**Phone:** (707) 441-8855 **Email:** info@shn-engr.com **Web:** shn-engr.com
812 W. Wabash Avenue, Eureka, CA 95501-2138

Reference: 022138

April 24, 2023

Danco Group
c/o McKenzie Dibble
5251 Erickson Way
Arcata, CA 95521

**Subject: Updated Geotechnical Recommendations, Proposed Three-Story Building on Taylor Way, Blue Lake, California**

Dear McKenzie Dibble:

# Introduction

This letter report presents our updated geotechnical recommendations for seismic design parameters to be used in the design of the subject project. When the 2022 California Building Code (CBC) took effect this year, there was confusion as to the correct American Society of Civil Engineers (ASCE) 7 Standard to use, and we began using ASCE 7-22 to determine seismic design parameters. We now understand that the 2022 CBC did not adopt ASCE 7-22, and it continues to use ASCE 7-16. Our March 22, 2023, report provided parameters from ASCE 7-22 instead of ASCE 7-16. Below are the updated seismic design parameters from the ASCE 7-16 Standard.

## Seismic Design Parameters

Based on the subsurface conditions encountered at our exploration locations, laboratory test results, and our interpretation of soil conditions within 100 feet of the ground surface, we classify the site as a Site Class D consisting of a "Stiff Soil Profile" in accordance with Chapter 20 of ASCE 7-16. On this basis, the mapped and design spectral response accelerations were determined using the ASCE 7 Hazard Tool (accessed April 24, 2023) in conjunction with the site class and site coordinates at the location of the proposed building. Calculated values for ASCE 7-16 are presented in the table below.

**Table 1. ASCE 7-16 Spectral Acceleration Parameters (40.879528$^0$, -123.996769$^0$)**

| Parameter | 0.2 Second | 1 Second |
|---|---|---|
| Maximum Considered Earthquake Spectral Acceleration (MCE$_R$) | $S_S$ = 2.817 | $S_1$ = 1.058 |
| Site Class | D | |
| Site Amplification factor | $F_a$ = 1.0 | $F_v$ = N/A |
| Site-modified spectral acceleration | $S_{MS}$ = 2.817 | $S_{M1}$ = N/A |
| Numeric seismic design value | $S_{DS}$ = 1.878 | $S_{D1}$ = N/A |
| MCE$_G$ peak ground acceleration (PGA) | 1.167 | |
| Site amplification factor at PGA (F$_{PGA}$) | 1.1 | |
| Site modified peak ground acceleration (PGA$_M$) | 1.284 | |

# Closure

We trust this provides the updated recommendations you require. We apologize for any inconvenience. If you have any comments or concerns, please call me at (707) 441-8855.

Sincerely,

**SHN**

REGISTERED PROFESSIONAL ENGINEER
JOHN H. DAILEY
No. 256
Exp. 3/31/24
GEOTECHNICAL
STATE OF CALIFORNIA

4/24/23

John H. Dailey, PE, GE
Senior Geotechnical Engineer

JHD:ame

# Geotechnical Investigation and Geologic Hazards Evaluation for Proposed Three-Story Building

## APNs 312-161-018 and 312-161-015
## Taylor Way, Blue Lake, California

**Prepared for:**

Danco Group

**March 2023**

**022138**

**Phone:** (707) 441-8855 **Email:** info@shn-engr.com
**Web:** shn-engr.com • 812 W. Wabash Ave, Eureka, CA 95501

**Phone:** (707) 459-4518 **Email:** info@shn-engr.com **Web:** shn-engr.com
812 W. Wabash Ave, Eureka, CA 95502

Reference: 022138

March 22, 2023

Danco Group
C/O McKenzie Dibble
5251 Ericson Way
Arcata, CA 95521

**Subject: Geotechnical Investigation and Geologic Hazards Evaluation for Proposed Three-Story Building on Taylor Way, Blue Lake, California**

To McKenzie Dibble:

This report presents the results of a geotechnical investigation and geologic hazards evaluation conducted by SHN for the proposed mixed residential and commercial building to be constructed on Assessor's Parcel Numbers (APNs) 312-161-015 and 312-161-018, located on Taylor Way in Blue Lake, California. The primary purpose of this investigation was to assess site subsurface conditions and to develop geotechnical recommendations in support of the design and construction of the proposed new building.

We appreciate this opportunity to work with you on this project. If additional information or clarification is required, please contact us at 707-441-8855.

Sincerely,insert

**SHN**

Jason Buck, CEG
Senior Engineering Geologist

John H. Dailey, PE, GE
Senior Geotechnical Engineer

JPD:JHD:dkl

Enclosure: Report

# Geotechnical Investigation and Geologic Hazards Evaluation for Proposed Three-Story Building, Taylor Way, Blue Lake, California

Prepared for:

**Danco Group**

CERTIFIED ENGINEERING GEOLOGIST
JASON PAUL
BUCK
No. 2641
STATE OF CALIFORNIA

3/22/23

Expiration July 31, 2024

REGISTERED PROFESSIONAL ENGINEER
JOHN H. DAILEY
No. 256
Exp. 3/31/24
GEOTECHNICAL
STATE OF CALIFORNIA

3/22/23

Prepared by:

812 W. Wabash Avenue
Eureka, CA 95502
(707) 441-8855

March 2023

QA/QC: GDS GDS
Reference: 022138

# Table of Contents

**Appendices**

# List of Illustrations

# Abbreviations and Acronyms

## Units of Measure

| Term | Definition |
|---|---|
| bpf | blows per foot |
| g | acceleration of gravity |
| mm | millimeters |
| ohms-cm | ohms-centimeter |
| pcf | pounds per cubic foot |
| pci | pounds per cubic inch |
| ppm | parts per million |
| psf | pounds per square foot |
| psi | pounds per square inch |
| μm | micrometers |

## Additional Terms

| Term | Definition |
|---|---|
| ACI | American Concrete Institute |
| ASCE | American Society of Civil Engineers |
| ASTM | ASTM-International |
| B-# | boring number |
| Caltrans | California Department of Transportation |
| CBC | California Building Code |
| CGS | California Geological Survey |
| DSA | California Division of the State Architect |
| $F_{PGA}$ | Site amplification factor at PGA |
| H:V | horizontal to vertical |
| ID | internal diameter |
| $MCE_R$ | Maximum Considered Earthquake Spectral Acceleration |
| MCS | Modified California Sampler |
| NR | no reference |
| OD | outside diameter |
| OSHA | Occupational Safety and Health Administration |
| PGA | $MCE_G$ peak ground acceleration |
| $PGA_M$ | site modified peak ground acceleration |
| SDC | Seismic Design Category |
| SPT | standard penetration test |
| USGS | United States Geological Survey |

# 1.0 Introduction

## 1.1 General

This geotechnical report presents the results of SHN's field and laboratory investigation for the proposed three-story mixed residential and commercial building to be constructed on APNs 312-161-018 and 312-161-015 on Taylor Way in Blue Lake, California (Figure 1 and Figure 2). This report was prepared for the sole use of Danco Group and its design consultants. The report is intended to comply with criteria presented in Section 1803, "Chapter 18A: Geotechnical Investigations," of the 2022 California Building Code (CBC), the requirements of Humboldt County and the City of Blue Lake (as appropriate).

The conclusions and recommendations presented in this report are provided to assist the project design consultants in addressing the design and construction of the proposed building. This report is based on the data obtained from our field investigation, the results of laboratory testing performed on samples obtained from the geotechnical borings, and a review of previous reporting, published geologic literature and mapping in the vicinity of the project site.

## 1.2 Site History and Previous Work

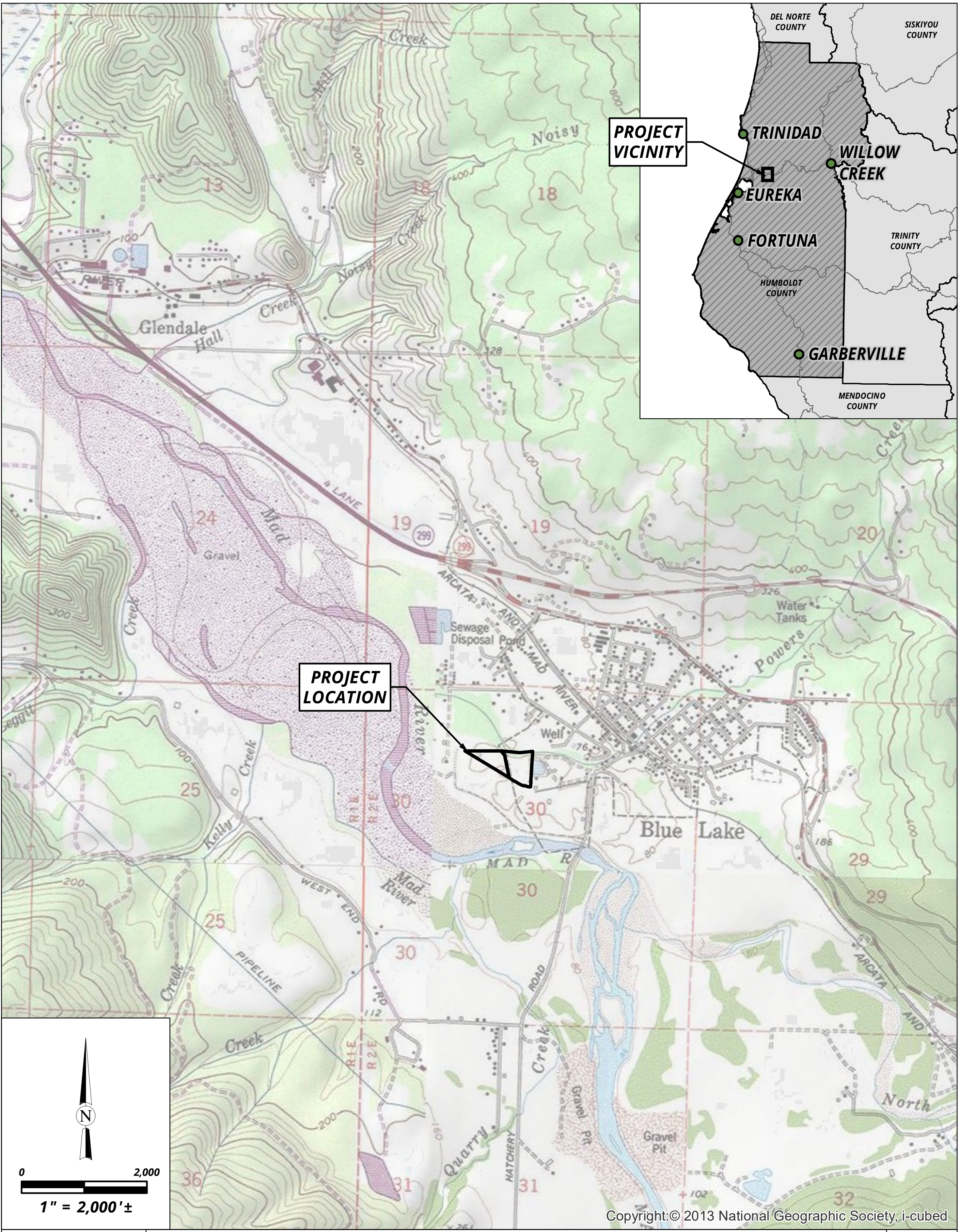
Reporting in the project vicinity (NGS, 1981; SHN, 2008), as well as historic aerial photography from 1956 (Figure 3) indicate that previously, the portion of the subject parcels north of Taylor Way (in its present location) were mostly occupied by a log pond associated with a lumber mill. The lumber mill (which included the pond) was constructed between 1941 and 1954 and a major portion of the pond was backfilled prior to 1974 (NGS, 1981). The area of the proposed building footprint is within the boundaries of the former log pond, and as such, the existing ground surface is considered non-native.

As part of our work, we reviewed the results of previous geologic/geotechnical studies in the project vicinity. Northern Geotechnical Services (NGS, 1981) conducted a preliminary soils investigation to support the development of the Blue Lake Industrial Park, within which the current project is located. In 2013, SHN conducted a site investigation (borings and test pits) to assess subsurface conditions that may have been impacted by historic site activities (SHN, 2013). SHN also conducted a geotechnical investigation for a commercial development off Mondo Way, east of the project site (SHN, 2008, 2013). The locations of exploratory test pits and borings on the subject parcels investigated during the NGS (1981) and SHN (2013) studies are shown on Figure 2.

## 1.3 Project Description

Our understanding of the project is based on communications with Danco Group and their design architect, and on our review of the architect's "Site Plan Study" (December 3, 2022), which contains a site map depicting the proposed building location and dimensions (shown on Figure 2). We understand that the project will include the construction of a three-story mixed residential and commercial building. The proposed building will be located on the north side of Taylor Way, south of Powers Creek. Based on discussions with the project team, we understand the owner intends to utilize a reinforced shallow foundation system for support of the building.

The project site is generally flat and minimal grading will be necessary to develop the parking areas, sidewalks and access roadways.



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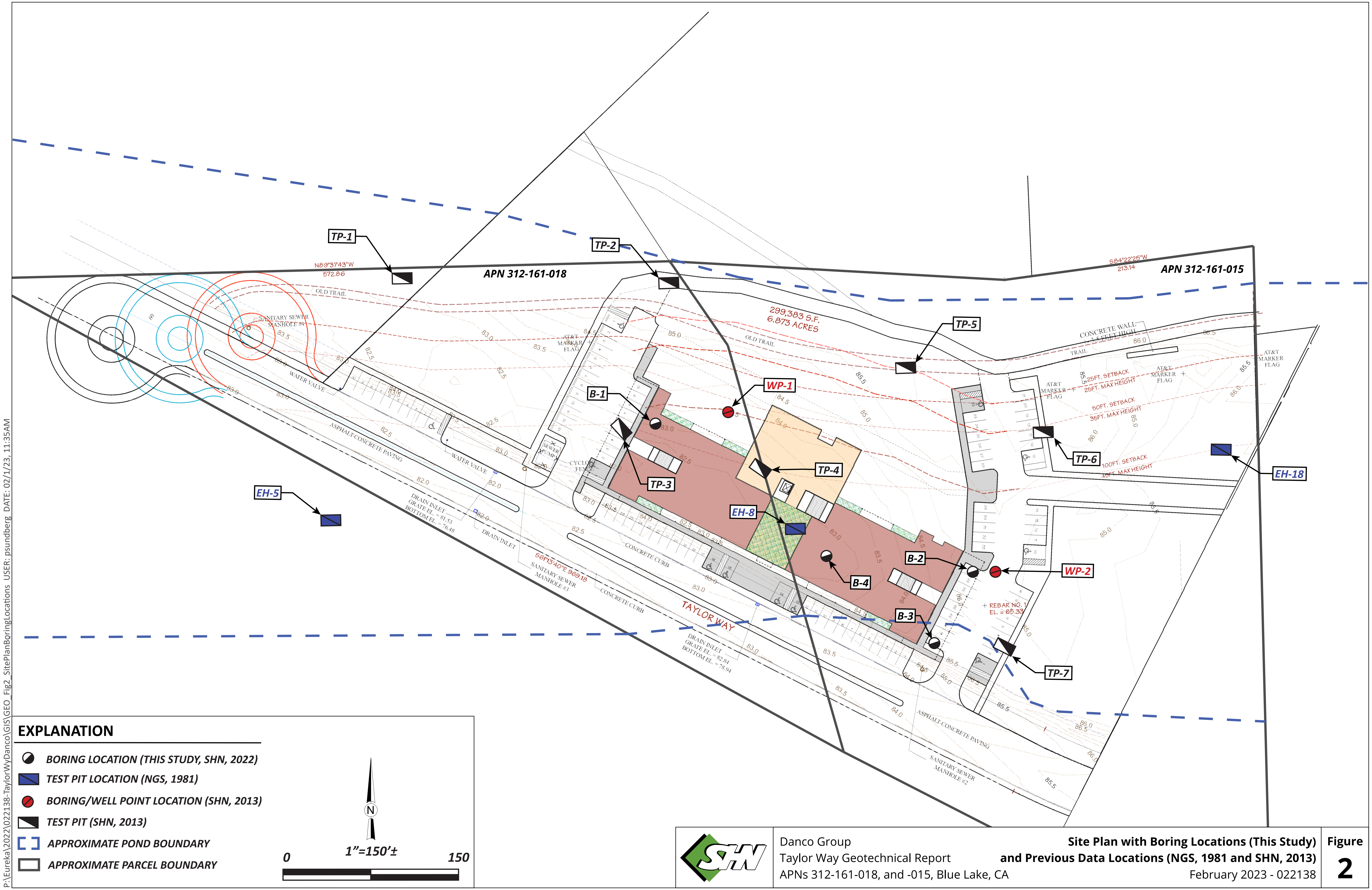
Danco Report
Taylor Way Geotechnical Report
APNs 312-161-018, and -015, Blue Lake, CA

**Project Location Map**

February 2023 - 022138

**Figure**

**1**



## EXPLANATION

◐ BORING LOCATION (THIS STUDY, SHN, 2022)
▨ TEST PIT LOCATION (NGS, 1981)
● BORING/WELL POINT LOCATION (SHN, 2013)
◩ TEST PIT (SHN, 2013)
[ ] APPROXIMATE POND BOUNDARY
☐ APPROXIMATE PARCEL BOUNDARY

0 1"=150'± 150

Danco Group
Taylor Way Geotechnical Report
APNs 312-161-018, and -015, Blue Lake, CA

**Site Plan with Boring Locations (This Study) and Previous Data Locations (NGS, 1981 and SHN, 2013)**
February 2023 - 022138

**Figure 2**

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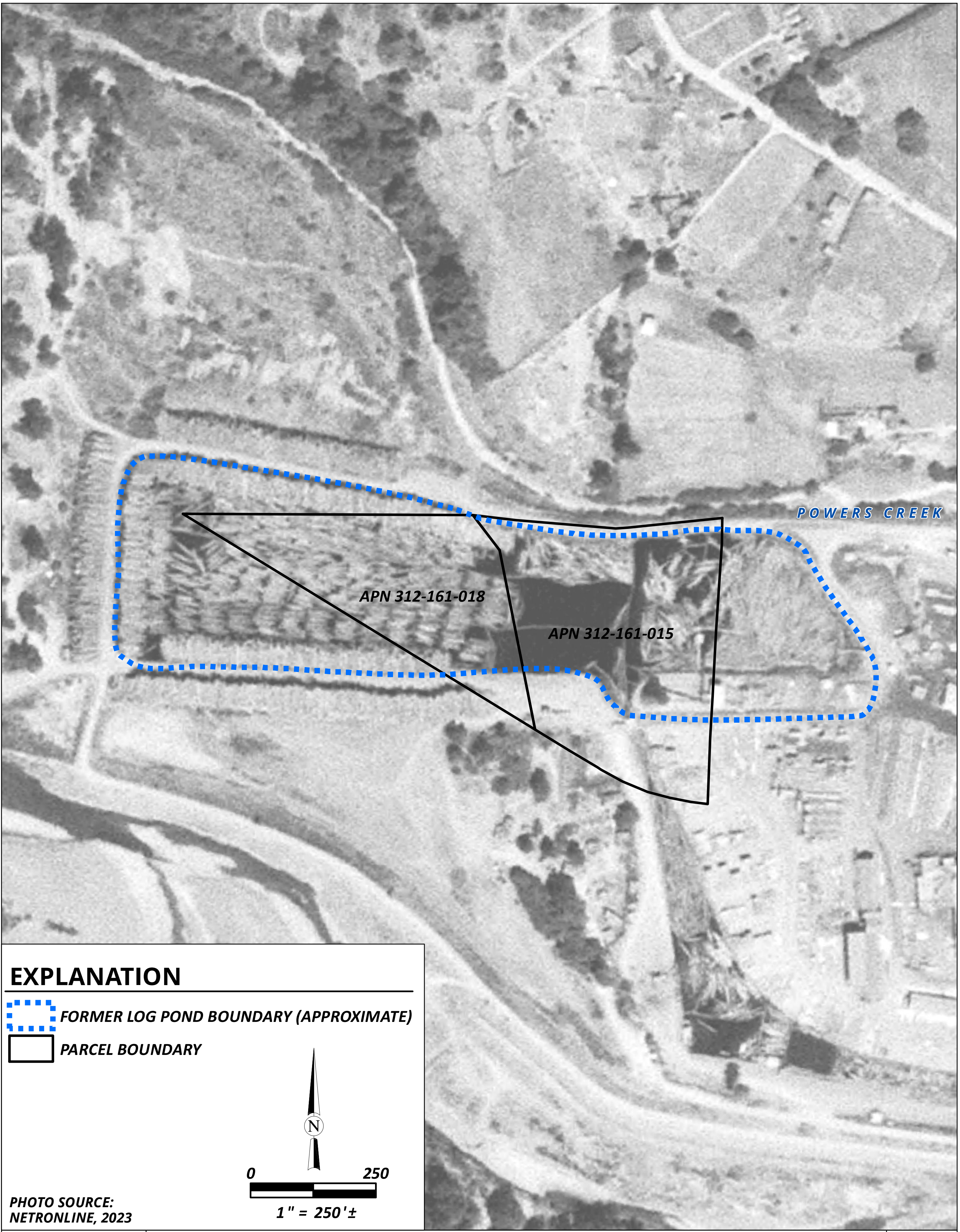

Danco Group
Taylor Way Geotechnical Report
APNs 312-161-018, and -015, Blue Lake, CA

**Former Log Pond (1956)**

February 2023 - 022138

**Figure**
**3**

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# 2.0 Scope of Work

The scope of our services included the following:

- Review nearby geotechnical and geologic reports of the property and published geologic and geologic hazard maps.
- Perform site reconnaissance to observe existing site conditions and mark the exploration areas for USA (Underground Service Alert).
- Drill four (4) borings near the proposed building. Three (3) borings were advanced to approximately 25 to 35 feet below the ground surface. One (1) boring was advanced to a depth of 50 feet below the ground surface to address the liquefaction potential beneath the site. Samples from each boring were collected using standard penetration test (SPT) and modified California split spoon samplers.
- Samples collected were returned to SHN's soils testing laboratory for geotechnical analysis. Tests included dry density and moisture content, percent passing the #200 sieve, Atterberg Limits, and R-Value.
- Assessment of potential earthquake-related geologic/geotechnical hazards (for example, strong earthquake ground shaking, surface fault rupture, liquefaction, and differential settlement), and other potential geologic/geotechnical hazards.
- Perform engineering analyses in order to provide conclusions and recommendations regarding a.) earthwork, including site and subgrade preparation, fill material specifications, and fill compaction requirements, b.) discussion of appropriate foundation options, including allowable bearing capacities, estimates of settlement (total and differential), minimum footing depth, and allowable lateral capacities, c.) support of concrete slabs-on-grade, and d.) recommendations for observation of site preparation and grading, observation of foundation installation, and other geotechnical construction considerations.
- Preparation of this report summarizing our findings and recommendations, complete with field and laboratory data.

# 3.0 Field Investigation and Laboratory Testing

The geotechnical field exploration and laboratory testing programs carried out for this study are summarized below. Results of the field and laboratory testing programs are presented in Appendices 1 and 2, respectively.

## 3.1 Field Exploration Program

Four mud-rotary and solid flight auger exploratory borings were drilled on December 13-14, 2022, by Taber Drilling of Sacramento, using a track-mounted CME-300 drill rig equipped with solid flight augers and an automatic hammer for standard penetration testing (SPT). The borings were advanced to total depths of approximately 35 feet (B-1 and B-2), 25 feet (B-3), and 50 feet (B-4) below the existing ground surface (BGS). The locations of the exploratory borings are shown on Figure 2.

Visual classifications of the earth materials encountered in the borings were made in general accordance with the Manual-Visual Classification Method (ASTM-International [ASTM] D 2488). Final geotechnical boring logs were prepared based on conditions encountered in the field, examination of samples in the laboratory, and the results of laboratory testing. Boring logs are included as Appendix 1.

Relatively undisturbed soil samples were obtained by driving a 2.5-inch internal diameter (ID), 3.0-inch outside diameter (OD), Modified California Sampler (MCS) containing steel liners and a 1.5-inch ID, 2.0-inch OD SPT sampler without liners in accordance with ASTM D1586 standards. The samplers were advanced using a 140-pound auto-hammer falling 30 inches per blow. The number of hammer blows required to drive the samplers the last 12 inches of an 18-inch drive is provided on the boring logs reflecting the penetration resistance of the material (shown as blows per foot [bpf]). The penetration resistance values (bpf) recorded for SPT sampler drives and provided on the boring logs are actual penetration resistance (N-values) that are uncorrected for depth and the energy transfer ratio of the automatic hammers used. The penetration resistance values provided on boring logs for the MCS sampler drives are field blow counts and should not be construed as SPT N-values. Approximate equivalent SPT N-values for the MCS sampler should be multiplied by a factor of 0.64.

## 3.2 Laboratory Testing

Selected soil samples were submitted to SHN's soils testing laboratory in Eureka to determine index properties and strength characteristics of the subsurface materials. Samples were tested for in-place moisture content, dry density, percent fines, liquid limit, plasticity index, and R-value. Results of the tests are provided at the corresponding sample locations on the boring logs (Appendix 1) and included as Appendix 2.

