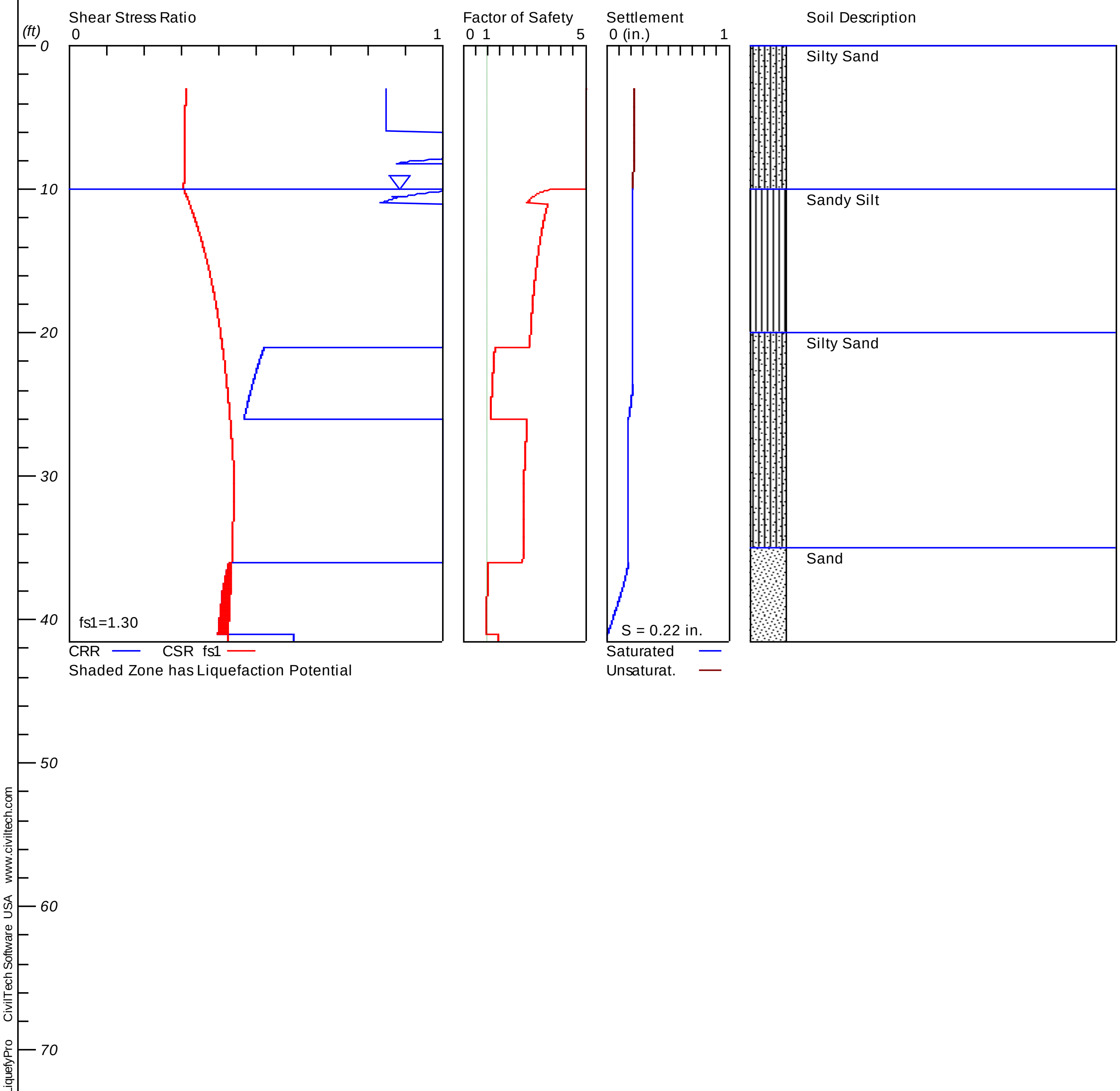

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Figure 3

****

LIQUEFACTION ANALYSIS SUMMARY
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Input File Name: P:\FRS\Active\GEO\G1922311F - Turlock Well 38 Arsenic Treatment\Data\B-4.liq
Title: Turlock Well 38 Arsenic Treatment
Subtitle:

Surface Elev.=
Hole No.=B-4
Depth of Hole= 41.50 ft
Water Table during Earthquake= 10.00 ft
Water Table during In-Situ Testing= 38.00 ft
Max. Acceleration= 0.37 g
Earthquake Magnitude= 5.50

Input Data:

Surface Elev.=
Hole No.=B-4
Depth of Hole=41.50 ft
Water Table during Earthquake= 10.00 ft
Water Table during In-Situ Testing= 38.00 ft
Max. Acceleration=0.37 g
Earthquake Magnitude=5.50
No-Liquefiable Soils: Based on Analysis

1. SPT or BPT Calculation.
2. Settlement Analysis Method: Tokimatsu, M-correction
3. Fines Correction for Liquefaction: Stark/Olson et al.*
4. Fine Correction for Settlement: During Liquefaction*
5. Settlement Calculation in: All zones*
6. Hammer Energy Ratio, Ce = 1.3
7. Borehole Diameter, Cb= 1
8. Sampling Method, Cs= 1
9. User request factor of safety (apply to CSR) , User= 1.3
   Plot one CSR curve (fs1=User)
10. Use Curve Smoothing: No

* Recommended Options

In-Situ Test Data:

Depth ft	SPT	gamma pcf	Fines %
3.00	14.00	115.00	30.00
6.00	16.00	112.00	30.00
11.00	27.00	124.00	50.00
16.00	31.00	129.00	50.00
21.00	17.00	129.00	15.00
26.00	100.00	129.00	15.00
31.00	49.00	129.00	15.00
36.00	21.00	129.00	5.00
41.00	30.00	129.00	5.00

Output Results:

Settlement of Saturated Sands=0.21 in.
Settlement of Unsaturated Sands=0.01 in.
Total Settlement of Saturated and Unsaturated Sands=0.22 in.
Differential Settlement=0.111 to 0.147 in.