GEOTECHNICAL DESIGN REPORT

Pioneer Meadows Fire Station

Sparks, Washoe County, NV

Submitted To:

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CITY OF SPARKS

431 Prater Way

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Project Number

8523020

Date

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EXECUTIVE SUMMARY

The project is in the Wingfield Springs area in Sparks, Nevada, adjacent to the proposed Pioneer Meadows Business Park. The project consists of constructing a City of Sparks' fire station; the structure is anticipated to be one to two stories in height, steel framed and masonry construction with concrete slab-on-grade flooring. Standard spread footings will support the overall structure. Street improvements, underground services, and project access for the mass graded pad and street improvements have been designed and constructed to City of Sparks' standards.

The site has been mass graded with approximately 5-feet of structural fill as part of the grading for the Pioneer Meadows Business Park. The underlying soils consist of Quaternary-age basin fill deposits consisting of clayey and silty sandy soils. Soils encountered during the performance of the geotechnical investigation typically consisted of nonplastic well graded sands with silt and medium plasticity silty sands in the upper 15-feet +- of the soil profile. Deeper soils consist of complexly blended and intercalated mixtures of sand, silt, and clay. Free water was encountered at an average depth of 10 ½-feet, or approximate elevation of 4467-feet (NAVD 88).

Based on the soil conditions encountered, the fire station may be supported on standard spread foundations. Structural pavement sections for the fire truck driveway and loading area have been developed based on AASHTO Guide for Design of Pavement Structures (1993) and Caltrans' Highway Design Manual for reinforced and unreinforced concrete pavements.

Per ACI 318-19, corrosion testing yielded sulfate results in the S0 negligible range and therefore Type II cement is suitable for use.

1. INTRODUCTION

Presented herein are the results of Wood Rodgers' geotechnical exploration, laboratory testing, and associated geotechnical design recommendations for the proposed Pioneer Meadows Fire Station in Sparks, Nevada. The assessments and recommendations presented in this geotechnical report have been determined, in part, around the surface and subsurface conditions identified by our exploration program which was developed to be consistent with locally accepted industry practices regarding exploratory means and methods for geotechnical investigations of similar projects. The proposed structural elements, topography, grading design, soils, and geology are all unique; therefore, the engineering judgment employed by those in responsible charge of geotechnical design is in general conformance with the accepted standards of care for engineering analyses as defined by the Nevada State Board of Engineers and Land Surveyors.

This report has been prepared in consideration of the applicable provisions of the International Building Code (IBC, 2018), ASCE 7, and the amendments and modifications adopted by the City of Sparks. These documents establish the minimum requirements to safeguard the public health, safety and general welfare of the occupants as well as the minimum level of structural integrity, life safety, and fire safety for inhabitants of new and existing structures. Geotechnical considerations for public improvements have been formulated around the requirements of the City of Sparks Public Works Design Manual and the Standard Specifications for Public Works Construction. Performance standards around which our primary recommendations have been framed are based upon the requirements of the referenced documents. Any expectations of performance inconsistent with, outside the purview of, or exceeding the requirements of the referenced documents are subjective and, therefore, a function of materials, design, workmanship, and ownership. Unless these expectations of performance are specifically stipulated or quantified herein, they are considered in excess to the scope and design standards of this report.

The objectives of this study were to:

1. Explore, test, and assess general soil, geology, and ground water conditions pertaining to design and construction considerations for the proposed development as required by the IBC.
   2. Provide recommendations associated with the design and construction of the project, as related to the identified geotechnical conditions and the stipulated design levels and performance standards established herein,

2. SCOPE OF SERVICES

As indicated in our proposal, upon completion of our field and office studies, a geotechnical investigation report consistent with the requirements of the 2018 International Building Code (IBC) will be completed for the project and will present the following:

- Description of the project site with the approximate locations of our explorations, shown on a Site Plan.
- Descriptive logs of the explorations performed for this study.
- General summary of the site soils and geology.
- Summary of surface and ground water conditions encountered.
- Summary of seismic hazards including site seismicity, potential for surface fault rupture, landslides, and liquefaction susceptibility.
- Site preparation and grading recommendations based on the results of our field exploration and laboratory testing for standard spread foundations.
- Allowable bearing pressures, appropriate footing depths and widths, and anticipated settlement.
- Lateral earth pressures and design parameters, as applicable for retaining walls and planned structures.
- Concrete and concrete slab-on-grade support options.
- Special concrete considerations due to corrosivity and potential environmental exposure.
- Drainage considerations that may affect foundation and concrete slab-on-grade performance.
- Structural pavement sections.

Our study included field exploration, laboratory testing, and engineering analyses to identify the physical and mechanical properties of the various on-site materials. Results of our field exploration and testing programs are included in this report. In consideration of the stated design levels and performance standards, these results form the basis for our conclusions and recommendations.

3. PROJECT CHARACTERIZATION

3.1. Project Description

- The project is in the Wingfield Springs area of Sparks, Nevada. (Appendix A, Figure 1 – Vicinity Map)
- The project consists of constructing a fire station at the proposed Pioneer Meadows Business Park, just north of the Wingfield Hills Road intersection with Vista Boulevard.
- The fire station structure is anticipated to be one to two stories in height, steel framed and masonry construction with concrete slab-on-grade flooring.
- Heavy vehicle loads are anticipated for the fire truck access zones and driveways.
- Street improvements, underground services, and project access for the mass graded pad and street improvements have been designed to be consistent with the Pioneer Meadows' handbook and City of Spark's standards.
- Overall project layout is presented on Figure 2 – Improvement Map (Appendix A).

3.2. Site Conditions

- The overall site encompasses an area of approximately 2 acres.
- The development area is located at a central latitude and longitude of 39.6194°N and 119.7076°W, respectively.
- The proposed Pioneer Meadows Business Park borders the site to the north, south, and west. Single family residential developments border the property to the east.
- The site has been previously mass graded and therefore cuts and fills are anticipated to be limited (i.e., less than 3-feet).
- Except where recently graded, vegetation across the site typically consists of large brush and grasses.

3.3. Exploration

3.3.1. Drilling and Sampling

- The project was explored in February 2023 by advancing two geotechnical borings, referred to as B-2 and B-3, with a CME 55 drill rig, flight auger and mud rotary techniques (Appendix A, Figure 3 – Site Map & Approximate Exploration Locations). Boring B-1 was abandoned due to access constraints.

- Boring B-3a, from the geotechnical investigation completed at the site for the proposed Pioneer Meadows Business Park investigation, was utilized in development of geotechnical design considerations and is included in Appendix B (B-1c).

- Maximum depth of boring advance extended to 51 ½-feet.

- Standard Penetration Test (SPT) sampling consisted of driving a spilt spoon sampler into the ground and measuring the number of blows to advance the sampler a vertical distance of 18-inches. A drop weight system, utilizing a 140-pound hammer falling 30-inches, is used to drive three successive 6-inch increments. The first increment is considered "seating" the sampler, and the subsequent number of blows recorded to advance the second and third increments are the N-value blow counts or SPT-resistance. Uncorrected SPT blow counts are presented on B-1 in Appendix B of this report.

- In addition to SPT sampling, soils were also sampled in-place with a 3-inch outer diameter (OD), 2 ½-inch inner diameter (ID) split-spoon sampler driven by a standard 140-pound drive hammer with a 30-inch stroke (California Modified Sampler, CMS). Thin-walled brass liners, 2 ½ -inch OD by 2.42-inch ID, were used within the split-spoon sampler to collect disturbed samples. The reported blow counts were corrected for sampler size to roughly correlate (Caltrans, 2021) to N-value (Standard Penetration Test (SPT), ASTM D1586).

- Wood Rodgers' personnel examined and classified soils in the field in general accordance with ASTM D2488 (Description and Identification of Soils).

- As reported on the Logs of Borings (B-1a through B-1c in Appendix B of this report) blow counts, both reported SPT and CMS, have not been corrected for overburden, hole diameter, or hammer energy efficiency.

3.3.2. Geophysical Methods

- Seismic refraction methods (ReMi ™) were performed to measure shear wave velocity and to establish $V_{S\ 100}$.

- Shear wave velocity measurements have been relied upon to aid in the determination of an appropriate Site Class (ASCE 7). B-3 (Appendix B) presents the shear wave geophysical profile.

3.4. Sampling, Classification and Reporting

- Wood Rodgers' personnel examined and classified soils in the field in general accordance with ASTM D2488 (Description and Identification of Soils).

- During exploration, representative bulk samples were placed in sealed plastic bags, brass tubes, and buckets and returned to our Reno, Nevada laboratory for testing.

- Additional soil classifications, as well as verification of the field classifications, were performed in accordance with ASTM D2487 (Unified Soil Classification System [USCS]) upon completion of laboratory testing as described in the Laboratory Testing section.
- Logs of the explorations are presented as B-1a through B-1c in Appendix B.
- The USCS explanatory chart of soil unit symbols and related descriptions has been included as B-2a - Unified Soil Classification and Key to Soil Descriptions (Appendix B).
- It should be noted that ASTM D2488 and ASTM D2487 are specific to geotechnical engineering characterization of soils and do not meet the scientifically based quantitative sampling protocols necessary for consideration in bidding.

3.5. Laboratory Testing

- Soil testing performed in the Wood Rodgers' laboratory was conducted in general accordance with the standards and methods described in Volume 4.08 (Soil and Rock; Dimension Stone; Geosynthetics) of the ASTM Standards.
- Samples of significant soil types were analyzed to determine in-situ moisture contents (ASTM D2216), grain size distributions (ASTM D6913), and plasticity indices (ASTM D4318).
- Test results were used to classify the soils according the USCS (ASTM D2487) and to verify the field logs which were then updated.

Results of the soil testing is presented in Appendix C on C-1a through C-1b. Table 1 also presents a summary of the test data.

Table 1 – Summary of Test Data

Test Hole	Depth (Ft.)	Moisture (%)	%Gravel (+ #4)*	% Sand (#4-#200)	%Fines (-#200)	Liquid Limit	Plastic Index	USCS
ASTM Standard		D2216	D6913			D4318		D2487
B-2	2.5	17.2	--	--	28.1	34	18	SC[2]
B-2	7.5	6.0	5.3	84.6	10.1	NP	NP	SW-SM
B-2	13.5	41.5	0.7	51.8	47.5	38	12	SM
B-2	21	16.6	0.0	76.6	23.4	--	--	SM[2]

Test Hole	Depth (Ft.)	Moisture (%)	%Gravel (+ #4)*	% Sand (#4-#200)	%Fines (-#200)	Liquid Limit	Plastic Index	USCS
ASTM Standard		D2216	D6913			D4318		D2487
B-3	10	26.8	--	--	26.8	27	5	SM[2]

[1] Since ASTM D2487 is limited by a maximum particle size of 3", the gradation test data presented is based on a maximum particle size of 3". Larger particles (i.e., 8 to 12" in diameter) if observed in our test holes would be documented on the logs and should be anticipated as part of grading.
[2] Samples classified via ASTM D2488 – Visual – Manual procedure.

- Chemical testing was performed to indicate the potential for corrosion to concrete and steel elements which is presented on C-2.
- As a courtesy, resistivity, pH, chlorides, oxidation-reduction potential, sulfides and moisture were also tested to aid others in the assessment of potential corrosivity to ductile iron pipe and/or steel reinforcement; refer to Appendix C, C-2 for test methods and results.

Wood Rodgers, Inc. is not a corrosion engineering firm. Therefore, a corrosion engineer or structural engineer knowledgeable in the project steel specifications should be consulted for final assessments of corrosion potential at the site.

3.6. Geologic and General Soil and Groundwater Conditions

- Based on the National Geologic Map Database (NGMDB, 2011), the site is mapped in an area of Holocene (Qby) and Late Pleistocene (Qbi) basin fill deposits described by Bell and Bonham as generally fine-grained deposits (silty, clayey sand and sandy silt and clay) derived from volcanic and granitic alluvial-fan deposits to the east and west.
- A 5-feet thick ± fill layer consisting of clayey sand was encountered across the site in borings B-2, B-3 and B-3a.
- The soils encountered in our explorations typically consisted of nonplastic well graded sands with silt and medium plasticity silty sands with an intermittent low plasticity silt layer to the depths explored.
- Subsurface conditions encountered are consistent with the geologic map.
- Groundwater was encountered at an average depth of 10 ½-feet, or approximate elevation of 4467-feet (NAVD 88).

4. SEISMIC HAZARDS

The International Building Code (IBC) requires that structures assigned to Seismic Design Category D, E, or F be evaluated for the following potential geologic and seismic hazards: surface rupture or displacement due to faulting, slope instability, liquefaction including the potential for seismically induced spreading or lateral flow, and slope instability. The following sections present our discussion and assessments of the stipulated hazards. Discussions regarding total and differential settlement are incorporated with the foundation design considerations.

4.1. Surface Rupture

4.1.1. Evaluation Guidelines

In 1998, the Nevada Earthquake Safety Council formulated guidelines for evaluating potential surface rupture due to faulting. The intent of the guidelines is to provide *a standardized minimum level of investigation for fault rupture in Nevada*; these guidelines have been adopted with the 2018 Northern Nevada Amendments of the IBC. Specifically, the guidelines state that investigation of sites for potential surface rupture or hazards shall be included in all geotechnical investigations. Further, if any Quaternary age surface rupture is mapped or otherwise interpreted to be present on the site, the feature is to be investigated further.

In addition to establishing the minimum level of investigation for fault rupture, the guidelines also offer recommendations for dealing with or mitigating identified hazards, including:

- Holocene active faults (evidence of movement within the past 10,000 years) shall be set-back a minimum distance of 50-feet for occupied structures.
- Late Quaternary (evidence of movement within the past 130,000 years) faults shall not be spanned by any critical facilities (hospitals, schools, fire stations, etc.). The facility under investigation does not meet the requisite requirements to be considered critical.

These guidelines allow for set-back distances to be adjusted by the competent professional. No additional constraints regarding fault-structure location are presented in the guidelines.

4.1.2. Investigation

- The United States Geological Survey's (USGS) interactive fault map was accessed to determine the presence of any mapped features transecting the property. No faults have been mapped crossing, intersecting, or trending toward the property. The closest mapped Quaternary fault is approximately 1-mile to the west of the property. This fault has been identified as a segment of the Spanish Springs Valley fault, aged *latest Quaternary active (< 15,000 years) regarding recency of movement.* Therefore, this structure is sufficiently distant and of an age that offsets or additional considerations are not recommended or presented by the Guidelines; surface rupture due to the identified structure is considered unlikely.

4.2. Soil Profile Type Amplification Factors

- Seismic design values were determined based on a representative latitude and longitude of 39.6194°N and 119.7076°W, respectively.
- Site Class D has been assigned to the project based on our measurement of shear wave velocity from the ReMi geophysical survey at the site (B-3, S-Wave ReMi Results, Appendix B).
- In accordance with ASCE 7-16 (Table 20.3-1) and the Northern Nevada Amendments of the 2018 IBC, Seismic Risk Category IV has been evaluated for soil profile type amplification factors. (Appendix D)
- Per ASCE 7-16, the site's modified Peak Ground Acceleration (PGA_M) to be used for engineering analyses is equal to 0.55g.

4.3. Liquefaction

Liquefaction is a loss of soil shear strength that can occur during a seismic event as excessive pore water pressure between the soil grains is induced by cyclic shear stresses. This phenomenon is limited to poorly consolidated (Standard Penetration Test less than 30, overburden stress corrected shear wave velocity less than 700 fps) clean to silty sand/sandy silt lying below the ground water table (typically less than 50 feet deep). In addition, we are using AASHTO's recommendation of Bray and Sancio (2006) criteria for assessing liquefaction susceptibility in clays and silts which suggests that a soil with a plasticity index less than 12 and a water content to liquid limit ratio greater than 0.85 will be susceptible to liquefaction.

- A liquefaction analysis has been performed as a part of this study by advancing boring B-2 to a depth of 51 ½-feet below the existing ground surface.
- A maximum magnitude of 6.52 from the Spanish Springs Valley Fault Zone was estimated for the project area using the USGS Unified Hazard Tool (USGS, 2022) and the appropriate parameters identified during this investigation.
- ASCE 7-16 indicates the modified site peak ground acceleration is 0.55 g which is the design value around which liquefaction was examined.
- Following the NCEER 1998 method, GeoLogismiki's LiqSVs software was used in our liquefaction analysis by incorporating the SPT data and SPT correlated CMS data obtained during the geotechnical investigation.
- Based on our analyses, the potential for liquefaction is discontinuous, intermittent, and limited in magnitude (i.e., ≤ ¼ – inch), and below the level for which mitigation would be required and below the threshold for which overall site behavior would be impacted. Therefore, Site Class F was not assigned to the project.

