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GEOTECHNICAL INVESTIGATION-DESIGN PHASE

Proposed Steel Building
1400 Ranport Road
Watsonville, Santa Cruz County, California
A.P.N.: 052-511-08

For:
David Joseph
193 Vega Road
Watsonville, California 95076

Project No. 14047
November 10, 2014

1100 Main Street, Suite A, Watsonville, CA 95076 • (831) 724-5868 • Fax: (831) 763-1578 • Email: rocksolid@cruzio.com

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193 Vega Road
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SUBJECT: **GEOTECHNICAL INVESTIGATION - DESIGN PHASE**
Proposed Steel Building
1400 Ranport Road, Watsonville, Santa Cruz County, California
APN 052-511-08

Dear Mr. Joseph:

In accordance with your authorization, we have completed a geotechnical investigation for the proposed steel building at 1400 Ranport Road, in Watsonville, California. This report summarizes the findings, conclusions, and recommendations from our field exploration, laboratory testing, and engineering analysis. The conclusions and recommendations included herein are based upon applicable standards at the time this report was prepared.

It is a pleasure being associated with you on this project. If you have any questions, or if we may be of further assistance, please do not hesitate to contact our office.

Sincerely,

ROCK SOLID ENGINEERING, INC.

Signed: 11/17/2014

Yvette M. Wilson, P.E.
Principal Engineer
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Staff Engineer

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TABLE OF CONTENTS

1. **INTRODUCTION** 1
   - 1.1 Purpose 1
   - 1.2 Proposed Development 1
   - 1.3 Scope of Services 1
   - 1.4 Authorization 2
   - 1.5 Exclusions 2

2. **FIELD EXPLORATION AND LABORATORY TESTING PROGRAM** 2

3. **SITE DESCRIPTION** 2
   - 3.1 Location 2
   - 3.2 Surface Conditions 2
   - 3.3 Subsurface Conditions 2

4. **GEOTECHNICAL HAZARDS** 3
   - 4.1 General 3
   - 4.2 Liquefaction Analysis 4
     - 4.2.1 General 4
     - 4.2.2 Results 5
     - 4.2.3 Discussion 5

5. **CONCLUSIONS AND RECOMMENDATIONS** 6
   - 5.1 General 6
   - 5.2 Grading 7
     - 5.2.1 General 7
     - 5.2.2 Site Clearing 7
     - 5.2.3 Excavating Conditions 8
     - 5.2.4 Fill Material 8
     - 5.2.5 Fill Placement and Compaction 8
     - 5.2.6 Preparation of On-Site Soils 9
     - 5.2.7 Groundwater Table 10
     - 5.2.8 Expansive Soils 10
     - 5.2.9 Sulfate Content 10
     - 5.2.10 Resistivity 10
     - 5.2.11 Surface Drainage 11
     - 5.2.12 Utility Trenches 11
   - 5.3 Foundations 12
     - 5.3.1 General 12
     - 5.3.2 Helical Screw Piles 12
   - 5.4 Settlements 13
   - 5.5 Slabs-on-Grade 14

6. **LIMITATIONS** 15

REFERENCES 16

Appendix A: Field Exploration and Laboratory Testing Program
Appendix B: Liquefaction Analysis

1. INTRODUCTION

1.1 Purpose

The purpose of our investigation is to provide preliminary geotechnical design parameters and recommendations for development of the site. Conclusions and recommendations related to site grading and foundations are presented herein.

1.2 Proposed Development

a. Based on our conversations with you, it is our understanding that the project consists of the construction of a new prefabricated steel building along the southwest side of the subject site.

b. The subject site is comprised of two parcels APN 052-511-08 and 052-511-06. At your request we did a subsurface investigation on both parcels, in the event that one parcel was deemed un-buildable. The site which is intended for construction is APN 052-511-08, this site is considered suitable from a geotechnical prospective. The boring logs for the APN 052-511-06 are included in the appendix, however we will not address this site further in this report.

c. Anticipated construction consists of prefabricated steel construction supported at columns. The structure will either have earthen or concrete slab-on-grade floors. Exact wall, column, and foundation loads are unavailable, but are expected to be typical of such construction.

d. Final grading and foundation plans were unavailable at the time of this report. It is our understanding that the information obtained during our investigation will be used in the development of a finalized plan set.

1.3 Scope of Services

The scope of services provided during the course of our investigation included:

a. Review of the referenced geotechnical, geologic, and seismological reports and maps pertinent to the development of the site (available in our files).

b. Field exploration consisting of 5 borings, drilled to depths between 10 and 50 feet below existing grade in the area of the proposed development.

c. Logging and sampling of the borings by our Field Engineer, including the collection of soil samples for laboratory testing.

d. Laboratory testing of soil samples considered representative of subsurface conditions.

e. Geotechnical analyses of field and laboratory data.

f. Quantitative liquefaction analysis.

g. Preparation of a report (6 copies) presenting our findings, conclusions and recommendations.

1.4 Authorization

This investigation, as outlined in our Proposal dated August15, 2014, was performed in accordance with your written authorization on September 9, 2014.