# 4.0 Site Conditions

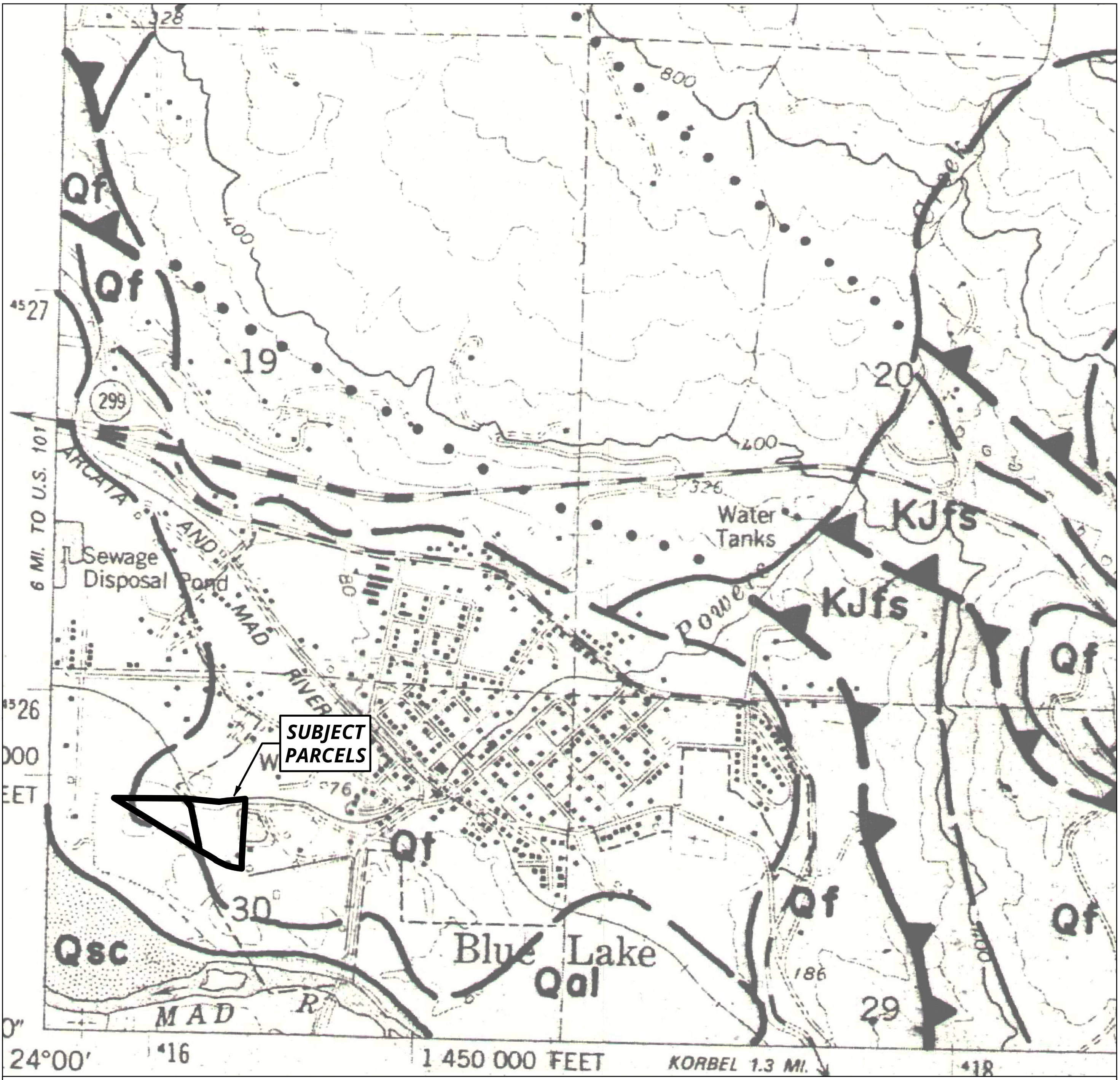
The following sections describe the project site and surface conditions, the geologic setting of the site, and subsurface soil and groundwater conditions encountered at the time of our field investigation.

## 4.1 Site Surface Description

The site is in the western Blue Lake area, approximately 1000 feet north of the active channel of the Mad River, on a relatively level portion of the river flood plain. The proposed building footprint is situated on a generally flat, grass covered surface adjacent to a public walking path along Powers Creek (to the north) and Taylor Way (to the south). Elevations in the proposed building area range between 80 to 85 feet (Figure 2).

## 4.2 Geologic Setting

Basement rock within the region is composed of late Jurassic to late Cretaceous age mélange of the Franciscan Complex (McLaughlin et al., 2000). In the Blue Lake region, Franciscan rock is overlain by early to middle Pleistocene age marine and continental deposits of the Falor formation (Carver, Stephens, and Young, 1985). In the project vicinity, Franciscan basement rock and Falor Formation deposits are overlain by a veneer of late Quaternary river terrace deposits associated with ancestral alignments of the Mad River. These terraces typically consist of an abrasion platform cut across bedrock, with river terrace sediments consisting of alluvial deposits (interbedded sand, gravel, and silt). Review of published geologic mapping by Carver, Stephens, and Young (1985; Figure 4) indicates that the site is underlain by these river terrace deposits.



## EXPLANATION

*Qsc - Quaternary stream channel*

*Qal - Quaternary alluvium*

*Qt - Quaternary terrace*

*Qf - Quaternary Falor Formation*

*KJfs - Cretaceous-Jurassic Fransiscan Sandstone*

*Thrust fault*
*(Dotted where concealed)*

N

0 1,200

*1" = 1,200'±*

**DATA SOURCE:**
**COUNTY GIS, 2023**

Danco Group
Taylor Way Geotechnical Report
APNs 312-161-018, and -015, Blue Lake, CA

**Geologic Map**
**Carver, Stephens, and Young (1985)**
February 2023 - 022138

**Figure**
**4**

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## 4.3 Subsurface Soil Conditions

Our understanding of the subsurface soil conditions is based on review of previous work in the area (NGS, 1981; SHN, 2008; SHN, 2013) and the results of our subsurface investigation conducted in December 2022. Figure 2 shows the locations of the relevant subsurface explorations from previous studies and those from the current study. The logs associated with the subsurface explorations shown on Figure 2 are included as Appendix 1.

As discussed above, the proposed project overlies the site of an old log pond active in the 1950s/60s that has subsequently been backfilled. We used an aerial image from 1956 to estimate the boundary of the pond which is shown on Figure 2 relative to the proposed project. The proposed building location appears to be almost entirely located over the top of the backfilled pond with a small portion in the southeast corner that extends outside. The pond has been backfilled for some time now, but we are not aware of any records of the backfill activities; preparation of subgrade, methods for placement, and compaction effort.

During our subsurface exploration we encountered up to 12 feet of undocumented fill, which we associate with the backfilling of the former log pond. At the location of B-4, the fill was 6 feet thick. Fill may not be present at all in the southeast corner of the proposed building (outside log pond location). The fill materials encountered in our borings primarily consisted of gravels and sands. A 2-foot layer of fine sand and silt with mixed organics was encountered at the base of the fill in B-2 and B-3, interpreted to be sediment that had settled at the bottom of the pond. Soil densities in the fill were generally moderate to high except for the mid to lower portion of the fill where penetration resistance values (blows per foot) were below 10 (loose/soft). This low-density interval was observed in B-1, B-2 and B-3. B-4 had 6 feet of fill, with no observed loose/soft intervals noted. It should be noted that the presence of gravels can influence the field blows per foot recorded during the standard penetration test such that they can be erroneously high. Soil densities may be lower than the recorded values would indicate. Only minor organic materials were noted in the borings for this study, however previous explorations encountered wood/logs (TP-4) and layers of bark and/or mixtures of gravel and burned cinders (EH-5 and EH-18).

Beneath the fill, we encountered medium dense to dense, well-graded, interbedded sands and gravels we interpret to be native alluvial deposits of the ancestral Mad River. In the deepest boring (B-4), the alluvial deposits are underlain by Falor formation bedrock at an approximate depth of 40 feet BGS.

Groundwater was encountered during our site investigation (December 13-14, 2022) at approximately 7 feet (B-1, B-2, and B-3), and at 6 feet (B-4) BGS. Groundwater levels at the time of our investigation (mid-December) would be expected to be at or near the seasonal high. Groundwater levels can be expected to be higher during periods of intense precipitation.

# 5.0 Geologic Hazards

Potential geologic/geotechnical hazards common to the local area include seismic ground shaking, surface fault rupture, and adverse soil conditions. The assessment of these potential hazards is presented in this section.

## 5.1 Seismic Ground Shaking

The entire North Coast region is a seismically active area where strong seismic shaking presents a significant hazard. That hazard is present at the proposed building site, but it is no greater than that present elsewhere in the region. The site is approximately 1.1 miles west of the McKinleyville fault, which is considered active by the State of California. Additionally, the Cascadia subduction zone is located approximately 45 miles to the west, offshore. Based on the proximity to these active faults, the site can be expected to experience strong seismic ground shaking during the design life of the project.

## 5.2 Surface Fault Rupture

The site is not located within an Alquist-Priolo Earthquake Fault Hazard Zone (Bryant and Hart, 2007). The McKinleyville fault, which is the closest recognized active fault, is approximately 1.1 miles east of the project site. The project site is located on a planar terrace surface that exhibits no geomorphic evidence that would suggest previous surface rupture. It is our opinion that the potential for surface fault rupture at the site is negligible.

## 5.3 Soil Liquefaction Potential

Liquefaction is the sudden loss of soil shear strength due to a rapid increase of soil pore water pressures that occurs in response to strong seismic ground shaking. The adverse effects of liquefaction include the amplification of seismic shaking, localized ground settlement and ground cracking, the expulsion of water and sand, the partial or complete loss of bearing and confining forces used to support building loads and mitigate lateral spreading.

Based on the published results of geotechnical testing and post-earthquake studies, a soil's susceptibility to liquefaction can be directly correlated to the type, origin, and age of the deposit. Materials that are most susceptible to liquefaction are geologically young, unconsolidated loose sands and soft silt-rich deposits located in river valleys, bay margins, and along ocean shorelines. Post-liquefaction studies indicate that the likelihood of liquefaction occurring decreases with increasing geologic age (Youd and Perkins, 1978).

Qualitatively, some intervals in the fill materials meet the criteria for deposits susceptible to liquefaction; that is, young (historically placed), loosely consolidated sandy soils that are saturated. These conditions are observed towards the mid to lower portion of the fill materials, generally at depths between 7 to 10 feet below grade. The liquefaction potential in these saturated fill materials is considered moderate to high during a relatively rare, very strong or prolonged earthquake. The upper fill is suitably dense and generally not saturated. The native soils underlying the fill materials appear to be suitably dense and well graded (include coarse gravels) such that the liquefaction potential is considered low.

Quantitative liquefaction modeling was completed for this project to evaluate susceptible layers and the magnitude of seismic settlement predicted. The results of the analysis are included as Appendix 3 and indicate that each boring location has some intervals falling into a category of liquefaction potential. The soil profiles in B-1 and B-2 show the highest liquefaction potential with seismic settlement potential on the order of 1.75 to 3.5 inches, respectively. The soil profiles in B-3 and B-4 have lower liquefaction potential with seismic settlement potential (within the fill) on the order of 0.3 to 0.5 inches, respectively. It should be noted that the blows per foot may be artificially high due to the presence of gravels in the soil profile (discussed in Section 4.3, above). Therefore, the seismic settlement potential may be higher than that calculated. The liquefaction analysis was performed with a modeled seismic event with an earthquake magnitude ($M_W$) of 9.1 and a peak ground acceleration of 1.49g.

Liquefaction, which in our opinion is likely to be associated only with a relatively rare, very strong or prolonged earthquake, presents the following estimated risks:

- a low to moderate risk of sand boils at the ground surface;
- a moderate to high risk of a few inches of co-seismic subsidence, including a high potential for differential seismic settlement; and
- a low to moderate risk of differential ground movement beneath the building site from lateral spreading.

We provide recommendations to reduce these risks in Section 7 below.

## 5.4 Seismic Design Parameters

Based on the subsurface conditions encountered in the borings advanced on the site, laboratory test results, and our interpretation of soil conditions within 100 feet of the ground surface, we classify the site as a Site Class D, consisting of a "stiff soil profile" in accordance with Chapter 20 of ASCE 7-22. On this basis, the mapped and design spectral response accelerations were determined using the ASCE 7 Hazard Tool (accessed 01/31/23) in conjunction with the site class and site coordinates at the location of the proposed building. Calculated values for ASCE 7-22 are presented in the table below.

**Table 1. ASCE 7-22 Spectral Acceleration Parameters (40.879528$^{0}$, -123.996769$^{0}$)**

| Parameter | 0.2 Second | 1 Second |
|---|---|---|
| Maximum Considered Earthquake Spectral Acceleration ($MCE_R$) | $S_S = 3.39$ | $S_1 = 1.15$ |
| Site Class | D = Stiff Soil | |
| Site-modified spectral acceleration | $S_{MS} = 3.52$ | $S_{M1} = 2.45$ |
| Numeric seismic design value | $S_{DS} = 2.34$ | $S_{D1} = 1.64$ |
| Seismic Design Category (SDC) | E | |
| Site modified peak ground acceleration ($PGA_M$) | 1.36 | |
| Long-period transition period ($T_L$) | 8 | |
| Time averaged shear wave velocity to 30 meters depth ($V_{S30}$) | 260 | |

# 6.0 Geotechnical Discussion and Conclusions

Based on the results of our investigation, SHN concludes the site can be developed as planned for the proposed building construction, provided the recommendations presented in this report are followed and that associated geologic and geotechnical risks are acknowledged. The primary geotechnical consideration affecting the design and construction of the project is the presence of up to 12 feet of undocumented fill of variable density underlying most of the proposed project. The variability of fill materials, intervals of low-density sands, soft silts and organics within the fill, and the variation of fill thicknesses all contribute to a risk of total and differential settlement of structural elements over time, including seismically-induced settlement associated with earthquakes.

The site is located in an area susceptible to a multitude of seismically induced hazards, including strong seismic ground shaking, liquefaction susceptibility, and total and differential settlement (both under static and seismic conditions). The above-noted hazards pose a significant risk to the structural integrity of the new building over the course of its design life, particularly because it is located on a backfilled pond site. Static settlement under structural loads and seismically-induced differential settlements associated with a rare, very large or prolonged earthquake is estimated at up to 2 to 3 inches, or more, if a typical shallow foundation system were to be used. Alternatives discussed with the project team for reducing this risk and providing uniform foundation support under the proposed structure include:

1. Removal and replacement of the fill materials with engineered fill; this option eliminates the problem soils and ensures that no layers or pockets of unsuitable soils remain. Susceptibility to settlement would be mitigated for all project elements supported on the engineered fill.
2. Support the structure on a deep foundation system that penetrates through the fill; this option mitigates the hazard of static and seismic settlement for project elements supported by the deep foundation. Sidewalks, parking areas and other project elements that are not supported by the deep foundation remain susceptible to settlement.
3. Preloading the building footprint and/or project site; this option mitigates the potential for static settlement, but does not mitigate seismic settlement associated with liquefaction
4. Support the structure on a mat slab foundation underlain by a reinforced soil mat; this system does not mitigate the potential for total settlement (seismic or static), but it reduces the potential for concentrated differential settlement.

It is our understanding that the owner has chosen to utilize a reinforced shallow foundation system, and we therefore provide specific recommendations for the use of this type of system. If other alternatives are considered, we should be consulted to provide appropriate recommendations.

A shallow mat foundation system can be used if it is suitably sized and reinforced and supported on a minimum 4-foot-thick layer of geogrid-reinforced engineered fill mat below the entire structure. The placement of a geogrid–reinforced engineered fill mat below the proposed structure is intended to minimize (but not eliminate) the estimated differential settlements caused by any settlement of the remaining undocumented fills, and any underlying liquefaction-susceptible soils that undergo volumetric strain due to post-liquefaction reconsolidation. In addition, the high tensile strength of the geogrid reinforcement is expected to reduce the potentially damaging effects associated with liquefaction-induced ground surface deformation, if they were to occur.

All geotechnical-related work should be performed in accordance with the recommendations of the Geotechnical Engineer-of-Record during construction. Where the recommendations of this report and the cited sections of Title 24 are in conflict, the Owner and Architect should request clarification from the Geotechnical Engineer-of-Record. The recommendations in this report should not be waived without the consent of the Geotechnical Engineer-of-Record for the project. The following subsections present recommendations for the geotechnical-related work.

# 7.0 Recommendations

## 7.1 Site Preparation and Grading

### 7.1.1 General Recommendations

Site preparation includes removal of debris, organics, organic topsoil, loose soil, and any other unsuitable material. Site preparation operations should extend at least 5 feet beyond the limits of improvements. We anticipate that stripping to a depth of about 2 to 4 inches will be required to remove the organics and topsoil. Deeper stripping may be locally required to remove concentrations of vegetation, such as brush. The cleared vegetation and debris should be removed from the site, but the strippings can be stockpiled for reuse in landscape areas.

Any vegetation and organic topsoil with more than 2 percent organic material by dry weight should be removed. The Geotechnical Engineer or qualified representative should observe and approve the prepared site prior to any excavation, subgrade preparation, and placement of fill or improvements.

We expect that the site soils will be excavatable with conventional grading and trenching equipment. If grading commences in the winter or spring, or after a period of excessive rainfall, it is likely that the surficial soils may become saturated. Wet or saturated soil may cause difficulties in access with grading and trenching equipment and difficulties in loading, spreading, and compaction of fill material. Moisture conditioning and/or aerating of the site soils may be required. The time required for drying can be reduced by disking, ripping, or otherwise aerating the soil.

The contractor shall be responsible for the stability of all temporary excavations and should comply with applicable Occupational Safety and Health Administration (OSHA) regulations (California Construction Safety Orders, Title 8). The Contractor should periodically monitor all open cuts for evidence of incipient stability failures.

### 7.1.2 Reinforced Soil Mat Construction

The area to contain the proposed building and for a horizontal distance of at least 5 feet beyond, should be over-excavated to a minimum depth of 4 feet below proposed subgrade elevation. This will allow for the removal of a substantial portion of the undocumented fill soils of variable density. The over-excavated subgrade should be scarified to a depth of 6 inches, moisture conditioned or aerated and recompacted to 90 percent relative compaction[1]. To prevent pumping of the subgrade, compaction should be conducted under static mode only (that is, no vibratory compaction). The Geotechnical Engineer or qualified representative should observe and approve the over-excavation, and prior to subgrade preparation and placement of engineered fill or improvements.

Following recompaction of the over-excavated subgrade, we recommend that a layer of geogrid (Tensar InterAx™ NX750 or equivalent) be placed on the exposed subgrade prior to backfilling the overexcavated area with engineered fill. A second layer of geogrid should be placed at the midpoint of the 4 feet of replaced engineered fill. We, therefore, anticipate that approximately 4 feet of engineered fill, with 2 layers of geogrid, will be placed below the proposed building footprint.

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[1] Relative compaction refers to the in-place dry density of a soil expressed as a percentage of the maximum dry density of the same soil, as determined by the ASTM D1557-12 Test Method. Optimum moisture content is the water content (percentage by dry weight) corresponding to the maximum dry density.

## 7.2 Select Engineered Fills

Fill placed in areas to support proposed foundations should meet the requirements for select engineered fill. Select engineered fill should have less than 2 percent by dry weight of vegetation and deleterious material and should meet the gradation requirements presented in Table 2.

**Table 2. Fill Gradation Criteria**

| Sieve Designation | Percent Passing by Dry Weight |
|---|---|
| 3-inch (50 mm)[a] | 100 |
| 2½-inch (37.5 mm) | 85 minimum |
| ¾-inch (19 mm) | 70 minimum |
| No. 4 (4.75 mm) | 60 minimum |
| No. 200 (75 µm)[b] | 5 minimum, 30 maximum |

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[a] mm: millimeters

[b] µm: micrometers

We anticipate that onsite soils will be suitable for reuse as select engineered fill following removal of debris, organics, and any other unsuitable material. Fine-grained soil with a liquid limit greater than 40 and a plasticity index greater than 15 should not be used as select engineered fill. If clayey soils do not meet the plasticity requirements, mixing of the clayey soils with sandier soils may be required. Crushing and/or removal of rock particles greater than 3 inches in size will be required. Select engineered fill should have a low corrosion potential, which is defined as a minimum resistivity of 2,000 ohms-centimeter (ohms-cm) and maximum sulfate and chloride concentrations of 250 parts per million (ppm). In addition, we do not recommend using river-run material as select engineered fill; crushed, angular material is preferred with at least 50 percent of the material (as determined by the material's dry weight) containing a minimum of two fractured faces.

Engineered fill should be placed in loose lifts not exceeding 8 inches in thickness and compacted to a minimum of 90 percent relative compaction. The Geotechnical Engineer or qualified representative should approve all fill prior to placement. A qualified field technician should be present to observe fill placement and perform field density tests in accordance with ASTM D 6938 at random locations throughout each lift to verify that the specified compaction is being achieved.

Samples of any proposed import fill materials should be submitted to SHN for approval at least 3 business days prior to use at the site.

## 7.3 Wet Weather Subgrade Protection

Contractors should expect high soil moisture conditions in the near-surface soils throughout the wet season and into the late spring months following a typical winter wet season, and in the common perennially wet areas at the site. The wet season in coastal northern California generally begins in the month of November and continues through May. Heavy rains are also not uncommon during the months of October and June. Beginning construction activities and earthwork immediately prior to the onset of the wet season is not advised and will likely lead to delays if measures are not taken to stabilize and protect the exposed subgrade.

Protection of the subgrade, if necessary, is the responsibility of the contractor. Track-mounted excavating equipment may be required during and following wet weather. The contractor will be responsible for constructing an all-weather access road and staging area, as necessary. The thickness of the haul road to access the site for construction and staging areas will depend on the amount and type of construction traffic. The materials used for haul roads or site access drives should be stabilization material consisting of pit or quarry run rock that is well-graded, angular, crushed rock consisting of 4- to 6-inch minus material with less than 5 percent passing the U.S. Standard No. 4 Sieve. The material should be free of organic matter and other deleterious material. A minimum 6- to 12-inch-thick mat of stabilization material should be used for light staging areas. The stabilization material for haul roads and areas with repeated heavy construction traffic will likely need to be increased to between 12- to 18-inches. The actual thickness of haul roads and staging areas is the contractor's responsibility and should be based on the contractor's approach to site work and the amount and type of construction traffic. The stabilization material should be placed in one lift over the prepared, undisturbed subgrade and compacted using a smooth-drum, non-vibratory roller. Additionally, a geotextile fabric should be placed as a barrier between the subgrade and stabilization material. The geotextile should meet specifications for soil separation and stabilization, such as Mirafi 600X or equivalent.

## 7.4 Surface and Subsurface Drainage Control

Surface drainage should be planned to prevent ponding and enable water to drain away from foundations, slabs-on-grade, and edges of pavements, and towards suitable collection or discharge facilities. A positive surface drainage of at least 4 percent is recommended within 10 feet of all building foundations in unpaved areas. In paved areas, a positive surface drainage of at least 2 percent is recommended to allow for rapid removal of surface water. Roof drainage systems should be planned to direct rainwater away from building foundations.

Concentrated water should not be discharged onto bare ground but should be carried in pipes or lined channels to suitable disposal points. The use of water-intensive landscaping around the perimeter of structures should be avoided to reduce the amount of water introduced to the subgrade. Irrigation of landscaping around structures should be limited to drip or bubbler-type systems. Trees with large roots should also be avoided since they can dry out the soil beneath foundations and cause settlement. The purpose of these recommendations is to avoid large differential moisture changes adjacent to foundations, which have been known to cause large differential movement over short horizontal distances in expansive soils, resulting in cracking of slabs and architectural damage.

In addition, surface drainage should adhere to the setbacks for low-impact development (LID) features, if required, as shown in Table 3.

**Table 3. Recommended Setbacks for LID[a] Features**

| Type of LID Feature | Setback from Building Foundations | Setback from Pavement Sections and Exterior Slabs-on-Grade |
|---|---|---|
| Designed to infiltrate collected and concentrated stormwater (that is, dry wells, vegetated swales, bioretention facilities) | 10 feet | 3 feet[b] |
| Alternative engineered hardscaping (that is, porous asphalt, permeable pavers) subject only to incidental rainfall (*not* subject to re-routed, concentrated stormwater) | 5 feet | 3 feet |

---

[a] LID- low impact development

[b] Setback is not required *only if* an effective barrier is installed (such as a concrete-filled cutoff trench that prevents moisture from traveling from the LID feature to below the pavement section/slab-on-grade).

## 7.5 Utility Trench Backfill

New utility trenches excavated parallel to spread footing foundations should be set back from the footings such that the trench bottoms lie outside a projected hypothetical 1.5H:1V (horizontal to vertical) line extending downward from the footing bottom.

Unless concrete bedding is required around utilities, bedding should consist of sand having a sand equivalent (SE) of at least 30. The bedding should extend from 6 inches below to 1 foot above the conduit or pipe. Sand bedding should not be jetted or ponded into place and should be mechanically compacted to a minimum of 90 percent relative compaction.

In areas to support improvements (such as adjacent-to-structure foundations), backfill placed above the bedding in utility trenches (including culvert and sprinkler lines) should be properly placed and adequately compacted to minimize settlement and provide a stable subgrade. If possible, the trench backfill should be compacted following rough grading, but prior to final grading and compaction. Onsite inorganic soils meeting the requirements for engineered fill may be used as trench backfill. Backfill consisting of onsite soils should be placed in layers not exceeding 8 inches in loose thickness, moisture-conditioned, and compacted to at least 90 percent relative compaction as described for engineered fill. Trench backfill need only be compacted to 85 percent relative compaction in landscape areas or in areas more than 5 feet beyond the limits of building foundations.

Where utility trenches cross underneath buildings, we recommend that a plug be placed within the trench backfill to minimize the normally granular backfill from acting as a conduit for water to enter beneath the building. The plug should be constructed using sand cement slurry (minimum 28-day compressive strength of 500 pounds per square inch [psi]) or relatively impermeable native soil for pipe bedding or backfill. We recommend that the plug extend a distance of at least 3 feet in each direction from the point where the utility enters the building perimeter.

## 7.6 Mat Slab Foundation

As discussed in Section 6.0 above, a reinforced shallow foundation system is the preferred alternative chosen for supporting the structure. We recommend that a suitably sized and reinforced mat slab foundation be constructed on a geogrid-reinforced soil mat at least 4 feet thick.