- The SPT blow count-based liquefaction analysis report is presented in Appendix E of this report.

4.3.1. Lateral Spreading

- Liquefaction induced horizontal ground displacement from lateral spreading or flow failure would be considered negligible due to site and surrounding topography and the discontinuous and isolated zones of low susceptibility liquefaction soils.

4.4. Slope Instability

- When evaluating for slope instability, the terms incidence and susceptibility are used. Susceptibility is the likelihood a landslide would occur based on local terrain. Incidence reflects the number of known landslides, irrespective of the age or climate at the time they occurred.

- MyHAZARDS – Nevada places the property in an area of low landslide incidence, low susceptibility. The classification of low indicates less than 1.5-percent of the land area has been involved in landsliding and therefore the potential for slope instability at the project site is considered very low.

4.5. Seismic Compression

- Seismic compression is an accrual of volumetric strains during seismic events in unsaturated soil and is typically confined to poorly compacted engineered fills and Holocene soils. Therefore, the settlement potential due to seismic compression is considered negligible.

- Significant slopes and deep, loose engineering fills are not anticipated. Therefore, the potential for significant settlement due to seismic compression is considered limited.

5. DISCUSSION AND RECOMMENDATIONS

5.1. General Information

- *Conformance with the Geotechnical Report:* The recommendations provided herein, particularly under Site Preparation, Grading and Filling, Foundations, Site Drainage, and Construction Observations and Testing Services are intended to reduce risks of structural distress related to consolidation or expansion of native soils and/or structural fills. These recommendations, along with proper design and construction of the planned structure(s) and associated improvements, work together as a system to improve overall performance. If any aspect of this system is ignored or poorly implemented, the performance of the project will suffer.

- *Clarification of Structural Areas:* Structural areas referred to in this report include all areas of buildings, concrete slabs, asphalt pavements, as well as pads for any minor structures or retaining walls. Areas which extend behind or under rockery or other retaining structures are considered structural zones. In addition, the structural zone shall be considered to extend at a 1:1 (H:V) slope out from the edge of the structural footprint. All compaction requirements presented in this report are relative to ASTM D 1557 (ASTM D Volume 4, 2022).

- *Hazardous Materials:* Any evaluation of the site for the presence of surface or subsurface hazardous substances is beyond the scope of this study. When suspected hazardous substances are encountered during routine geotechnical investigations, they are noted in the exploration logs and reported to the client. No such substances were identified during our exploration.

- *SWPPP Requirements:* The site-specific Stormwater Pollution Prevention Plan (SWPPP), as required by the State of Nevada, will be the responsibility of the general contractor and/or owner. Recommendations presented herein regarding moisture conditioning are for the benefit of creating a targeted fill behavior. Moisture conditioning recommendations are not intended to direct the contractor in their means and methods for dust and SWPPP control.

- *Public Improvements:* Recommendations for paved improvements in right-of-way areas will be consistent with City of Sparks' standards. On-site parking and driveway recommendations are in general conformance with AASHTO's Low Volume Road design protocols, the Portland Cement Association (PCA), the American Concrete Institute (ACI) recommendations, and Jointed Plain Concrete Pavement considerations offered by Caltrans'. Underground utilities will be provided by a variety of public and private companies; trenching and backfill recommendations presented herein are generally consistent with OSHA and City of Sparks' requirements, respectively.

5.2. Earthwork

5.2.1. Clarification of Geomaterials

The following definitions characterize terms utilized in this report:

- Fine-grained soil possesses more than 40 percent by weight passing the number 200 sieve and exhibits a plasticity index lower than 15.
- Clay soil possesses more than 30 percent passing the number 200 sieve and exhibits a plasticity index greater than 15.
- Granular soil does not meet the above criteria and has a maximum particle size less than 6-inches.

It should be noted these definitions have been formulated around anticipated soil behavior and may not coincide with classifications provided by the Unified Soil Classification System.

5.2.2. Clearing and Grubbing

- All vegetation and topsoil are to be cleared and grubbed from structural areas. A limited stripping depth is anticipated.
- Vegetation and organic debris should be disposed of offsite.

5.2.3. Existing Fills

- Approximately 5-feet of fill was encountered across the site.
- To document construction observation and field density testing during mass grading of the site, a mass grading certification was issued by Wood Rodgers in February 2012 (Appendix F).

5.2.4. Subgrade Preparation

- All subgrade soils shall be scarified for a minimum depth of 12-inches, moisture conditioned to within 3-percent of optimum and compacted to not less than 90-percent of the soil's maximum dry density (ASTM D1557) prior to placing fill or constructing improvements.
- Subgrade soils exposed in the bottom of footing excavations shall be moisture conditioned to within 3-percent of optimum and compacted to not less than 95-percent of the soil's maximum dry density.
- In all cases, the final subgrade shall be smooth, firm, and relatively unyielding as determined by the testing agencies' qualified representative.

5.2.5. Subgrade Stabilization

- Site soils may tend to pump and or destabilize when moisture content exceeds optimum.
- Care should be taken during grading to assure irrigation water, precipitation, or construction activities do not lead to an increase in or ponding of water on exposed grade.
- Pumping soils may be scarified and allowed to dry or removed and replaced with a layer of clean, angular, 12-inch minus rock fill or stabilized with a geogrid.

- The size of the rock could vary depending on the soil's consistency and depth of soft, saturated soils.
- Typically, a stabilization depth of 12 to 18-inches is adequate to develop a firm and relatively unyielding subgrade, but variations may exist.
- The rock zone should be separated from the adjacent soils by encasing the rock in a geomembrane such as Mirafi 180N installed per the manufacturer's instructions.
- As an alternative, the use of a stabilizing geogrid (such as Tensar TX160), complemented by an aggregate layer to bridge unstable and/or pumping subgrade, may be used to stabilize subgrade.
- Subgrade stabilization is a trial-and-error process, and it is recommended that a test section of suitable depth and length be conducted.
- The contractor should propose a stabilization protocol that is consistent with their readily available means and methods, and this proposal presented for review, by the owner, the general contractor, and grading inspector.
- Subgrade stabilization is considered adequate if the subgrade is firm and relatively unyielding (as approved by the engineer) when proof-rolled with a fully loaded water truck.
- Subgrade stabilization may not be required for walkways or private improvements subject solely to foot traffic providing the required compaction levels are achieved.

5.2.6. Site Grading

- Verification testing of onsite soils to be qualified as structural fill shall be in accordance with ASTM D75, ASTM D6913, and ASTM D4318 and shall meet the requirements of Table 2.
- Import structural fill if required, shall meet the requirements of Table 2 and shall be obtained from a commercial source.
- Prior to importing material to the site, a submittal verifying import quality shall be received and approved by Wood Rodgers.

Table 2 - Guideline Specification for Import Structural Fill

Sieve Size (ASTM D6913)	Percent by Weight Passing Sieve	
6 Inch	100	
4 Inch	90 - 100	
¾ Inch[2]	50 - 100	
No. 200	5 - 20	5 - 50
Maximum Liquid Limit (ASTM D4318)[1]	40	40

Table 2 - Guideline Specification for Import Structural Fill

Sieve Size (ASTM D6913)	Percent by Weight Passing Sieve	
Maximum Plasticity Index (ASTM D4318)[1]	20	12
Soluble Sulfate Level (ACI 318, Table 4.3.1)	Negligible	
R-Value (ASTM D2844)[2]	30 Min.	

[1] Dry Method

[2] Within parking and drive areas.

- Recycled asphalt concrete or recycled concrete from the razed structures should meet the requirements of Table 200.01.04-I of the Standard Specification for Public Works Construction (SSPWC, 2016).
- Structural fill shall be moisture conditioned to within 3-percent of optimum, placed in 12-inch maximum loose lifts, and compacted to not less than 90-percent of the soil's maximum dry density (ASTM D1557).

5.2.7. Trenching and Excavations

- OSHA - Regulations amended in Part 1926, Volume 54, Number 209 of the Federal Register (Table B-1, October 31, 1989) require that the temporary sidewall slopes be limited to maintain trench stability. Minimum sidewall slopes and acceptable trench configurations are also presented in the referenced register.
- Groundwater and seepage were encountered as shallow as 10 ½-feet. Groundwater and seepage can lead to instability within trench sidewalls and excavations and, therefore, should be appropriately considered by the contractor's person of knowledge.
- Should any large precipitation events be forecast, it is imperative that open excavations be protected from flooding. Covering trenches with plates or tarps, constructing berms around the excavations, daylighting trenches to drain or temporary backfilling should be considered by the contractor to prevent flooding damage and erosion in general.
- Based on the results of our exploration program, it is our opinion that the bulk of the native site soils appear to be predominately Type C, although variations exist.
- All fills should conservatively be considered Type C unless directed otherwise by the contractor's competent individual trained in trench safety.
- All trenching should be performed and stabilized in accordance with local, state, and OSHA standards. Bank stability is the responsibility of the contractor or contractor's qualified representative who is present at the site, able to observe changes in ground conditions, and has control over personnel and equipment.

- Pipe bedding, initial backfill, and trench backfill shall be consistent with the approved civil and/or utility plans.

5.2.8. Earthwork Testing and Observation

- Verification of subgrade and fills should be performed by a firm that is AMRL accredited in ASTM E329.
- Special inspection of fill soils is required during mass grading; the Special Inspector should be ICC certified in soils or NAQTC certified in Sampling and Density disciplines.
- The special inspector shall verify and document that placement of fill is consistent with the Site Grading (Section 5.2.6) of this report.
- Density testing of fills should be in accordance with ASTM D6938 (Standard Test Methods for In-Place Density and Water Content of Soil and Soil-Aggregate by Nuclear Methods) or ASTM D1556 (Standard Test Method for Density and Unit Weight of Soil in Place by Sand-Cone Method).
- Subgrade should be density tested approximately every 500 square yards.
- Fill should be density tested once for every 1,000 square yards per lift of material placed during mass grading and one test per 300 feet of footing trenches or overexcavation of footings.
- One density test should be performed for each 500 square yards or per each lift for smaller, localized fill zones.
- Full time construction observation is required for mass graded fills.
- The testing frequency should be increased if the contractor is having difficulty achieving and maintaining the required moisture levels.
- Utility bedding and trench backfill should be density tested per the requirements of the governing agency. Typically, agencies prescribe density testing per foot of bedding/backfill thickness, at a frequency of one test between manholes or valves, or one test every 500 lineal feet, including laterals.
- Nonstructural fills should be density tested for every 2,000 yards or for every 2-feet of fill for smaller, localized fill zones.

5.3. Structures

The following sections have been developed based on the understanding and expectation that the requirements specified in Section 5.2 Earthwork of this report have been met.

5.3.1. Standard Spread Foundations

The following table and supporting statements present design values and considerations for use in the development of standard spread foundation design.

Table 3 - Allowable Foundation Bearing Pressures for Standard Spread Foundations

Loading Condition	Maximum Net Allowable Bearing Pressure (PSF)[1]
Dead Load Plus Full Time Live Load	3,000
Dead Load Plus Live Loads, Plus Transient Wind or Seismic Loads	4,000

[1] Net allowable bearing pressure is that pressure at the base of the footing is in addition to the adjacent overburden pressure.

- For frost protection, footings should be founded at least two feet below adjacent outside or unheated interior finish grades.
- Interior footings not located within frost prone areas should be founded at least 12 inches below surrounding ground or slab level for confinement.
- Regardless of loading, individual pad foundations and continuous spread foundations should be at least 18 and 12 inches wide, respectively, or as required by code.
- The minimum footing sizes recommended are based on the ability to develop bearing capacity. Footing dimensions should be determined by the engineer responsible for structural design.
- Lateral loads, such as wind or seismic, may be resisted by passive soil pressure and friction on the bottom of the footing. A coefficient of base friction of 0.45 is typical to the structural fills.
- Design values for active and passive equivalent fluid pressures of 35 and 425 pounds per square foot per foot of depth, respectively, can be utilized.
- When designing for passive pressure, the upper one foot of the soil profile should not be included unless confined by a concrete slab, or pavement.
- The integrity of the foundation support soils shall be maintained until concrete is placed.
- Total post-construction settlement for the structures is anticipated to be on the order of 1-inch, or less.
- Differential settlement between foundations with similar loads and sizes is anticipated to be half of the total settlement.

5.3.1.a. Slab-on-Grade Flooring

- A moisture vapor retarder (Stego Wrap 15 mil) should be considered for slab areas covered by moisture sensitive floor coverings or equipment. Moisture vapor layers should be placed in accordance with the manufacturer's instructions and below the base layer on a properly prepared subgrade.
- Interior slabs-on-grade should be supported on a 6-inch, minimum, layer of Type 2, Class B aggregate base layer compacted to not less than 95-percent of the soil's maximum dry density.

5.3.2. Retaining Structures

- Recommended lateral earth pressures for consideration in the design of retaining walls less than 6-feet in total height and landscaping walls are presented in Table 4.

Table 4 – Lateral Earth Pressures

Condition	Active (psf/f)		Passive (psf/f)		At Rest
	Static	Pseudo-Static	Static	Pseudo-Static	
Level, Drained	35	57	350	300	55

- The values presented in Table 4 do not consider hydrostatic pressures or surcharge loading.
- In addition, it has been assumed that some displacement is allowable during the design event, and our recommended values have therefore been based on 50% of the USGS' predicted PGA.
- Excessive wall pressures can be developed due to heavy compaction equipment proximate to the wall during backfill placement. Therefore, due care during placement and compaction of backfill is required.
- Backfill behind retaining structures should be granular and compacted to not less than 90 percent of the soils' maximum dry density.
- Site preparation and foundations for retaining structures shall be consistent with Section 5.2 Earthwork and Section 5.3.1 Standard Spread Foundations.

5.4. Concrete

The American Concrete Institute (ACI) considers three different exposure classes to address the degree of severity for environmental conditions: freezing and thawing (Exposure Category F), concrete in contact with soil that contains water soluble sulfate ions (Exposure Category S), and Exposure Class W for concrete in contact with water. Exposure categories F and S, and potential mitigation, are discussed in Section 5.4.1.

5.4.1. *Materials*

The concrete material recommendations vary between Exposure Class F and Exposure Class S. Therefore, those specifying concrete subject to both freezing and thawing and water-soluble sulfate ions should consider the potential exposure conditions for Class F and Class S and select the most appropriate.

5.4.1.a. Exposure Class F (Exterior Slabs-on-Grade)

- The potential exists for some portions of concrete to absorb sufficient water to be saturated prior to freezing (Exposure Class F2).
- Where deicing chemicals are anticipated to be used (civil improvements), exposure Class F3 would be assigned.
- Therefore, as presented in the Standard Specifications for Public Works Construction concrete requirements for Portland cement concrete exposed to freeze-thaw cycles, should include air entrainment (4 – 7%), maximum water: binder ratio of 0.45, and a minimum 28-day compressive strength of 4,000 psi.

5.4.1.b. Exposure Class S

Mitigation for sulfate exposure is required based on the test results. The summary below presents excerpts from ACI and SSPWC. Refer to the referenced standards in their entirety when specifying sulfate resistance exceeding the presented geotechnical or civil considerations.

- Sulfate testing on the native soils yielded results in the S0 negligible range (ACI 318, < 0.10 percent SO_4^{2-} by mass).
- Unless sulfate testing indicates a soil profile presenting severe to very severe sulfate levels, Type II cement, a maximum water: binder ratio of 0.50, and a minimum 28-day compressive strength of 4,000 psi should be observed.
- Per the SSPWC, no special concrete provisions are required to address sulfate resistance. Type II cement is recommended for use.

5.4.1.c. Chlorides

- Chemical testing on the native soils presented chloride ion concentrations of 87 mg/kg.
- The structural engineer will identify any required mitigation due to the presence of chlorides.
- The structural engineer should weigh the criticality of the chloride ion concentration and call for additional verification testing once the pads have been mass graded.

5.4.2. Construction

- All concrete placement and curing should be performed in accordance with procedures outlined by the American Concrete Institute. Special considerations should be given to concrete placed and cured during hot or cold weather conditions.

- The occurrence of concrete shrinkage cracking may be reduced and/or controlled by limiting the amount of water within the mix (water cement ratio of 0.45 or less), the incorporation of crack control joints and proper concrete placing and curing practices including ACI 318 provisions for areas subject to freeze thaw conditions.

- Proper curing, finishing, control joints and reinforcing should be provided to minimize any damage resulting from shrinkage including cracks and slab curling.

- Western Nevada is a region with absorptive aggregates and exceptionally low relative humidity. Therefore, concrete flatwork will shrink and curl in a manner which is not typical of many other US regions.

- Proper site preparation and placement of reinforcement are also important to the performance of slab-on-grade improvements.