1.5 Exclusions

Our services on this project are limited to the proposed steel building. Our services specifically exclude issues related to the overall stability of the parcel and adjacent parcels, and excludes all existing structures, foundations, retaining walls, driveways and other improvements.

2. FIELD EXPLORATION AND LABORATORY TESTING PROGRAM

Details of the field exploration and laboratory testing are presented in Appendix A.

3. SITE DESCRIPTION

3.1 Location

The subject project is located at 1400 Ranport Road, in Watsonville, Santa Cruz County, California. The location is shown on the Location Map, **Figure 1**.

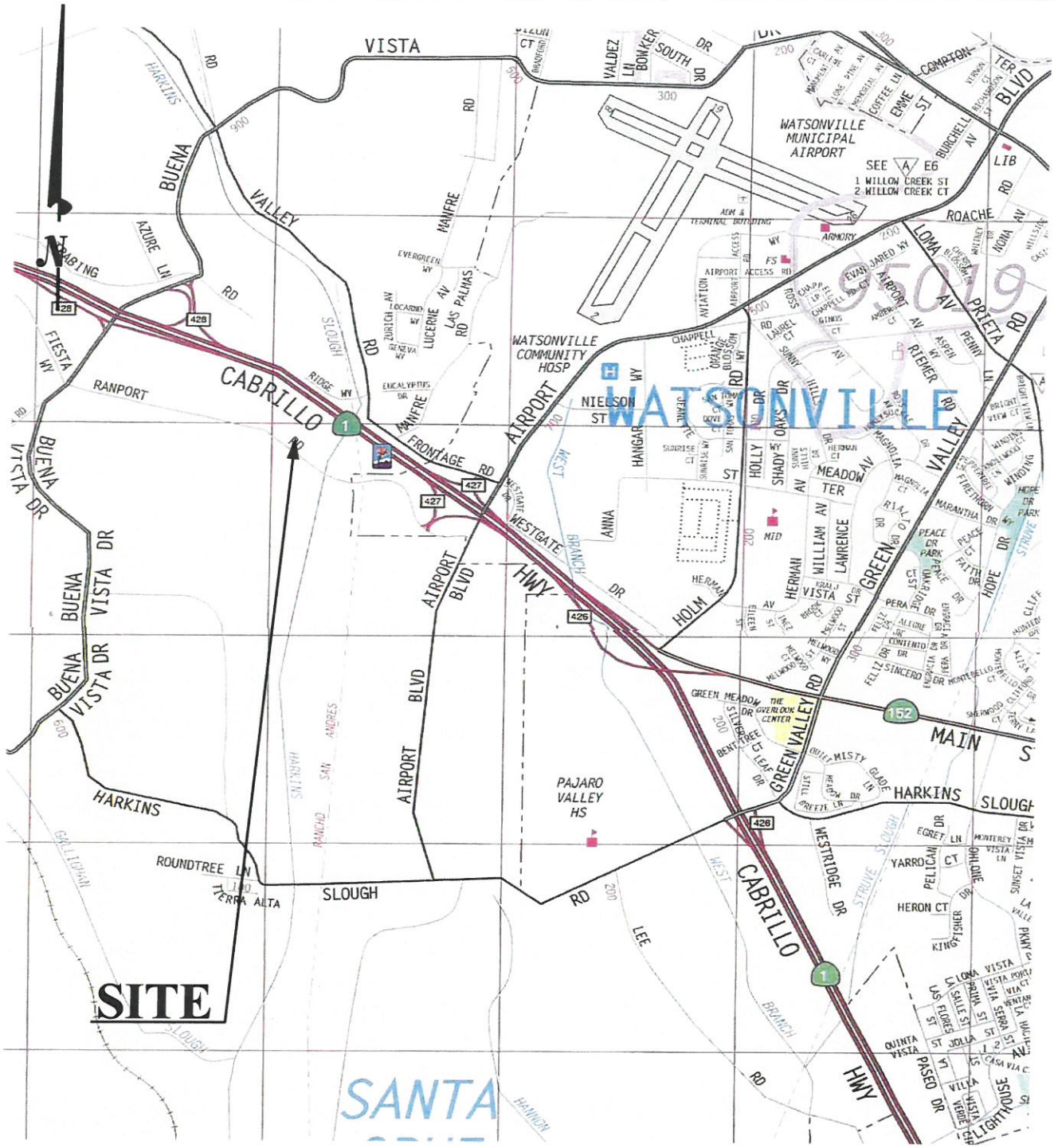
3.2 Surface Conditions

The subject site is located in a valley adjacent to Harkins Slough. The site slopes gently toward the east and toward Harkins Slough. The proposed building pad is currently used to stockpile trees and is currently not developed with any structures.

It is our understanding that the new structures will not be located in the flood zone of the Harkins Slough.

3.3 Subsurface Conditions

a. The results of our field exploration indicate that the subsurface soils present on the site are relatively consistent, however, there are variations in color, moisture content, and density.