The foundation should be designed using a maximum allowable bearing capacity of 2,500 pounds per square foot (psf) for dead plus normal duration live loads. The allowable bearing capacity may be increased by one-third when considering short-term wind and seismic loads.

The mat foundation system should be constructed on a compacted geogrid-reinforced soil mat with two layers of triaxial geogrid reinforcement designed and constructed as described in Section 7.1.2, above. It is important that the foundation excavations are moist, clean, and free of drying cracks, debris, loose sand and gravel, and water at the time the foundation is cast. Foundation excavations should be checked and approved by the Geotechnical Engineer or qualified representative immediately prior to placing concrete.

For the geogrid-reinforced soil mat and foundation using the allowable bearing values given above, we estimate a maximum settlement under static loading conditions of less than 1 inch. Differential settlement is not expected to exceed half the estimated maximum.

### 7.6.1 Subgrade Modulus for Mat Design

For mat design, we recommend using the following equation to estimate the subgrade modulus:

$$K_s = k_1 \left\{ \frac{(B+1)}{2B} \right\}^2$$

where:

$k_1$ = coefficient of subgrade reaction for 1-foot square plate = 300 pounds per cubic inch (pci)

B = width beneath column or bearing wall, in feet, where stresses are imposed on ground

The value of B and the corresponding $K_s$ value should be consistent with the calculated deflected shape of the foundation beneath columns and bearing walls.

### 7.6.2 Lateral Resistance

Base friction resistance may be calculated using a friction coefficient of 0.35 (ultimate value for concrete on engineered fill material). The ultimate friction coefficient may be as low as 0.15 if waterproofing is used, depending on the waterproofing. Passive resistance may be calculated using an equivalent fluid unit weight of 300 pounds per cubic foot (pcf). This value is reduced by a factor of 1.5 from the ultimate value to limit movement required to mobilize ultimate passive pressure. Both the ultimate base friction and allowable passive pressure may be combined in calculating total lateral resistance. The passive resistance contributed by fill material within 1 foot of the ground surface should be neglected unless these materials are protected and confined by a slab-on-grade or pavement.

The mat foundation should be cast neat against the engineered fill to develop the design passive resistance. Alternatively, any gap between the foundation and the adjacent ground should be completely backfilled using lean concrete.

## 7.7 Sidewalks and Other Flatwork Areas

In general, we recommend that exterior concrete flatwork be supported on a minimum of 4 inches of Class II crushed aggregate base compacted to a minimum of 90 percent relative compaction.

## 7.8 Asphalt Pavement Areas

Pavement construction should conform to the requirements of the Caltrans Standard Specifications, latest edition. Recommendations for both flexible pavements (asphalt concrete) and rigid pavements (Portland cement concrete) are provided below.

Recommended minimum pavement sections for standard flexible asphalt concrete are given below in Table 4 for various traffic loading conditions. The recommended pavement sections are based on a laboratory R-Value of 59 for the gravelly sand with clay that currently surfaces the site. Pavement sections for other traffic loading should be designed on a case-by-case basis.

**Table 4. Minimum Pavement Sections, Standard Flexible Asphalt Concrete Pavement**

| Traffic Index | Asphalt Concrete Thickness (inches) | Class 2 Aggregate Base Thickness (inches) |
|---|---|---|
| 5 and below | 2.5 | 6 |
| 6 | 3 | 6 |
| 7 | 4 | 6 |

Aggregate used for asphalt concrete surfacing should conform to the grading specified in Caltrans Standard Specifications Section 39 for 9.5 millimeters (mm) or 12.5 mm (⅜ inch or ½ inch, respectively) maximum, medium grading. Asphalt concrete surfacing should be placed in a single lift.

We recommend that rigid concrete pavements consist of at least 6 inches of Class 2 Aggregate Base beneath at least 6 inches of concrete. For durability and wear resistance, all Portland cement concrete pavements should have a minimum compressive strength of 4,000 pounds per square inch (psi). A modulus of subgrade reaction, $k_v$ (30-inch circular plate) of 200 psi may be used for design of Portland cement concrete pavements.

Paved areas should be sloped and adequately drained to prevent surface water or subsurface seepage from saturating the pavement subgrade soil. All curbs surrounding landscape areas should be embedded at least 6 inches into the soil subgrade to minimize the migration of water beneath pavement sections.

Heavy construction traffic on new pavements or partial pavement sections (such as, the base course over the prepared subgrade) will likely exceed the design loads and could potentially damage or shorten the pavement life. Therefore, we recommend construction traffic not be allowed on new pavements, or that the contractor takes appropriate precautions to protect the subgrade and pavement during construction.

If construction traffic is to be allowed on newly constructed road sections, an allowance for this additional traffic will need to be made in the design pavement section.

# 8.0 Additional Services

We suggest communications be maintained during the design phase between the design team and SHN to optimize compatibility between the design, soil, and groundwater conditions. We also recommend that SHN be retained during the construction phase to verify the implementation of our recommendations related to earthwork.

## 8.1 Plan and Specification Review

We have assumed in preparing our recommendations that SHN will be retained to review those portions of the plans and specifications that pertain to earthwork and foundations. The purpose of this review is to confirm that our earthwork and foundation recommendations have been properly interpreted and implemented during design. If we are not provided with this opportunity for review of the plans and specifications, our recommendations could be misinterpreted. If SHN does not review the geotechnical elements of the plans and specifications, the reviewing Geotechnical Engineer should thoroughly review this report and should agree with its conclusions and recommendations or otherwise provide alternative recommendations. Furthermore, if another geotechnical consultant is retained for follow-up services to this report, SHN will at that time cease to be the Geotechnical Engineer-of-Record. SHN cannot assume responsibility or liability for the adequacy of our geotechnical recommendations unless SHN is retained to observe the soil-related portions of the construction.

## 8.2 Construction Phase Monitoring

We recommend that SHN be retained during the construction phase to verify the implementation of our recommendations related to earthwork and to perform the following tasks:

1. Monitor site clearing, including removal of loose fill material, and any other unsuitable material if it is determined that this is required.
2. Monitor over excavation and subgrade preparation.
3. Observe and test placement of the geogrid reinforced engineered fill mat and backfill.
4. Observe foundation excavations.
5. Observe construction of asphalt-paved parking areas

This construction phase monitoring is important as it provides the stakeholders and SHN the opportunity to verify anticipated site conditions and recommend appropriate changes in design or construction procedures if site conditions encountered during construction vary from those described in this report. It also allows SHN to recommend appropriate changes in design or construction procedures if construction methods adversely affect the competence of onsite soils to support the structural improvements.

# 9.0 Closure

The opinions presented in this report are valid as of the present date for the property evaluated. Changes in the condition of a property can occur with the passage of time, whether due to natural processes or the works of human, on this or adjacent properties. In addition, changes in applicable standards of practice can occur, whether from legislation or the broadening of knowledge.

Accordingly, the opinions presented in this report may be invalidated, wholly or partially, by changes outside of our control. Therefore, this report is subject to review and should not be relied upon after a period of 3 years. In addition, this report should not be used and is not applicable for any property other than that evaluated.

# 10.0 References

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# 1

# Boring Logs

# BORING NUMBER B-1

PAGE 1 OF 2

**CLIENT** Danco

**PROJECT NAME** Taylor Way Geotechnical Study

**PROJECT NUMBER** 022138

**PROJECT LOCATION** Blue Lake, Humboldt County

**DATE STARTED** 12/14/22 **COMPLETED** 12/14/22

**GROUND ELEVATION** 83 ft (approx.) **HOLE SIZE** 4"

**DRILLING CONTRACTOR** Taber Drilling

**GROUND WATER LEVELS:**

**DRILLING METHOD** Solid Flight Augers/ Mud Rotary

▽ **AT TIME OF DRILLING** 7.00 ft / Elev 76.00 ft

**LOGGED BY** A. Troia **CHECKED BY** J. Buck

**AT END OF DRILLING** ---

**NOTES** Boring backfilled with cement grout and bentonite chips.

**AFTER DRILLING** ---

| DEPTH (ft) | GRAPHIC LOG | MATERIAL DESCRIPTION | SAMPLE TYPE NUMBER | RECOVERY % (RQD) | BLOW COUNTS (N VALUE) | POCKET PEN. (tsf) | DRY UNIT WT. (pcf) | MOISTURE CONTENT (%) | ATTERBERG LIMITS: LIQUID LIMIT | ATTERBERG LIMITS: PLASTIC LIMIT | ATTERBERG LIMITS: PLASTICITY INDEX | FINES CONTENT (%) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | (SW) GRAVELLY SAND with SILT, dense to medium dense, very dark gray (2.5Y 3/1), moist, well graded gravel, well graded sand, weak cementation, (FILL). | | | | | | | | | | |
| | | | SPT S1 | 44 | 21-17-17 (34) | | | | | | | |
| | | | SPT S2 | 56 | 11-9-8 (17) | | | | | | | 8 |
| 5 | | | SPT S3 | 100 | 10-11-9 (20) | | | | | | | 36 |
| | | ▽ Becomes saturated. | SPT S4 | 22 | 7-2-1 (3) | | | | | | | |
| 10 | | Hole caving at 10'; switch to mud-rotary. | MCS | 0 | 4-6-14 (20) | | | | | | | |
| | | (SW) GRAVELLY SAND, very dense to medium dense, dark grayish brown (2.5Y 4/2), wet, poorly sorted, well graded, subangular to subrounded gravel, mostly medium to coarse sand, some fine sand, moderate cementation, trace silt, non-stratified, (NATIVE ALLUVIAL DEPOSITS). | | | | | | | | | | |
| 15 | | Driller notes significantly harder (15-20'); broken cobble fragments in sample. | MCS S5 | 67 | 30-26-26 (52) | | | | | | | |
| 20 | | | | | | | | | | | | |

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# BORING NUMBER B-1

**CLIENT** Danco
**PROJECT NAME** Taylor Way Geotechnical Study
**PROJECT NUMBER** 022138
**PROJECT LOCATION** Blue Lake, Humboldt County

| DEPTH (ft) | GRAPHIC LOG | MATERIAL DESCRIPTION | SAMPLE TYPE NUMBER | RECOVERY % (RQD) | BLOW COUNTS (N VALUE) | POCKET PEN. (tsf) | DRY UNIT WT. (pcf) | MOISTURE CONTENT (%) | ATTERBERG LIMITS | | | FINES CONTENT (%) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | LIQUID LIMIT | PLASTIC LIMIT | PLASTICITY INDEX | |
| 20 | | | SPT<br>S7 | 56 | 15-21-30<br>(51) | | | | | | | |
| | | Rig chatter (23') | | | | | | | | | | |
| 25 | | | SPT<br>S8 | 56 | 19-10-13<br>(23) | | | | | | | |
| 30 | | (SW) WELL GRADED SAND with GRAVEL, dense, dark grayish brown (2.5Y 4/2), wet, moderate cementation, poorly sorted, weak stratification, angular, medium to coarse sand, broken cobbles and gravels in sampler. | SPT<br>S9 | 56 | 21-22-20<br>(42) | | | | | | | |
| 35 | | Abundant broken chert gravels in cuttings. | SPT<br>S10 | 56 | 31-27-24<br>(51) | | | | | | | |

Bottom of borehole at 36.5 feet.

# BORING NUMBER B-2

**CLIENT** Danco

**PROJECT NAME** Taylor Way Geotechnical Study

**PROJECT NUMBER** 022138

**PROJECT LOCATION** Blue Lake, Humboldt County

**DATE STARTED** 12/14/22 **COMPLETED** 12/14/22

**GROUND ELEVATION** 85 ft (approx.) **HOLE SIZE** 4"

**DRILLING CONTRACTOR** Taber Drilling

**GROUND WATER LEVELS:**

**DRILLING METHOD** Rotary Hollow Stem Auger

▽ **AT TIME OF DRILLING** 7.00 ft / Elev 78.00 ft

**LOGGED BY** A. Troia **CHECKED BY** J. Buck

**AT END OF DRILLING** ---

**NOTES** Boring backfilled with cement grout and bentonite chips.

**AFTER DRILLING** ---

| DEPTH (ft) | GRAPHIC LOG | MATERIAL DESCRIPTION | SAMPLE TYPE NUMBER | RECOVERY % (RQD) | BLOW COUNTS (N VALUE) | POCKET PEN. (tsf) | DRY UNIT WT. (pcf) | MOISTURE CONTENT (%) | ATTERBERG LIMITS LIQUID LIMIT | ATTERBERG LIMITS PLASTIC LIMIT | ATTERBERG LIMITS PLASTICITY INDEX | FINES CONTENT (%) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | (GW) WELL GRADED GRAVEL, dence, very dark gray (2/5Y 4/1), dry, subangular gravels, trace silt, weak cementation, cobbles up to 6" in upper 2', (FILL). | | | | | | | | | | |
| | | | SPT<br>S1 | 100 | 19-20-21<br>(41) | | | | | | | |
| 5 | | (SW) GRAVELLY SAND, dense, very dark gray (2.5Y 4/1), dry to moist, well graded sand, subangular, fine to coarse gravel, weak cementation, (FILL). | SPT<br>S2 | 83 | 16-18-15<br>(33) | | | | | | | |
| | | (SW) WELL GRADED SAND with GRAVEL, medium dense, becomes wet at 7', weak cementation, (FILL).<br>▽ | SPT<br>S3 | 100 | 10-8-8<br>(16) | | | | | | | 6 |
| | | Gravel stuck in sample shoe. | SPT | 0 | 5-4-2<br>(6) | | | | | | | |
| 10 | | (ML) SILTY SAND, loose, very dark gray (2/5Y 3/1), moist, very fine sand with minor coarse angular sand, (FILL).<br>3" wood fragment. | SPT<br>S4 | 100 | 2-1-2<br>(3) | | | | | | | 38 |
| | | (SM) SILTY SAND with GRAVEL, medium dense, dark gray, wet, angular sand, strong stratification, subrounded to subangular, mostly fine gravel with medium and coarse gravel, chert-rich, weak to no cementation, (NATIVE ALLUVIAL DEPOSITS). | | | | | | | | | | |
| 15 | | | SPT<br>S5 | 100 | 6-9-14<br>(23) | | | | | | | 32 |
| 20 | | | | | | | | | | | | |

*(Continued Next Page)*

# BORING NUMBER B-2

**CLIENT** Danco
**PROJECT NAME** Taylor Way Geotechnical Study
**PROJECT NUMBER** 022138
**PROJECT LOCATION** Blue Lake, Humboldt County

| DEPTH (ft) | GRAPHIC LOG | MATERIAL DESCRIPTION | SAMPLE TYPE NUMBER | RECOVERY % (RQD) | BLOW COUNTS (N VALUE) | POCKET PEN. (tsf) | DRY UNIT WT. (pcf) | MOISTURE CONTENT (%) | ATTERBERG LIMITS | | | FINES CONTENT (%) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | LIQUID LIMIT | PLASTIC LIMIT | PLASTICITY INDEX | |
| 20 | | | SPT<br>S6 | 100 | 13-26-27<br>(53) | | | | | | | |
| | | Rig chatter (22-25'), gravel and cobbles likely. | | | | | | | | | | |
| 25 | | (SW) WELL GRADED SAND with GRAVEL, medium dense, dark gray, wet, stratified, subrounded to subangular gravel with weak to moderate cementation. | SPT<br>S7 | 100 | 9-14-23<br>(37) | | | | | | | 4 |
| | | Heaving sand (27') | | | | | | | | | | |
| 30 | | | SPT<br>S8 | 100 | 26-22-23<br>(45) | | | | | | | |
| 35 | | | SPT<br>S9 | 100 | 14-20-25<br>(45) | | | | | | | |

Bottom of borehole at 36.5 feet.

# BORING NUMBER B-3

**CLIENT** Danco

**PROJECT NAME** Taylor Way Geotechnical Study

**PROJECT NUMBER** 022138

**PROJECT LOCATION** Blue Lake, Humboldt County

**DATE STARTED** 12/14/22 **COMPLETED** 12/15/22

**GROUND ELEVATION** 85 ft (approx.) **HOLE SIZE** 4"

**DRILLING CONTRACTOR** Taber Drilling

**GROUND WATER LEVELS:**

**DRILLING METHOD** Rotary Hollow Stem Auger

▽ **AT TIME OF DRILLING** 7.00 ft / Elev 78.00 ft

**LOGGED BY** A. Troia **CHECKED BY** J. Buck

**AT END OF DRILLING** ---

**NOTES** Boring backfilled with cement grout and bentonite chips.

**AFTER DRILLING** ---

| DEPTH (ft) | GRAPHIC LOG | MATERIAL DESCRIPTION | SAMPLE TYPE NUMBER | RECOVERY % (RQD) | BLOW COUNTS (N VALUE) | POCKET PEN. (tsf) | DRY UNIT WT. (pcf) | MOISTURE CONTENT (%) | ATTERBERG LIMITS LIQUID LIMIT | ATTERBERG LIMITS PLASTIC LIMIT | ATTERBERG LIMITS PLASTICITY INDEX | FINES CONTENT (%) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | (GW) WELL GRADED GRAVEL, very dense, gray, dry, cobbles up to 8", minor silt, subrounded to rounded gravel, moderate cementation, compacted, (FILL). | | | | | | | | | | |
| | | (GW) SANDY GRAVEL, very dense, dark gray, dry to moist, medium to coarse, angular sand, broken cobbles and coarse gravel in sampler, weak cementation, (FILL). | SPT | 50 | 50/4" | | | | | | | |
| 5 | | (GW) WELL GRADED GRAVEL with SAND, very dense, dark gray, wet, abundand broken/crushed gravel, medium to coarse angular sand, minor silt, (FILL)<br>Rig grinding and hopping (5-7') | SPT | 0 | 50/4" | | | | | | | |
| | | ▽ | SPT | 56 | 13-30-45<br>(75) | | | | | | | 8 |
| | | | SPT | 22 | 7-2-1<br>(3) | | | | | | | |
| 10 | | (SM) SILTY SAND, medium dense, very dark gray (2.5YR 3/1), wet, minor fine to medium sand, few fine to coarse, subangular to subrounded gravels, slightly clayey, low plasticity, low toughness, no cementation, fibrous wood, stick fragments, (FILL). | SPT | 100 | 1-4-11<br>(15) | | | | 35 | 31 | 4 | 29 |
| | | (SW) WELL GRADED SAND with GRAVEL, medium dense, gray, dry to moist, medium to coarse sand, fine to coarse, angular to subangular gravel, no cementation, (NATIVE ALLUVIAL DEPOSITS). | SPT | 100 | 18-15-12<br>(27) | | | | | | | |
| 15 | | (SM) SILTY SAND with GRAVEL, medium dense, dark gray (2.5YR 4/1), wet, angular to subangular, medium to coarse sand, subangular, fine to coarse gravel, weak cementation. | SPT | 28 | 9-10-10<br>(20) | | | | | | | 36 |
| | | | SPT | 44 | 22-28-21<br>(49) | | | | | | | |
| 20 | | | | | | | | | | | | |

*(Continued Next Page)*

# BORING NUMBER B-3

**CLIENT** Danco

**PROJECT NAME** Taylor Way Geotechnical Study

**PROJECT NUMBER** 022138

**PROJECT LOCATION** Blue Lake, Humboldt County

| DEPTH (ft) | GRAPHIC LOG | MATERIAL DESCRIPTION | SAMPLE TYPE NUMBER | RECOVERY % (RQD) | BLOW COUNTS (N VALUE) | POCKET PEN. (tsf) | DRY UNIT WT. (pcf) | MOISTURE CONTENT (%) | ATTERBERG LIMITS | | | FINES CONTENT (%) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | LIQUID LIMIT | PLASTIC LIMIT | PLASTICITY INDEX | |
| 20 | | | | | | | | | | | | |
| | | (SW) WELL GRADED SAND with GRAVEL, dense, dark gray, wet, stratified, sand coarsens downward, angular, quartz-rich sand with moderate cementation, minor silt. | | | | | | | | | | |
| 25 | | | SPT | 100 | 19-20-22<br>(42) | | | | | | | |

Bottom of borehole at 26.5 feet.

# BORING NUMBER B-4

PAGE 1 OF 3

**CLIENT** Danco

**PROJECT NAME** Taylor Way Geotechnical Study

**PROJECT NUMBER** 022138

**PROJECT LOCATION** Blue Lake, Humboldt County

**DATE STARTED** 12/15/22 **COMPLETED** 12/15/22

**GROUND ELEVATION** 83 ft (approx.) **HOLE SIZE** 4"

**DRILLING CONTRACTOR** Taber Drilling

**GROUND WATER LEVELS:**

**DRILLING METHOD** Solid Flight Augers/ Mud Rotary

▽ **AT TIME OF DRILLING** 6.00 ft / Elev 77.00 ft

**LOGGED BY** A. Troia **CHECKED BY** J. Buck

**AT END OF DRILLING** ---

**NOTES** Boring backfilled with cement grout and bentonite chips.

**AFTER DRILLING** ---

| DEPTH (ft) | GRAPHIC LOG | MATERIAL DESCRIPTION | SAMPLE TYPE NUMBER | RECOVERY % (RQD) | BLOW COUNTS (N VALUE) | POCKET PEN. (tsf) | DRY UNIT WT. (pcf) | MOISTURE CONTENT (%) | ATTERBERG LIMITS LIQUID LIMIT | ATTERBERG LIMITS PLASTIC LIMIT | ATTERBERG LIMITS PLASTICITY INDEX | FINES CONTENT (%) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | (GW) WELL GRADED SANDY GRAVEL, dense, gray, dry, minor silt, angular to subangular fine to coarse gravel, compacted cobbles up to 8" in upper 18", (FILL).<br>**R-Value** = 59 | | | | | | | | | | |
| | | | SPT | 56 | 20-18-20 (38) | | | | | | | |
| 5 | | | SPT | 56 | 17-12-10 (22) | | | | | | | 7 |
| | ▽ | (SW) WELL GRADED SAND with GRAVEL, medium dense, dark brownish gray, wet, angular to subangular, quartz-rich sand, subangular to subrounded, fine to coarse gravels, weak cementation, trace silt, (NATIVE ALLUVIAL DEPOSITS). | SPT | 67 | 7-7-9 (16) | | | | | | | |
| 10 | | (SW) WELL GRADED GRAVELLY SAND. dense, dark gray, wet, angular, quartz-rich sand, moderate cementation. | MCS | 67 | 8-21-31 (52) | | 129 | 10 | | | | |
| 15 | | | SPT | 44 | 10-11-8 (19) | | | | | | | 15 |
| 20 | | | | | | | | | | | | |

*(Continued Next Page)*

**CLIENT** Danco
**PROJECT NAME** Taylor Way Geotechnical Study
**PROJECT NUMBER** 022138
**PROJECT LOCATION** Blue Lake, Humboldt County

| DEPTH (ft) | GRAPHIC LOG | MATERIAL DESCRIPTION | SAMPLE TYPE NUMBER | RECOVERY % (RQD) | BLOW COUNTS (N VALUE) | POCKET PEN. (tsf) | DRY UNIT WT. (pcf) | MOISTURE CONTENT (%) | ATTERBERG LIMITS LIQUID LIMIT | ATTERBERG LIMITS PLASTIC LIMIT | ATTERBERG LIMITS PLASTICITY INDEX | FINES CONTENT (%) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 20 | | | SPT | 56 | 19-21-23<br>(44) | | | | | | | |
| 25 | | | | | | | | | | | | |
| 30 | | Becomes well-cemented. | SPT | 67 | 26-28-31<br>(59) | | | | | | | |
| 35 | | | | | | | | | | | | |
| | | Contact based on change in drilling.<br>(CL) LEAN CLAY with SAND, very stiff, very dark gray (GLEY 1 3/N), dry to moist, high toughness, moderate to strong cementation, low to medium plasticity, well graded sand, non-stratified, very fine wood and charcoal fragments, plant fibers, occasional fine, subangular gravel. | | | | | | | | | | |
| 40 | | | SPT | 22 | 4-7-11<br>(18) | | | | 25 | 16 | 9 | |

*(Continued Next Page)*

# BORING NUMBER B-4

**CLIENT** Danco

**PROJECT NAME** Taylor Way Geotechnical Study

**PROJECT NUMBER** 022138

**PROJECT LOCATION** Blue Lake, Humboldt County

| DEPTH (ft) | GRAPHIC LOG | MATERIAL DESCRIPTION | SAMPLE TYPE NUMBER | RECOVERY % (RQD) | BLOW COUNTS (N VALUE) | POCKET PEN. (tsf) | DRY UNIT WT. (pcf) | MOISTURE CONTENT (%) | ATTERBERG LIMITS LIQUID LIMIT | ATTERBERG LIMITS PLASTIC LIMIT | ATTERBERG LIMITS PLASTICITY INDEX | FINES CONTENT (%) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 45 | ▨ | Rig chatter (45') | | | | | | | | | | |
| | ▨ | Contact estimated.<br>(SC) CLAYEY SAND with GRAVEL, very dense, dark yellowish brown (10YR 3/6), moist, fine sand with rounded, fine to coarse gravel, strong cementation with medium toughness, medium plasticity fines, (FALOR FORMATION) | | | | | | | | | | |
| 50 | ▨ | | SPT | 67 | 33-32-38 (70) | | | | | | | 26 |

Bottom of borehole at 51.5 feet.