- Interior slabs-on-grade should be supported on a 6-inch, minimum, layer of Type 2, Class B aggregate base layer compacted to not less than 95-percent of the soil's maximum dry density.

- A moisture vapor retarder (Stego Wrap 15 mil) should be considered for slab areas covered by moisture sensitive floor coverings or equipment. Moisture vapor layers should be placed in accordance with the manufacturer's instructions and below the base layer on a properly prepared subgrade.

- Joint spacing, locally, is typically on 10-to-12-foot centers for large slabs and no more than five feet for sidewalks.

- Cracking that occurs within the slab-on-grade floors will often reflect through overlying improvements even if adequate substrate preparation has occurred.

- Control of the rate of moisture loss in concrete slab-on-grade improvements by using curing compounds, fogging, or other suitable means is imperative to protecting the slab from excessive curling.

- Slabs-on-grade will still exhibit some cracking and curling. The occurrence of concrete shrinkage cracks is independent of the soil supporting characteristics.

5.5. Public Improvements

Unless noted otherwise, dedicated improvements shall be designed and constructed in accordance with the approved civil plans, the Standard Specifications for Public Works Construction, and the City of Sparks' standard details.

5.5.1. Structural Pavement Sections

- Materials and workmanship shall be consistent with the requirements of the jurisdiction and the Standard Specifications for Public Works Construction.
- Table 5 presents the recommended minimum structural pavement sections for the development based on planned use. Minimum sections required by the jurisdictions shall govern if more than specified in Table 5.
- Depending on final site grading, structural pavement sections may be re-evaluated by the Geotechnical Engineer in consideration of the subgrade R-Value (ASTM D2844 – Standard Test Method for Resistance R-Value and Expansion Pressure of Compacted Soils).

Table 5 - Structural Pavement Sections

Road Type	Pavement Thickness (In.)	Pavement Type[1]	Type II Class B Base Course Thickness (In.)
Main Access Drives	4	2" Type 3 + Lime / 2" Type 2	6
Parking and Automobile Traffic Driveways	3	Type 3 + Lime	6
Fire Truck Driveway Apron and Access	8	Portland Cement Concrete	6
Dumpster Aprons[2]	6	Reinforced Portland Cement Concrete	6

[1] Per the Standard Specifications for Public Works Construction
[2] Dumpster aprons should extend far enough from the trash enclosure so that the wheel loads are confined to the reinforced concrete pad.

- The Contractor should submit concrete and plantmix bituminous pavement mix designs to the Owner or Engineer, for approval, at least five working days prior to paving. When pavement is placed directly adjacent to concrete flatwork, the finish compacted grade of the pavement should be at least ½ of an inch higher than the edge of adjacent concrete surface to allow adequate compaction of the pavement without damaging the concrete.

5.5.2. Pavement Design Life

- Maintenance is mandatory to ensure long-term pavement performance and to meet or exceed the assumed 20-year design life.

- Maintenance refers to any activity performed on the pavement that is intended to preserve its original service life or load-carrying capacity. Examples of maintenance activities include patching, crack or joint sealing, and seal coats. If these maintenance activities are ignored or deferred, premature failure of the pavement will occur.
- Premature failure of asphaltic concrete frequently occurs adjacent to poorly graded ponding areas and/or landscape areas. Failures may occur due to excessive precipitation, irrigation and landscaping water infiltrating into the subgrade soils causing subgrade failure.
- In areas where saturation of the subgrade soils beneath asphaltic pavement may occur, it is strongly recommended the owner/project manager include provisions by design for a subdrain system to eliminate the potential for saturation of subgrade soils. The subdrain system should discharge into a permanent drainage area that will not impede drainage flow to cause the system to back-up and/or clog. Appropriate maintenance procedures should be implemented to ensure the subdrain system does not plug and allow for proper drainage of surface and subsurface water beneath paved areas. Subdrain location and configuration should be evaluated once final grading and landscaping plans have been prepared.
- If the ultimate traffic exceeds the anticipated levels, it may be necessary to reevaluate and overlay the pavement at some time in the future.
- The cost associated with proper maintenance is generally much less than the cost for reconstruction due to the premature failure of the pavement. Therefore, it is strongly recommended the owner/project manager implement a pavement management program.

6. CONSTRUCTION OBSERVATION AND TESTING SERVICES

- The recommendations presented in this report assume that the contractors perform their work as required by the project documents and that owner/project manager provides for sufficient field-testing and construction review during each phase of construction.
- Prior to construction, the owner/project manager should schedule a pre-construction conference including, but not limited to representatives of the owner, architect, civil engineer, the general contractor, earthwork and materials subcontractors, building official, and geotechnical engineer.
- It is the owner's/project manager responsibility to set-up this meeting and contact all responsible parties. The conference will allow parties to review the project plans, specifications, scheduling and recommendations presented in this report, and discuss applicable material quality and mix design requirements.
- Quality control reports should be submitted to the owner/project manager for review and distributed to the appropriate parties. It is essential that any changes or revisions to project plans be provided to Wood Rodgers in a timely fashion to ensure contractor compliance and avoid construction delays or the need to remove completed work.
- During construction, Wood Rodgers Incorporated should have the opportunity to provide sufficient on-site observation of site preparation and grading, over-excavation, fill placement, foundation installation, and paving.

These observations would allow us to document the geotechnical conditions are as anticipated and that the contractor's work meets with the criteria in the approved plans and specifications. Verification of horizontal and vertical control must be provided by whoever was responsible for establishing those boundaries and constructing associated improvements.

7. STANDARD LIMITATION CLAUSE

- This report has been prepared in accordance with generally accepted local geotechnical practices.
- The analyses and recommendations submitted are based upon field exploration performed at the specific locations identified and the conditions encountered, as discussed in our report.
- No guarantee or warranty as to the continuity of soil conditions between exploration points is implied or intended.
- Therefore, this report does not reflect soil variations that may become evident during the construction period, at which time re-evaluation of the recommendations may be necessary.
- Final plans and specifications should be reviewed by the design engineer responsible for this geotechnical report to determine if they have been prepared in accordance with the recommendations contained in this report prior to submitting to the building department for review.
- It is the owner's/project manager responsibility to provide the plans and specifications to the engineer.
- This report is issued with the understanding that it is the responsibility of the owner or their representative to ensure that the information and recommendations contained herein are brought to the attention of the design team for the project and incorporated into the plans and specifications, and that the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.
- We recommend our firm be retained to perform construction observation in all phases of the project related to geotechnical factors to document compliance with our recommendations.
- The owner/project manager is responsible for distribution of this geotechnical report to all designers and contractors whose work is related to geotechnical factors.
- It is the contractor’s responsibility for the grading and construction of the designed improvements. This responsibility includes the means, methods, techniques, sequence, and procedures of construction and safety of construction at the site.
- All construction shall conform to the requirements of the most recently adopted version of the Standard Specifications for Public Works Construction and the requirements of the City of Sparks and Washoe County, Nevada.
- Failure to inspect the work shall not relieve the contractor from his obligation to perform sound and reliable work as described herein and as described in the Standard Specifications for Public Works Construction.

- In the event of changes in the design, location, or ownership of the project after presentation of this report, our recommendations should be reviewed and possibly modified by the Geotechnical Engineer.
- If the Geotechnical Engineer is not accorded the privilege of making this recommended review, we can assume no responsibility for misinterpretation or misapplication of our recommendations or their validity in the event changes have been made in the original design concept without our prior review. The engineer makes no other warranties, either expressed or implied, as to the professional advice provided under the terms of this agreement and included in this report.
- This report was prepared by Wood Rodgers, Inc. for the benefit of the City of Sparks and their duly assigned agents or other responsible parties.
- The material in this report reflects Wood Rodgers' best judgment considering the information available to it at the time of preparation. Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties.
- Wood Rodgers accepts no responsibility for damages, if any, suffered by any third party because of decisions made by third parties or actions based on this report without consultation with Wood Rodgers and written approval for such actions.

8. REFERENCES

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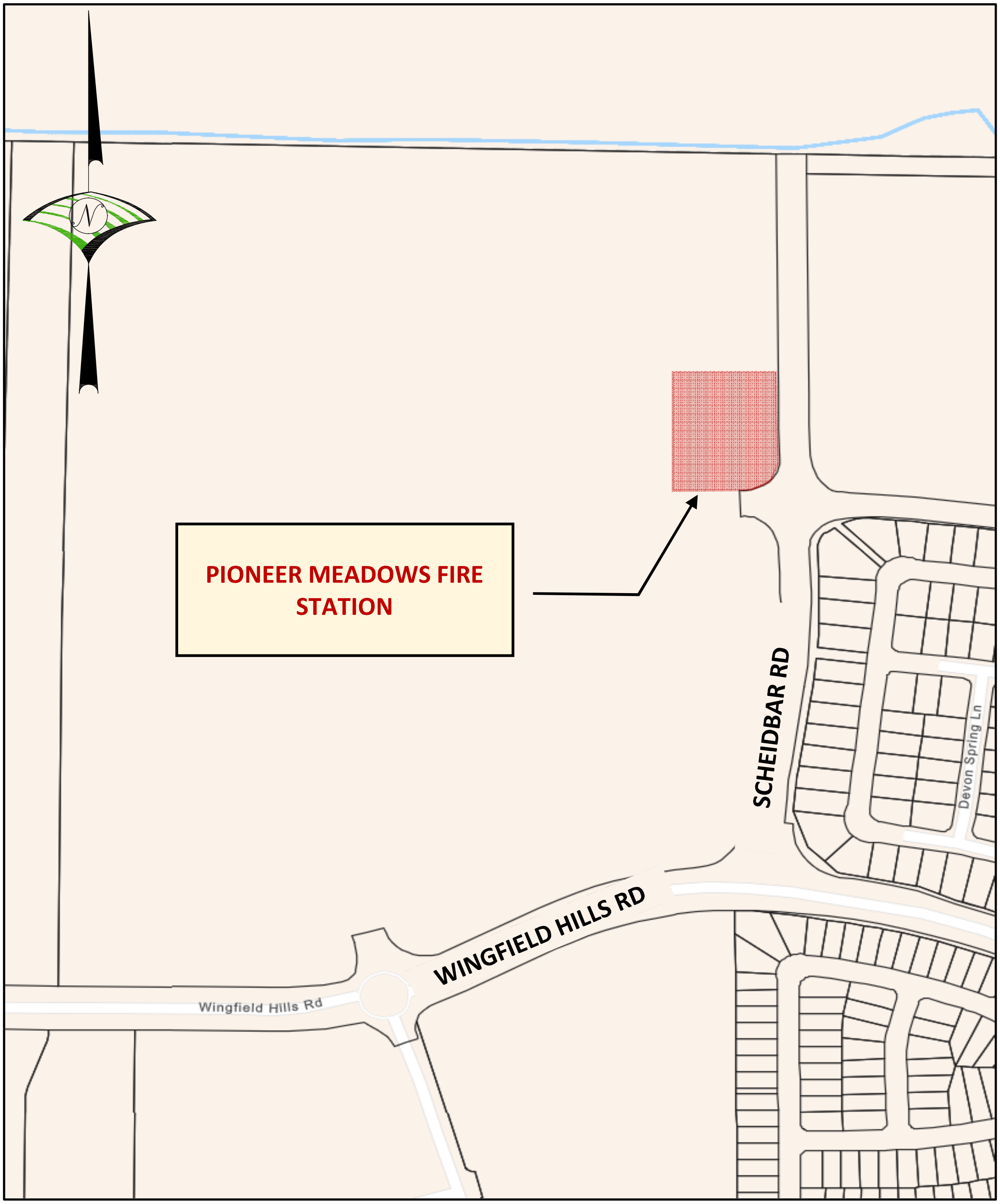
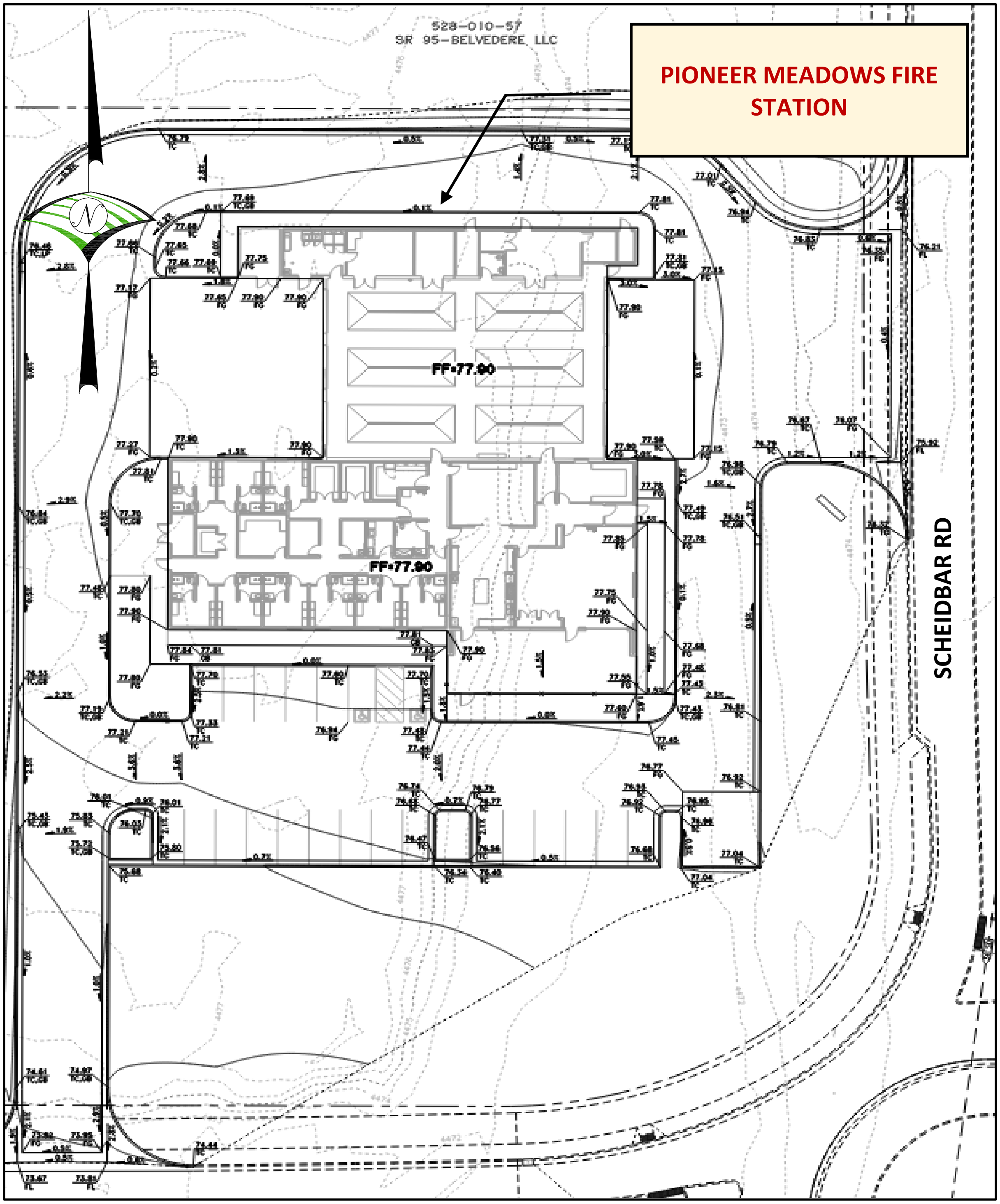
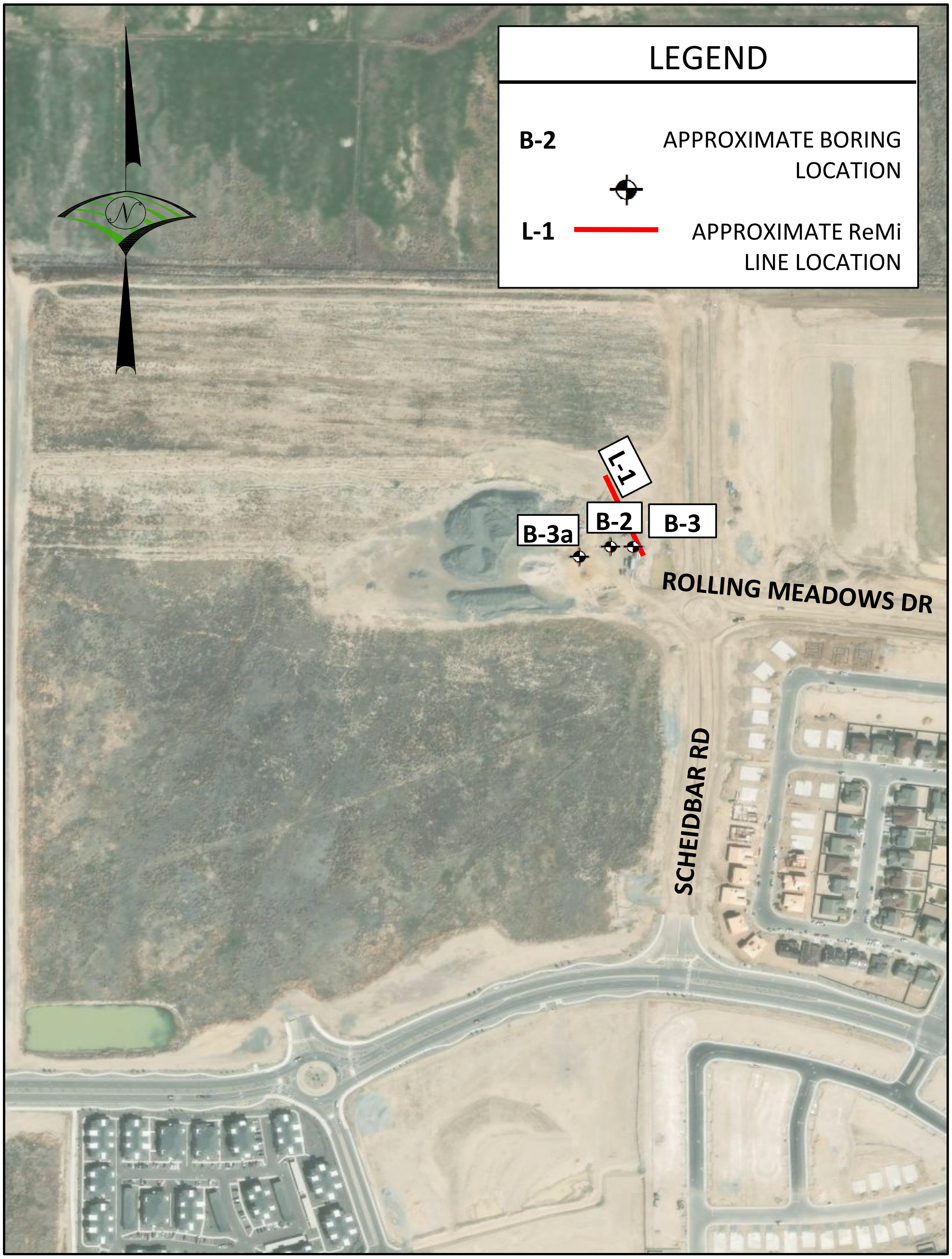
APPENDIX A
FIGURES