REFERENCE: The Thomas Guide, 2004.

ROCK SOLID ENGINEERING, INC.	LOCATION MAP	FIGURE
	1400 Ranport Road, Watsonville	1

b. **Perched groundwater was encountered during the course of our field exploration at 15 and 30 feet below existing grade.**

c. The upper stratum is fill, consisting of orange brown silty to clayey sand. The fill was observed from the surface to between 1.5 and 2.5 feet below existing grade. This material is generally dry to wet, medium dense, and non-plastic.

d. Underlying the fill stratum, black and grey laminated clayey sand and sandy clay with organics is present. The laminated clay and sand was observed to between 11.5 and 12 feet below existing grade. This material is generally moist, loose to very stiff, and medium plastic. This stratum consists of mottled sand, clay and organic material which may or may not be fill.

e. Underlying the black and grey clay and sand stratum, peat is present. The peat was observed to between 17 and 20 feet below existing grade.

f. Underlying the peat stratum grey clayey sand to sand with fines is present. The clayey sand to sand with fines was observed to the extent of our boring at approximately 50 feet below existing grade. This material is generally moist to saturated, loose to very dense, and non- to medium plastic.

g. Complete soil profiles are presented on the Logs of Exploratory Borings and the boring locations are shown on the Boring Location Plan in Appendix A.

4. GEOTECHNICAL HAZARDS

4.1 General

a. Potential geotechnical hazards to man made structures include ground shaking, surface rupture, landsliding, liquefaction, lateral spreading, and differential compaction. The potential for each of these to impact the site is discussed below.

b. Ground shaking caused by earthquakes is a complex phenomenon. Structural damage can result from the transmission of earthquake vibrations from the ground into the structure. The intensity of an earthquake at any given site depends on many variables including, the proximity of the site to the hypocenter, and the characteristics of the underlying soil and/or rock. The subject site is situated at the approximate latitude of 36°59' 57" and longitude -121°59' 18". The project location (latitude and longitude) were used in conjunction with the U.S. Geologic Survey website (Reference 10) to obtain the seismic design parameters presented in **Table 1**. All proposed structures at the subject site shall be designed with the corresponding seismic design parameters in accordance with the 2013 California Building Code (Reference 2).

Table 1
2013 CBC Seismic Design Criteria

SEISMIC DESIGN CRITERIA							
Site Class	Seismic Design Category	Spectral Response Accelerations					
		Ss	S1	SMs	SM1	SDs	SD1
E	D	1.766	0.685	1.589	1.643	1.059	1.096

c. Surface rupture usually occurs along lines of previous faulting. Based on our review of the Faults and Their Potential Hazards in Santa Cruz County map (Reference 5), no faults are shown to cross the property. Therefore, the potential for surface rupture should be considered low.

d. Landslides are generally mass movements of loose rock and soil, both dry and water saturated, and usually gravity driven. Based on our review of the Preliminary Map of Landslide Deposits if Santa Cruz County (Reference 3), no landslides are mapped on the subject parcel. In addition, the building pad is relatively level, therefore, the potential for landsliding to occur across the building pad a cause damage to structures should be considered low.

e. Liquefaction, lateral spreading, and differential compaction tend to occur in loose, unconsolidated, noncohesive soils with shallow groundwater. During our field exploration relatively loose, non-cohesive soils below the groundwater level were observed and a quantitative liquefaction analysis was deemed necessary. The results of our analysis are presented in Section 4.2 of this report, and the methodology and calculations are presented in Appendix B. The end bearing foundation anchors proposed for the steel building foundation will be located at a depth sufficient to be below the potentially liquefiable material on the site. Therefore the potential for damage, due to liquefaction, to the structure should be considered low.

4.2 Liquefaction Analysis

4.2.1 General

a. The liquefaction analysis uses empirical predictions of earthquake-induced liquefaction potential and is based on the published methods used by Seed and others (Reference 9).

b. The clayey sand and sand with fines stratum encountered below the groundwater table was generally characteristic of potentially liquefiable soil. The soil is composed of poorly graded sand with varying amounts of fines. The sand was observed from approximately 17 feet to approximately 47 feet below existing grade.

c. During the course of our field investigation, perched groundwater table was located at approximately 15 and 30 feet below existing grade in the area of the proposed development. Our quantitative liquefaction analysis conservatively reflects the groundwater elevation at a depth of 10 feet to account for a rise in groundwater during the wetter winter months.

4.2.2 Results

a. The results of our quantitative liquefaction analysis indicate that the underlying clayey sand and sand with fines situated below the groundwater level to a depth of approximately 17 feet below existing grade is **susceptible** to liquefaction during the design seismic event.

b. We have calculated the resulting surface deformation due to liquefaction during the design seismic event to be approximately 2.25 inches for structures founded above the liquefiable layers. This settlement can occur beneath the entire structure, or differentially, across the least dimension of the structure. The liquefaction calculations are presented in Appendix B.