# SUBSURFACE EXPLORATION LOG

PROJECT NAME: Blue Lake Industrial Park
PROJECT NUMBER: 20,034
HOLE NUMBER: EH- 5
HOLE SIZE: 1½ x 7 feet
EXCAVATION METHOD: Backhoe
DRILLING DATE(S): 6/4/81
LOGGED BY: DRB
HOLE ELEV: 78
DATUM: Plot Plan
SAMPLER & DRIVE: 2" O.D. Shelby Tube; 18 pound hand forced Hammer.

| LABORATORY DATA | | | | | | | SOIL CLASSIFICATION |
|---|---|---|---|---|---|---|---|
| OTHER TESTS (Key below) | UNCONFINED COMPRESSION (PSF) | MOISTURE CONTENT (%) | DRY DENSITY (PCF) | BLOWS PER FOOT | SAMPLES (Key below) | DEPTH (FEET) | UNIFIED SOIL CLASSIFICATION SYSTEM—SEE FIGURE 2 FIELD CLASSIFICATION AS MODIFIED BY TEST RESULTS TEXTURE, CONSISTENCY, MOISTURE, COLOR, SYMBOL, REMARKS |
| | | | | | | | 80% BARK, soft, moist, brown (PT) and 20% GRAVEL, sandy, silty, loose, moist, gray (GM) (fill) |
| | | | | | | 1 | 50% BARK and 50% GRAVEL |
| | | | | | | 2 | |
| | | | | | | 3 | |
| | | | | | | 4 | |
| | | | | | | 5 | |
| | | 24.3 | 103 | 8/6" | II | 6 | SILT, clayey, stiff, wet, yellowish-brown and gray, (ML) (fill) |
| | | | | | | 7 | SAND, fine, silty, medium dense, wet, gray (SM) (fill) |
| | | | | | | 8 | |
| | 620 | 15.9 | 108 | 9/6" | II | 9 | |
| | | | | | | 10 | Bottom at 10 feet. Backfilled following excavation. |

KEY
⊻ WATER LEVEL  I SPLIT SPOON  II SHELBY TUBE  X DISTURBED
LL LIQUID LIMIT  PI PLASTICITY INDEX  %F PERCENT FINES PASSING U.S. NO. 200 SIEVE

NGS NORTHCOAST GEOTECHNICAL SERVICES

FE-2 2/81

FIGURE 3E

# SUBSURFACE EXPLORATION LOG

PROJECT NAME: Blue Lake Industrial Park
PROJECT NUMBER: 20,034
HOLE NUMBER: EH- 8
HOLE SIZE: 1½ x 7 feet
EXCAVATION METHOD: Backhoe
DRILLING DATE(S): 6/4/81
LOGGED BY: DRB
HOLE ELEV: 80
DATUM Plot Plan
SAMPLER & DRIVE: 2" O.D. Shelby Tube; 18 pound hand forced Hammer.

| LABORATORY DATA | | | | | | | SOIL CLASSIFICATION |
|---|---|---|---|---|---|---|---|
| OTHER TESTS (Key below) | UNCONFINED COMPRESSION (PSF) | MOISTURE CONTENT (%) | DRY DENSITY (PCF) | BLOWS PER FOOT | SAMPLES (Key below) | DEPTH (FEET) | UNIFIED SOIL CLASSIFICATION SYSTEM—SEE FIGURE 2<br>FIELD CLASSIFICATION AS MODIFIED BY TEST RESULTS<br>TEXTURE, CONSISTENCY, MOISTURE, COLOR, SYMBOL, REMARKS |
| | | | | | | | GRAVEL, rounded, silty, sandy, medium dense, moist, gray (GM) (river-run) 50% bark in upper six inches (probably fill; hole is located in former pond area) |
| | | | | | | 1 | |
| | | | | | | 2 | |
| | | | | | | 3 | |
| | | | | | | 4 | |
| | | | | | | 5 | |
| | | | | | | 6 | Wet<br>▽ |
| | | | | | | 7 | Caving |
| | | | | | | 8 | Bottom. Backfilled following excavation. |
| | | | | | | 9 | |
| | | | | | | 10 | |

KEY
▽ WATER LEVEL I SPLIT SPOON II SHELBY TUBE X DISTURBED
LL LIQUID LIMIT PI PLASTICITY INDEX %F PERCENT FINES PASSING U.S. NO. 200 SIEVE

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FE-2 2/81

FIGURE 3

# SUBSURFACE EXPLORATION LOG

PROJECT NAME: Blue Lake Industrial Park
PROJECT NUMBER: 20,034
HOLE NUMBER: EH- 18
HOLE SIZE: 1½ x 7 feet
EXCAVATION METHOD: Backhoe
DRILLING DATE(S): 6/4/81
LOGGED BY: DRB
HOLE ELEV: 84
DATUM: Plot Plan
SAMPLER & DRIVE: 2" O.D. Shelby Tube; 18 pound hand forced Hammer.

| LABORATORY DATA | | | | | | | SOIL CLASSIFICATION |
|---|---|---|---|---|---|---|---|
| OTHER TESTS (Key below) | UNCONFINED COMPRESSION (PSF) | MOISTURE CONTENT (%) | DRY DENSITY (PCF) | BLOWS PER FOOT | SAMPLES (Key below) | DEPTH (FEET) | UNIFIED SOIL CLASSIFICATION SYSTEM—SEE FIGURE 2 FIELD CLASSIFICATION AS MODIFIED BY TEST RESULTS TEXTURE, CONSISTENCY, MOISTURE, COLOR, SYMBOL, REMARKS |
| | | | | | | | GRAVEL, rounded, with cobbles to 10", sandy, silty, medium dense (GM) (river-run) (fill) |
| | | | | | | 1 | |
| | | | | | | 2 | BARK, soft, moist, brown (PT) (fill) |
| | | | | | | 3 | SAND, gravelly, silty, medium dense, damp, black (SM) (fill) (light in weight, mixture of burner cinders and river-run gravel) |
| | — | 24.2 | 79 | 22/6" | II | 4 | |
| | | | | | | | wet |
| | | | | | | 5 | |
| | | | | | | 6 | |
| | | | | | | 7 | |
| | | | | | | 8 | BARK, soft, wet, dark brown (PT) (fill) |
| | | | | | ▽ | 9 | SAND, gravelly, silty, medium dense, damp, black, (SM) (fill) (burner cinders and river-run gravel) |
| | | | | | | | Bottom at 9½ feet. Backfilled following excavation. |
| | | | | | | 10 | |

KEY
▽ WATER LEVEL I SPLIT SPOON II SHELBY TUBE ☒ DISTURBED
LL LIQUID LIMIT PI PLASTICITY INDEX %F PERCENT FINES PASSING U.S. NO. 200 SIEVE

NGS NORTHCOAST GEOTECHNICAL SERVICES

FE-2 2/81

FIGURE 3 R

SHN Consulting Engineers & Geologists, Inc.
812 West Wabash, Eureka, CA 95501 ph. (707) 441-8855 fax. (707) 441-8877

**PROJECT:** Blue Lake Business Park
**LOCATION:** Blue Lake, CA
**EXCAVATION METHOD:** Backhoe
**LOGGED BY:** EJN

**JOB NUMBER:** 013066
**DATE:** 10/1/13
**TOTAL DEPTH OF TEST PIT:** 5.0 feet BGS
**SAMPLE TYPE:** Discrete

# TEST PIT NUMBER
# TP-1

| DEPTH (FT) | BULK SAMPLES | SS SAMPLES | PID (OVA ppm) | USCS | PROFILE | DESCRIPTION | % Moisture | Dry Density (pcf) | Unc. Com. (psf) | U.C. (psf) by P.P. | % Passing 200 | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.0 | | | | GP/SM | ⊠ | FILL: GRAVEL WITH SILT AND SAND, fine to coarse rounded gravel, dark gray to brown, medium dense, dry, non-plastic. | | | | | | No HC Odor or Sheen |
| -1.0 | | | | | ⊠ | | | | | | | |
| | ☒ | | 0.0 | | ⊠ | | | | | | | |
| -2.0 | | | | | ⊠ | | | | | | | |
| -3.0 | ☒ | | 0.0 | | ⊠ | | | | | | | |
| -4.0 | | | | | ⊠ | | | | | | | |
| -5.0 | | | | | | Bottom of TP-1 at 5.0 feet BGS. | | | | | | |
| -6.0 | | | | | | | | | | | | |
| -7.0 | | | | | | | | | | | | |
| -8.0 | | | | | | | | | | | | |
| -9.0 | | | | | | | | | | | | |
| -10.0 | | | | | | | | | | | | |
| -11.0 | | | | | | | | | | | | |
| -12.0 | | | | | | | | | | | | |

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

FIELD LOG

# Consulting Engineers & Geologists, Inc.

812 West Wabash, Eureka, CA 95501 ph. (707) 441-8855 fax. (707) 441-8877

**PROJECT:** Blue Lake Business Park
**LOCATION:** Blue Lake, CA
**EXCAVATION METHOD:** Backhoe
**LOGGED BY:** EJN

**JOB NUMBER:** 013066
**DATE:** 10/1/13
**TOTAL DEPTH OF TEST PIT:** 4.5 feet BGS
**SAMPLE TYPE:** Discrete

## TEST PIT NUMBER
## TP-2

| DEPTH (FT) | BULK SAMPLES | SS SAMPLES | PID (OVA ppm) | USCS | PROFILE | DESCRIPTION | % Moisture | Dry Density (pcf) | Unc. Com. (psf) | U.C. (psf) by P.P. | % Passing 200 | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.0 | | | | GP/ SM |  | FILL: GRAVEL WITH SILT AND SAND, fine to coarse rounded gravel, dark gray to brown, medium dense, dry, non-plastic. | | | | | | No HC Odor or Sheen |
| -1.0 | ▨ | | 0.0 | | | | | | | | | |
| -2.0 | | | | | | | | | | | | |
| -3.0 | ▨ | | 0.0 | | | | | | | | | |
| -4.0 | | | | | | | | | | | | |
| -5.0 | | | | | | Bottom of TP-2 at 4.5 feet BGS. | | | | | | |
| -6.0 | | | | | | | | | | | | |
| -7.0 | | | | | | | | | | | | |
| -8.0 | | | | | | | | | | | | |
| -9.0 | | | | | | | | | | | | |
| -10.0 | | | | | | | | | | | | |
| -11.0 | | | | | | | | | | | | |
| -12.0 | | | | | | | | | | | | |

FIELD LOG

**SHN Consulting Engineers & Geologists, Inc.**
812 West Wabash, Eureka, CA 95501 ph. (707) 441-8855 fax. (707) 441-8877

**PROJECT:** Blue Lake Business Park
**LOCATION:** Blue Lake, CA
**EXCAVATION METHOD:** Backhoe
**LOGGED BY:** EJN

**JOB NUMBER:** 013066
**DATE:** 10/1/13
**TOTAL DEPTH OF TEST PIT:** 4.5 feet BGS
**SAMPLE TYPE:** Discrete

## TEST PIT NUMBER
## TP-3

| DEPTH (FT) | BULK SAMPLES | SS SAMPLES | PID (OVA ppm) | USCS | PROFILE | DESCRIPTION | % Moisture | Dry Density (pcf) | Unc. Com. (psf) | U.C. (psf) by P.P. | % Passing 200 | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.0 | | | | GP/SM | ☒ | FILL: GRAVEL WITH SILT AND SAND, fine to coarse rounded gravel, dark gray to brown, medium dense, dry, non-plastic. | | | | | | No HC Odor or Sheen |
| -1.0 | ☒ | | 0.0 | | ☒ | | | | | | | |
| -2.0 | | | | | ☒ | | | | | | | |
| -3.0 | ☒ | | 0.0 | | ☒ | | | | | | | |
| -4.0 | | | | | ☒ | | | | | | | |
| -5.0 | | | | | | Bottom of TP-3 at 4.5 feet BGS. | | | | | | |
| -6.0 | | | | | | | | | | | | |
| -7.0 | | | | | | | | | | | | |
| -8.0 | | | | | | | | | | | | |
| -9.0 | | | | | | | | | | | | |
| -10.0 | | | | | | | | | | | | |
| -11.0 | | | | | | | | | | | | |
| -12.0 | | | | | | | | | | | | |

**SHN Consulting Engineers & Geologists, Inc.**

812 West Wabash, Eureka, CA 95501 ph. (707) 441-8855 fax. (707) 441-8877

**PROJECT:** Blue Lake Business Park

**LOCATION:** Blue Lake, CA

**EXCAVATION METHOD:** Backhoe

**LOGGED BY:** EJN

**JOB NUMBER:** 013066

**DATE:** 10/1/13

**TOTAL DEPTH OF TEST PIT:** 4.5 feet BGS

**SAMPLE TYPE:** Discrete

## TEST PIT NUMBER
## TP-4

| DEPTH (FT) | BULK SAMPLES | SS SAMPLES | PID (OVA ppm) | USCS | PROFILE | DESCRIPTION | % Moisture | Dry Density (pcf) | Unc. Com. (psf) | U.C. (psf) by P.P. | % Passing 200 | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.0 | | | | GP/SM | ☒ | FILL: GRAVEL WITH SILT AND SAND, fine to coarse rounded gravel, dark gray to brown, medium dense, dry, non-plastic. | | | | | | No HC Odor or Sheen |
| -1.0 | ▨ | | 1.1 | | ☒ | | | | | | | |
| -2.0 | | | | | ☒ | | | | | | | Wood/logs observed |
| -3.0 | ▨ | | 0.0 | | ☒ | | | | | | | |
| -4.0 | | | | | ☒ | | | | | | | |
| -5.0 | | | | | | Bottom of TP-4 at 4.5 feet BGS. | | | | | | |
| -6.0 | | | | | | | | | | | | |
| -7.0 | | | | | | | | | | | | |
| -8.0 | | | | | | | | | | | | |
| -9.0 | | | | | | | | | | | | |
| -10.0 | | | | | | | | | | | | |
| -11.0 | | | | | | | | | | | | |
| -12.0 | | | | | | | | | | | | |

FIELD LOG

SHN Consulting Engineers & Geologists, Inc.
812 West Wabash, Eureka, CA 95501 ph. (707) 441-8855 fax. (707) 441-8877

**PROJECT:** Blue Lake Business Park
**LOCATION:** Blue Lake, CA
**EXCAVATION METHOD:** Backhoe
**LOGGED BY:** EJN

**JOB NUMBER:** 013066
**DATE:** 10/1/13
**TOTAL DEPTH OF TEST PIT:** 5.0 feet BGS
**SAMPLE TYPE:** Discrete

# TEST PIT NUMBER
# TP-5

| DEPTH (FT) | BULK SAMPLES | SS SAMPLES | PID (OVA ppm) | USCS | PROFILE | DESCRIPTION | % Moisture | Dry Density (pcf) | Unc. Com. (psf) | U.C. (psf) by P.P. | % Passing 200 | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.0 | | | | GP/ SM | ☒ | FILL: GRAVEL WITH SILT AND SAND, fine to coarse rounded gravel, dark gray to brown, medium dense, dry, non-plastic. | | | | | | No HC Odor or Sheen |
| -1.0 | ▨ | | 0.0 | | | | | | | | | |
| -2.0 | | | | | | | | | | | | |
| -3.0 | ▨ | | 0.0 | | | | | | | | | |
| -4.0 | | | | | | | | | | | | |
| -5.0 | | | | | | Bottom of TP-5 at 5.0 feet BGS. | | | | | | |
| -6.0 | | | | | | | | | | | | |
| -7.0 | | | | | | | | | | | | |
| -8.0 | | | | | | | | | | | | |
| -9.0 | | | | | | | | | | | | |
| -10.0 | | | | | | | | | | | | |
| -11.0 | | | | | | | | | | | | |
| -12.0 | | | | | | | | | | | | |

FIELD LOG

**Consulting Engineers & Geologists, Inc.**
812 West Wabash, Eureka, CA 95501 ph. (707) 441-8855 fax. (707) 441-8877

**PROJECT:** Blue Lake Business Park
**LOCATION:** Blue Lake, CA
**EXCAVATION METHOD:** Backhoe
**LOGGED BY:** EJN

**JOB NUMBER:** 013066
**DATE:** 10/1/13
**TOTAL DEPTH OF TEST PIT:** 5.0 feet BGS
**SAMPLE TYPE:** Discrete

TEST PIT NUMBER
**TP-6**

| DEPTH (FT) | BULK SAMPLES | SS SAMPLES | PID (OVA ppm) | USCS | PROFILE | DESCRIPTION | % Moisture | Dry Density (pcf) | Unc. Com. (psf) | U.C. (psf) by P.P. | % Passing 200 | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.0 | | | | GP/SM | | FILL: GRAVEL WITH SILT AND SAND, fine to coarse rounded gravel, dark gray to brown, medium dense, dry, non-plastic. | | | | | | No HC Odor or Sheen |
| -1.0 | | | | | | | | | | | | |
| | ▨ | | 0.0 | | | | | | | | | |
| -2.0 | | | | | | | | | | | | |
| -3.0 | | | | | | | | | | | | |
| | ▨ | | 0.0 | | | | | | | | | |
| -4.0 | | | | | | | | | | | | |
| -5.0 | | | | | | Bottom of TP-6 at 5.0 feet BGS. | | | | | | |
| -6.0 | | | | | | | | | | | | |
| -7.0 | | | | | | | | | | | | |
| -8.0 | | | | | | | | | | | | |
| -9.0 | | | | | | | | | | | | |
| -10.0 | | | | | | | | | | | | |
| -11.0 | | | | | | | | | | | | |
| -12.0 | | | | | | | | | | | | |

FIELD LOG

SHN Consulting Engineers & Geologists, Inc.
812 West Wabash, Eureka, CA 95501 ph. (707) 441-8855 fax. (707) 441-8877

**PROJECT:** Blue Lake Business Park
**LOCATION:** Blue Lake, CA
**EXCAVATION METHOD:** Backhoe
**LOGGED BY:** EJN

**JOB NUMBER:** 013066
**DATE:** 10/1/13
**TOTAL DEPTH OF TEST PIT:** 5.0 feet BGS
**SAMPLE TYPE:** Discrete

## TEST PIT NUMBER
## TP-7

| DEPTH (FT) | BULK SAMPLES | SS SAMPLES | PID (OVA ppm) | USCS | PROFILE | DESCRIPTION | % Moisture | Dry Density (pcf) | Unc. Com. (psf) | U.C. (psf) by P.P. | % Passing 200 | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.0 | | | | GP/SM | ⊠ | FILL: GRAVEL WITH SILT AND SAND, fine to coarse rounded gravel, dark gray to brown, medium dense, dry, non-plastic. | | | | | | No HC Odor or Sheen |
| -1.0 | ☒ | | 0.0 | | ⊠ | | | | | | | |
| -2.0 | | | | | ⊠ | | | | | | | |
| -3.0 | ☒ | | 0.0 | | ⊠ | | | | | | | |
| -4.0 | | | | | ⊠ | | | | | | | |
| -5.0 | | | | | | Bottom of TP-7 at 5.0 feet BGS. | | | | | | |
| -6.0 | | | | | | | | | | | | |
| -7.0 | | | | | | | | | | | | |
| -8.0 | | | | | | | | | | | | |
| -9.0 | | | | | | | | | | | | |
| -10.0 | | | | | | | | | | | | |
| -11.0 | | | | | | | | | | | | |
| -12.0 | | | | | | | | | | | | |

FIELD LOG

**SHN Consulting Engineers & Geologists, Inc.**
812 West Wabash, Eureka, CA ph. (707) 441-8855 fax. (707) 441-8877

## WELL POINT LOG

## WP-1

**PROJ. NAME:** Blue Lake Business Park
**PROJ. NUMBER:** 013066
**DRILLER:** Fisch
**DRILLING METHOD:** GeoProbe
**SAMPLER TYPE:** DT-22
**LOGGED BY:** R. Rueber

**LOCATION:** Blue Lake, CA
**TOC ELEVATION:** --
**DEPTH OF BORING/WELL:** 26.0 feet BGS
**DEPTH TO FIRST WATER:** 17 feet BGS
**SCREEN INTERVAL:** 16-26 feet BGS
**DATE:** 9/25/13

| DEPTH (Feet BGS) | WATER LEVEL | SAMPLE | | | | USCS | LITHOLOGY PATTERN | SOIL DESCRIPTION | REMARKS | WELL POINT CONSTRUCTION |
|---|---|---|---|---|---|---|---|---|---|---|
| | | OVA READING (ppm) | RECOVERY (%) | DRILLING | LABORATORY | | | | | |
| 0 | | 0 | 50 | ☒ | | | | FILL: GRAVEL WITH SAND, fine gravel, rounded to subangular, well graded sand, with silt and clay, medium brown, dense, moist.<br>Becomes coarse gravelly. | | |
| 5 | | 0 | 50 | ☒ | | ML | | SILT WITH SAND, fine, dark gray, medium stiff, moist, with clay and organics. | | |
| 10 | | 0 | 60 | ☒ | | GW/<br>SW | | WELL GRADED SAND WIH GRAVEL, dark gray, dense, moist, trace silt. | | |
| 15 | | 0 | 50 | ☒ | ☒ | | | | | |
| | ▽ | 0 | 40 | ☒ | | | | Becomes moist to wet.<br>Becomes saturated. | | 0.010 PVC Well Screen 16 to 26 feet BGS |
| 20 | | | 60 | ☒ | | | | Becomes dark brown, increasing silt. | | |
| 25 | | | 80 | ☒ | | | | | | |
| 30 | | | | | | | | Bottom of WP-1 at 26 feet BGS. | | |

BORING LOG

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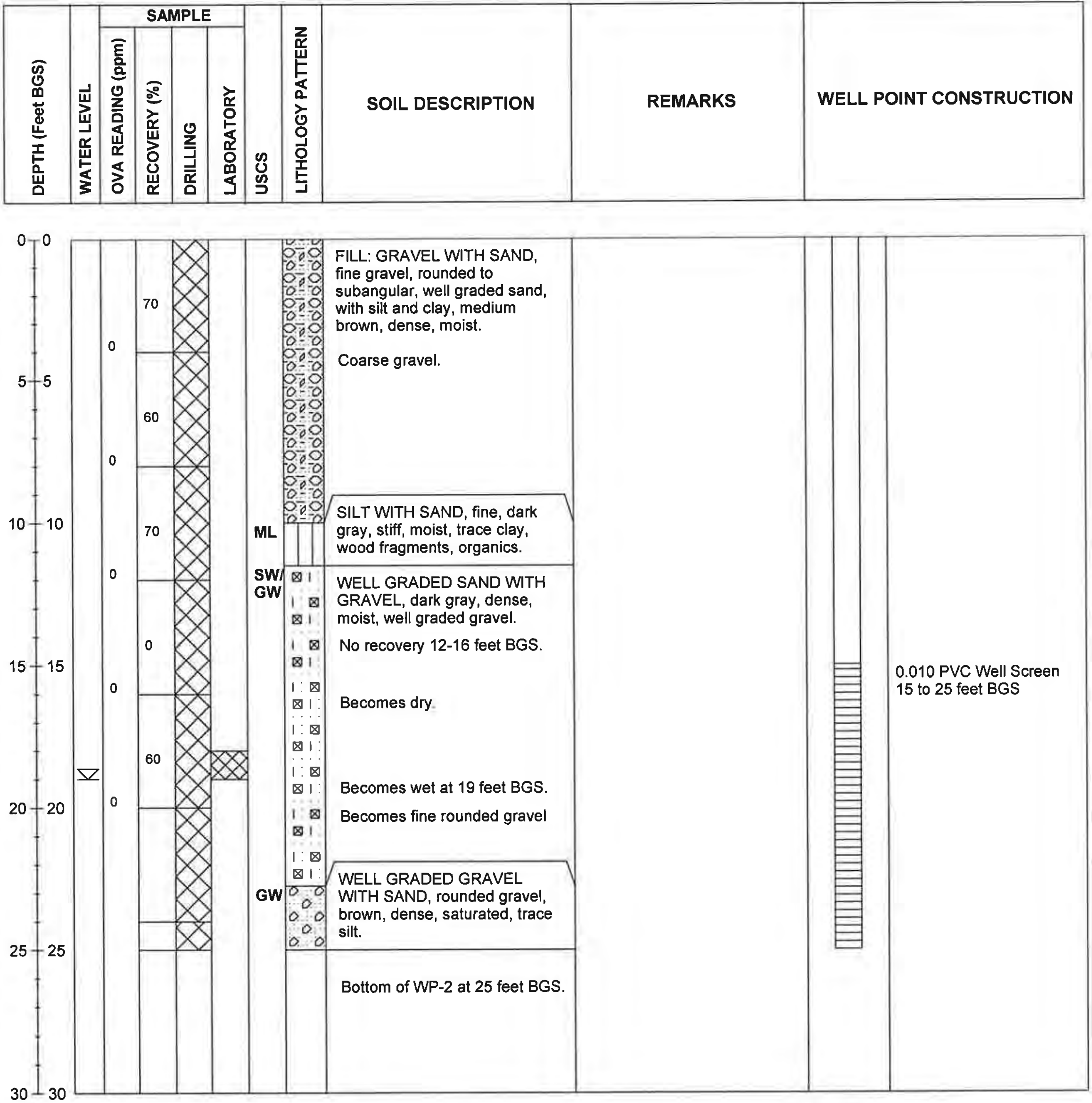
**812 West Wabash, Eureka, CA ph. (707) 441-8855 fax. (707) 441-8877**

## WELL POINT LOG

## WP-2

**PROJ. NAME:** Blue Lake Business Park
**PROJ. NUMBER:** 013066
**DRILLER:** Fisch
**DRILLING METHOD:** GeoProbe
**SAMPLER TYPE:** DT-22
**LOGGED BY:** R. Rueber
**LOCATION:** Blue Lake, CA
**TOC ELEVATION:** --
**DEPTH OF BORING/WELL:** 25.0 feet BGS
**DEPTH TO FIRST WATER:** 19 feet BGS
**SCREEN INTERVAL:** 15-25 feet BGS
**DATE:** 9/25/13



FILL: GRAVEL WITH SAND, fine gravel, rounded to subangular, well graded sand, with silt and clay, medium brown, dense, moist.

Coarse gravel.

SILT WITH SAND, fine, dark gray, stiff, moist, trace clay, wood fragments, organics.

WELL GRADED SAND WITH GRAVEL, dark gray, dense, moist, well graded gravel.

No recovery 12-16 feet BGS.

Becomes dry.

Becomes wet at 19 feet BGS.

Becomes fine rounded gravel

WELL GRADED GRAVEL WITH SAND, rounded gravel, brown, dense, saturated, trace silt.