PIONEER MEADOWS FIRE STATION	PROJECT NO. 8523020	VICNITY MAP	FIGURE 1



PIONEER MEADOWS FIRE STATION	PROJECT NO. 8523020	IMPROVEMENT PLAN	FIGURE 2



PIONEER MEADOWS FIRE STATION	PROJECT NO. 8523020	SITE MAP & APPROXIMATE EXPLORATION LOCATIONS	FIGURE 3

APPENDIX B
FIELD EXPLORATION

Wood Rodgers

BORING NUMBER B-2

CLIENT City of Sparks
PROJECT NAME Pioneer Meadows Fire Station
PROJECT NUMBER 8523020
PROJECT LOCATION Sparks, Nevada
DATE STARTED 2/8/23 **COMPLETED** 2/8/23
GROUND ELEVATION 4477 ft NAVD 88 **HOLE SIZE** 4 inches
DRILLING CONTRACTOR Taber Drilling
GROUND WATER LEVELS:
DRILLING METHOD CME 55 - 4" SSA to 14-ft, Mud Rotary to 51.5-ft
AT TIME OF DRILLING 11.5 ft
LOGGED BY Lilian Lorincz **CHECKED BY** Justin McDougal
AT END OF DRILLING ---
NOTES: Elevations: City of Sparks Benchmark 10, 39.6193, -119.7077
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	ADDITIONAL SAMPLE	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	R-VALUE	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		FILL - CLAYEY SAND, (SC) dense, dry, olive brown, medium plasticity											
		Medium dense, slightly moist, olive yellow	GB 2A	SPT 2B		3-4-7 (11)			17.2	34	16	18	28.1
5		WELL GRADED SAND WITH SILT, (SW-SM) medium dense, slightly moist, olive yellow, non plastic		SPT 2C		11-11-16 (27)							
		Very dense		SPT 2D		18-26-28 (54)			6.0	NP	NP	NP	10.1
10				SPT 2E		14-13-9 (22)							
		SILTY SAND, (SM) medium dense, moist, olive yellow, medium plasticity											
				MC 2F		6-6-8 (14)		78	41.5	38	26	12	47.5
15				MC 2G		7-7-9 (16)							
		Dense, wet, yellowish brown		MC 2H		9-14-50/6"							
20				MC 2I		18-24-35 (59)		114	16.6				23.4
25		Very dense		SPT 2J		20-29-40 (69)							
30		SILT, (ML) hard, wet, yellowish brown, low plasticity		SPT 2K		9-13-46 (59)							
35													

(Continued Next Page)

Wood Rodgers

BORING NUMBER B-2

CLIENT City of Sparks
PROJECT NAME Pioneer Meadows Fire Station
PROJECT NUMBER 8523020
PROJECT LOCATION Sparks, Nevada

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	ADDITIONAL SAMPLE	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	R-VALUE	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
35		SILTY SAND, (SM) very dense, wet, yellowish brown, non plastic		MC 2L		23-30-35 (65)							
40		SILT, (ML) stiff, wet, yellowish brown, low plasticity		SPT 2M		6-7-8 (15)							
		SILTY SAND, (SM) dense, wet, yellowish brown, non plastic											
45				SPT 2N		12-16-19 (35)							
50				SPT 2O		13-16-30 (46)							

Bottom of Borehole at 51.5 Feet.

Wood Rodgers

BORING NUMBER B-3

CLIENT City of Sparks
PROJECT NAME Pioneer Meadows Fire Station
PROJECT NUMBER 8523020
PROJECT LOCATION Sparks, Nevada
DATE STARTED 2/8/23 **COMPLETED** 2/8/23
GROUND ELEVATION 4477 ft NAVD 88 **HOLE SIZE** 4 inches
DRILLING CONTRACTOR Taber Drilling
GROUND WATER LEVELS:
DRILLING METHOD CME 55 - 4" SFA to 21.5-ft
▽ **AT TIME OF DRILLING** 14.0 ft
LOGGED BY Lilian Lorincz **CHECKED BY** Justin McDougal
AT END OF DRILLING ---
NOTES: Elevations: City of Sparks Benchmark 10, 39.6193, -119.7079
▼ **AFTER DRILLING** 9.50 ft / Elev 4467.50 ft

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	ADDITIONAL SAMPLE	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	R-VALUE	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		FILL - CLAYEY SAND, (SC) medium dense, slightly moist, olive brown, medium plasticity Yellowish brown	GB 3A										
5		WELL GRADED SAND WITH SILT, (SW-SM) medium dense, moist, yellowish brown to olive gray, non plastic		SPT 3B		6-12-13 (25)							
10	▼	SILTY SAND, (SM) medium dense, moist, olive yellow, low plasticity		SPT 3C		11-11-16 (27)			26.8	27	22	5	26.8
15	▽			MC 3D		15-15-13 (28)							
20		Wet, yellowish brown		SPT 3E		9-5-8 (13)							

Bottom of Borehole at 21.5 Feet.

Wood Rodgers Inc.
1361 Corporate Blvd
Reno NV 89521
Telephone: 775-823-4068
Fax: 775-823-4066

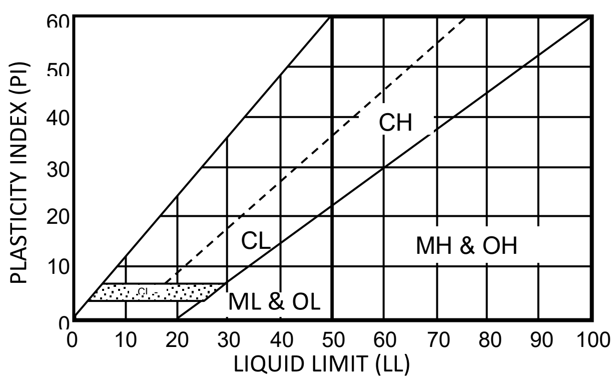
BORING NUMBER B-3a

CLIENT LDK Ventures
PROJECT NAME Pioneer Meadows Business Park Geotech
PROJECT NUMBER 1509079
PROJECT LOCATION Sparks, Nevada
DATE STARTED 6/15/22 **COMPLETED** 6/15/22
GROUND ELEVATION 4476.2 ft **HOLE SIZE** 4 inches
DRILLING CONTRACTOR Taber Drilling
GROUND WATER LEVELS:
DRILLING METHOD CME 45 - 4" SFA to 16.5-ft
AT TIME OF DRILLING 12.0 ft
LOGGED BY Jackson Beadell **CHECKED BY** Justin McDougal
AT END OF DRILLING ---
NOTES: Elevations: Washoe County 6ft DEM, 39.6191, -119.7082
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	ADDITIONAL SAMPLE	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	R-VALUE	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	FINES CONTENT (%)
0		FILL - CLAYEY GRAVEL, (GC) medium dense, slightly moist, grey, medium plasticity		SPT 3A		19-10-15 (25)							
		FILL - CLAYEY SAND, (SC) loose, moist, brown, medium plasticity	GB 3H	SPT 3B		3-3-4 (7)							
5		SILTY SAND, (SM) medium dense, moist, brown, non-plastic		SPT 3C		6-13-13 (26)							
		SILTY SAND, (SM) medium dense, moist, orange brown, non-plastic		SPT 3D		11-14-16 (30)							
10		Dense		SPT 3E		11-28-14 (42)							
		LEAN CLAY WITH SAND, (CL) hard, moist, light brown, medium plasticity											
		SANDY LEAN CLAY, (CL) stiff, wet, brown, low plasticity		MC 3F		5-8-11 (19)							
15				SPT 3G		4-5-5 (10)							

Bottom of Borehole at 16.5 Feet.

MAJOR DIVISION				TYPICAL NAMES
COARSE-GRAINED SOILS MORE THAN HALF IS COARSER THAN NO. 200 SIEVE	GRAVEL MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN SANDS WITH LITTLE OR NO FINES	GW	WELL GRADED GRAVELS WITH OR WITHOUT SAND, LITTLE OR NO FINES
			GP	POORLY GRADED GRAVELS WITH OR WITHOUT SAND, LITTLE OR NO FINES
		GRAVELS WITH OVER 12% FINES	GM	SILTY GRAVELS, SILTY GRAVELS WITH SAND
			GC	CLAYEY GRAVELS, CLAYEY GRAVELS WITH SAND
	SAND MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS WITH LITTLE OR NO FINES	SW	WELL GRADED SANDS WITH OR WITHOUT GRAVEL, LITTLE OR NO FINES
			SP	POORLY GRADED SAND WITH OR WITHOUT GRAVEL, LITTLE OR NO FINES
		SANDS WITH OVER 12% FINES	SM	SILTY SANDS WITH OR WITHOUT GRAVEL
			SC	CLAYEY SANDS WITH OR WITHOUT GRAVEL
FINE-GRAINED SOILS MORE THAN HALF IS FINER THAN NO. 200 SIEVE	SILT AND CLAY LIQUID LIMIT 50% OR LESS		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTS WITH SANDS AND GRAVELS
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY CLAYS WITH SANDS AND GRAVELS, LEAN CLAYS
			OL	ORGANIC SILTS OR CLAYS OF LOW PLASTICITY
	SILT AND CLAY LIQUID LIMIT GREATER THAN 50%		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOLID, ELASTIC SILTS
			CH	INORGANIC CLAYS OR HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC SILTS OR CLAYS MEDIUM TO HIGH PLASTICITY
HIGHLY ORGANIC SOILS			PT	PEAT AND OTHER HIGHLY ORGANIC SOILS



CONSISTENCY		RELATIVE DENSITY	
SILTS & CLAYS	SPT BLOW* COUNTS (N)	SANDS & GRAVELS	SPT BLOW* COUNTS (N)
VERY SOFT	0 - 2	VERY LOOSE	0 - 4
SOFT	3 - 4	LOOSE	5 - 10
MEDIUM STIFF	5 - 8	MD DENSE	11 - 30
STIFF	9 - 15	DENSE	31 - 50
VERY STIFF	16 - 30	VERY DENSE	50 +
HARD	30 +		

* The Standard Penetration Resistance (N) In blows per foot is obtained by the ASTM D1586 procedure using 2” O.D., 1 3/8” I.D. samplers.

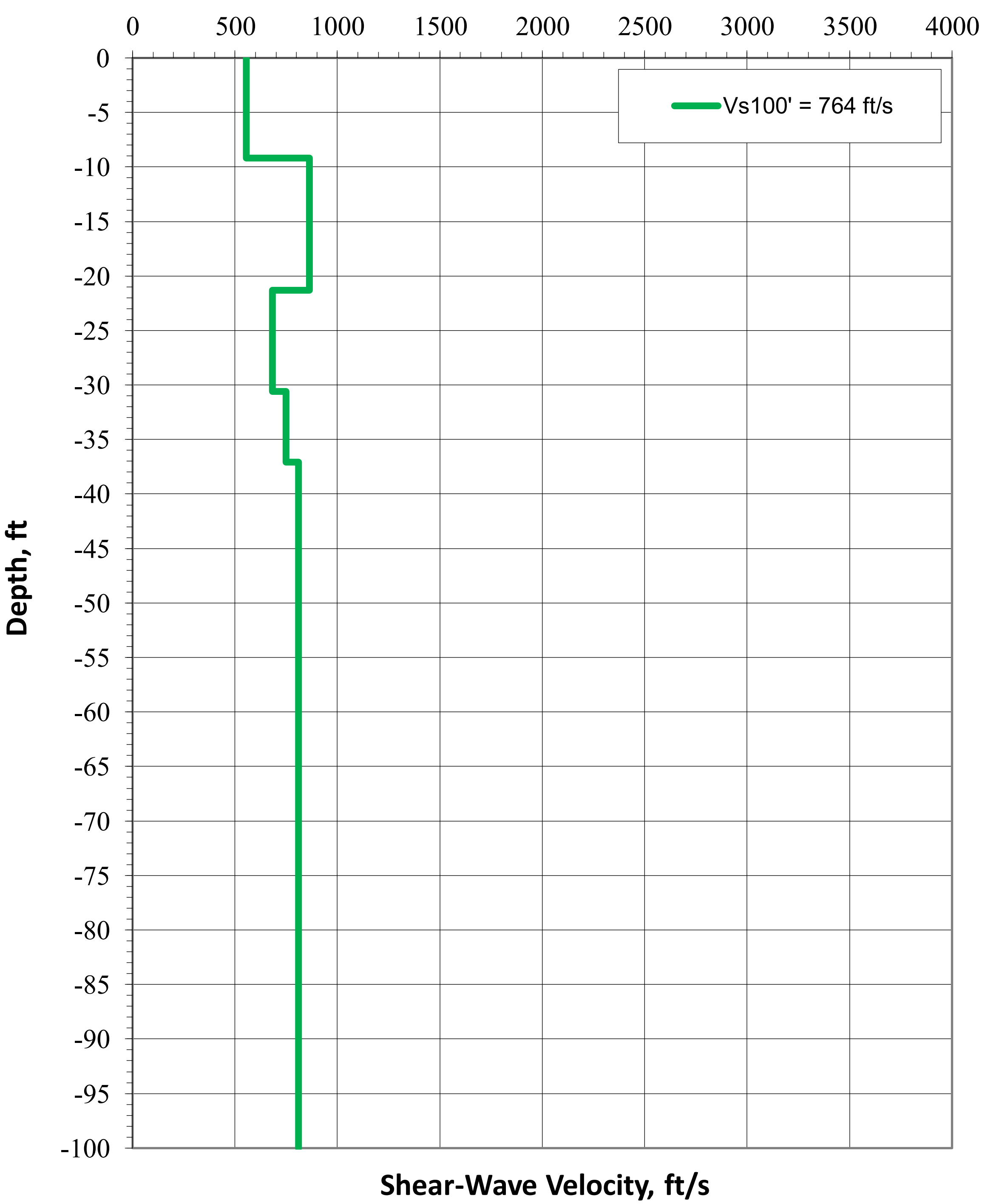
PLASTICITY DESCRIPTIONS			
DESCRIPTION	RANGE	DESCRIPTION	RANGE
NONPLASTIC	<5	MEDIUM	10-20
LOW	< 10	MEDIUM-HIGH	15 - 25
LOW-MEDIUM	5 - 15	HIGH	>25

DESCRIPTION OF ESTIMATED PERCENTAGES OF GRAVEL, SAND, AND FINES	
TRACE	Particles are present but est. < 5%
FEW	5% - 10%
LITTLE	15% - 20%
SOME	30% - 45%
MOSTLY	50% - 100%

DEFINITIONS OF SOIL FRACTIONS	
SOIL COMPONENT	PARTICLE SIZE RANGE
BOULDERS	> 12 INCHES
COBBLES	3 to 12 Inches
GRAVEL	3 IN. TO NO. 4 SIEVE
COARSE GRAVEL	3 IN. TO 3/4 IN.
FINE GRAVEL	3/4 IN. TO NO. 4 SIEVE
SAND	NO. 4 TO NO. 200
COARSE SAND	NO. 4 TO NO. 10
MEDIUM SAND	NO. 10 TO NO. 40
FINE SAND	NO. 40 TO NO. 200
FINES (SILT OR CLAY)	MINUS NO. 200 SIEVE

NOTE: Percentages are presented within soil description for soil horizon with laboratory tested soil samples.