4.2.3 Discussion

a. It must be cautioned that liquefaction analysis is an inexact science and the empirical predictions of earthquake-induced liquefaction potential are based on a comparison of the subject site with areas that have experienced liquefaction. The soil configuration analyzed contains many simplifying assumptions, not the least of which are isotropy and homogeneity. Soil stratums deemed "susceptible" to liquefaction during the design seismic event will not necessarily liquefy, but the probability will be greater than a stratum deemed "not susceptible".

b. Significant variations in the proposed grades may require that our analysis and the recommendations herein be reviewed and if necessary, amended.

c. Further discussion of our liquefaction analysis, methodology, and calculations are presented in Appendix B.

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 General

a. Based on the results of our investigation, it is our opinion that from the geotechnical standpoint, the subject site will be suitable for the proposed development provided the recommendations presented herein are implemented during grading and construction.

b. It is our opinion that the steel building may be supported by a **foundation system composed of helical screw piles** founded below the liquefiable stratum and peat layers. Recommendations for these foundation systems are provided in Section 5.3, Foundations.

c. It is our understanding that the proposed use of the structure as a lumber mill will allow for an earthen floor between the column supports. A significant layer of peat was encountered underlying the building pad. The peat layer may cause significant settlement of concrete slab floors. Should concrete slab-on-grade floors be required, the slab should be designed as structurally separate from the foundation system (floating) or to span all loads to the foundations (assume no soil support). Floating slabs should be expected to experience settlement and/or cracking. However, provided the slab is not structurally required then the settlement and cracking will not effect the structural integrity of the steel building. See section 5.5 Slabs-on-grade for further recommendations.

d. At the time we prepared this report, grading and foundation plans had not been finalized. We request an opportunity to review these plans during the design stages to determine if supplemental recommendations will be necessary.

e. The design recommendations of this report must be reviewed during the grading phase when subsurface conditions in the excavations become exposed.

f. **Field observation and testing must be provided by a representative of Rock Solid Engineering, Inc.**, to enable them to form an opinion regarding the adequacy of the site preparation, and the extent to which the earthwork is performed in accordance with the geotechnical conditions present, the requirements of the regulating agencies, the project specifications and the recommendations presented in this report. Any earthwork performed in connection with the subject project without the full knowledge of, and not under the direct observation of Rock Solid Engineering, Inc., the Geotechnical Consultant, will render the recommendations of this report invalid.

g. **The Geotechnical Consultant should be notified at least five (5) working days prior to any site clearing or other earthwork operations** on the subject project in order to observe the stripping and disposal of unsuitable materials and to ensure coordination with the grading contractor. During this period, a preconstruction conference should be held on the site to discuss project specifications, observation/testing requirements and responsibilities, and scheduling. This conference should include at least the Grading Contractor, the Architect, and the Geotechnical Consultant.

5.2 Grading

5.2.1 General

All grading and earthwork should be performed in accordance with the recommendations presented herein and the requirements of the regulating agencies.

5.2.2 Site Clearing

a. Prior to grading, the areas to be developed for structures, pavements and other improvements, should be stripped of any vegetation and cleared of any surface or subsurface obstructions, including any existing foundations, utility lines, basements, septic tanks, pavements, stockpiled fills, and miscellaneous debris.

b. All pipelines encountered during grading should be relocated as necessary to be completely removed from construction areas or be capped and plugged according to applicable code requirements.

c. Any wells encountered shall be capped in accordance with **Santa Cruz County** Health Department requirements. The strength of the cap shall be at least equal to the adjacent soil and shall not be located within 5 feet of any structural element.

d. Surface vegetation and organically contaminated topsoil should be removed from areas to be graded. The required depth of stripping will vary with the time of year the work is done and must be observed by the Geotechnical Consultant. It is generally anticipated that the required depth of stripping will be 6 to 12 inches.

e. Holes resulting from the removal of buried obstructions that extend below finished site grades should be backfilled with compacted engineered fill.

5.2.3 Excavating Conditions

a. We anticipate that excavation of the on-site soils may be accomplished with standard earthmoving and trenching equipment.

b. Perched groundwater was encountered during the course of our field exploration at depths of 15 and 30 feet below existing grade .

c. Although not anticipated, any excavations adjacent to existing structures should be reviewed, and recommendations obtained to prevent undermining or distress to these structures.

5.2.4 Fill Material

a. With the exception of the near surface sand (fill), the on-site soils **may not** be used as compacted fill.

b. All soils, both on-site and imported, to be used as fill, should contain less than 3% organics and be free of debris and cobbles over 6 inches in maximum dimension.

c. Any imported soil to be used as engineered fill shall meet the following requirements:

(i) free of organics, debris and other deleterious materials
(ii) be granular (sandy) in nature and have sufficient fines to allow for excavation of the foundation trenches.
(iii) free of rock and cobbles in excess of 3 inches
(iv) have an expansion potential not greater than low (EI<20)

d. Imported fill material should be approved by the Geotechnical Consultant prior to importing. The Geotechnical Consultant should be notified not less than 5 working days in advance of placing any fill or base course material proposed for import. Each proposed source of import material should be sampled, tested and approved by the Geotechnical Consultant prior to delivery of <u>any</u> soils imported for use on the site.