Bottom of WP-2 at 25 feet BGS.

0.010 PVC Well Screen 15 to 25 feet BGS

**BORING LOG**

# Laboratory Testing Results

# 2

# DENSITY BY DRIVE- CYLINDER METHOD (ASTM D2937)

| | | | |
|---|---|---|---|
| **Project Name:** | Danco Taylor Way | **Project Number:** | 022138 |
| **Performed By:** | JMA | **Date:** | 1/12/2023 |
| **Checked By:** | ALG | **Date:** | 1/30/2023 |
| **Project Manager:** | JB | | |

| | | | | | |
|---|---|---|---|---|---|
| **Lab Sample Number** | 23-034 | | | | |
| **Boring Label** | B4 | | | | |
| **Sample Depth (ft)** | 10-10.5 | | | | |
| **Diameter of Cylinder, in** | 2.42 | | | | |
| **Total Length of Cylinder, in.** | 6.00 | | | | |
| **Length of Empty Cylinder A, in.** | 0.00 | | | | |
| **Length of Empty Cylinder B, in.** | 0.07 | | | | |
| **Length of Cylinder Filled, in** | 5.93 | | | | |
| **Volume of Sample, in$^3$** | 27.28 | | | | |
| **Volume of Sample, cc.** | 446.97 | | | | |

| | | | | | |
|---|---|---|---|---|---|
| **Pan #** | ss15 | | | | |
| **Weight of Wet Soil and Pan** | 1206.0 | | | | |
| **Weight of Dry Soil and Pan** | 1117.9 | | | | |
| **Weight of Water** | 88.1 | | | | |
| **Weight of Pan** | 194.2 | | | | |
| **Weight of Dry Soil** | 923.7 | | | | |
| **Percent Moisture** | 9.5 | | | | |
| **Dry Density, g/cc** | 2.07 | | | | |
| **Dry Density, lb/ft$^3$** | 129.0 | | | | |

# PERCENT PASSING # 200 SIEVE (ASTM - D1140)

| | | | |
|---|---|---|---|
| **Project Name:** | Danco Taylor Way | **Project Number:** | **022138** |
| **Performed By:** | **JMA** | **Date:** | **1/16/2022** |
| **Checked By:** | **ALG** | **Date:** | **1/30/2023** |
| **Project Manager:** | **JB** | | |

| **Lab Sample Number** | 23-003 | 23-014 | 23-015 | 23-018 | 23-023 |
|---|---|---|---|---|---|
| **Boring Label** | B1 | B2 | B2 | B2 | B3 |
| **Sample Depth** | 3-4.5 | 6-7.5 | 10-11.5 | 25-26.5 | 6-7.5 |
| **Pan Number** | ss3 | ss7 | s11 | ss11 | ss1 |
| **Dry Weight of Soil & Pan** | 551.5 | 839.9 | 426.8 | 354.1 | 614.9 |
| **Pan Weight** | 197.1 | 192.9 | 192.6 | 192.6 | 194.8 |
| **Weight of Dry Soil** | 354.4 | 647.0 | 234.2 | 161.5 | 420.1 |
| **Soil Weight Retained on #200&Pan** | 523.4 | 800.8 | 338.9 | 347.3 | 579.5 |
| **Soil Weight Passing #200** | 28.1 | 39.1 | 87.9 | 6.8 | 35.4 |
| **Percent Passing #200** | **7.9** | **6.0** | **38** | **4.2** | **8.4** |

| **Lab Sample Number** | 23-025 | 23-031 | 23-035 | 23-039 | |
|---|---|---|---|---|---|
| **Boring Label** | B3 | B4 | B4 | B4 | |
| **Sample Depth** | 10-11.5 | 4-5.5 | 15-16.5 | 50-51.5 | |
| **Pan Number** | ss14 | ss9 | ss12 | ss10 | |
| **Dry Weight of Soil & Pan** | 527.6 | 600.9 | 556.5 | 494.0 | |
| **Pan Weight** | 192.6 | 196.4 | 194.2 | 195.4 | |
| **Weight of Dry Soil** | 335.0 | 404.5 | 362.3 | 298.6 | |
| **Soil Weight Retained on #200&Pan** | 430.9 | 574.6 | 503.2 | 416.7 | |
| **Soil Weight Passing #200** | 96.7 | 26.3 | 53.3 | 77.3 | |
| **Percent Passing #200** | **29** | **6.5** | **15** | **26** | |

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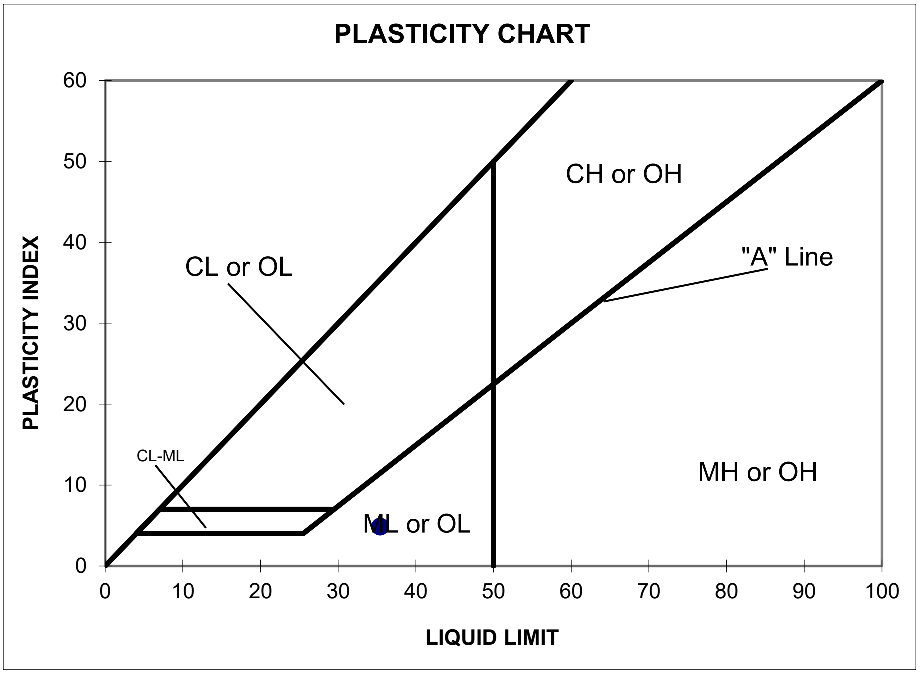
812 W. Wabash  Eureka, CA 95501-2138  Tel: 707/441-8855  FAX: 707/441-8877 E-mail: shninfo@shn-engr.com

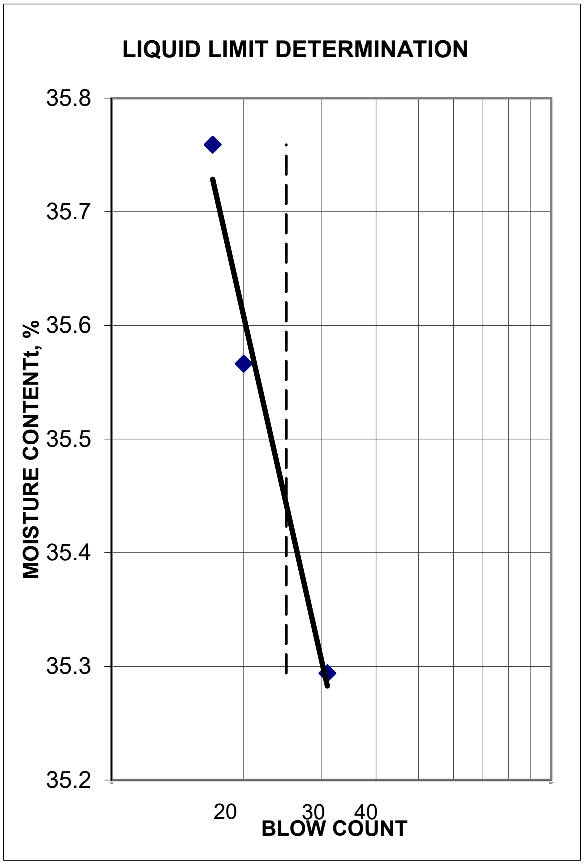
# LIQUID LIMIT, PLASTIC LIMIT, and PLASTICITY INDEX (ASTM-D4318)

JOB NAME: **Danco Taylor Way** JOB #: **022138** LAB SAMPLE #: **23-025**
SAMPLE ID: **B3 @ 10-11.5** PERFORMED BY: **JMA** DATE: 1/17/2023
PROJECT MANAGER: **JB** CHECKED BY: **ALG** DATE: 1/30/2023
Taylor Way, Blue Lake California

| LINE NO. | | TRIAL NO. 1 | TRIAL NO. 2 | TRIAL NO. 1 | TRIAL NO. 2 | TRIAL NO. 3 |
|---|---|---|---|---|---|---|
| A | PAN # | 15 | 16 | 4 | 5 | 6 |
| B | PAN WT. (g) | 21.150 | 20.330 | 29.210 | 28.770 | 29.520 |
| C | WT. WET SOIL & PAN (g) | 27.420 | 26.850 | 40.020 | 39.900 | 40.340 |
| D | WT. DRY SOIL & PAN (g) | 25.930 | 25.350 | 37.200 | 36.980 | 37.490 |
| E | WT. WATER (C-D) | 1.490 | 1.500 | 2.820 | 2.920 | 2.850 |
| F | WT. DRY SOIL (D-B) | 4.780 | 5.020 | 7.990 | 8.210 | 7.970 |
| G | BLOW COUNT | -- | -- | 31 | 20 | 17 |
| H | MOISTURE CONTENT (E/F*100) | 31.2 | 29.9 | 35.3 | 35.6 | 35.8 |

| LIQUID LIMIT | **PLASTIC INDEX** | PLASTIC LIMIT |
|---|---|---|
| 35 | **5** | 31 |





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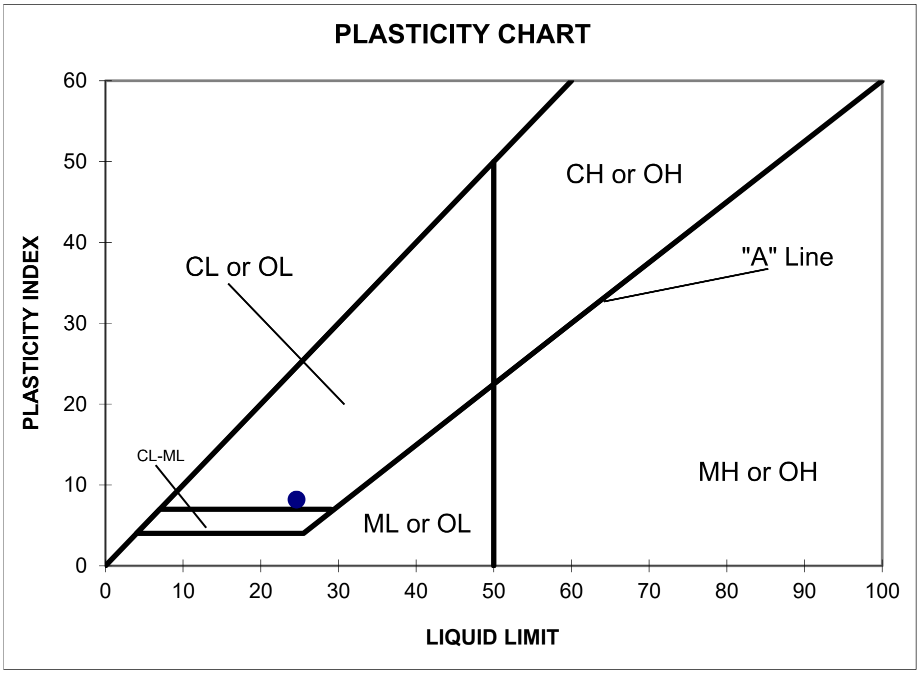
812 W. Wabash Eureka, CA 95501-2138 Tel: 707/441-8855 FAX: 707/441-8877 E-mail: shninfo@shn-engr.com

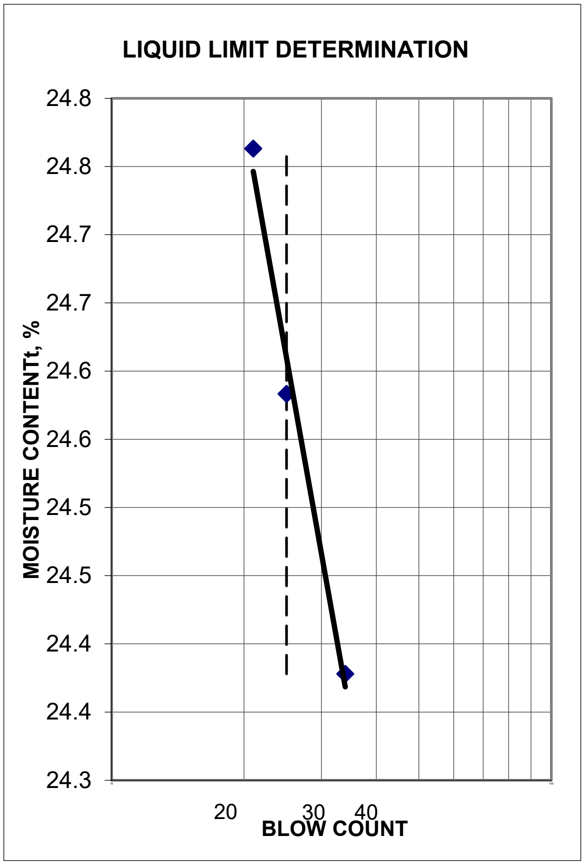
# LIQUID LIMIT, PLASTIC LIMIT, and PLASTICITY INDEX (ASTM-D4318)

| | | | | | |
|---|---|---|---|---|---|
| JOB NAME: | **Danco Taylor Way** | JOB #: | **022138** | LAB SAMPLE #: | **23-038** |
| SAMPLE ID: | **B4 @ 41-41.5** | PERFORMED BY: | **JMA** | DATE: | 1/17/2023 |
| PROJECT MANAGER: | **JB** | CHECKED BY: | **ALG** | DATE: | 1/30/2023 |
| | Taylor Way, Blue Lake California | | | | |

| LINE NO. | | TRIAL NO. 1 | TRIAL NO. 2 | TRIAL NO. 1 | TRIAL NO. 2 | TRIAL NO. 3 |
|---|---|---|---|---|---|---|
| A | PAN # | 19 | 20 | 10 | 11 | 12 |
| B | PAN WT. (g) | 16.870 | 17.150 | 29.570 | 28.690 | 29.330 |
| C | WT. WET SOIL & PAN (g) | 23.570 | 23.710 | 39.570 | 37.660 | 38.550 |
| D | WT. DRY SOIL & PAN (g) | 22.640 | 22.770 | 37.610 | 35.890 | 36.720 |
| E | WT. WATER (C-D) | 0.930 | 0.940 | 1.960 | 1.770 | 1.830 |
| F | WT. DRY SOIL (D-B) | 5.770 | 5.620 | 8.040 | 7.200 | 7.390 |
| G | BLOW COUNT | -- | -- | 34 | 25 | 21 |
| H | MOISTURE CONTENT (E/F*100) | 16.1 | 16.7 | 24.4 | 24.6 | 24.8 |

| LIQUID LIMIT | **PLASTIC INDEX** | PLASTIC LIMIT |
|---|---|---|
| 25 | **8** | 16 |





# *Resistance, R-Value*

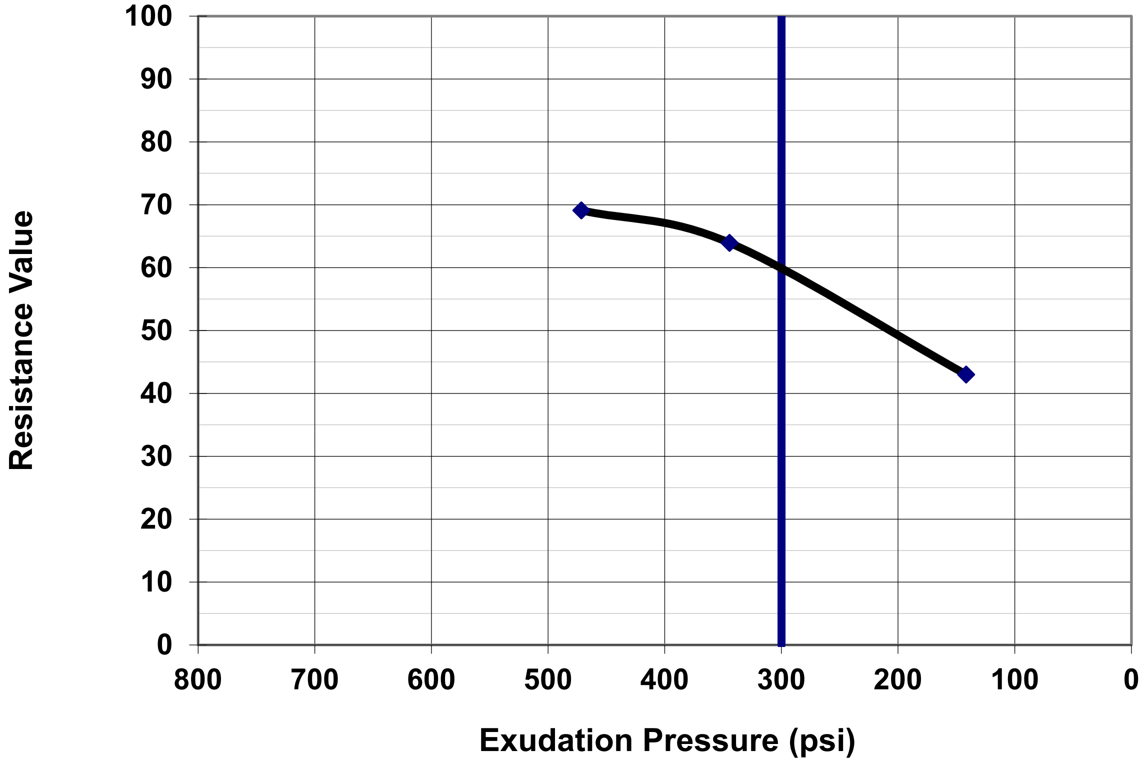
Caltrans Method 301

**Project :** Taylor Way
**Client :** Danco
**Sample Location :** Composite 0-1'
**Sample Description :** Gravelly CLAY with sand

**Project No. :** 022138
**Sampled By :** AT
**Test Date :** 1/20/2023
**Sample Number :** 23-147

| Test Specimen | 1 | 2 | 3 |
|---|---|---|---|
| Moisture Content (%) | 10.1 | 10.2 | 9.6 |
| Dry Density (pcf) | 127.8 | 125.5 | 126.6 |
| Expansion Pressure (psf) | 86.6 | 112.6 | 52.0 |
| Exudation Pressure (psi) | 142 | 345 | 472 |
| Resistance Value | 43 | 64 | 69 |

**R Value at 300 psi Exudation Pressure:** **59**

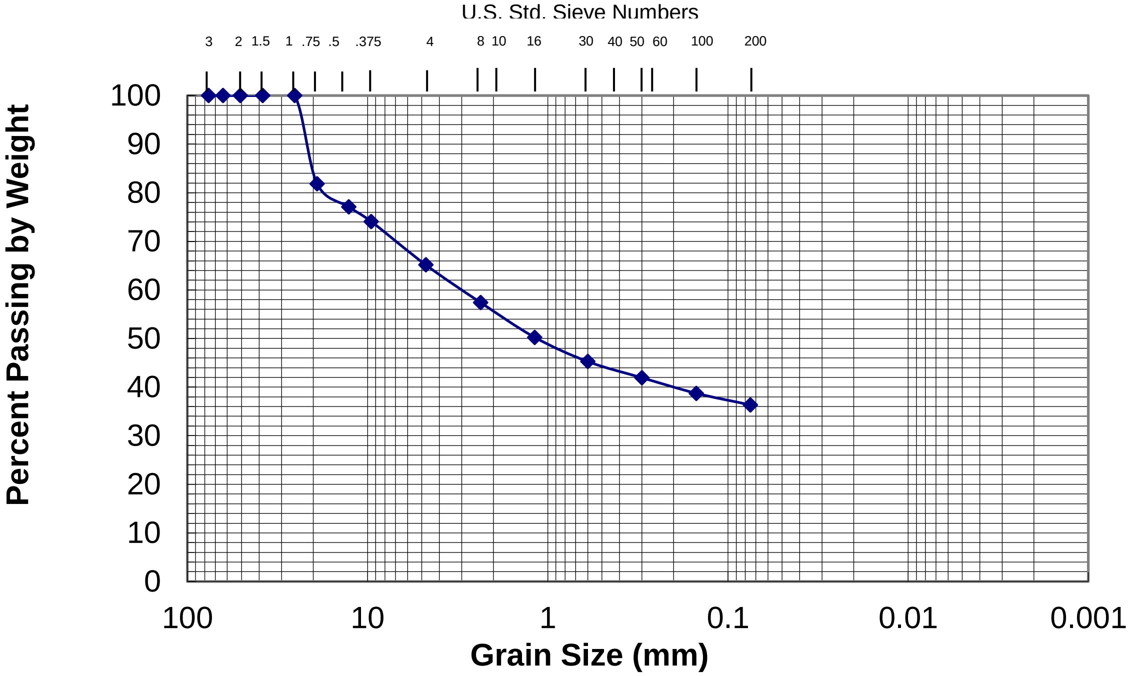


**Phone:** (707) 441-8855 **Email:** info@shn-engr.com **Web:** shn-
812 W. Wabash Avenue, Eureka, CA 95

| | | | |
|---|---|---|---|
| PROJECT NAME: | Taylor Way Geotech | PROJECT NUMBER: | 022138 |
| SAMPLE ID: | B-1, 5-6.5' | LAB SAMPLE: | 23-004 |
| DATE TESTED: | 1/16/23 | CLIENT: | Danco |

| SIEVE | 3" | 2 1/2" | 2" | 1 1/2" | 1" | 3/4" | 1/2" | 3/8" | #4 | #8 | #16 | #30 | #50 | #100 | #200 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| SIEVE SIZE (mm) | 76.2 | 63.5 | 50.8 | 38.1 | 25.4 | 19.1 | 12.7 | 9.53 | 4.75 | 2.36 | 1.18 | 0.600 | 0.300 | 0.150 | 0.075 |
| PERCENT PASSING | 100 | 100 | 100 | 100 | 100 | 81.9 | 77.1 | 74.1 | 65.2 | 57.4 | 50.2 | 45.3 | 41.9 | 38.7 | 36.3 |

## Gradation Test Results



| | | | | | | |
|---|---|---|---|---|---|---|
| | GRAVEL | | SAND | | | SILT or CLAY |
| | Coarse | Fine | Coarse | Medium | Fine | |

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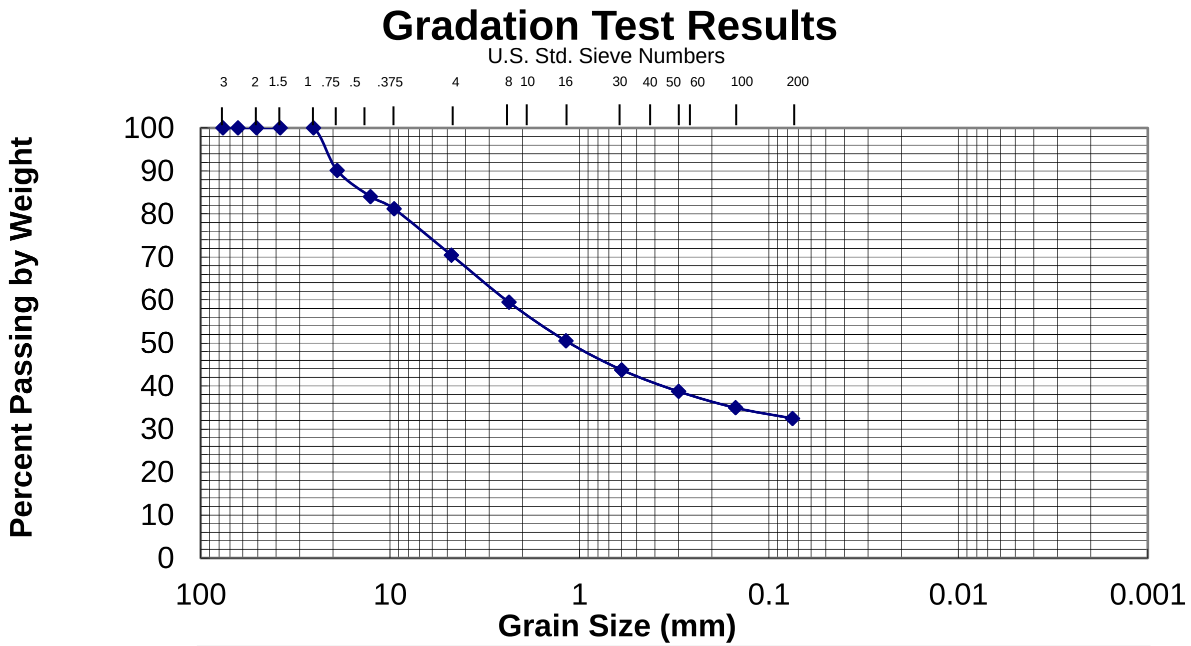
PROJECT NAME: Taylor Way Geotech
SAMPLE ID: B-2, 15-16.5'
DATE TESTED: 1/16/23

PROJECT NUMBER: 022138
LAB SAMPLE: 23-016
CLIENT: Danco

| SIEVE | 3" | 2 1/2" | 2" | 1 1/2" | 1" | 3/4" | 1/2" | 3/8" | #4 | #8 | #16 | #30 | #50 | #100 | #200 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| SIEVE SIZE (mm) | 76.2 | 63.5 | 50.8 | 38.1 | 25.4 | 19.1 | 12.7 | 9.53 | 4.75 | 2.36 | 1.18 | 0.600 | 0.300 | 0.150 | 0.075 |
| PERCENT PASSING | 100 | 100 | 100 | 100 | 100 | 90.1 | 84.1 | 81.2 | 70.4 | 59.5 | 50.5 | 43.8 | 38.7 | 34.9 | 32.5 |



| | GRAVEL | | SAND | | | SILT or CLAY |
|---|---|---|---|---|---|---|
| | Coarse | Fine | Coarse | Medium | Fine | |