Pioneer Meadows Fire Station, L-1, 165': Vs Model



PIONEER MEADOWS FIRE STATION	PROJECT NO. 8523020	S-WAVE ReMi RESULTS	B-3

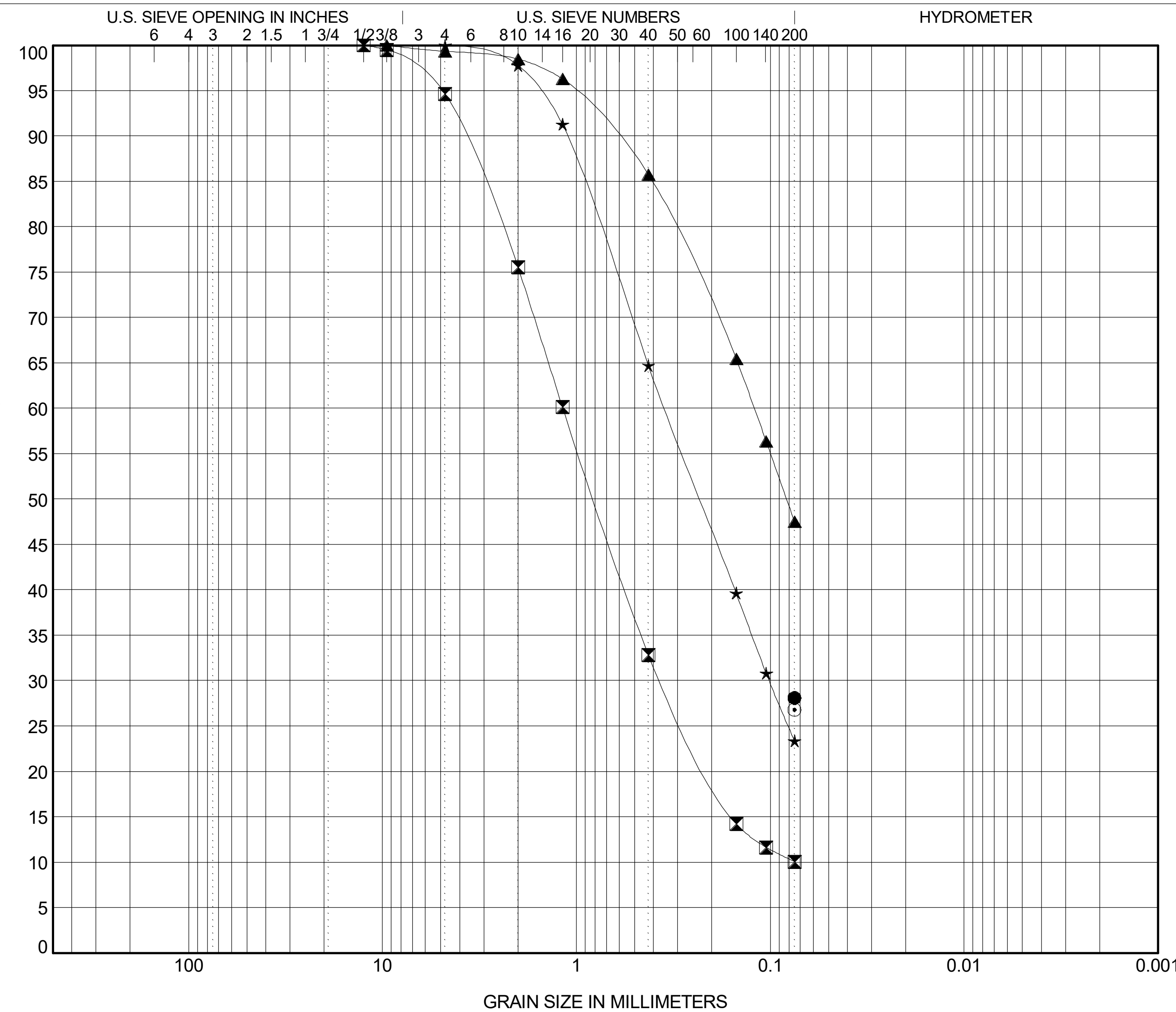
APPENDIX C
LABORATORY TESTING RESULTS

Wood Rodgers Inc.
1361 Corporate Blvd
Reno NV 89521
Telephone: 775-823-4068
Fax: 775-823-4066

GRAIN SIZE DISTRIBUTION

CLIENT City of Sparks **PROJECT NAME** Pioneer Meadows Fire Station

PROJECT NUMBER 8523020 **PROJECT LOCATION** Sparks, Nevada



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BORING	DEPTH	Classification	LL	PL	PI	Cc	Cu
●	**B-2**	**2.5**	**CLAYEY SAND(SC)**	**34**	**16**	**18**		
⧗	**B-2**	**7.5**	**WELL-GRADED SAND with SILT(SW-SM)**	**NP**	**NP**	**NP**	**1.51**	**15.84**
▲	**B-2**	**13.5**	**SILTY SAND(SM)**	**38**	**26**	**12**		
★	**B-2**	**21.0**	**SILTY SAND(SM)**					
⊙	**B-3**	**10.0**	**SILTY SAND(SM)**	**27**	**22**	**5**		

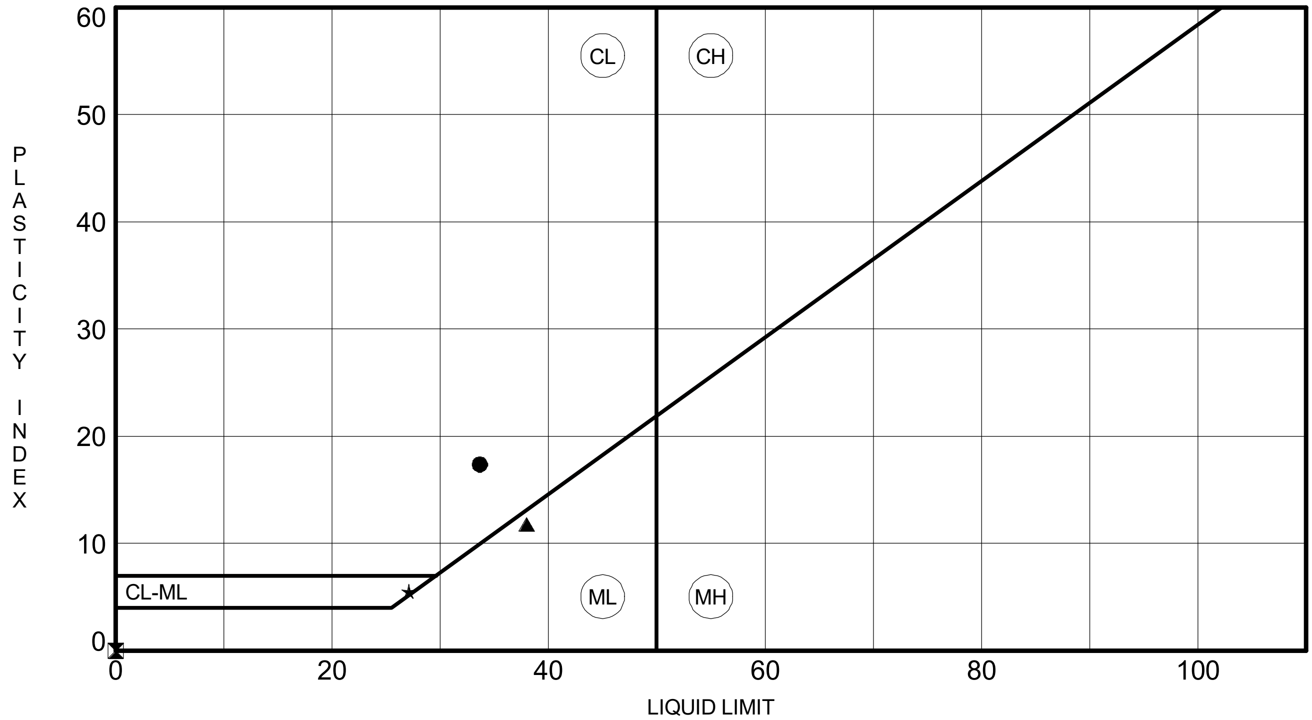
	BORING	DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-2**	**2.5**	**0.075**						**28.1**	
⧗	**B-2**	**7.5**	**12.5**	**1.174**	**0.363**		**5.3**	**84.6**	**10.1**	
▲	**B-2**	**13.5**	**9.5**	**0.121**			**0.7**	**51.8**	**47.5**	
★	**B-2**	**21.0**	**4.75**	**0.349**	**0.101**		**0.0**	**76.6**	**23.4**	
⊙	**B-3**	**10.0**	**0.075**						**26.8**	

GRAIN SIZE - BORING - GINT STD US LAB.GDT - 2/24/23 13:12 - \\WOODRODGERS.LOC\PRODUCTIONDATA\JOBS-RENO\JOBS\8523_CITYOFSPARKS\020_PM_FIRE_STATION\GEOTECH\GEOTECH\06 GINT\PIONEER MEADOWS FIRE STATION.GPJ

Wood Rodgers Inc.
1361 Corporate Blvd
Reno NV 89521
Telephone: 775-823-4068
Fax: 775-823-4066

ATTERBERG LIMITS' RESULTS

CLIENT City of Sparks
PROJECT NAME Pioneer Meadows Fire Station
PROJECT NUMBER 8523020
PROJECT LOCATION Sparks, Nevada



	BORING	DEPTH	LL	PL	PI	Fines	Classification
●	**B-2**	**2.5**	**34**	**16**	**18**	**28.1**	**CLAYEY SAND(SC)**
⧗	**B-2**	**7.5**	**NP**	**NP**	**NP**	**10.1**	**WELL-GRADED SAND with SILT(SW-SM)**
▲	**B-2**	**13.5**	**38**	**26**	**12**	**47.5**	**SILTY SAND(SM)**
★	**B-3**	**10.0**	**27**	**22**	**5**	**26.8**	**SILTY SAND(SM)**

SGS Silver State Analytical Laboratories
1135 Financial Blvd
Reno, NV 89502
(775) 857-2400 FAX: (888) 398-7002
www.ssalabs.com

Analytical Report

Workorder#: 23020832
Date Reported: 3/3/2023

Client: Wood Rodgers
Project Name: 1509079 / Pioneer Meadows Fire Station / B - 2 @ 0 - 5'
PO #: LAB 3961
Sampled By: W. Musnicki

Laboratory Accreditation Number: NV015/CA2990

Laboratory ID	Client Sample ID	Date/Time Sampled	Date Received
23020832-01	B - 2 @ 0 - 5'	02/15/2023 14:00	2/15/2023

Parameter	Method	Result	Units	PQL	Analyst	Date/Time Analyzed	Data Flag
Chloride	EPA 9056	87	mg/Kg	50	SR	02/25/2023 0:48	
Oxidation-Reduction Potential	SM 2580B	329	mV		AC	02/28/2023 11:26	
pH	SW-846 9045D	8.23	pH Units		AC	02/28/2023 9:16	
pH Temperature	SW-846 9045D	18.0	°C		AC	02/28/2023 9:16	
Resistivity	EPA 120.1	1700	Ohms-cm		AC	02/28/2023 14:50	
Sodium	ASTM D2791	< 0.01	%	0.01	AC	03/02/2023 10:08	
Sodium Sulfate as Na2SO4	Calculation	< 0.01	%	0.01	AC	03/02/2023 14:04	
Sulfate	SM4500 SO4E	< 0.01	%	0.01	AC	03/02/2023 14:08	
Sulfide	AWWA C105	Negative	POS/NEG		AC	02/28/2023 14:47	

Laboratory Accreditation Number: NV015/CA2990

Laboratory ID	Client Sample ID	Date/Time Sampled	Date Received
23020832-02	B - 3 @ 5'	02/15/2023 14:00	2/15/2023

Parameter	Method	Result	Units	PQL	Analyst	Date/Time Analyzed	Data Flag
Sodium	ASTM D2791	< 0.01	%	0.01	AC	03/02/2023 10:08	
Sodium Sulfate as Na2SO4	Calculation	< 0.01	%	0.01	AC	03/02/2023 14:04	
Sulfate	SM4500 SO4E	< 0.01	%	0.01	AC	03/02/2023 14:08	

APPENDIX D
ASCE 7 HAZARDS REPORT

ASCE 7 Hazards Report

Address:
No Address at This Location

Standard: ASCE/SEI 7-16
Risk Category: IV
Soil Class: D - Stiff Soil

Latitude: 39.619379
Longitude: -119.707614
Elevation: 4473.64 ft (NAVD 88)

ASCE
AMERICAN SOCIETY OF CIVIL ENGINEERS

Seismic

Site Soil Class: D - Stiff Soil

Results:

S_S :	1.358	S_{D1} :	N/A
S_1 :	0.47	T_L :	6
F_a :	1	PGA :	0.5
F_v :	N/A	PGA_M :	0.55
S_{MS} :	1.358	F_{PGA} :	1.1
S_{M1} :	N/A	I_e :	1.5
S_{DS} :	0.905	C_v :	1.372

Ground motion hazard analysis may be required. See ASCE/SEI 7-16 Section 11.4.8.

Data Accessed: Mon Feb 13 2023

Date Source: USGS Seismic Design Maps

APPENDIX E
LIQUEFACTION ASSESSMENT

Wood Rodgers, Inc.