5.2.5 Fill Placement and Compaction

a. Any fill or backfill required should be placed in accordance with the recommendations presented below.

b. Material to be compacted or reworked should be moisture-conditioned or dried to achieve near-optimum conditions, and compacted to achieve the following minimum relative compaction:
   (a) All fill and compacted building subgrade: 90%
   (b) Upper 6 inches of subgrade in pavement/drive areas: 95%
   (c) Baserock and subbase: 95%.

c. The relative compaction and required moisture content shall be based on the maximum dry density and optimum moisture content obtained in accordance with ASTM D-1557.

d. The in-place dry density and moisture content of the compacted fill shall be tested in accordance with ASTM D-6780 or ASTM D-2922/ASTM D-3017.

e. The number and frequency of field tests required will be based on applicable county standards and at the discretion of the Geotechnical Consultant. As a minimum standard every 1 vertical foot of engineered fill placed within a building pad area, and every 2 vertical feet in all other areas shall be tested, unless specified otherwise by a Rock Solid Engineering, Inc. representative.

f. Fill should be compacted by mechanical means in uniform horizontal loose lifts not exceeding 8 inches in thickness.

g. All fill should be placed and all grading performed in accordance with applicable codes and the requirements of the regulating agency.

5.2.6 Preparation of On-Site Soils

a. Laboratory consolidation test results indicate that the native, near-surface soils are moderately compressible. Site preparation, consisting of over excavation and recompaction of the native subgrade will be required prior to placement of slabs-on-grade and pavements.

b. The native subgrade beneath **slabs-on-grade** should be reworked to a depth sufficient to provide a zone of compacted fill extending at least 12 inches below the bottom of the capillary break.

c. The native subgrade beneath **pavements** should be reworked to a depth sufficient to provide a zone of compacted fill extending at least 12 inches below the bottom of aggregate base coarse. The zone of compacted fill must extend a minimum of 3 feet laterally beyond all pavements.

d. A representative of our firm shall observe the bottom of the excavation once the required depth of overexcavation has been achieved to verify suitability. Prior to replacing the excavated soil, the exposed surface should be scarified to a depth of 6 to 8 inches, moisture conditioned, and compacted.

e. The depths of reworking required are subject to review by the Geotechnical Consultant during grading when subsurface conditions become exposed.

5.2.7 Groundwater Table

Perched groundwater **was** encountered during the course of our exploration between 15 and 30 feet below the existing grade.

5.2.8 Expansive Soils

Our laboratory testing shows that the expansion index of the near surface soils are equal to 14, this indicates that the expansion potential of the near surface soils should be considered **very low**.

5.2.9 Sulfate Content

The results of our laboratory testing indicate that the soluble sulfate content of the on-site soils likely to come into contact with concrete is below the 150 ppm generally considered to constitute an adverse sulfate condition. **Type II cement** is therefore considered adequate for use in concrete in contact with the on-site soils.

5.2.10 Resistivity

The results of our laboratory testing indicate that the resistivity of the on-site soils likely to come into contact with helical foundation piers is 2100 ohm-cm. Based on the literature available for Chance brand helical piers (Table 7.2, Reference 7), this resistivity corresponds to a **Moderate** corrosion potential. As these recommendations may vary by manufacturer, we recommend that the **helical foundation piers be protected from corrosion as specified by the manufacturer of the pier brand chosen for the project.**

5.2.11 Surface Drainage

a. The ground immediately adjacent to the foundation shall be slopes away from the building at slope of not less than one unit vertical in 20 units horizontal (**5 percent slope) for a distance of no less than 10 feet** measured perpendicular to the wall face.

b. If 10 horizontal feet can not be satisfied due to lot lines or physical constraints, the drainage shall be designed in accordance with the requirements of Section 1804.3 of the 2013 California Building Code.

c. Swales and impervious surfaces shall be sloped a minimum of 2 percent towards an approved drainage inlet or discharge point or as specified by the Project Civil Engineer.

d. All roof eaves should be guttered with downspouts provided. The downspouts shall discharge to either splash blocks or solid pipe to carry the storm water away from the structure to reduce the possibility of soil saturation and erosion.

e. Splash blocks may be used provided the final grade slopes away from the structures at 5 percent as required above. As the site slopes gently across the building pad, swales may be necessary to carry the runoff from the high side of the site around the structure.

f. Drainage patterns approved at the time of construction should be maintained throughout the life of the structures. The building and surface drainage facilities must not be altered nor any grading, filling, or excavation conducted in the area without prior review by the Geotechnical Consultant.

g. Irrigation activities at the site should be controlled and reasonable. Planter areas should not be sited adjacent to walls without implementing approved measures to contain irrigation water and prevent it from seeping into walls and under foundations and slabs-on-grade. Large trees should be planted a minimum distance of ½ their mature height away from the foundation.