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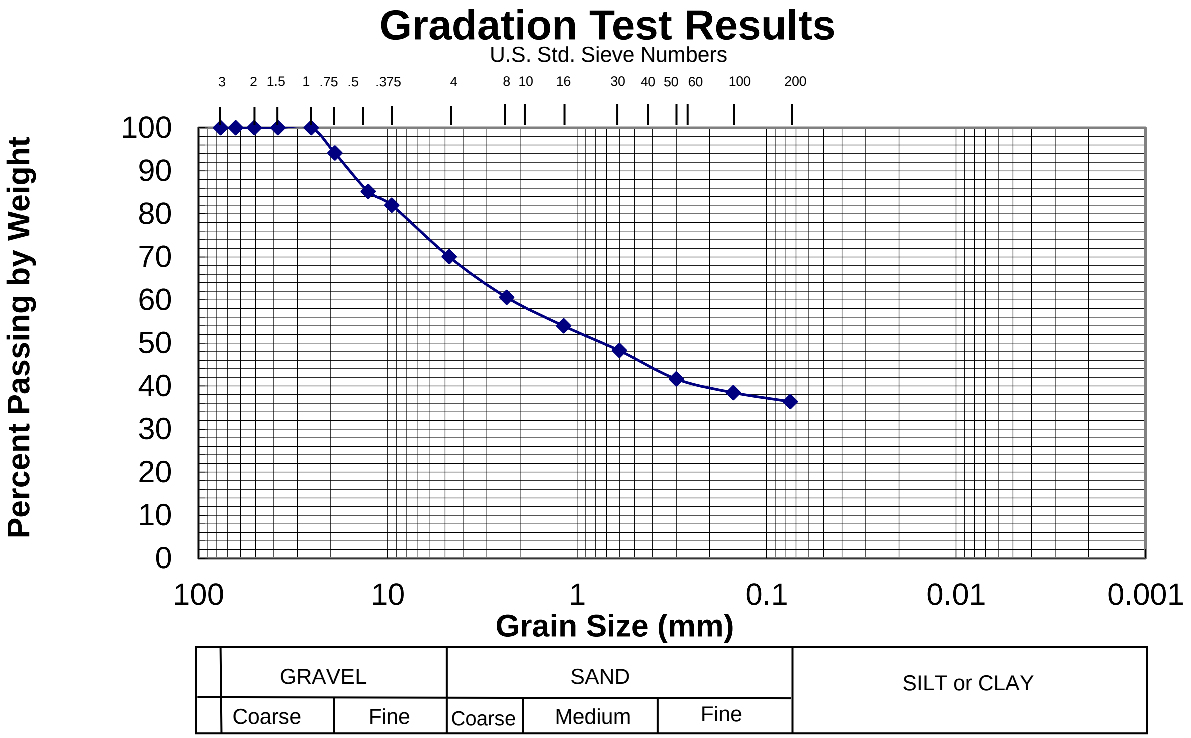
812 W. Wabash Avenue, Eureka, CA 95501-2138

| | | | | |
|---|---|---|---|---|
| PROJECT NAME: | Taylor Way Geotech | | PROJECT NUMBER: | 022138 |
| SAMPLE ID: | B-3, 15-16.5' | | LAB SAMPLE: | 23-027 |
| DATE TESTED: | 1/16/23 | | CLIENT: | Danco |

| SIEVE | 3" | 2 1/2" | 2" | 1 1/2" | 1" | 3/4" | 1/2" | 3/8" | #4 | #8 | #16 | #30 | #50 | #100 | #200 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| SIEVE SIZE (mm) | 76.2 | 63.5 | 50.8 | 38.1 | 25.4 | 19.1 | 12.7 | 9.53 | 4.75 | 2.36 | 1.18 | 0.600 | 0.300 | 0.150 | 0.075 |
| PERCENT PASSING | 100 | 100 | 100 | 100 | 100 | 94.1 | 85.2 | 82 | 70 | 60.6 | 54.0 | 48.3 | 41.6 | 38.5 | 36.4 |



# Liquefaction Analysis Results

# 3

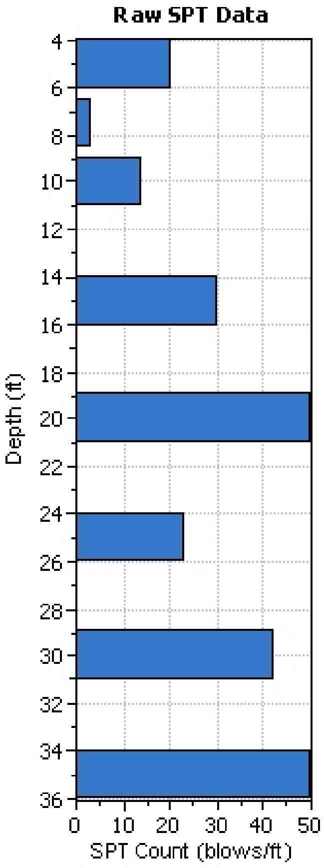
# SPT BASED LIQUEFACTION ANALYSIS REPORT

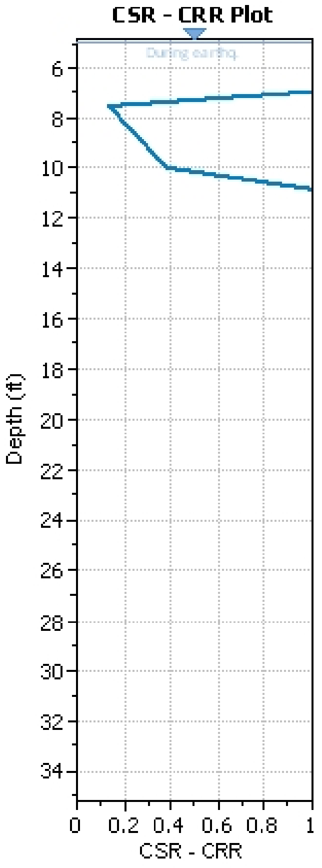
**Project title : DANCO-Taylor Way Geotechnical Investigation** **SPT Name: B-1**

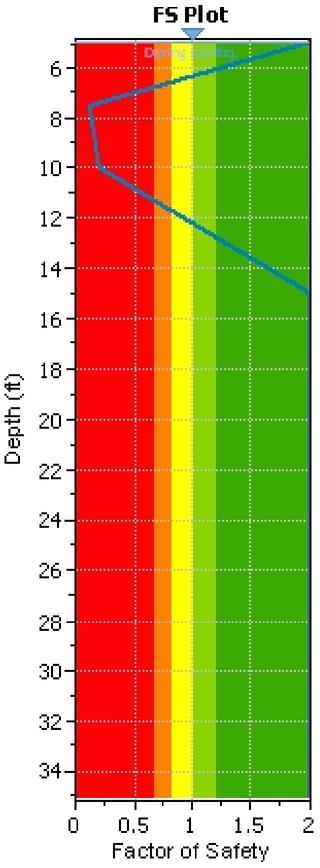
**Location : Blue Lake, Humboldt County, CA**

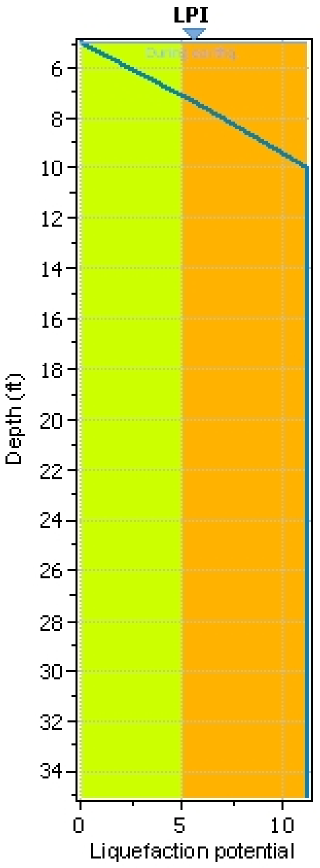
## :: Input parameters and analysis properties ::

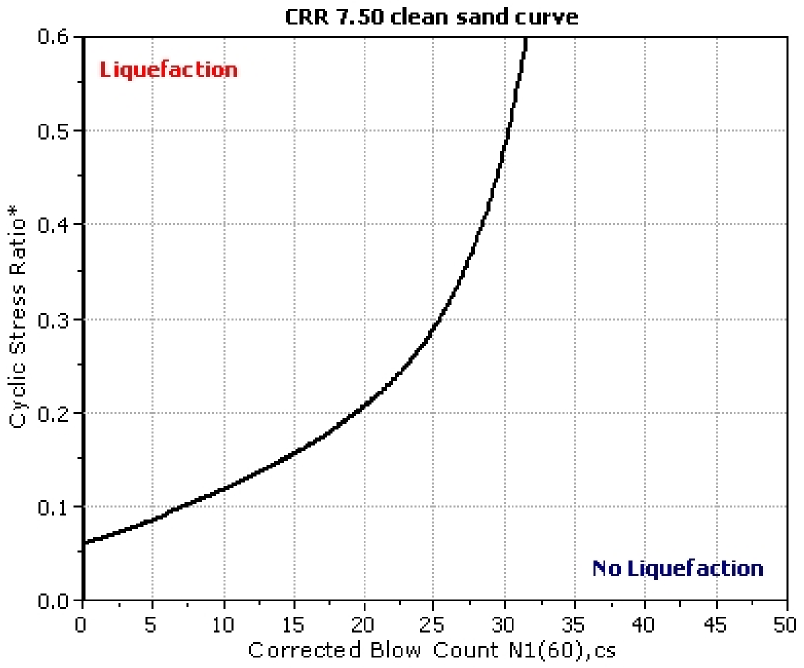
| | | | |
|---|---|---|---|
| Analysis method: | Boulanger & Idriss, 2014 | G.W.T. (in-situ): | 7.00 ft |
| Fines correction method: | Boulanger & Idriss, 2014 | G.W.T. (earthq.): | 5.00 ft |
| Sampling method: | Sampler wo liners | Earthquake magnitude $M_w$: | 9.10 ft |
| Borehole diameter: | 65mm to 115mm | Peak ground acceleration: | 1.49 g |
| Rod length: | 3.00 ft | Eq. external load: | 0.00 tsf |
| Hammer energy ratio: | 1.25 | | |


Raw SPT Data


CSR - CRR Plot


FS Plot


LPI


CRR 7.50 clean sand curve

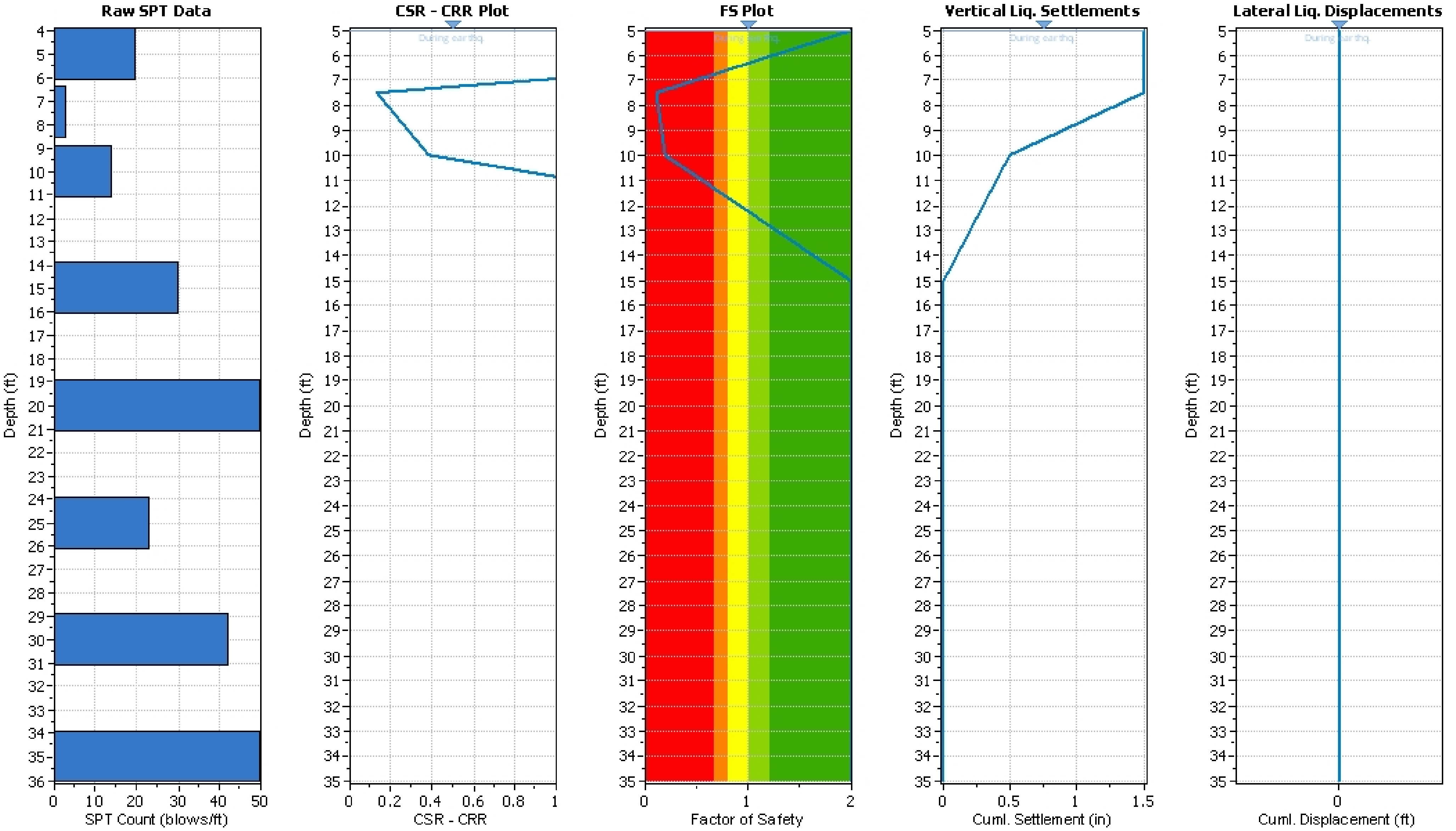
**F.S. color scheme**

- 🟥 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 | 20 | 36.00 | 120.00 | 2.50 | Yes |
| 7.50 | 3 | 36.00 | 120.00 | 2.50 | Yes |
| 10.00 | 14 | 36.00 | 120.00 | 5.00 | Yes |
| 15.00 | 30 | 36.00 | 120.00 | 5.00 | Yes |
| 20.00 | 50 | 36.00 | 120.00 | 5.00 | Yes |
| 25.00 | 23 | 36.00 | 120.00 | 5.00 | Yes |
| 30.00 | 42 | 36.00 | 120.00 | 5.00 | Yes |
| 35.00 | 50 | 36.00 | 120.00 | 5.00 | 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) | $\sigma_v$ (tsf) | $u_o$ (tsf) | $\sigma'_{vo}$ (tsf) | m | $C_N$ | $C_E$ | $C_B$ | $C_R$ | $C_S$ | $(N_1)_{60}$ | FC (%) | $\Delta(N_1)_{60}$ | $(N_1)_{60cs}$ | $CRR_{7.5}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 5.00 | 20 | 120.00 | 0.30 | 0.00 | 0.30 | 0.29 | 1.45 | 1.25 | 1.00 | 0.75 | 1.20 | 33 | 36.00 | 5.52 | 39 | 4.000 |
| 7.50 | 3 | 120.00 | 0.45 | 0.02 | 0.43 | 0.49 | 1.55 | 1.25 | 1.00 | 0.80 | 1.20 | 6 | 36.00 | 5.52 | 12 | 0.132 |
| 10.00 | 14 | 120.00 | 0.60 | 0.09 | 0.51 | 0.37 | 1.31 | 1.25 | 1.00 | 0.80 | 1.20 | 22 | 36.00 | 5.52 | 28 | 0.384 |
| 15.00 | 30 | 120.00 | 0.90 | 0.25 | 0.65 | 0.28 | 1.14 | 1.25 | 1.00 | 0.85 | 1.20 | 44 | 36.00 | 5.52 | 50 | 4.000 |
| 20.00 | 50 | 120.00 | 1.20 | 0.41 | 0.79 | 0.10 | 1.03 | 1.25 | 1.00 | 0.95 | 1.20 | 73 | 36.00 | 5.52 | 79 | 4.000 |
| 25.00 | 23 | 120.00 | 1.50 | 0.56 | 0.94 | 0.30 | 1.04 | 1.25 | 1.00 | 0.95 | 1.20 | 34 | 36.00 | 5.52 | 40 | 4.000 |
| 30.00 | 42 | 120.00 | 1.80 | 0.72 | 1.08 | 0.18 | 1.00 | 1.25 | 1.00 | 1.00 | 1.20 | 63 | 36.00 | 5.52 | 69 | 4.000 |
| 35.00 | 50 | 120.00 | 2.10 | 0.87 | 1.23 | 0.10 | 0.99 | 1.25 | 1.00 | 1.00 | 1.20 | 74 | 36.00 | 5.52 | 80 | 4.000 |

**Abbreviations**

| | |
|---|---|
| $\sigma_v$: | Total stress during SPT test (tsf) |
| $u_o$: | Water pore pressure during SPT test (tsf) |
| $\sigma'_{vo}$: | Effective overburden pressure during SPT test (tsf) |
| m: | Stress exponent normalization factor |
| $C_N$: | 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 |
| $\Delta(N_1)_{60}$ | Equivalent clean sand adjustment |
| $N_{1(60)cs}$: | Corected $N_{1(60)}$ value for fines content |
| $CRR_{7.5}$: | Cyclic 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) | $r_d$ | CSR | $MSF_{max}$ | $(N_1)_{60cs}$ | MSF | $CSR_{eq,M=7.5}$ | $K_{sigma}$ | $CSR^*$ | FS | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 5.00 | 120.00 | 0.30 | 0.00 | 0.30 | 1.01 | 0.973 | 2.20 | 39 | 0.48 | 2.046 | 1.10 | 1.860 | 2.000 | 🟢 |
| 7.50 | 120.00 | 0.45 | 0.08 | 0.37 | 1.01 | 1.178 | 1.24 | 12 | 0.90 | 1.313 | 1.10 | 1.194 | 0.111 | 🔴 |
| 10.00 | 120.00 | 0.60 | 0.16 | 0.44 | 1.01 | 1.317 | 1.88 | 28 | 0.62 | 2.140 | 1.10 | 1.945 | 0.197 | 🔴 |
| 15.00 | 120.00 | 0.90 | 0.31 | 0.59 | 1.01 | 1.494 | 2.20 | 50 | 0.48 | 3.140 | 1.10 | 2.855 | 2.000 | 🟢 |
| 20.00 | 120.00 | 1.20 | 0.47 | 0.73 | 1.01 | 1.603 | 2.20 | 79 | 0.48 | 3.369 | 1.10 | 3.062 | 2.000 | 🟢 |

### :: 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) | $r_d$ | CSR | $MSF_{max}$ | $(N_1)_{60cs}$ | MSF | $CSR_{eq,M=7.5}$ | $K_{sigma}$ | $CSR^*$ | FS | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 25.00 | 120.00 | 1.50 | 0.62 | 0.88 | 1.01 | 1.676 | 2.20 | 40 | 0.48 | 3.524 | 1.06 | 3.338 | 2.000 | ● |
| 30.00 | 120.00 | 1.80 | 0.78 | 1.02 | 1.01 | 1.730 | 2.20 | 69 | 0.48 | 3.635 | 1.01 | 3.596 | 2.000 | ● |
| 35.00 | 120.00 | 2.10 | 0.94 | 1.16 | 1.01 | 1.769 | 2.20 | 80 | 0.48 | 3.719 | 0.97 | 3.826 | 2.000 | ● |

**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
CSR : Cyclic Stress Ratio
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
FS: Calculated factor of safety against soil liquefaction

### :: Liquefaction potential according to Iwasaki ::

| Depth (ft) | FS | F | wz | Thickness (ft) | $I_L$ |
|---|---|---|---|---|---|
| 5.00 | 2.000 | 0.00 | 9.24 | 2.50 | 0.00 |
| 7.50 | 0.111 | 0.89 | 8.86 | 2.50 | 6.00 |
| 10.00 | 0.197 | 0.80 | 8.48 | 2.50 | 5.19 |
| 15.00 | 2.000 | 0.00 | 7.71 | 5.00 | 0.00 |
| 20.00 | 2.000 | 0.00 | 6.95 | 5.00 | 0.00 |
| 25.00 | 2.000 | 0.00 | 6.19 | 5.00 | 0.00 |
| 30.00 | 2.000 | 0.00 | 5.43 | 5.00 | 0.00 |
| 35.00 | 2.000 | 0.00 | 4.67 | 5.00 | 0.00 |

**Overall potential $I_L$ : 11.19**

$I_L$ = 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 & Lateral displ.acements estimation for saturated sands ::

| Depth (ft) | $(N_1)_{60cs}$ | $\gamma_{lim}$ (%) | $F_{\alpha}$ | $FS_{liq}$ | $\gamma_{max}$ (%) | $e_v$ (%) | dz (ft) | $S_{v-1D}$ (in) | LDI (ft) |
|---|---|---|---|---|---|---|---|---|---|
| 5.00 | 39 | 1.07 | -0.73 | 2.000 | 0.00 | 0.00 | 2.50 | 0.000 | 0.00 |
| 7.50 | 12 | 38.03 | 0.86 | 0.111 | 38.03 | 3.34 | 2.50 | 8.356 | 0.00 |
| 10.00 | 28 | 6.08 | 0.04 | 0.197 | 3.92 | 0.83 | 5.00 | 4.172 | 0.00 |
| 15.00 | 50 | 0.04 | -1.59 | 2.000 | 0.00 | 0.00 | 5.00 | 0.000 | 0.00 |
| 20.00 | 79 | 0.00 | -4.11 | 2.000 | 0.00 | 0.00 | 5.00 | 0.000 | 0.00 |
| 25.00 | 40 | 0.87 | -0.80 | 2.000 | 0.00 | 0.00 | 5.00 | 0.000 | 0.00 |
| 30.00 | 69 | 0.00 | -3.21 | 2.000 | 0.00 | 0.00 | 5.00 | 0.000 | 0.00 |
| 35.00 | 80 | 0.00 | -4.20 | 2.000 | 0.00 | 0.00 | 5.00 | 0.000 | 0.00 |

**:: Vertical & Lateral displ.acements estimation for saturated sands ::**

| Depth (ft) | $(N_1)_{60cs}$ | $\gamma_{lim}$ (%) | $F_\alpha$ | $FS_{liq}$ | $\gamma_{max}$ (%) | $e_v$ (%) | dz (ft) | $S_{v\text{-}1D}$ (in) | LDI (ft) |
|---|---|---|---|---|---|---|---|---|---|

**Cumulative settlements: 12.527 0.00**

**Abbreviations**

| | |
|---|---|
| $\gamma_{lim}$: | Limiting shear strain (%) |
| $F_\alpha$/N: | Maximun shear strain factor |
| $\gamma_{max}$: | Maximum shear strain (%) |
| $e_v$:: | Post liquefaction volumetric strain (%) |
| $S_{v\text{-}1D}$: | Estimated vertical settlement (in) |
| LDI: | Estimated lateral displacement (ft) |

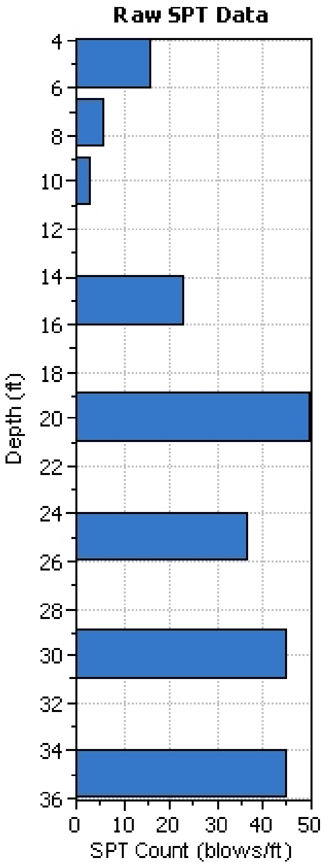
## SPT BASED LIQUEFACTION ANALYSIS REPORT

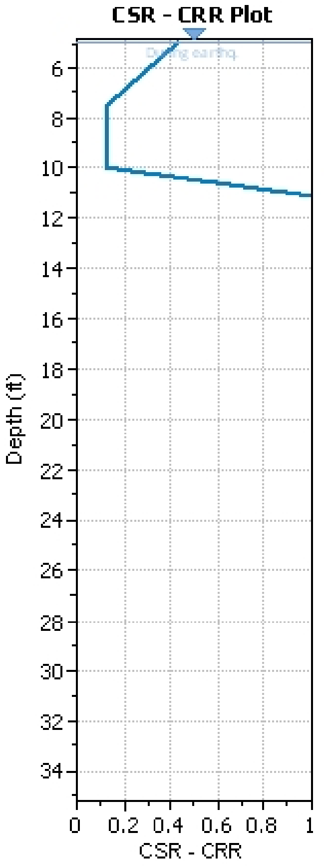
**Project title : DANCO-Taylor Way Geotechnical Investigation** **SPT Name: B-2**