SPT BASED LIQUEFACTION ANALYSIS REPORT

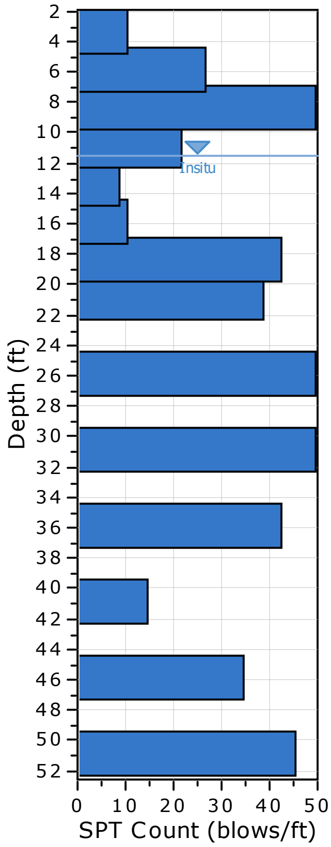
Project title : Pioneer Meadows Fire Station **SPT Name: B-2**

Location : Reno, NV

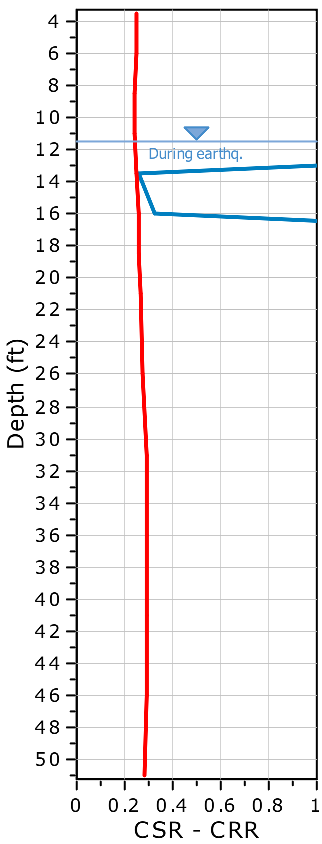
:: Input parameters and analysis properties ::

Analysis method:	NCEER 1998	G.W.T. (in-situ):	11.50 ft
Fines correction method:	NCEER 1998	G.W.T. (earthq.):	11.50 ft
Sampling method:	Sampler wo liners	Earthquake magnitude M_w:	6.52
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.55 g
Rod length:	3.28 ft	Eq. external load:	1.50 tsf
Hammer energy ratio:	1.45		

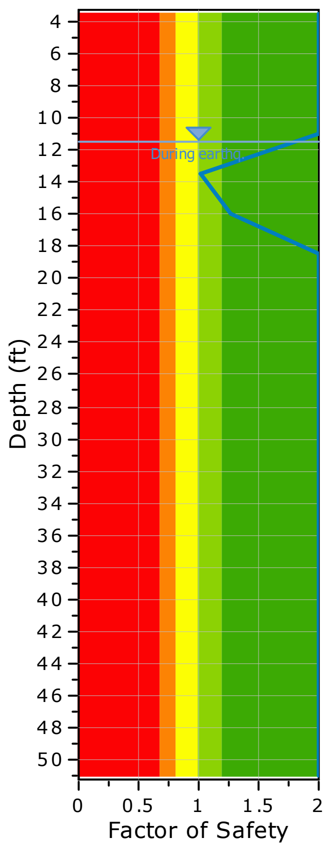
Raw SPT Data



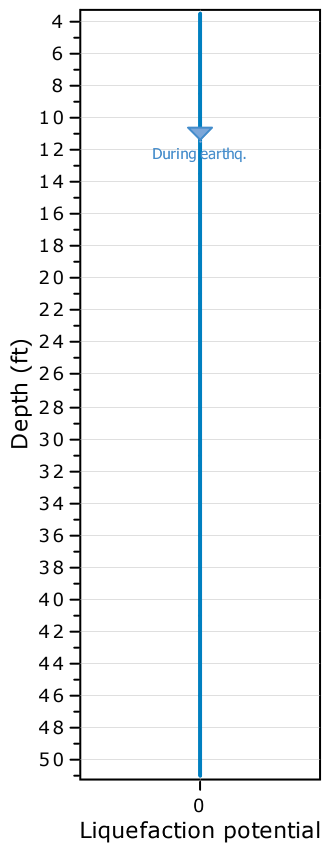
CSR - CRR Plot



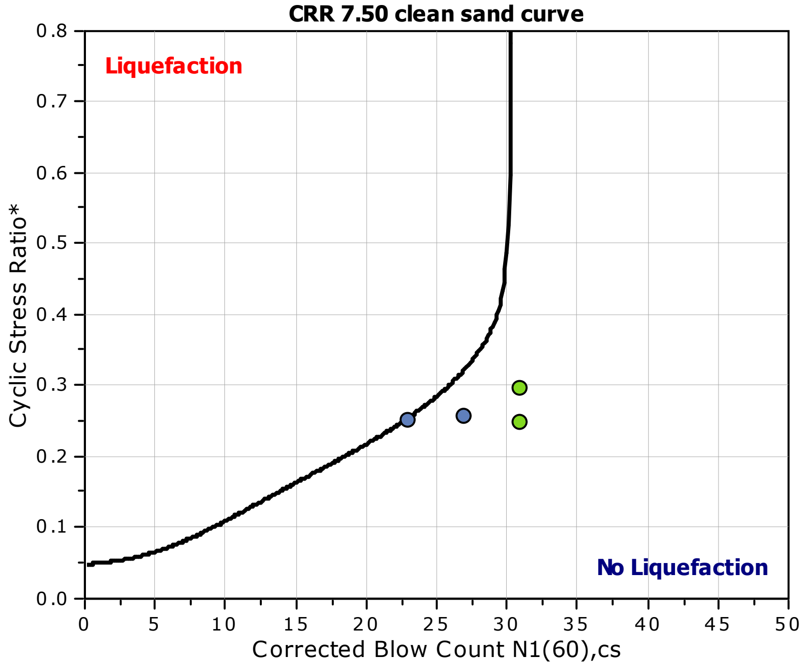
FS Plot



LPI



CRR 7.50 clean sand curve



F.S. color scheme

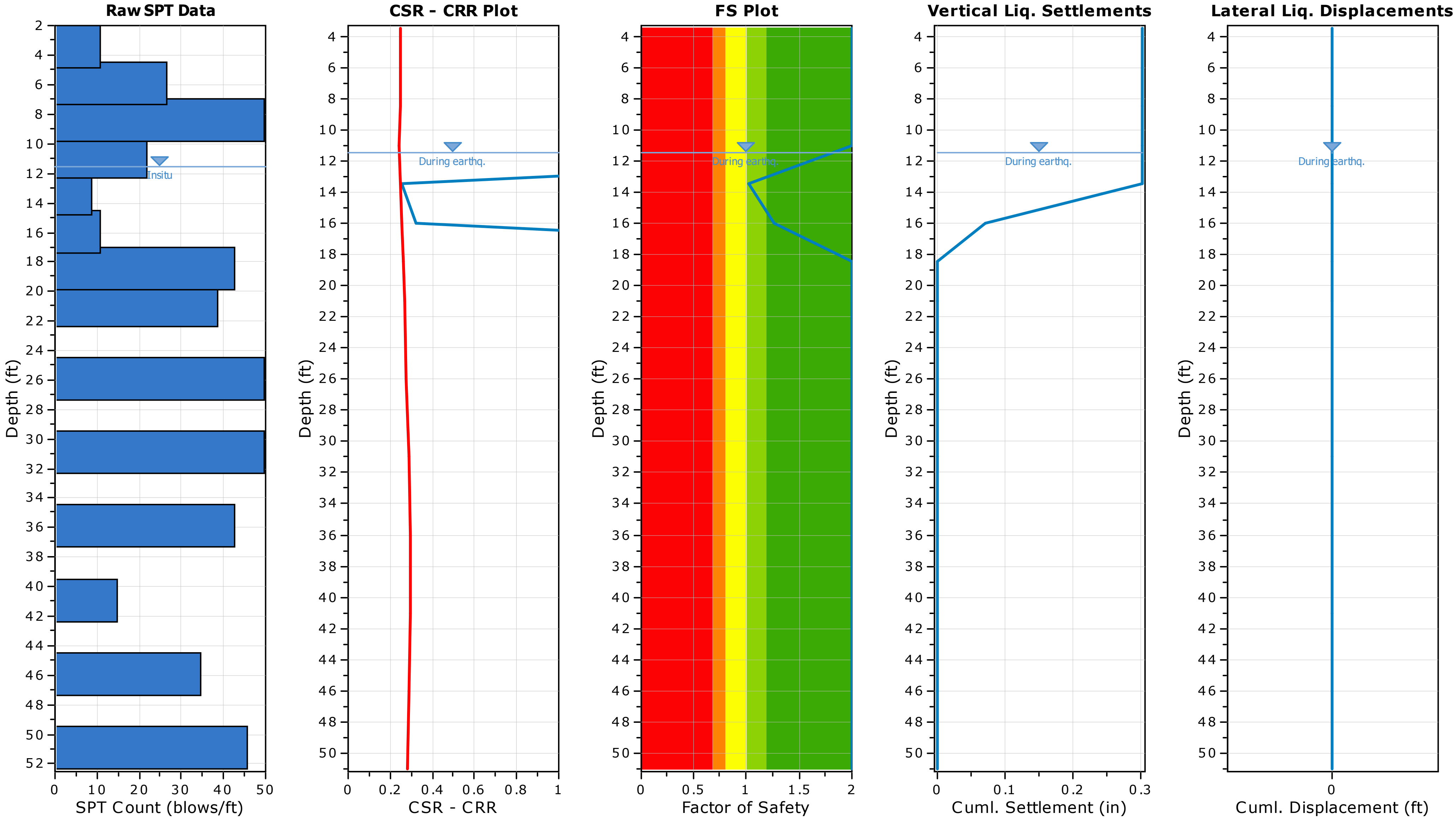
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

Project File: \\woodrodgers.loc\ProductionData\Jobs-Reno\Jobs\8523_CityofSparks\020_PM_Fire_Station\Geotech\Geotech\05 Analysis\20230213 Pioneer Meadows Fire.lsvs

:: 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
3.50	11	28.00	120.00	3.50	Yes
6.00	27	10.00	120.00	2.50	Yes
8.50	50	10.00	120.00	2.50	Yes
11.00	22	48.00	120.00	2.50	Yes
13.50	9	48.00	120.00	2.50	Yes
16.00	11	48.00	120.00	2.50	Yes
18.50	43	23.00	120.00	2.50	Yes
21.00	39	23.00	120.00	2.50	Yes
26.00	50	23.00	120.00	5.00	Yes
31.00	50	86.00	120.00	5.00	No
36.00	43	15.00	120.00	5.00	No
41.00	15	86.00	120.00	5.00	No
46.00	35	15.00	120.00	5.00	Yes
51.00	46	15.00	120.00	6.50	Yes

Abbreviations

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

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

Depth (ft)	SPT Field Value	Unit Weight (pcf)	σ_v (tsf)	u_o (tsf)	σ'_{vo} (tsf)	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	Fines Content (%)	α	β	$(N_1)_{60cs}$	$CRR_{7.5}$
3.50	11	120.00	0.21	0.00	0.21	1.57	1.45	1.00	0.75	1.20	23	28.00	4.56	1.14	31	4.000
6.00	27	120.00	0.36	0.00	0.36	1.43	1.45	1.00	0.75	1.20	50	10.00	0.87	1.02	52	4.000
8.50	50	120.00	0.51	0.00	0.51	1.31	1.45	1.00	0.75	1.20	85	10.00	0.87	1.02	88	4.000
11.00	22	120.00	0.66	0.00	0.66	1.21	1.45	1.00	0.85	1.20	39	48.00	5.00	1.20	52	4.000
13.50	9	120.00	0.81	0.06	0.75	1.15	1.45	1.00	0.85	1.20	15	48.00	5.00	1.20	23	0.255
16.00	11	120.00	0.96	0.14	0.82	1.11	1.45	1.00	0.85	1.20	18	48.00	5.00	1.20	27	0.323
18.50	43	120.00	1.11	0.22	0.89	1.08	1.45	1.00	0.95	1.20	77	23.00	4.06	1.10	89	4.000
21.00	39	120.00	1.26	0.30	0.96	1.04	1.45	1.00	0.95	1.20	67	23.00	4.06	1.10	78	4.000
26.00	50	120.00	1.56	0.45	1.11	0.98	1.45	1.00	0.95	1.20	81	23.00	4.06	1.10	93	4.000
31.00	50	120.00	1.86	0.61	1.25	0.92	1.45	1.00	1.00	1.20	80	86.00	5.00	1.20	101	4.000
36.00	43	120.00	2.16	0.76	1.40	0.87	1.45	1.00	1.00	1.20	65	15.00	2.50	1.05	71	4.000
41.00	15	120.00	2.46	0.92	1.54	0.83	1.45	1.00	1.00	1.20	22	86.00	5.00	1.20	31	4.000
46.00	35	120.00	2.76	1.08	1.68	0.79	1.45	1.00	1.00	1.20	48	15.00	2.50	1.05	53	4.000
51.00	46	120.00	3.06	1.23	1.83	0.75	1.45	1.00	1.00	1.20	60	15.00	2.50	1.05	65	4.000

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

Depth (ft)	SPT Field Value	Unit Weight (pcf)	σ_v (tsf)	u_o (tsf)	σ'_{vo} (tsf)	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	Fines Content (%)	α	β	$(N_1)_{60cs}$	$CRR_{7.5}$

Abbreviations

σ_v: Total stress during SPT test (tsf)
u_o: Water pore pressure during SPT test (tsf)
σ'_{vo}: Effective overburden pressure during SPT test (tsf)
C_N: Overburden corretion factor
C_E: Energy correction factor
C_B: Borehole diameter correction factor
C_R: Rod length correction factor
C_S: Liner correction factor
$N_{1(60)}$: Corrected N_{SPT} to a 60% energy ratio
α, β: Clean sand equivalent clean sand formula coefficients
$N_{1(60)cs}$: Corected $N_{1(60)}$ value for fines content
$CRR_{7.5}$: 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	α	CSR	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS	
3.50	120.00	0.21	0.00	1.71	0.99	1.00	0.355	1.43	0.248	1.00	0.248	2.000	●
6.00	120.00	0.36	0.00	1.86	0.99	1.00	0.353	1.43	0.247	1.00	0.247	2.000	●
8.50	120.00	0.51	0.00	2.01	0.98	1.00	0.351	1.43	0.245	1.00	0.245	2.000	●
11.00	120.00	0.66	0.00	2.16	0.98	1.00	0.349	1.43	0.244	1.00	0.244	2.000	●
13.50	120.00	0.81	0.06	2.25	0.97	1.00	0.357	1.43	0.250	1.00	0.250	1.022	●
16.00	120.00	0.96	0.14	2.32	0.97	1.00	0.366	1.43	0.256	1.00	0.256	1.262	●
18.50	120.00	1.11	0.22	2.39	0.96	1.00	0.375	1.43	0.262	1.00	0.262	2.000	●
21.00	120.00	1.26	0.30	2.46	0.95	1.00	0.382	1.43	0.267	1.00	0.267	2.000	●
26.00	120.00	1.56	0.45	2.61	0.94	1.00	0.394	1.43	0.275	0.99	0.278	2.000	●
31.00	120.00	1.86	0.61	2.75	0.92	1.00	0.400	1.43	0.279	0.97	0.289	2.000	●
36.00	120.00	2.16	0.76	2.90	0.88	1.00	0.399	1.43	0.279	0.95	0.295	2.000	●
41.00	120.00	2.46	0.92	3.04	0.84	1.00	0.392	1.43	0.274	0.93	0.296	2.000	●
46.00	120.00	2.76	1.08	3.18	0.79	1.00	0.380	1.43	0.265	0.91	0.291	2.000	●
51.00	120.00	3.06	1.23	3.33	0.74	1.00	0.364	1.43	0.254	0.90	0.284	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
α: Improvement factor due to stone columns
CSR : Cyclic Stress Ratio (adjusted for improvement)
MSF : Magnitude Scaling Factor
$CSR_{eq,M=7.5}$: CSR adjusted for M=7.5
K_{sigma}: Effective overburden stress factor
CSR^*: CSR fully adjusted (user FS applied)***
FS: Calculated factor of safety against soil liquefaction

*** User FS: 1.00

:: Liquefaction potential according to Iwasaki ::

Depth (ft)	FS	F	wz	Thickness (ft)	I_L
3.50	2.000	0.00	9.47	2.50	0.00
6.00	2.000	0.00	9.09	2.50	0.00
8.50	2.000	0.00	8.70	2.50	0.00
11.00	2.000	0.00	8.32	2.50	0.00

:: Liquefaction potential according to Iwasaki ::

Depth (ft)	FS	F	wz	Thickness (ft)	I_L
13.50	1.022	0.00	7.94	2.50	0.00
16.00	1.262	0.00	7.56	2.50	0.00
18.50	2.000	0.00	7.18	2.50	0.00
21.00	2.000	0.00	6.80	2.50	0.00
26.00	2.000	0.00	6.04	5.00	0.00
31.00	2.000	0.00	5.28	5.00	0.00
36.00	2.000	0.00	4.51	5.00	0.00
41.00	2.000	0.00	3.75	5.00	0.00
46.00	2.000	0.00	2.99	5.00	0.00
51.00	2.000	0.00	2.23	5.00	0.00

Overall potential I_L : 0.00

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

:: Vertical settlements estimation for dry sands ::

Depth (ft)	$(N_1)_{60}$	τ_{av}	p	G_{max} (tsf)	α	b	γ	ε_{15}	N_c	ε_{Nc} (%)	Δh (ft)	ΔS (in)
3.50	23	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	3.50	0.000
6.00	50	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.50	0.000
8.50	85	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.50	0.000
11.00	39	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.50	0.000

Cumulative settlemetns: 0.000

Abbreviations

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

:: Vertical settlements estimation for saturated sands ::

Depth (ft)	D_{50} (in)	q_c/N	e_v weight factor	e_v (%)	Δh (ft)	s (in)
13.50	0.00	5.00	1.00	0.78	2.50	0.233
16.00	0.00	5.00	1.00	0.23	2.50	0.070
18.50	0.00	5.00	1.00	0.00	2.50	0.000
21.00	0.00	5.00	1.00	0.00	2.50	0.000
26.00	0.00	5.00	1.00	0.00	5.00	0.000
31.00	0.00	5.00	1.00	0.00	5.00	0.000
36.00	0.00	5.00	1.00	0.00	5.00	0.000
41.00	0.00	5.00	1.00	0.00	5.00	0.000
46.00	0.00	5.00	1.00	0.00	5.00	0.000

:: Vertical settlements estimation for saturated sands ::

Depth (ft)	D_{50} (in)	q_c/N	e_v weight factor	e_v (%)	Δh (ft)	s (in)
51.00	0.00	5.00	1.00	0.00	6.50	0.000

Cumulative settlements: 0.303

Abbreviations

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

:: Lateral displacements estimation for saturated sands ::

Depth (ft)	$(N_1)_{60}$	D_r (%)	γ_{max} (%)	d_z (ft)	LDI	LD (ft)
3.50	23	67.14	0.00	3.50	0.000	0.00
6.00	50	100.00	0.00	2.50	0.000	0.00
8.50	85	100.00	0.00	2.50	0.000	0.00
11.00	39	87.43	0.00	2.50	0.000	0.00
13.50	15	54.22	3.68	2.50	0.000	0.00
16.00	18	59.40	1.28	2.50	0.000	0.00
18.50	77	100.00	0.00	2.50	0.000	0.00
21.00	67	100.00	0.00	2.50	0.000	0.00
26.00	81	100.00	0.00	5.00	0.000	0.00
31.00	80	100.00	0.00	5.00	0.000	0.00
36.00	65	100.00	0.00	5.00	0.000	0.00
41.00	22	65.67	0.00	5.00	0.000	0.00
46.00	48	100.00	0.00	5.00	0.000	0.00
51.00	60	100.00	0.00	6.50	0.000	0.00

Cumulative lateral displacements: 0.00

Abbreviations

D_r: Relative density (%)
γ_{max}: Maximum amplitude of cyclic shear strain (%)
d_z: Soil layer thickness (ft)
LDI: Lateral displacement index (ft)
LD: Actual estimated displacement (ft)

Wood Rodgers, Inc.