5.2.12 Utility Trenches

a. Bedding material may consist of sand with SE not less than 20 which may then be jetted, unless local jurisdictional requirements govern.

b. Existing on-site soils may not be utilized for trench backfill, provided they are free of organic material and rocks over 6 inches in diameter.

c. If sand is used, a 3 foot concrete plug should be placed in each trench where it passes under the exterior footings.

d. Backfill of all exterior and interior trenches should be placed in thin lifts and mechanically compacted to achieve a relative compaction of not less than 95% in paved areas and 90% in other areas per ASTM D-1557. Care should be taken not to damage utility lines.

e. Utility trenches that are parallel to the sides of a building should be placed so that they do not extend below a line sloping down and away at an inclination of 2:1 (H:V) from the bottom outside edge of all footings.

f. Trenches should be capped with 1.5± feet of impermeable material. Import material must be approved by the Geotechnical Consultant prior to its use.

g. Trenches must be shored as required by the local regulatory agency, the State Of California Division of Industrial Safety Construction Safety Orders, and Federal OSHA requirements.

5.3 Foundations

5.3.1 General

a. It is our opinion that the residence may be supported by a **foundation system composed of helical screw piles** founded below the liquefiable stratum and peat layers. As an increase in the peat thickness was noted in Boring B-1 closest to Harkins Slough, we recommend the buildings be setback 100 feet from the centerline of Harkins Slough. Building closer to the slough will require more investigation and possible deeper helical piers.

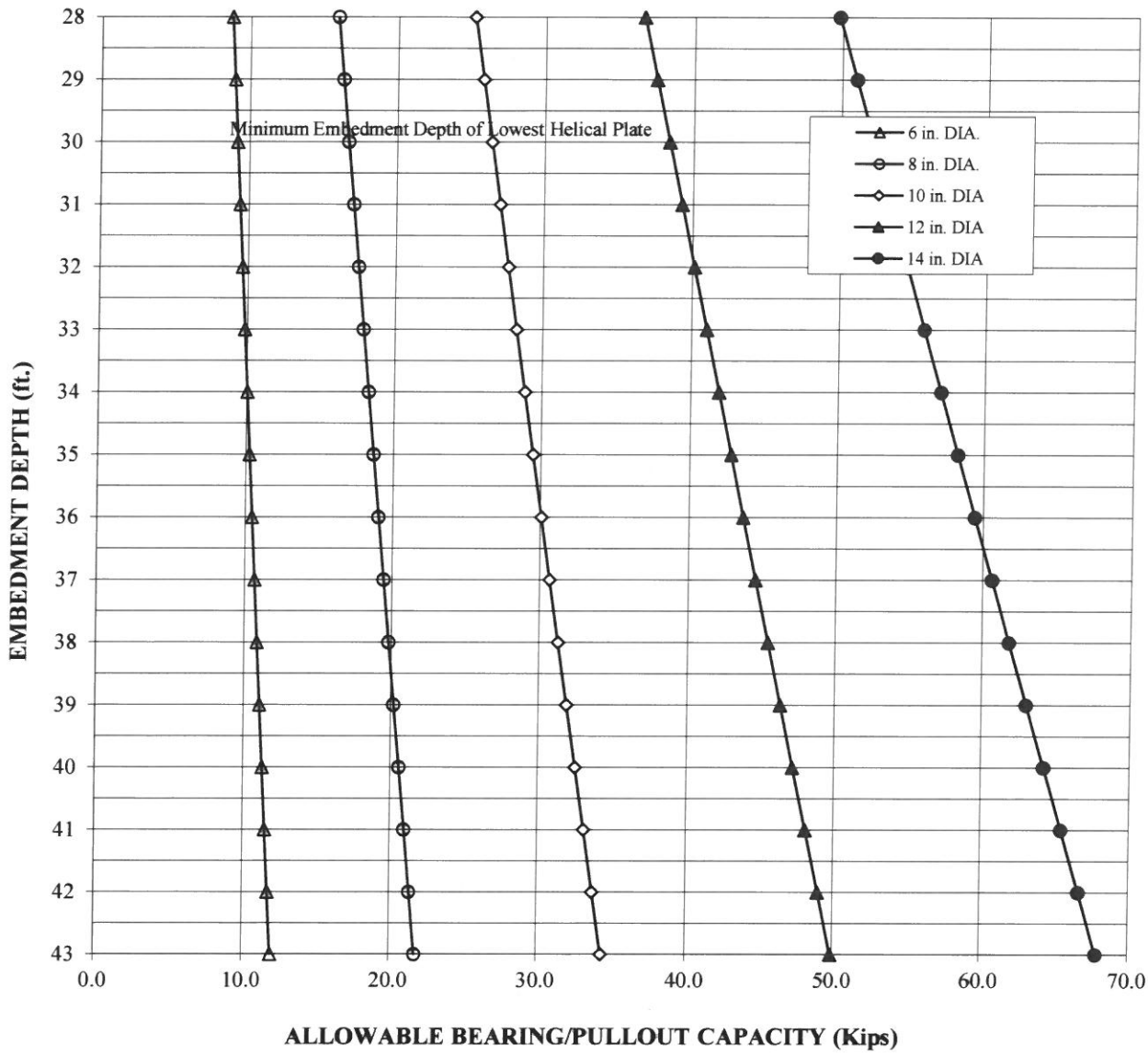
b. At the time we prepared this report, grading and foundation plans had not been finalized. We request an opportunity to review these plans during the design stages to determine if supplemental recommendations will be necessary.