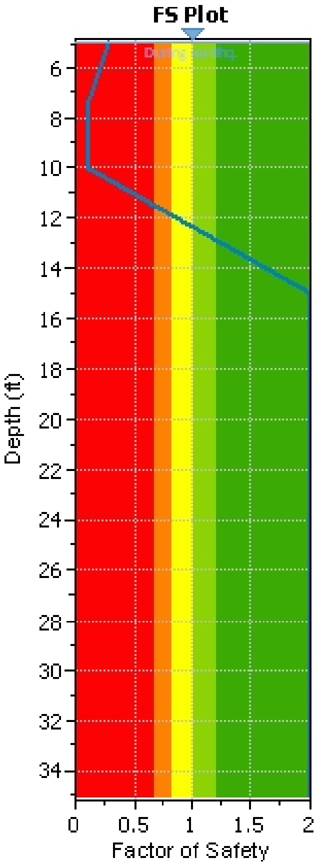
**Location : Blue Lake, Humboldt County, CA**

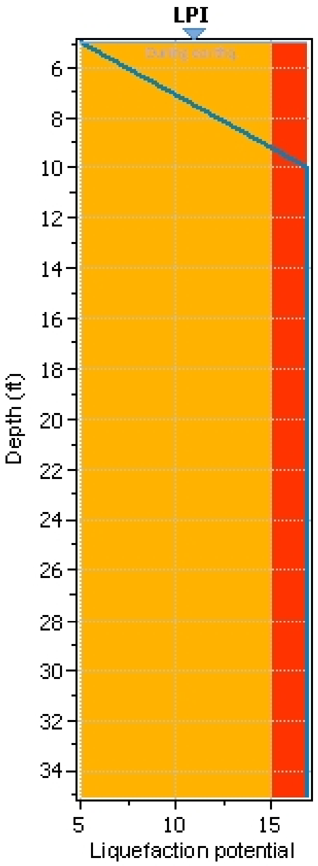
### :: Input parameters and analysis properties ::

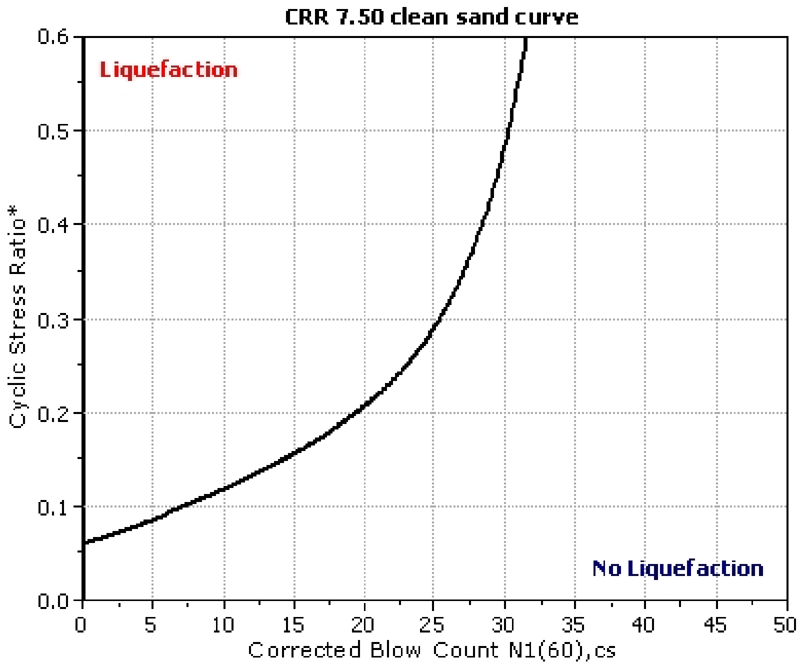
| | | | |
|---|---|---|---|
| Analysis method: | Boulanger & Idriss, 2014 | G.W.T. (in-situ): | 7.00 ft |
| Fines correction method: | Boulanger & Idriss, 2014 | G.W.T. (earthq.): | 5.00 ft |
| Sampling method: | Sampler wo liners | Earthquake magnitude $M_w$: | 9.10 ft |
| Borehole diameter: | 65mm to 115mm | Peak ground acceleration: | 1.49 g |
| Rod length: | 3.00 ft | Eq. external load: | 0.00 tsf |
| Hammer energy ratio: | 1.25 | | |











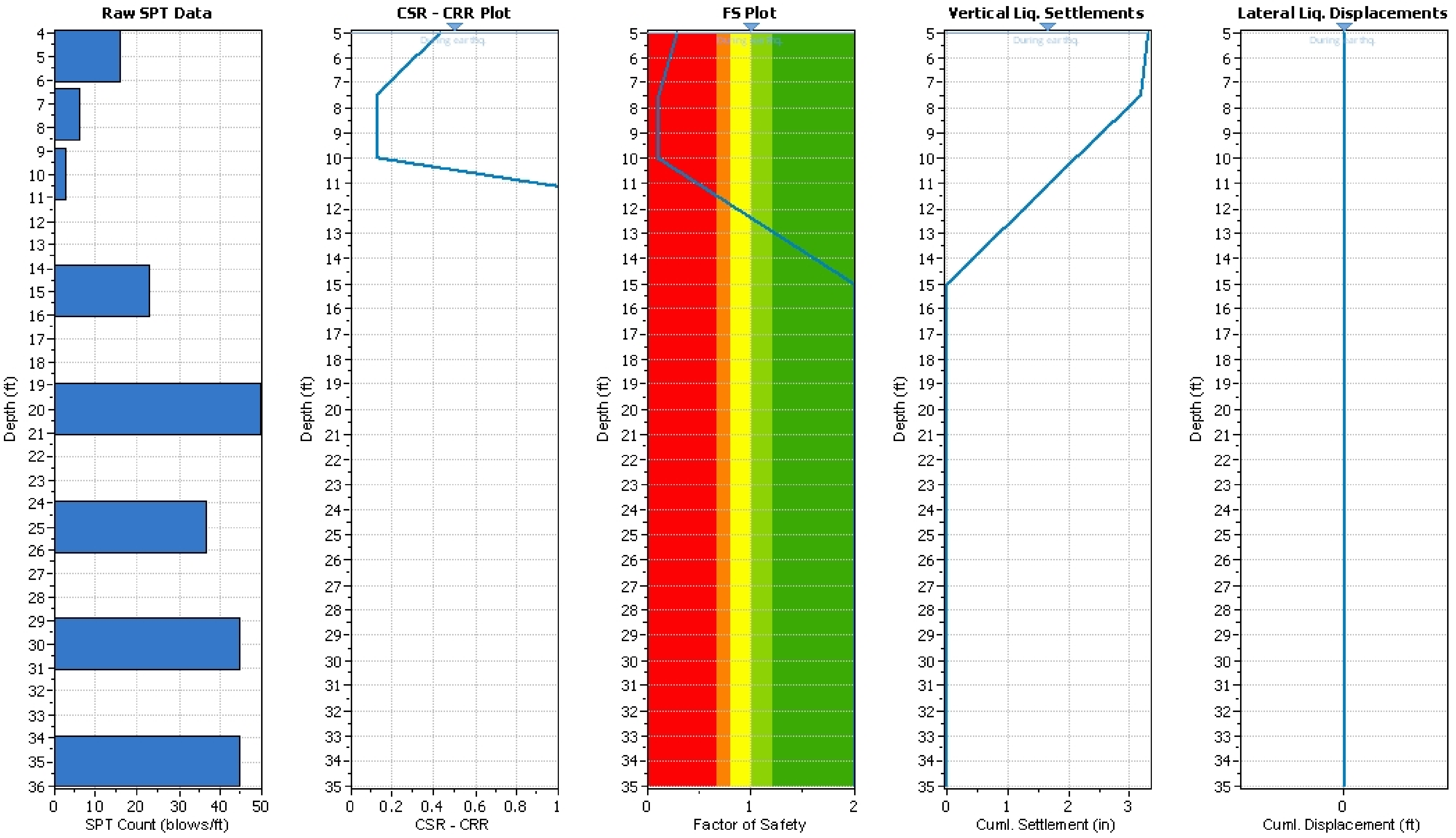
**F.S. color scheme**

- 🟥 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 | 16 | 6.00 | 120.00 | 2.50 | Yes |
| 7.50 | 6 | 6.00 | 120.00 | 2.50 | Yes |
| 10.00 | 3 | 38.00 | 120.00 | 5.00 | Yes |
| 15.00 | 23 | 32.00 | 120.00 | 5.00 | Yes |
| 20.00 | 50 | 32.00 | 120.00 | 5.00 | Yes |
| 25.00 | 37 | 4.00 | 120.00 | 5.00 | Yes |
| 30.00 | 45 | 4.00 | 120.00 | 5.00 | Yes |
| 35.00 | 45 | 4.00 | 120.00 | 5.00 | 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) | $\sigma_v$ (tsf) | $u_o$ (tsf) | $\sigma'_{vo}$ (tsf) | m | $C_N$ | $C_E$ | $C_B$ | $C_R$ | $C_S$ | $(N_1)_{60}$ | FC (%) | $\Delta(N_1)_{60}$ | $(N_1)_{60cs}$ | $CRR_{7.5}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 5.00 | 16 | 120.00 | 0.30 | 0.00 | 0.30 | 0.37 | 1.60 | 1.25 | 1.00 | 0.75 | 1.20 | 29 | 6.00 | 0.03 | 29 | 0.429 |
| 7.50 | 6 | 120.00 | 0.45 | 0.02 | 0.43 | 0.52 | 1.59 | 1.25 | 1.00 | 0.80 | 1.20 | 11 | 6.00 | 0.03 | 11 | 0.125 |
| 10.00 | 3 | 120.00 | 0.60 | 0.09 | 0.51 | 0.50 | 1.45 | 1.25 | 1.00 | 0.80 | 1.20 | 5 | 38.00 | 5.55 | 11 | 0.125 |
| 15.00 | 23 | 120.00 | 0.90 | 0.25 | 0.65 | 0.30 | 1.16 | 1.25 | 1.00 | 0.85 | 1.20 | 34 | 32.00 | 5.43 | 39 | 4.000 |
| 20.00 | 50 | 120.00 | 1.20 | 0.41 | 0.79 | 0.10 | 1.03 | 1.25 | 1.00 | 0.95 | 1.20 | 73 | 32.00 | 5.43 | 78 | 4.000 |
| 25.00 | 37 | 120.00 | 1.50 | 0.56 | 0.94 | 0.22 | 1.03 | 1.25 | 1.00 | 0.95 | 1.20 | 54 | 4.00 | 0.00 | 54 | 4.000 |
| 30.00 | 45 | 120.00 | 1.80 | 0.72 | 1.08 | 0.15 | 1.00 | 1.25 | 1.00 | 1.00 | 1.20 | 67 | 4.00 | 0.00 | 67 | 4.000 |
| 35.00 | 45 | 120.00 | 2.10 | 0.87 | 1.23 | 0.16 | 0.98 | 1.25 | 1.00 | 1.00 | 1.20 | 66 | 4.00 | 0.00 | 66 | 4.000 |

### Abbreviations

| | |
|---|---|
| $\sigma_v$: | Total stress during SPT test (tsf) |
| $u_o$: | Water pore pressure during SPT test (tsf) |
| $\sigma'_{vo}$: | Effective overburden pressure during SPT test (tsf) |
| m: | Stress exponent normalization factor |
| $C_N$: | 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 |
| $\Delta(N_1)_{60}$ | Equivalent clean sand adjustment |
| $N_{1(60)cs}$: | Corected $N_{1(60)}$ value for fines content |
| $CRR_{7.5}$: | Cyclic 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) | $r_d$ | CSR | $MSF_{max}$ | $(N_1)_{60cs}$ | MSF | $CSR_{eq,M=7.5}$ | $K_{sigma}$ | $CSR^*$ | FS | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 5.00 | 120.00 | 0.30 | 0.00 | 0.30 | 1.01 | 0.973 | 1.94 | 29 | 0.59 | 1.649 | 1.10 | 1.499 | 0.286 | 🔴 |
| 7.50 | 120.00 | 0.45 | 0.08 | 0.37 | 1.01 | 1.178 | 1.21 | 11 | 0.91 | 1.299 | 1.10 | 1.180 | 0.106 | 🔴 |
| 10.00 | 120.00 | 0.60 | 0.16 | 0.44 | 1.01 | 1.317 | 1.21 | 11 | 0.91 | 1.452 | 1.08 | 1.340 | 0.093 | 🔴 |
| 15.00 | 120.00 | 0.90 | 0.31 | 0.59 | 1.01 | 1.494 | 2.20 | 39 | 0.48 | 3.140 | 1.10 | 2.855 | 2.000 | 🟢 |
| 20.00 | 120.00 | 1.20 | 0.47 | 0.73 | 1.01 | 1.603 | 2.20 | 78 | 0.48 | 3.369 | 1.10 | 3.062 | 2.000 | 🟢 |

## :: 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) | $r_d$ | CSR | $MSF_{max}$ | $(N_1)_{60cs}$ | MSF | $CSR_{eq,M=7.5}$ | $K_{sigma}$ | $CSR^*$ | FS | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 25.00 | 120.00 | 1.50 | 0.62 | 0.88 | 1.01 | 1.676 | 2.20 | 54 | 0.48 | 3.524 | 1.06 | 3.338 | 2.000 | 🟢 |
| 30.00 | 120.00 | 1.80 | 0.78 | 1.02 | 1.01 | 1.730 | 2.20 | 67 | 0.48 | 3.635 | 1.01 | 3.596 | 2.000 | 🟢 |
| 35.00 | 120.00 | 2.10 | 0.94 | 1.16 | 1.01 | 1.769 | 2.20 | 66 | 0.48 | 3.719 | 0.97 | 3.826 | 2.000 | 🟢 |

**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 |
| CSR : | Cyclic Stress Ratio |
| 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 |
| FS: | Calculated factor of safety against soil liquefaction |

## :: Liquefaction potential according to Iwasaki ::

| Depth (ft) | FS | F | wz | Thickness (ft) | $I_L$ |
|---|---|---|---|---|---|
| 5.00 | 0.286 | 0.71 | 9.24 | 2.50 | 5.03 |
| 7.50 | 0.106 | 0.89 | 8.86 | 2.50 | 6.03 |
| 10.00 | 0.093 | 0.91 | 8.48 | 2.50 | 5.86 |
| 15.00 | 2.000 | 0.00 | 7.71 | 5.00 | 0.00 |
| 20.00 | 2.000 | 0.00 | 6.95 | 5.00 | 0.00 |
| 25.00 | 2.000 | 0.00 | 6.19 | 5.00 | 0.00 |
| 30.00 | 2.000 | 0.00 | 5.43 | 5.00 | 0.00 |
| 35.00 | 2.000 | 0.00 | 4.67 | 5.00 | 0.00 |

**Overall potential $I_L$ : 16.91**

$I_L$ = 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 & Lateral displ.acements estimation for saturated sands ::

| Depth (ft) | $(N_1)_{60cs}$ | $\gamma_{lim}$ (%) | $F_\alpha$ | $FS_{liq}$ | $\gamma_{max}$ (%) | $e_v$ (%) | dz (ft) | $S_{v-1D}$ (in) | LDI (ft) |
|---|---|---|---|---|---|---|---|---|---|
| 5.00 | 29 | 5.33 | -0.02 | 0.286 | 1.99 | 0.41 | 2.50 | 1.022 | 0.00 |
| 7.50 | 11 | 42.40 | 0.89 | 0.106 | 42.40 | 3.53 | 2.50 | 8.823 | 0.00 |
| 10.00 | 11 | 42.40 | 0.89 | 0.093 | 42.40 | 3.53 | 5.00 | 17.646 | 0.00 |
| 15.00 | 39 | 1.07 | -0.73 | 2.000 | 0.00 | 0.00 | 5.00 | 0.000 | 0.00 |
| 20.00 | 78 | 0.00 | -4.01 | 2.000 | 0.00 | 0.00 | 5.00 | 0.000 | 0.00 |
| 25.00 | 54 | 0.00 | -1.92 | 2.000 | 0.00 | 0.00 | 5.00 | 0.000 | 0.00 |
| 30.00 | 67 | 0.00 | -3.03 | 2.000 | 0.00 | 0.00 | 5.00 | 0.000 | 0.00 |
| 35.00 | 66 | 0.00 | -2.94 | 2.000 | 0.00 | 0.00 | 5.00 | 0.000 | 0.00 |

## :: Vertical & Lateral displ.acements estimation for saturated sands ::

| Depth (ft) | $(N_1)_{60cs}$ | $\gamma_{lim}$ (%) | $F_\alpha$ | $FS_{liq}$ | $\gamma_{max}$ (%) | $e_v$ (%) | dz (ft) | $S_{v-1D}$ (in) | LDI (ft) |
|---|---|---|---|---|---|---|---|---|---|

**Cumulative settlements: 27.491 0.00**

**Abbreviations**

| | |
|---|---|
| $\gamma_{lim}$: | Limiting shear strain (%) |
| $F_\alpha$/N: | Maximun shear strain factor |
| $\gamma_{max}$: | Maximum shear strain (%) |
| $e_v$:: | Post liquefaction volumetric strain (%) |
| $S_{v-1D}$: | Estimated vertical settlement (in) |
| LDI: | Estimated lateral displacement (ft) |

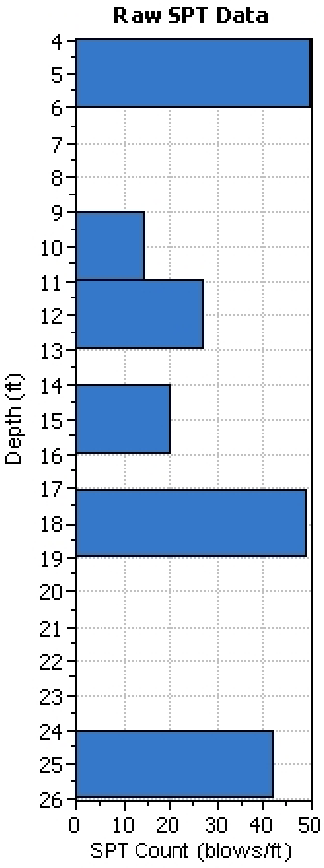
# SPT BASED LIQUEFACTION ANALYSIS REPORT

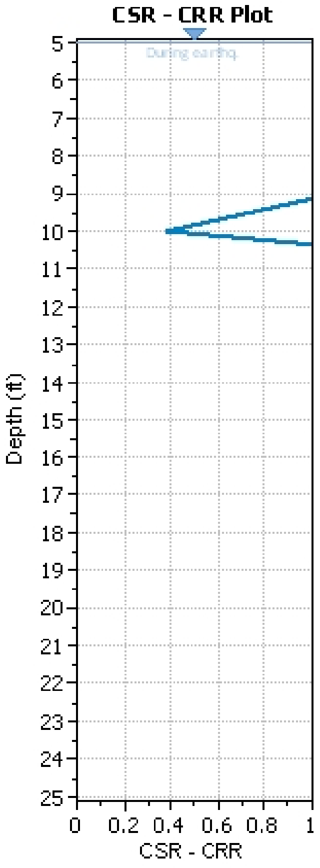
**Project title : DANCO-Taylor Way Geotechnical Investigation** **SPT Name: B-3**

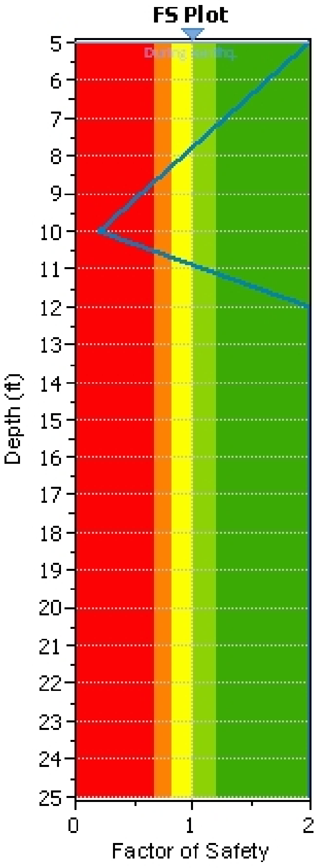
**Location : Blue Lake, Humboldt County, CA**

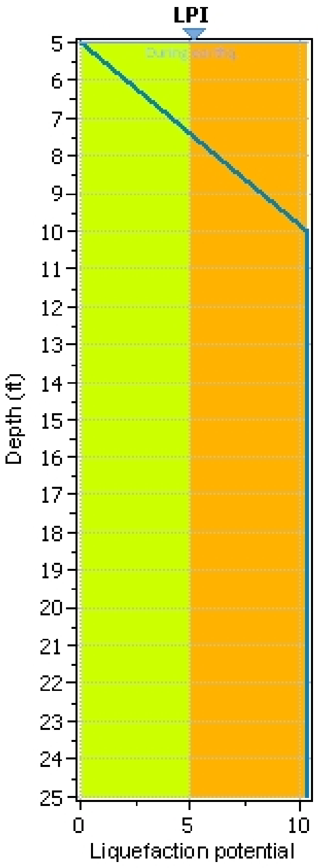
### :: Input parameters and analysis properties ::

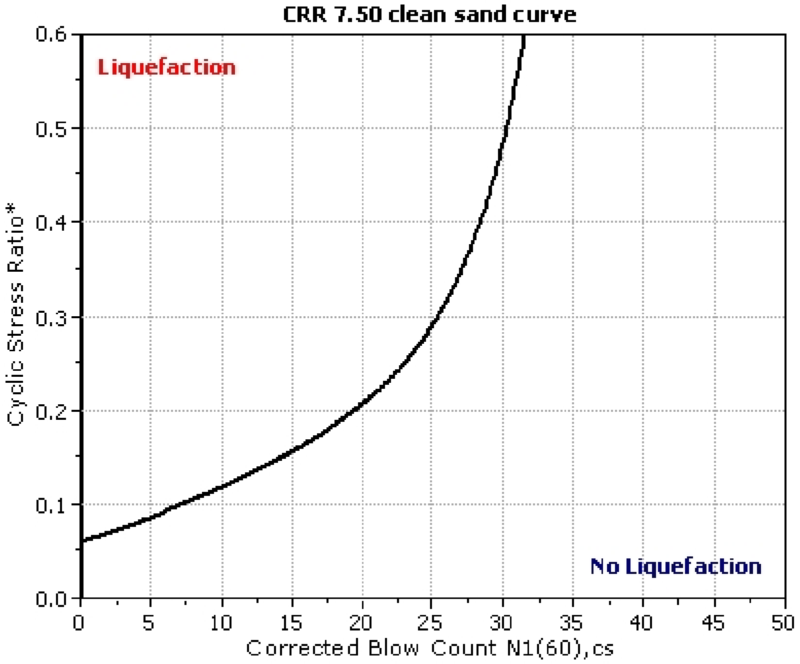
| | | | |
|---|---|---|---|
| Analysis method: | Boulanger & Idriss, 2014 | G.W.T. (in-situ): | 7.00 ft |
| Fines correction method: | Boulanger & Idriss, 2014 | G.W.T. (earthq.): | 5.00 ft |
| Sampling method: | Sampler wo liners | Earthquake magnitude $M_w$: | 9.10 ft |
| Borehole diameter: | 65mm to 115mm | Peak ground acceleration: | 1.49 g |
| Rod length: | 3.00 ft | Eq. external load: | 0.00 tsf |
| Hammer energy ratio: | 1.25 | | |