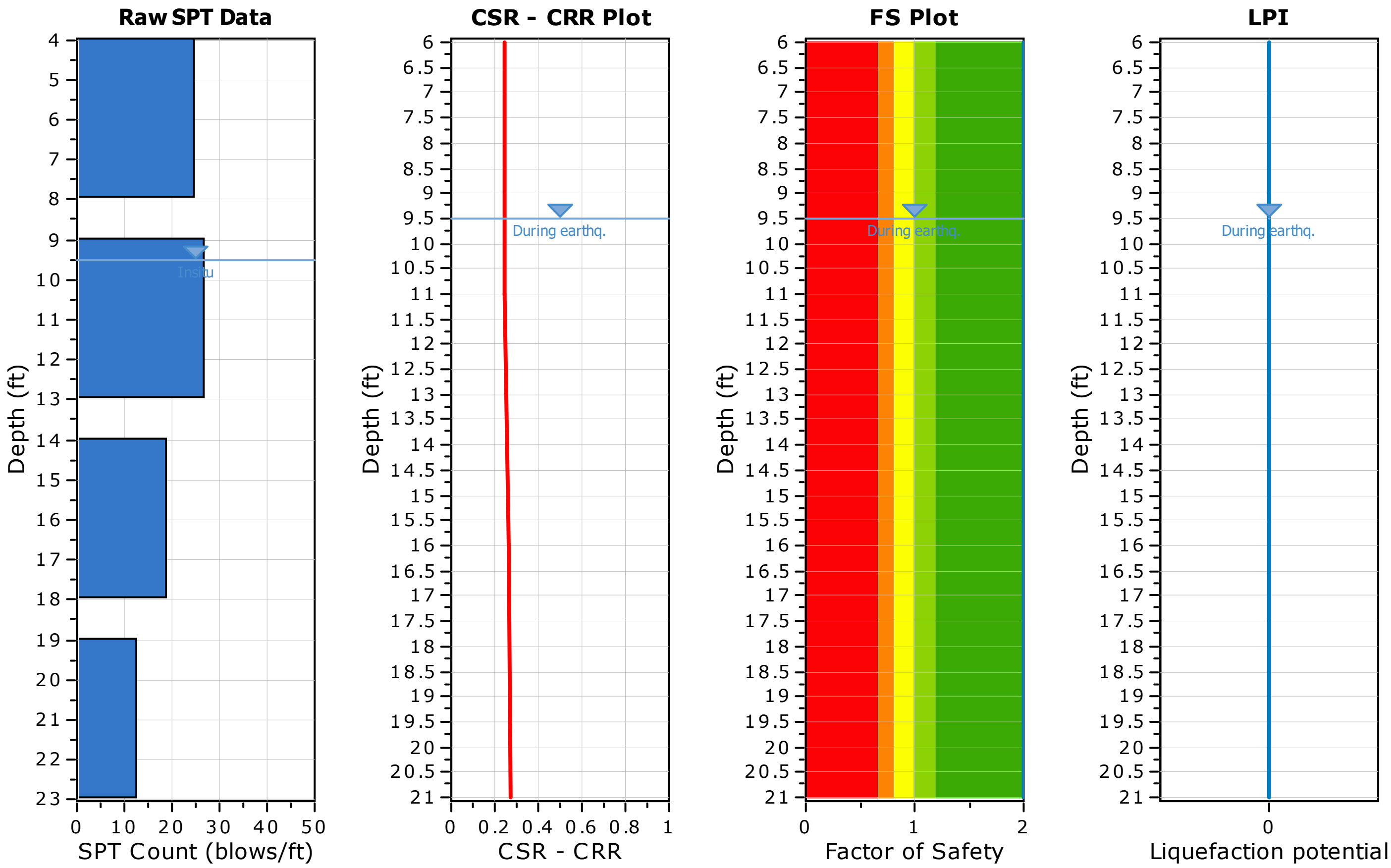
SPT BASED LIQUEFACTION ANALYSIS REPORT

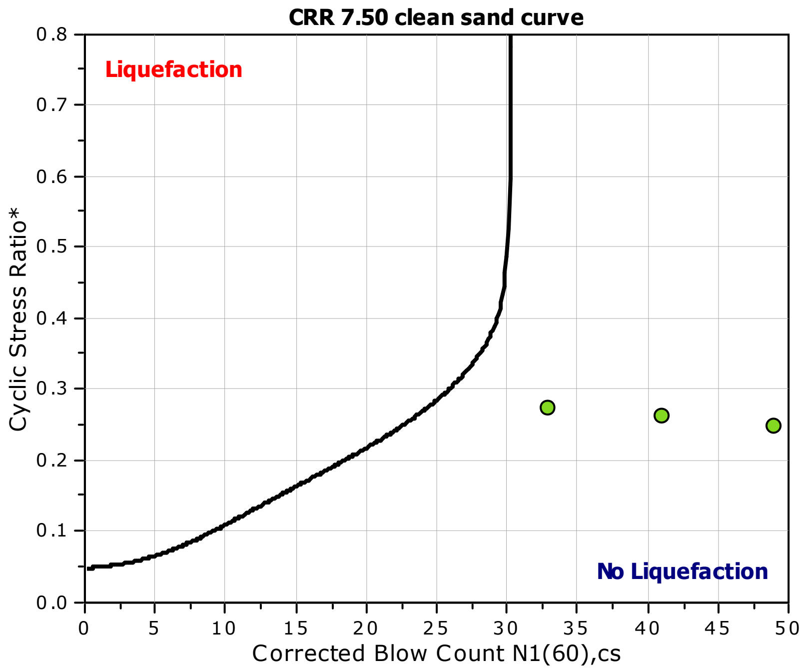
Project title : Pioneer Meadows Fire Station **SPT Name: B-3**

Location : Reno, NV

:: Input parameters and analysis properties ::

Analysis method:	NCEER 1998	G.W.T. (in-situ):	9.50 ft
Fines correction method:	NCEER 1998	G.W.T. (earthq.):	9.50 ft
Sampling method:	Sampler wo liners	Earthquake magnitude M_w:	6.52
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.55 g
Rod length:	3.28 ft	Eq. external load:	1.50 tsf
Hammer energy ratio:	1.45		





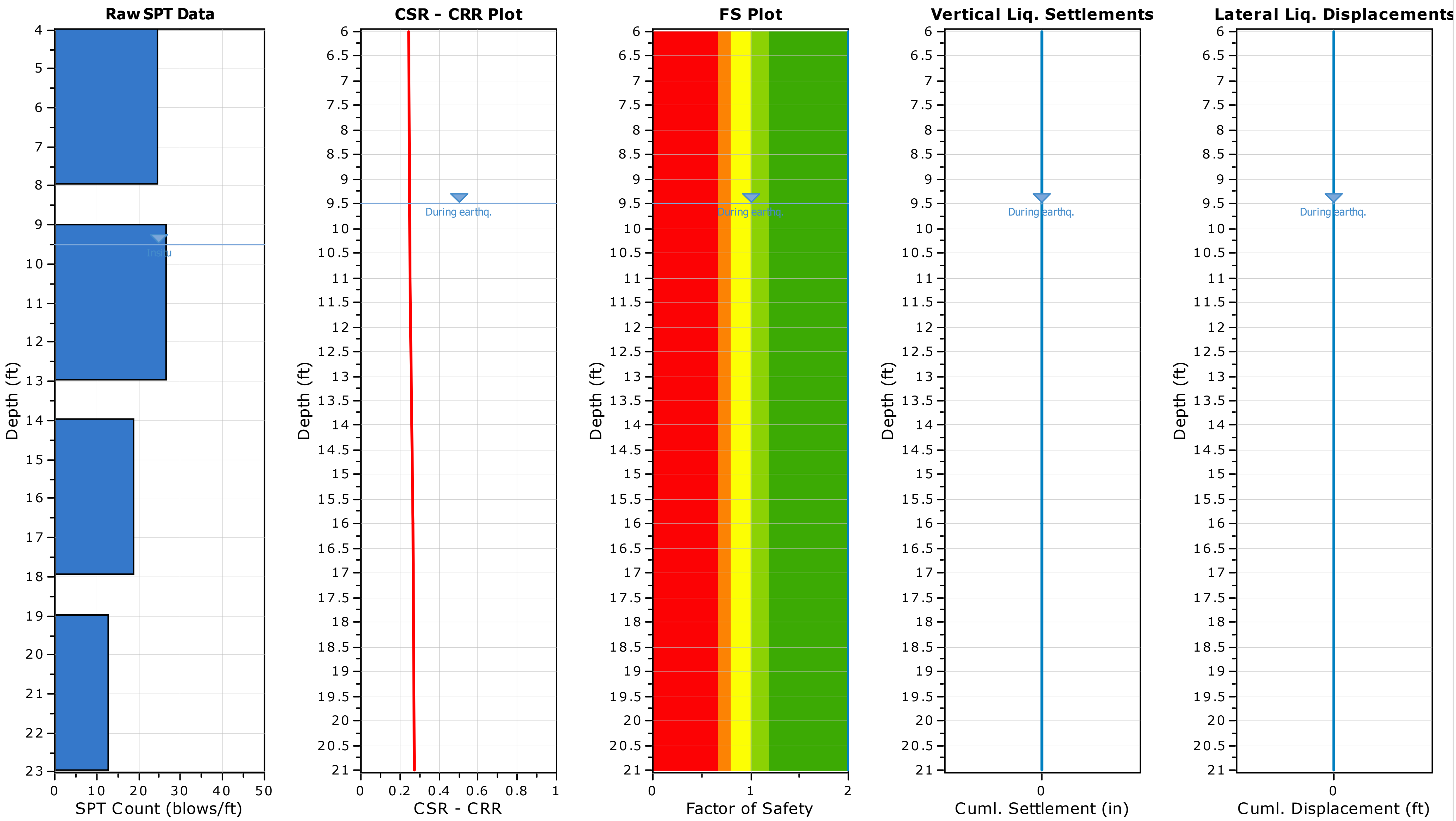
F.S. color scheme

- 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
6.00	25	10.00	120.00	6.00	Yes
11.00	27	27.00	120.00	5.00	Yes
16.00	19	27.00	120.00	5.00	Yes
21.00	13	70.00	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)	σ_v (tsf)	u_o (tsf)	σ'_{vo} (tsf)	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	Fines Content (%)	α	β	$(N_1)_{60cs}$	$CRR_{7.5}$
6.00	25	120.00	0.36	0.00	0.36	1.43	1.45	1.00	0.75	1.20	47	10.00	0.87	1.02	49	4.000
11.00	27	120.00	0.66	0.05	0.61	1.24	1.45	1.00	0.85	1.20	49	27.00	4.48	1.13	60	4.000
16.00	19	120.00	0.96	0.20	0.76	1.15	1.45	1.00	0.85	1.20	32	27.00	4.48	1.13	41	4.000
21.00	13	120.00	1.26	0.36	0.90	1.07	1.45	1.00	0.95	1.20	23	70.00	5.00	1.20	33	4.000

Abbreviations

σ_v: Total stress during SPT test (tsf)
u_o: Water pore pressure during SPT test (tsf)
σ'_{vo}: Effective overburden pressure during SPT test (tsf)
C_N: Overburden corretion factor
C_E: Energy correction factor
C_B: Borehole diameter correction factor
C_R: Rod length correction factor
C_S: Liner correction factor
$N_{1(60)}$: Corrected N_{SPT} to a 60% energy ratio
α, β: Clean sand equivalent clean sand formula coefficients
$N_{1(60)cs}$: Corected $N_{1(60)}$ value for fines content
$CRR_{7.5}$: 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	α	CSR	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS	
6.00	120.00	0.36	0.00	1.86	0.99	1.00	0.353	1.43	0.247	1.00	0.247	2.000	●
11.00	120.00	0.66	0.05	2.11	0.98	1.00	0.357	1.43	0.250	1.00	0.250	2.000	●
16.00	120.00	0.96	0.20	2.26	0.97	1.00	0.377	1.43	0.263	1.00	0.263	2.000	●
21.00	120.00	1.26	0.36	2.40	0.95	1.00	0.392	1.43	0.274	1.00	0.274	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
α: Improvement factor due to stone columns
CSR : Cyclic Stress Ratio (adjusted for improvement)
MSF : Magnitude Scaling Factor
$CSR_{eq,M=7.5}$: CSR adjusted for M=7.5
K_{sigma}: Effective overburden stress factor
CSR^*: CSR fully adjusted (user FS applied)***
FS: Calculated factor of safety against soil liquefaction

*** User FS: 1.00

:: Liquefaction potential according to Iwasaki ::

Depth (ft)	FS	F	wz	Thickness (ft)	I_L
6.00	2.000	0.00	9.09	5.00	0.00
11.00	2.000	0.00	8.32	5.00	0.00
16.00	2.000	0.00	7.56	5.00	0.00
21.00	2.000	0.00	6.80	5.00	0.00

Overall potential I_L : 0.00

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

:: Vertical settlements estimation for dry sands ::

Depth (ft)	$(N_1)_{60}$	τ_{av}	p	G_{max} (tsf)	α	b	γ	ε_{15}	N_c	ε_{Nc} (%)	Δh (ft)	ΔS (in)
6.00	47	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	6.00	0.000

Cumulative settlemetns: 0.000

Abbreviations

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

:: Vertical settlements estimation for saturated sands ::

Depth (ft)	D_{50} (in)	q_c/N	e_v weight factor	e_v (%)	Δh (ft)	s (in)
11.00	0.00	5.00	1.00	0.00	5.00	0.000
16.00	0.00	5.00	1.00	0.00	5.00	0.000
21.00	0.00	5.00	1.00	0.00	5.00	0.000

Cumulative settlements: 0.000

Abbreviations

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

:: Lateral displacements estimation for saturated sands ::

Depth (ft)	$(N_1)_{60}$	D_r (%)	γ_{max} (%)	d_z (ft)	LDI	LD (ft)
6.00	47	100.00	0.00	6.00	0.000	0.00
11.00	49	100.00	0.00	5.00	0.000	0.00
16.00	32	79.20	0.00	5.00	0.000	0.00

:: Lateral displacements estimation for saturated sands ::

Depth (ft)	$(N_1)_{60}$	D_r (%)	γ_{max} (%)	d_z (ft)	LDI	LD (ft)
21.00	23	67.14	0.00	5.00	0.000	0.00

Cumulative lateral displacements: 0.00

Abbreviations

D_r: Relative density (%)
γ_{max}: Maximum amplitude of cyclic shear strain (%)
d_z: Soil layer thickness (ft)
LDI: Lateral displacement index (ft)
LD: Actual estimated displacement (ft)

Wood Rodgers, Inc.