5.3.2 Helical Screw Piles

a. We recommend that the proposed helical screw piles be **embedded such that the lowest helical plate has a minimum embedment of 25 feet below lowest adjacent grade**. The final depth should be below the peat layers and will be determined from monitoring the installation torque.

b. The estimated allowable bearing/pullout capacities for 6 inch, 8 inch, 10 inch, 12 inch, and 14 inch diameter, helical plates are presented in **Figure 2**. These values were computed assuming a minimum embedment depth of 30 feet.

c. If multi-plate anchors are proposed, the total allowable bearing/pull-out capacity of each anchor is calculated by summing the capacity of each helical plate on the anchor. These capacities do not include the weight of the shaft.



ROCK SOLID ENGINEERING, INC.	**ALLOWABLE BEARING/PULLOUT CAPACITY for INDIVIDUAL HELICAL PLATES**	FIGURE 2

d. The allowable bearing capacity values above may be increased by one-third in the case of short duration loads, such as those induced by wind or seismic forces.

e. Lateral support may be mobilized by helical piles drilled at an incline.

f. The recommended allowable bearing/pullout capacities may be higher than the design strength of the helical foundation piles or their intermediate shaft connections depending on the product chosen. These factors may effect the design capacity of the helical foundation piles and govern in the design.

g. The monitoring of installation torque during installation is recommended. Installation torque should not exceed the anchor rating. Installation torque has been empirically related to bearing/pull-out capacity. The minimum bearing/pull-out capacity to installation torque ratio generally recommended is between 9 and 10, subject to verification in the field.

h. Installation tolerances should be within 2% with regards to plumbness and to within 2 inches in location.

i. In general, installation procedures should be per the manufacturer's specifications.

j. **It is recommended that at least one helical screw pile be installed and tested prior to full scale production** in order to verify both design loads and installation torque requirements.

k. **All helical screw pile installation must be observed and approved by the Geotechnical Consultant. Any helical screw piles installed without the full knowledge and continuous observation by a representative of Rock Solid Engineering, Inc. will render the recommendations of this report invalid.**

5.4 Settlements

Total and differential settlements beneath foundation elements founded on helix anchors are expected to be within tolerable limits. Vertical movements are not expected to exceed 1 inch. Differential movements are expected to be within the normal range (½ inch) for the anticipated loads and spacings. These preliminary estimates should be reviewed by the Geotechnical Consultant when foundation plans for the proposed structures become available.

Foundation elements founded on existing grade are expected to have more than 2.25 inches of settlement across the least dimension of the structure.

5.5 Slabs-on-Grade

a. It is our understanding that the proposed use of the structure as a lumber mill will allow for an earthen floor between the column supports. A significant layer of peat was encountered underlying the building pad. The peat layer may cause significant settlement of concrete slab floors. Should concrete slab-on-grade floors be required, the slab should be designed as structurally separate from the foundation system (floating) or to span all loads to the foundations (assume no soil support). Floating slabs should be expected to experience settlement and/or cracking. However, provided the slab is not structurally required then the settlement and cracking will not effect the structural integrity of the steel building. See section 5.5 Slabs-on-grade for further recommendations.

b. Concrete floor slabs may be founded on compacted engineered fill per the recommendations in section 5.2.6. The subgrade should be proof-rolled just prior to construction to provide a firm, relatively unyielding surface, especially if the surface has been loosened by the passage of construction traffic.

c. It is important that the subgrade soils be thoroughly saturated for 24 to 48 hours prior to the time the concrete is poured. **For compacted engineered fill with a low expansion potential, the subgrade should be presoaked 4 percentage points above optimum to a depth of 1.0 feet**.

d. The slab-on-grade section should incorporate a minimum 4 inch capillary break consisting of 3/4 inch, clean, crushed rock, or approved equivalent. Class II baserock is not recommended. Structural considerations may govern the thickness of the capillary break.

e. Where moisture sensitive floor coverings are anticipated or vapor transmission may be a problem, a 10 mil waterproof membrane should be placed between the floor slab and the capillary break in order to reduce moisture condensation under the floor coverings. Place a 2-inch layer of moist sand on top of the membrane. This will help protect the membrane and will assist in equalizing the curing rate of the concrete.

f. Slab thickness, reinforcement, and doweling should be determined by the Project Structural Engineer, based on the design live and dead loads, including vehicles.