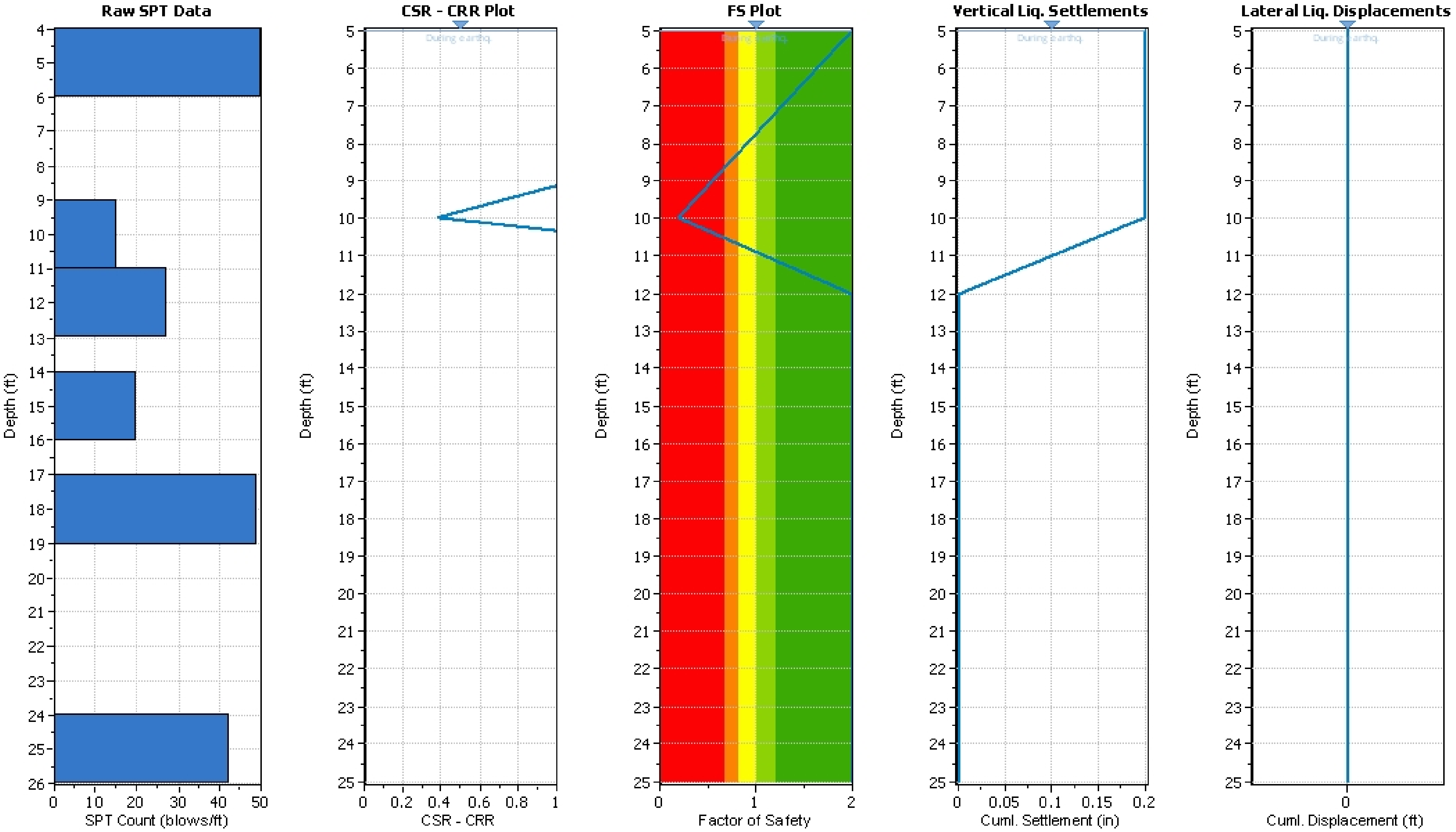
**F.S. color scheme**

- 🟥 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 | 50 | 8.00 | 120.00 | 5.00 | Yes |
| 10.00 | 15 | 29.00 | 120.00 | 2.00 | Yes |
| 12.00 | 27 | 29.00 | 120.00 | 3.00 | Yes |
| 15.00 | 20 | 36.00 | 120.00 | 3.00 | Yes |
| 18.00 | 49 | 36.00 | 120.00 | 7.00 | Yes |
| 25.00 | 42 | 36.00 | 120.00 | 5.00 | 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) | $\sigma_v$ (tsf) | $u_o$ (tsf) | $\sigma'_{vo}$ (tsf) | m | $C_N$ | $C_E$ | $C_B$ | $C_R$ | $C_S$ | $(N_1)_{60}$ | FC (%) | $\Delta(N_1)_{60}$ | $(N_1)_{60cs}$ | $CRR_{7.5}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 5.00 | 50 | 120.00 | 0.30 | 0.00 | 0.30 | 0.15 | 1.21 | 1.25 | 1.00 | 0.75 | 1.20 | 68 | 8.00 | 0.37 | 68 | 4.000 |
| 10.00 | 15 | 120.00 | 0.60 | 0.09 | 0.51 | 0.36 | 1.30 | 1.25 | 1.00 | 0.80 | 1.20 | 23 | 29.00 | 5.32 | 28 | 0.384 |
| 12.00 | 27 | 120.00 | 0.72 | 0.16 | 0.56 | 0.26 | 1.18 | 1.25 | 1.00 | 0.85 | 1.20 | 41 | 29.00 | 5.32 | 46 | 4.000 |
| 15.00 | 20 | 120.00 | 0.90 | 0.25 | 0.65 | 0.32 | 1.17 | 1.25 | 1.00 | 0.85 | 1.20 | 30 | 36.00 | 5.52 | 36 | 4.000 |
| 18.00 | 49 | 120.00 | 1.08 | 0.34 | 0.74 | 0.10 | 1.04 | 1.25 | 1.00 | 0.95 | 1.20 | 73 | 36.00 | 5.52 | 79 | 4.000 |
| 25.00 | 42 | 120.00 | 1.50 | 0.56 | 0.94 | 0.16 | 1.02 | 1.25 | 1.00 | 0.95 | 1.20 | 61 | 36.00 | 5.52 | 67 | 4.000 |

**Abbreviations**

| | |
|---|---|
| $\sigma_v$: | Total stress during SPT test (tsf) |
| $u_o$: | Water pore pressure during SPT test (tsf) |
| $\sigma'_{vo}$: | Effective overburden pressure during SPT test (tsf) |
| m: | Stress exponent normalization factor |
| $C_N$: | 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 |
| $\Delta(N_1)_{60}$ | Equivalent clean sand adjustment |
| $N_{1(60)cs}$: | Corected $N_{1(60)}$ value for fines content |
| $CRR_{7.5}$: | Cyclic 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) | $r_d$ | CSR | $MSF_{max}$ | $(N_1)_{60cs}$ | MSF | $CSR_{eq,M=7.5}$ | $K_{sigma}$ | $CSR^*$ | FS | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 5.00 | 120.00 | 0.30 | 0.00 | 0.30 | 1.01 | 0.973 | 2.20 | 68 | 0.48 | 2.046 | 1.10 | 1.860 | 2.000 | 🟢 |
| 10.00 | 120.00 | 0.60 | 0.16 | 0.44 | 1.01 | 1.317 | 1.88 | 28 | 0.62 | 2.140 | 1.10 | 1.945 | 0.197 | 🔴 |
| 12.00 | 120.00 | 0.72 | 0.22 | 0.50 | 1.01 | 1.400 | 2.20 | 46 | 0.48 | 2.942 | 1.10 | 2.675 | 2.000 | 🟢 |
| 15.00 | 120.00 | 0.90 | 0.31 | 0.59 | 1.01 | 1.494 | 2.20 | 36 | 0.48 | 3.140 | 1.10 | 2.855 | 2.000 | 🟢 |
| 18.00 | 120.00 | 1.08 | 0.41 | 0.67 | 1.01 | 1.565 | 2.20 | 79 | 0.48 | 3.289 | 1.10 | 2.990 | 2.000 | 🟢 |
| 25.00 | 120.00 | 1.50 | 0.62 | 0.88 | 1.01 | 1.676 | 2.20 | 67 | 0.48 | 3.524 | 1.06 | 3.338 | 2.000 | 🟢 |

## :: 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) | $r_d$ | CSR | $MSF_{max}$ | $(N_1)_{60cs}$ | 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 |
| CSR : | Cyclic Stress Ratio |
| 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 |
| FS: | Calculated factor of safety against soil liquefaction |

## :: Liquefaction potential according to Iwasaki ::

| Depth (ft) | FS | F | wz | Thickness (ft) | $I_L$ |
|---|---|---|---|---|---|
| 5.00 | 2.000 | 0.00 | 9.24 | 5.00 | 0.00 |
| 10.00 | 0.197 | 0.80 | 8.48 | 5.00 | 10.37 |
| 12.00 | 2.000 | 0.00 | 8.17 | 2.00 | 0.00 |
| 15.00 | 2.000 | 0.00 | 7.71 | 3.00 | 0.00 |
| 18.00 | 2.000 | 0.00 | 7.26 | 3.00 | 0.00 |
| 25.00 | 2.000 | 0.00 | 6.19 | 7.00 | 0.00 |

**Overall potential $I_L$ : 10.37**

$I_L$ = 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 & Lateral displ.acements estimation for saturated sands ::

| Depth (ft) | $(N_1)_{60cs}$ | $\gamma_{lim}$ (%) | $F_\alpha$ | $FS_{liq}$ | $\gamma_{max}$ (%) | $e_v$ (%) | dz (ft) | $S_{v-1D}$ (in) | LDI (ft) |
|---|---|---|---|---|---|---|---|---|---|
| 5.00 | 68 | 0.00 | -3.12 | 2.000 | 0.00 | 0.00 | 5.00 | 0.000 | 0.00 |
| 10.00 | 28 | 6.08 | 0.04 | 0.197 | 3.92 | 0.83 | 2.00 | 1.669 | 0.00 |
| 12.00 | 46 | 0.19 | -1.27 | 2.000 | 0.00 | 0.00 | 3.00 | 0.000 | 0.00 |
| 15.00 | 36 | 1.86 | -0.51 | 2.000 | 0.00 | 0.00 | 3.00 | 0.000 | 0.00 |
| 18.00 | 79 | 0.00 | -4.11 | 2.000 | 0.00 | 0.00 | 7.00 | 0.000 | 0.00 |
| 25.00 | 67 | 0.00 | -3.03 | 2.000 | 0.00 | 0.00 | 5.00 | 0.000 | 0.00 |

**Cumulative settlements: 1.669 0.00**

**Abbreviations**

| | |
|---|---|
| $\gamma_{lim}$: | Limiting shear strain (%) |
| $F_\alpha$/N: | Maximun shear strain factor |
| $\gamma_{max}$: | Maximum shear strain (%) |
| $e_v$:: | Post liquefaction volumetric strain (%) |
| $S_{v-1D}$: | Estimated vertical settlement (in) |
| LDI: | Estimated lateral displacement (ft) |

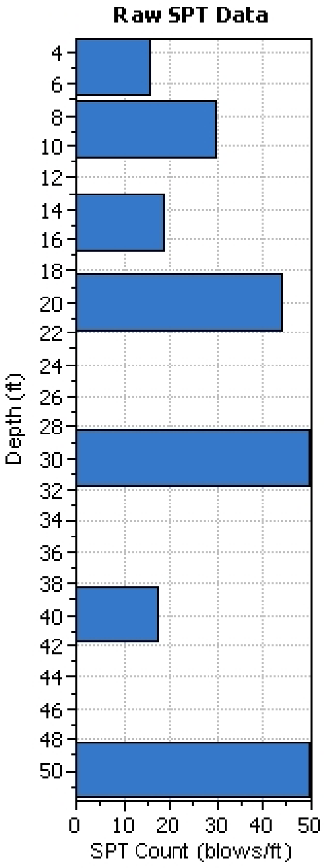
# SPT BASED LIQUEFACTION ANALYSIS REPORT

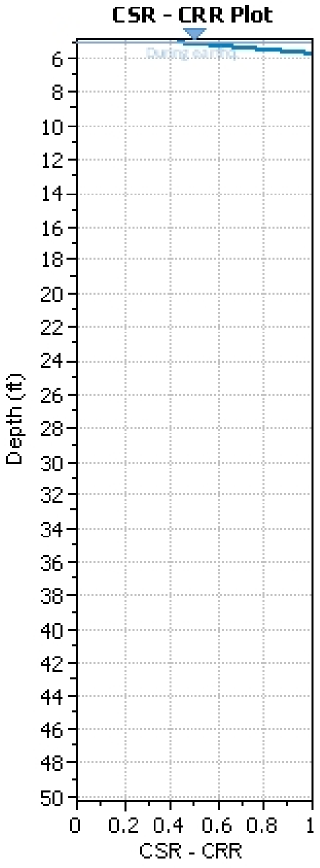
**Project title : DANCO-Taylor Way Geotechnical Investigation** **SPT Name: B-4**

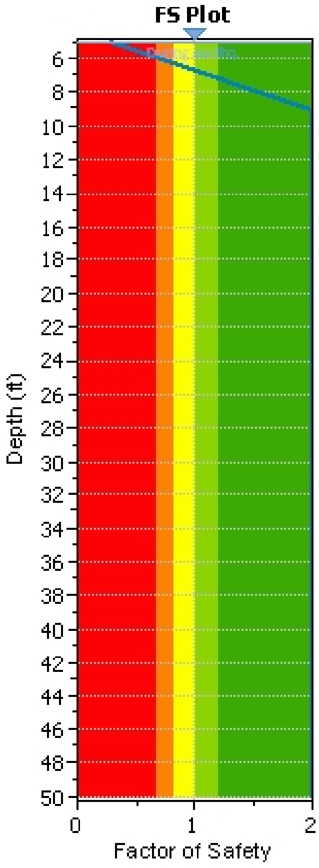
**Location : Blue Lake, Humboldt County, CA**

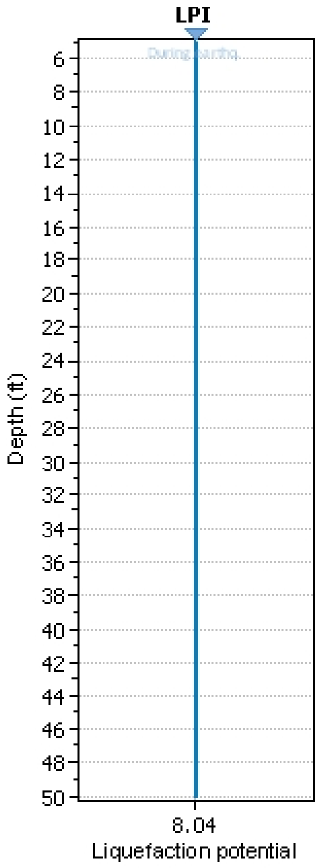
**:: Input parameters and analysis properties ::**

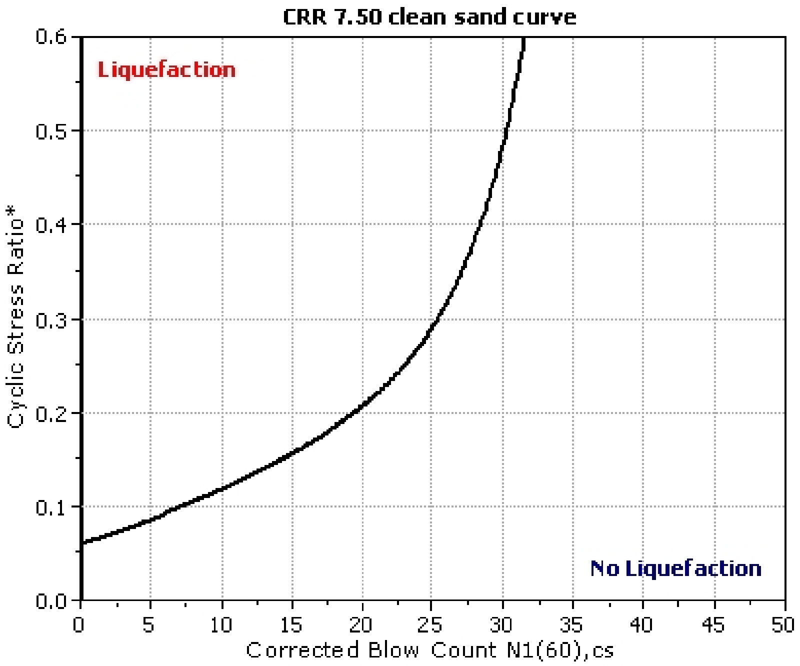
| | | | |
|---|---|---|---|
| Analysis method: | Boulanger & Idriss, 2014 | G.W.T. (in-situ): | 7.00 ft |
| Fines correction method: | Boulanger & Idriss, 2014 | G.W.T. (earthq.): | 5.00 ft |
| Sampling method: | Sampler wo liners | Earthquake magnitude $M_w$: | 9.10 ft |
| Borehole diameter: | 65mm to 115mm | Peak ground acceleration: | 1.49 g |
| Rod length: | 3.00 ft | Eq. external load: | 0.00 tsf |
| Hammer energy ratio: | 1.25 | | |











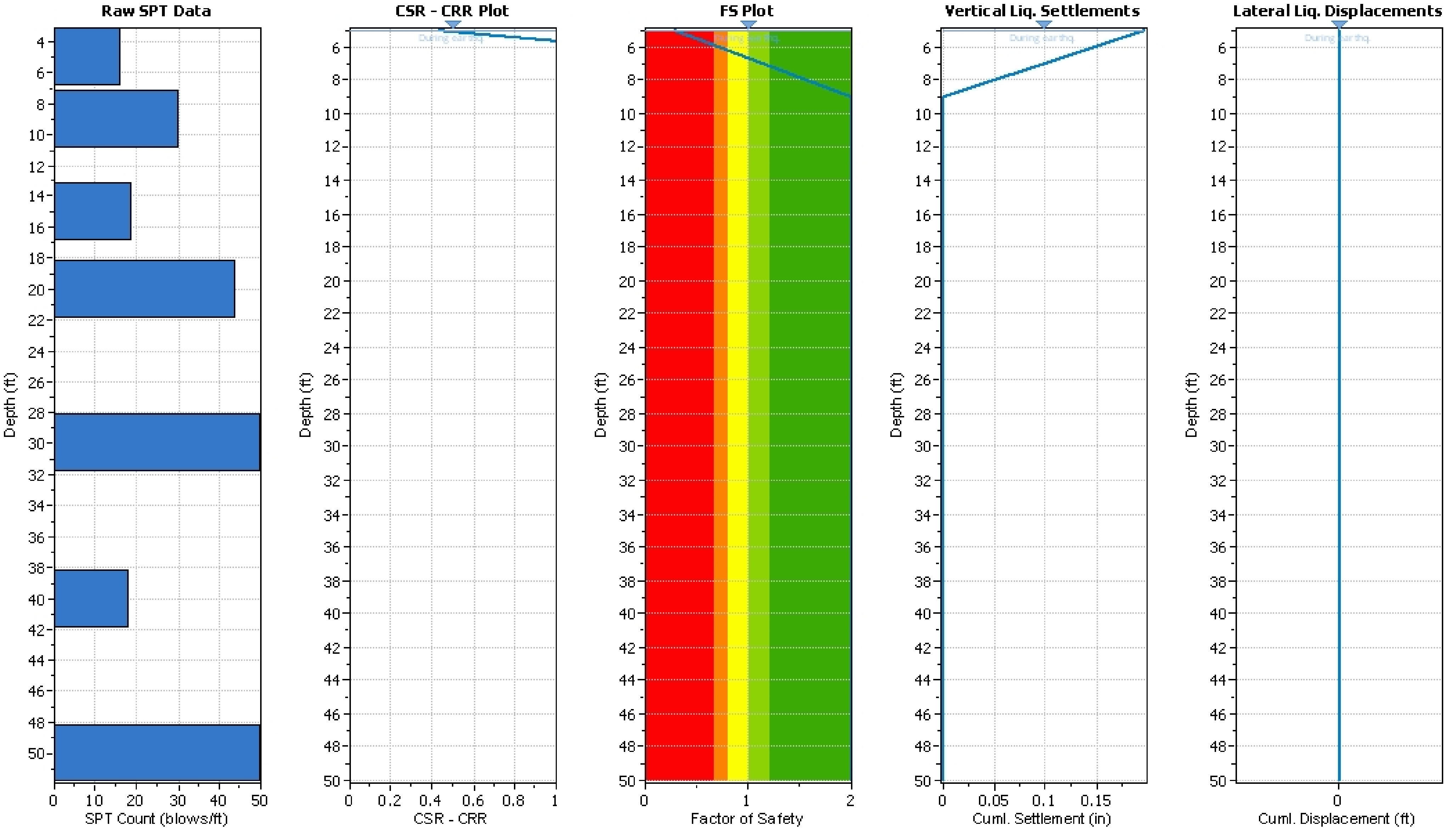
**F.S. color scheme**

- 🟥 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 | 16 | 7.00 | 120.00 | 4.00 | Yes |
| 9.00 | 30 | 7.00 | 129.00 | 6.00 | Yes |
| 15.00 | 19 | 15.00 | 129.00 | 5.00 | Yes |
| 20.00 | 44 | 15.00 | 129.00 | 10.00 | Yes |
| 30.00 | 50 | 15.00 | 129.00 | 10.00 | Yes |
| 40.00 | 18 | 75.00 | 129.00 | 10.00 | No |
| 50.00 | 50 | 26.00 | 129.00 | 5.00 | No |

**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) | $\sigma_v$ (tsf) | $u_o$ (tsf) | $\sigma'_{vo}$ (tsf) | m | $C_N$ | $C_E$ | $C_B$ | $C_R$ | $C_S$ | $(N_1)_{60}$ | FC (%) | $\Delta(N_1)_{60}$ | $(N_1)_{60cs}$ | $CRR_{7.5}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 5.00 | 16 | 120.00 | 0.30 | 0.00 | 0.30 | 0.37 | 1.60 | 1.25 | 1.00 | 0.75 | 1.20 | 29 | 7.00 | 0.14 | 29 | 0.429 |
| 9.00 | 30 | 129.00 | 0.56 | 0.06 | 0.50 | 0.27 | 1.23 | 1.25 | 1.00 | 0.80 | 1.20 | 44 | 7.00 | 0.14 | 44 | 4.000 |
| 15.00 | 19 | 129.00 | 0.94 | 0.25 | 0.70 | 0.35 | 1.16 | 1.25 | 1.00 | 0.85 | 1.20 | 28 | 15.00 | 3.26 | 31 | 4.000 |
| 20.00 | 44 | 129.00 | 1.27 | 0.41 | 0.86 | 0.15 | 1.03 | 1.25 | 1.00 | 0.95 | 1.20 | 65 | 15.00 | 3.26 | 68 | 4.000 |
| 30.00 | 50 | 129.00 | 1.91 | 0.72 | 1.19 | 0.11 | 0.99 | 1.25 | 1.00 | 1.00 | 1.20 | 74 | 15.00 | 3.26 | 77 | 4.000 |
| 40.00 | 18 | 129.00 | 2.56 | 1.03 | 1.53 | 0.38 | 0.87 | 1.25 | 1.00 | 1.00 | 1.20 | 24 | 75.00 | 5.56 | 30 | 4.000 |
| 50.00 | 50 | 129.00 | 3.20 | 1.34 | 1.86 | 0.12 | 0.93 | 1.25 | 1.00 | 1.00 | 1.20 | 70 | 26.00 | 5.15 | 75 | 4.000 |

**Abbreviations**

$\sigma_v$: Total stress during SPT test (tsf)
$u_o$: Water pore pressure during SPT test (tsf)
$\sigma'_{vo}$: Effective overburden pressure during SPT test (tsf)
m: Stress exponent normalization factor
$C_N$: 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
$\Delta(N_1)_{60}$ Equivalent clean sand adjustment
$N_{1(60)cs}$: Corected $N_{1(60)}$ value for fines content
$CRR_{7.5}$: Cyclic 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) | $r_d$ | CSR | $MSF_{max}$ | $(N_1)_{60cs}$ | MSF | $CSR_{eq,M=7.5}$ | $K_{sigma}$ | $CSR^*$ | FS | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 5.00 | 120.00 | 0.30 | 0.00 | 0.30 | 1.01 | 0.973 | 1.94 | 29 | 0.59 | 1.649 | 1.10 | 1.499 | 0.286 | 🔴 |
| 9.00 | 129.00 | 0.56 | 0.12 | 0.43 | 1.01 | 1.255 | 2.20 | 44 | 0.48 | 2.638 | 1.10 | 2.398 | 2.000 | 🟢 |
| 15.00 | 129.00 | 0.94 | 0.31 | 0.63 | 1.01 | 1.457 | 2.06 | 31 | 0.54 | 2.711 | 1.10 | 2.464 | 2.000 | 🟢 |
| 20.00 | 129.00 | 1.27 | 0.47 | 0.80 | 1.01 | 1.550 | 2.20 | 68 | 0.48 | 3.258 | 1.08 | 3.009 | 2.000 | 🟢 |
| 30.00 | 129.00 | 1.91 | 0.78 | 1.13 | 1.01 | 1.655 | 2.20 | 77 | 0.48 | 3.479 | 0.98 | 3.550 | 2.000 | 🟢 |
| 40.00 | 129.00 | 2.56 | 1.09 | 1.47 | 1.01 | 1.711 | 2.00 | 30 | 0.56 | 3.032 | 0.93 | 3.246 | 2.000 | 🟢 |
| 50.00 | 129.00 | 3.20 | 1.40 | 1.80 | 1.01 | 1.740 | 2.20 | 75 | 0.48 | 3.658 | 0.84 | 4.337 | 2.000 | 🟢 |

### :: 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) | $r_d$ | CSR | $MSF_{max}$ | $(N_1)_{60cs}$ | 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
CSR : Cyclic Stress Ratio
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
FS: Calculated factor of safety against soil liquefaction

### :: Liquefaction potential according to Iwasaki ::

| Depth (ft) | FS | F | wz | Thickness (ft) | $I_L$ |
|---|---|---|---|---|---|
| 5.00 | 0.286 | 0.71 | 9.24 | 4.00 | 8.04 |
| 9.00 | 2.000 | 0.00 | 8.63 | 4.00 | 0.00 |
| 15.00 | 2.000 | 0.00 | 7.71 | 6.00 | 0.00 |
| 20.00 | 2.000 | 0.00 | 6.95 | 5.00 | 0.00 |
| 30.00 | 2.000 | 0.00 | 5.43 | 10.00 | 0.00 |
| 40.00 | 2.000 | 0.00 | 3.90 | 10.00 | 0.00 |
| 50.00 | 2.000 | 0.00 | 2.38 | 10.00 | 0.00 |

**Overall potential $I_L$ : 8.04**

$I_L$ = 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 & Lateral displ.acements estimation for saturated sands ::

| Depth (ft) | $(N_1)_{60cs}$ | $\gamma_{lim}$ (%) | $F_\alpha$ | $FS_{liq}$ | $\gamma_{max}$ (%) | $e_v$ (%) | dz (ft) | $S_{v-1D}$ (in) | LDI (ft) |
|---|---|---|---|---|---|---|---|---|---|
| 5.00 | 29 | 5.33 | -0.02 | 0.286 | 1.99 | 0.41 | 4.00 | 1.636 | 0.00 |
| 9.00 | 44 | 0.34 | -1.11 | 2.000 | 0.00 | 0.00 | 6.00 | 0.000 | 0.00 |
| 15.00 | 31 | 4.04 | -0.16 | 2.000 | 0.00 | 0.00 | 5.00 | 0.000 | 0.00 |
| 20.00 | 68 | 0.00 | -3.12 | 2.000 | 0.00 | 0.00 | 10.00 | 0.000 | 0.00 |
| 30.00 | 77 | 0.00 | -3.92 | 2.000 | 0.00 | 0.00 | 10.00 | 0.000 | 0.00 |
| 40.00 | 30 | 0.00 | 0.00 | 2.000 | 0.00 | 0.00 | 10.00 | 0.000 | 0.00 |
| 50.00 | 75 | 0.00 | 0.00 | 2.000 | 0.00 | 0.00 | 5.00 | 0.000 | 0.00 |

**Cumulative settlements: 1.636 0.00**

**Abbreviations**

$\gamma_{lim}$: Limiting shear strain (%)
$F_\alpha$/N: Maximun shear strain factor
$\gamma_{max}$: Maximum shear strain (%)
$e_v$:: Post liquefaction volumetric strain (%)
$S_{v-1D}$: Estimated vertical settlement (in)
LDI: Estimated lateral displacement (ft)

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