GEOLOGISMIKI Geotechnical Software

SPT BASED LIQUEFACTION ANALYSIS REPORT

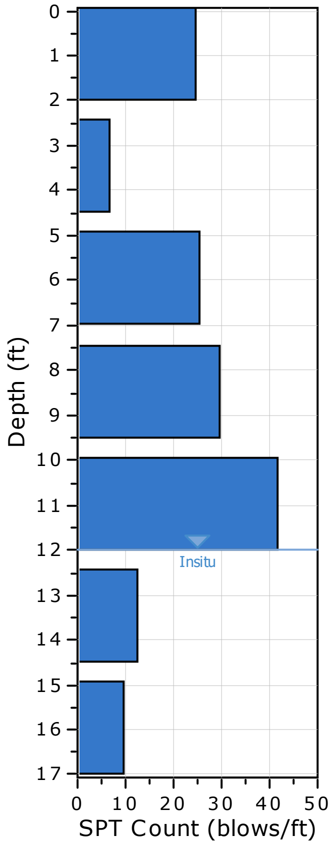
Project title : Pioneer Meadows Fire Station **SPT Name: B-3 (2022)**

Location : Reno, NV

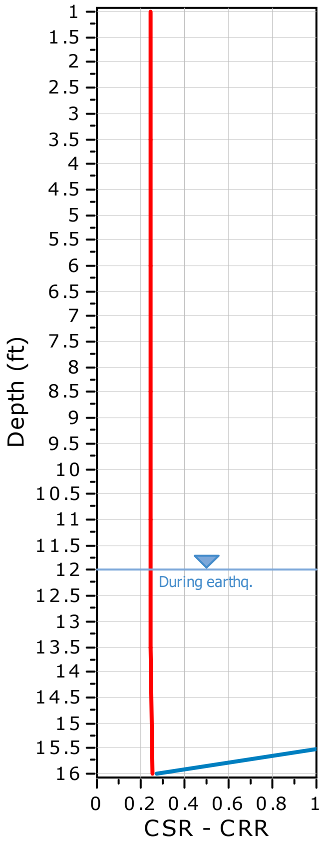
:: Input parameters and analysis properties ::

Analysis method:	NCEER 1998	G.W.T. (in-situ):	12.00 ft
Fines correction method:	NCEER 1998	G.W.T. (earthq.):	12.00 ft
Sampling method:	Sampler wo liners	Earthquake magnitude M_w:	6.52
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.55 g
Rod length:	3.28 ft	Eq. external load:	1.50 tsf
Hammer energy ratio:	1.45		

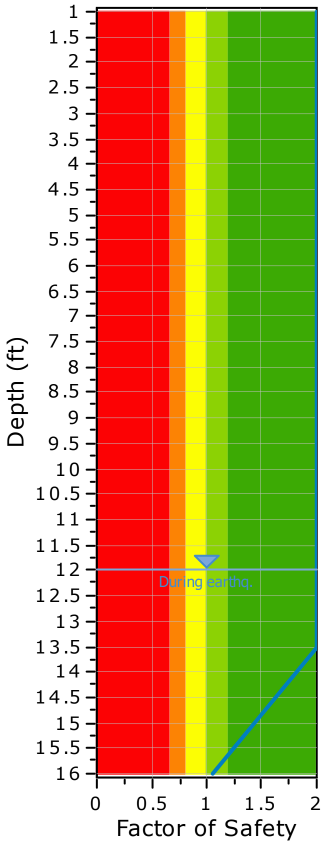
Raw SPT Data



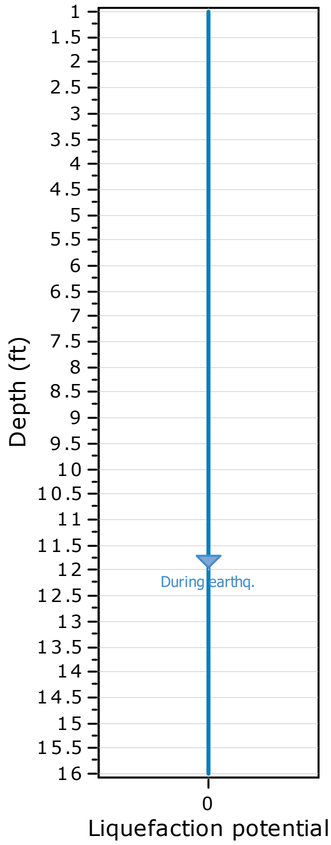
CSR - CRR Plot



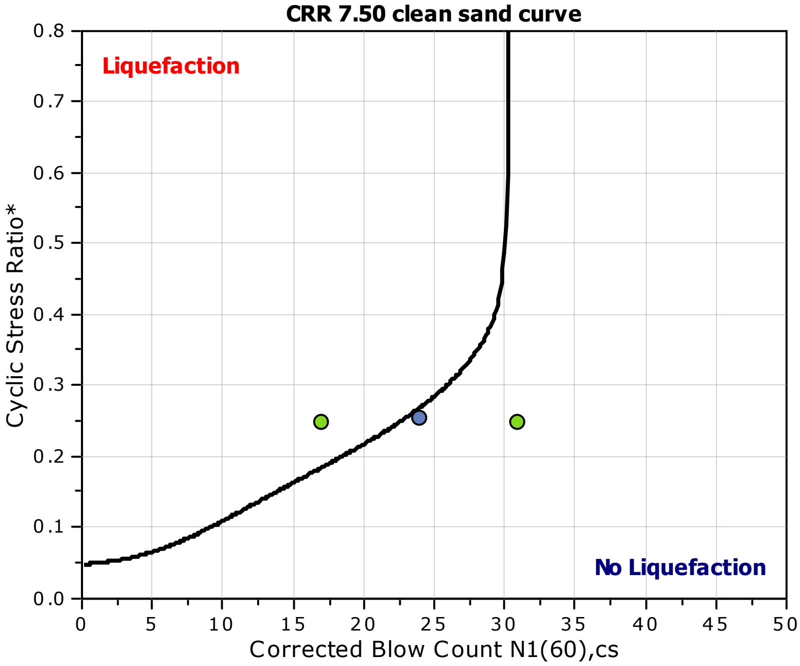
FS Plot



LPI



CRR 7.50 clean sand curve



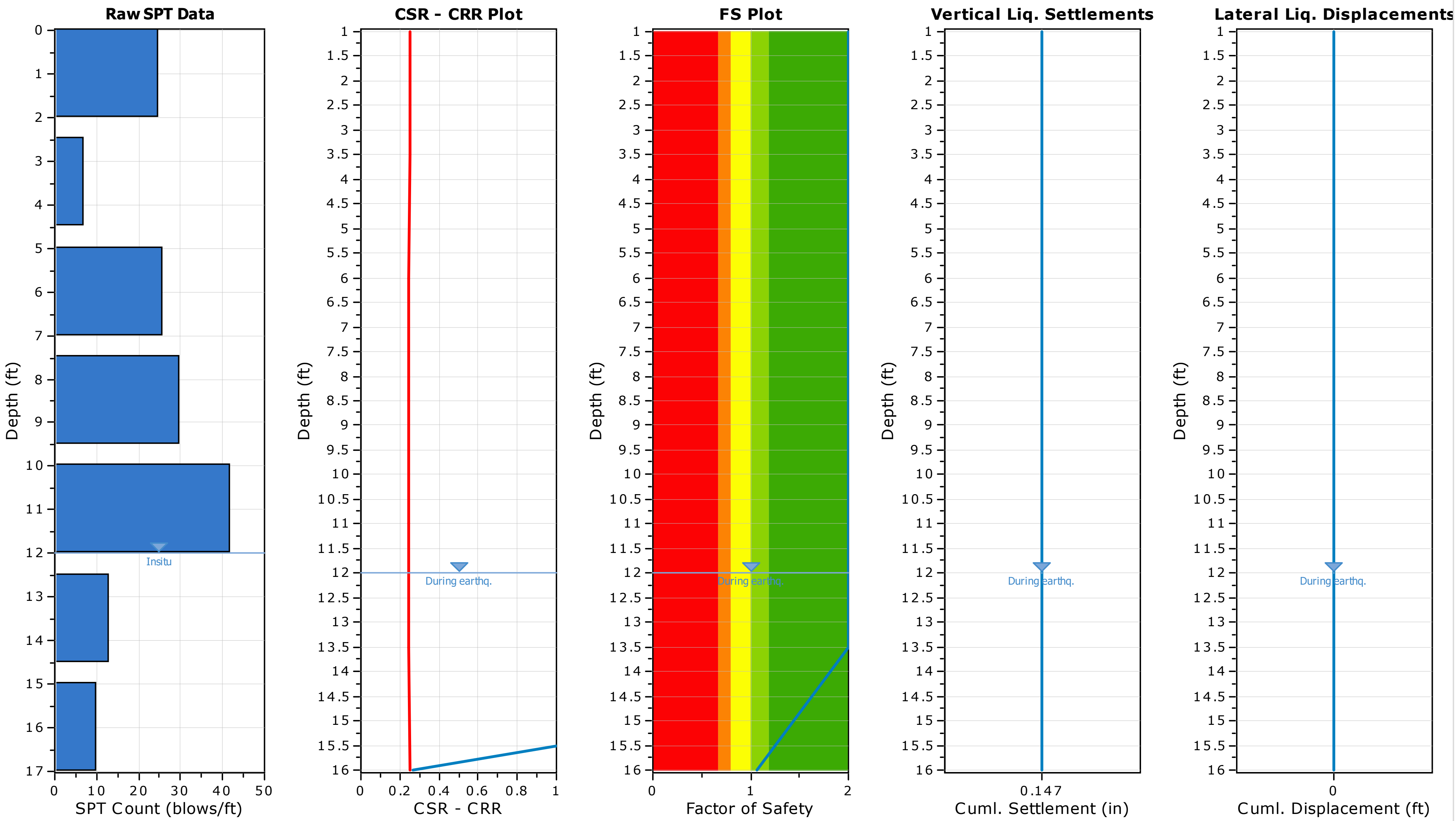
F.S. color scheme

- 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
1.00	25	15.00	120.00	2.50	Yes
3.50	7	15.00	120.00	2.50	Yes
6.00	26	15.00	120.00	2.50	Yes
8.50	30	15.00	120.00	2.50	Yes
11.00	42	15.00	120.00	2.50	Yes
13.50	13	70.00	120.00	2.50	Yes
16.00	10	70.00	120.00	2.50	Yes

Abbreviations

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

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

Depth (ft)	SPT Field Value	Unit Weight (pcf)	σ_v (tsf)	u_o (tsf)	σ'_{vo} (tsf)	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	Fines Content (%)	α	β	$(N_1)_{60cs}$	$CRR_{7.5}$
1.00	25	120.00	0.06	0.00	0.06	1.70	1.45	1.00	0.75	1.20	55	15.00	2.50	1.05	60	4.000
3.50	7	120.00	0.21	0.00	0.21	1.57	1.45	1.00	0.75	1.20	14	15.00	2.50	1.05	17	4.000
6.00	26	120.00	0.36	0.00	0.36	1.43	1.45	1.00	0.75	1.20	48	15.00	2.50	1.05	53	4.000
8.50	30	120.00	0.51	0.00	0.51	1.31	1.45	1.00	0.75	1.20	51	15.00	2.50	1.05	56	4.000
11.00	42	120.00	0.66	0.00	0.66	1.21	1.45	1.00	0.85	1.20	75	15.00	2.50	1.05	81	4.000
13.50	13	120.00	0.81	0.05	0.76	1.15	1.45	1.00	0.85	1.20	22	70.00	5.00	1.20	31	4.000
16.00	10	120.00	0.96	0.12	0.84	1.11	1.45	1.00	0.85	1.20	16	70.00	5.00	1.20	24	0.269

Abbreviations

σ_v: Total stress during SPT test (tsf)
u_o: Water pore pressure during SPT test (tsf)
σ'_{vo}: Effective overburden pressure during SPT test (tsf)
C_N: Overburden corretion factor
C_E: Energy correction factor
C_B: Borehole diameter correction factor
C_R: Rod length correction factor
C_S: Liner correction factor
$N_{1(60)}$: Corrected N_{SPT} to a 60% energy ratio
α, β: Clean sand equivalent clean sand formula coefficients
$N_{1(60)cs}$: Corected $N_{1(60)}$ value for fines content
$CRR_{7.5}$: 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	α	CSR	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS
1.00	120.00	0.06	0.00	1.56	1.00	1.00	0.357	1.43	0.250	1.00	0.250	2.000
3.50	120.00	0.21	0.00	1.71	0.99	1.00	0.355	1.43	0.248	1.00	0.248	2.000
6.00	120.00	0.36	0.00	1.86	0.99	1.00	0.353	1.43	0.247	1.00	0.247	2.000
8.50	120.00	0.51	0.00	2.01	0.98	1.00	0.351	1.43	0.245	1.00	0.245	2.000
11.00	120.00	0.66	0.00	2.16	0.98	1.00	0.349	1.43	0.244	1.00	0.244	2.000
13.50	120.00	0.81	0.05	2.26	0.97	1.00	0.355	1.43	0.248	1.00	0.248	2.000
16.00	120.00	0.96	0.12	2.34	0.97	1.00	0.364	1.43	0.254	1.00	0.254	1.059

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

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

Abbreviations

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

*** User FS: 1.00

:: Liquefaction potential according to Iwasaki ::

Depth (ft)	FS	F	wz	Thickness (ft)	I_L
1.00	2.000	0.00	9.85	2.50	0.00
3.50	2.000	0.00	9.47	2.50	0.00
6.00	2.000	0.00	9.09	2.50	0.00
8.50	2.000	0.00	8.70	2.50	0.00
11.00	2.000	0.00	8.32	2.50	0.00
13.50	2.000	0.00	7.94	2.50	0.00
16.00	1.059	0.00	7.56	2.50	0.00

Overall potential I_L : 0.00

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

:: Vertical settlements estimation for dry sands ::

Depth (ft)	$(N_1)_{60}$	τ_{av}	p	G_{max} (tsf)	α	b	γ	ε_{15}	N_c	ε_{Nc} (%)	Δh (ft)	ΔS (in)
1.00	55	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.50	0.000
3.50	14	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.50	0.000
6.00	48	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.50	0.000
8.50	51	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.50	0.000
11.00	75	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.50	0.000

Cumulative settlemetns: 0.000

Abbreviations

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

:: Vertical settlements estimation for saturated sands ::

Depth (ft)	D_{50} (in)	q_c/N	e_v weight factor	e_v (%)	Δh (ft)	s (in)
13.50	0.00	5.00	1.00	0.00	2.50	0.000
16.00	0.00	5.00	1.00	0.49	2.50	0.147

Cumulative settlements: 0.147

Abbreviations

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

:: Lateral displacements estimation for saturated sands ::

Depth (ft)	$(N_1)_{60}$	D_r (%)	γ_{max} (%)	d_z (ft)	LDI	LD (ft)
1.00	55	100.00	0.00	2.50	0.000	0.00
3.50	14	52.38	0.00	2.50	0.000	0.00
6.00	48	100.00	0.00	2.50	0.000	0.00
8.50	51	100.00	0.00	2.50	0.000	0.00
11.00	75	100.00	0.00	2.50	0.000	0.00
13.50	22	65.67	0.00	2.50	0.000	0.00
16.00	16	56.00	2.78	2.50	0.000	0.00

Cumulative lateral displacements: 0.00

Abbreviations

D_r: Relative density (%)
γ_{max}: Maximum amplitude of cyclic shear strain (%)
d_z: Soil layer thickness (ft)
LDI: Lateral displacement index (ft)
LD: Actual estimated displacement (ft)

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APPENDIX F
MASS GRADING CERTIFICATION

WOOD RODGERS

February 8, 2012
Project No. 1509.056

Mr. Tim Scheideman
Operations Manager
LENNAR RENO, LLC
10345 Professional Circle, Suite 100
Reno, Nevada 89521

RE: PIONEER MEADOWS
MASS GRADING CERTIFICATION

Dear Mr. Scheideman:

Wood Rodgers provided construction observation and field density testing during mass grading of the referenced project. The project consisted of excavating the borrow material from the on site Pioneer Meadows Pit and placing the material in Pioneer Meadows Villages 7B & 7C, Village 5, Village 6, Village 11, Village 12 and the future Pioneer Meadows Business park. The materials placed met or exceeded 90% relative compaction when tested in accordance with ASTM D 6938. The material was placed roughly to the grades as shown on the Wood Rodgers Grading Cut Plan, dated June, 2011.

The engineered fill primarily consisted of materials whose properties consisted of less than 30% passing the number 200 sieve and a maximum plastic index of 15. Because the material was obtained from a noncommercial source, due care was exercised during borrow excavation to attempt to segregate clay rich zones from the interbedded granular soils. The segregated clay material was placed in nonstructural areas. However, due to limitations inherent in mass grading operations, the engineered fills supporting the planned improvements may possess isolated clay rich pockets or zones. The presence of these pockets or zones should be considered part of the overall fill structure and their presence does not equate to an inadequate or poorly constructed fill. The potential for isolated pockets of moisture conditioned and compacted clay rich zones should be considered by the geotechnical engineer as a condition upon which he must base his analyses and final designs.

Please contact our office should you have any related questions or comments.

Sincerely,

WOOD RODGERS, INCORPORATED

James G. Smith, P.E.
Principal

JGS:dh