6. LIMITATIONS

a. Our investigation was performed in accordance with the usual and current standards of the profession, as they relate to this and similar localities. No other warranty, expressed or implied, is provided as to the conclusions and professional advice presented in this report.

b. The samples taken and tested, and the observations made, are considered to be representative of the site; however, soil and geologic conditions can vary significantly between sample locations.

c. As in most projects, conditions revealed during construction excavation may be at variance with preliminary findings. If this occurs, the changed conditions must be evaluated by the Project Geotechnical Consultant, and revised recommendations be provided as required.

d. This report is issued with the understanding that it is the responsibility of the Owner, or of his Representative, to ensure that the information and recommendations contained herein are brought to the attention of the Architect and Engineer for the project and incorporated into the plans, and that it is ensured that the Contractor and Subcontractors implement such recommendations in the field.

e. This firm does not practice or consult in the field of safety engineering. We do not direct the Contractor's operations, and we are not responsible for other than our own personnel on the site; therefore, the safety of others is the responsibility of the Contractor. The Contractor should notify the Owner if he considers any of the recommended actions presented herein to be unsafe.

f. The findings of this report are considered valid as of the present date. However, changes in the conditions of a site can occur with the passage of time, whether they be due to natural events or to human activities on this or adjacent sites. In addition, changes in applicable or appropriate codes and standards may occur, whether they result from legislation or the broadening of knowledge.

g. Accordingly, this report may become invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and revision as changed conditions are identified.

REFERENCES

1. Brabb, E.E., 1989, Geologic Map of Santa Cruz County, California, U.S. Geological Survey Miscellaneous Investigations Series Map I-1905, Scale: 1:62,500.

2. California Building Standards Commission, July 2013, 2013 California Building Code, California Code of Regulations, Title 24, Part 2, Effective January 1, 2014.

3. Cooper-Clark and Associates, 1975, Preliminary Map of Landslide Deposits in Santa Cruz County, California, Santa Cruz County Planning Dept., Scale: 1:62,500.

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APPENDIX A

FIELD EXPLORATION AND LABORATORY TESTING PROGRAM

- Field Exploration Procedures — Page A-1
- Laboratory Testing Procedures — Page A-2
- Boring Location Plan — Figure A-1
- Key to Logs — Figure A-2
- Logs of Exploratory Borings — Figures A-3 thru A-7
- Summary of Laboratory Test Results — Figure A-8
- Direct Shear Test Results — Figures A-9
- Consolidation Test Results — Figure A-10
- Grain Size Distribution Test Results — Figure A-11 thru A-13

FIELD EXPLORATION PROCEDURES

A-1. Subsurface conditions were explored by drilling 5 borings to depths between 10 and 50 feet below existing grade. The borings were advanced with a truck mounted drill rig equipped with 8 inch hollow stem augers. The approximate locations of the borings are shown on the Boring Location Plan, **Figure A-1**. The Key to Logs, **Figure A-2**, gives definitions of the terms used in the Logs of Exploratory Borings. The Logs of Exploratory Borings are presented in **Figures A-3** through **A-7**.

A-2. Drilling of the borings was observed by our Field Engineer who logged the soils and obtained bulk and relatively undisturbed samples for classification and laboratory testing. The soils were classified, based on field observations and laboratory testing, in accordance with Unified Soil Classification System.

A-3. Relatively undisturbed soil samples were obtained by means of a drive sampler. The hammer weight and drop being 140 pounds and 30 inches, respectively. The number of "Blows/Foot"required to drive samplers are indicated on the logs.

A-4. Exploratory borings were located in the field by measuring from know landmarks. The locations, as shown, are therefore within the accuracy of such a measurement.

A-5. Perched groundwater was encountered at depths of 15 and 30 feet below existing grade during the course of our field exploration.

LABORATORY TESTING PROCEDURES

A-6. Classification

Soils were classified in accordance with the Unified Soil Classification System. Moisture content and in-situ density determinations were made from relatively undisturbed soil samples. The results are presented in the Logs of Exploratory Borings and in the Summary of Laboratory Test Results, **Figure A-8**.

A-7. Direct Shear

Direct shear strength tests were performed on representative samples of the on-site soils in accordance with laboratory test standard ASTM D 3080-98. Samples were relatively undisturbed, or remolded as specified. To simulate possible adverse field conditions, the samples were saturated prior to testing unless otherwise noted. A saturating device was used which permitted the samples to absorb moisture while preventing volume change. The direct shear test results are presented in **Figure A-9**.

A-8. Consolidation

Consolidation tests were performed on representative, relatively undisturbed samples of the underlying soils to determine compressibility characteristics. The samples were saturated during the tests to simulate possible adverse field conditions. The test results are presented in **Figure A-10**.

A-9. Expansion Index

Expansion tests were performed on representative, remolded samples of the on-site soils in accordance with laboratory test standard ASTM D 4829-95. The test results are presented in **Figure A-8**.

A-10. Amount of Materials in Soil Finer than the No. 200 Sieve

Determination of the amount of materials in the soil finer than the No. 200 sieve analyses were performed on samples considered representative of the on-site soils. The laboratory test was performed in accordance with ASTM: D 1140. The test results are presented in **Figure A-8**.

A-11. Soluble Sulfates

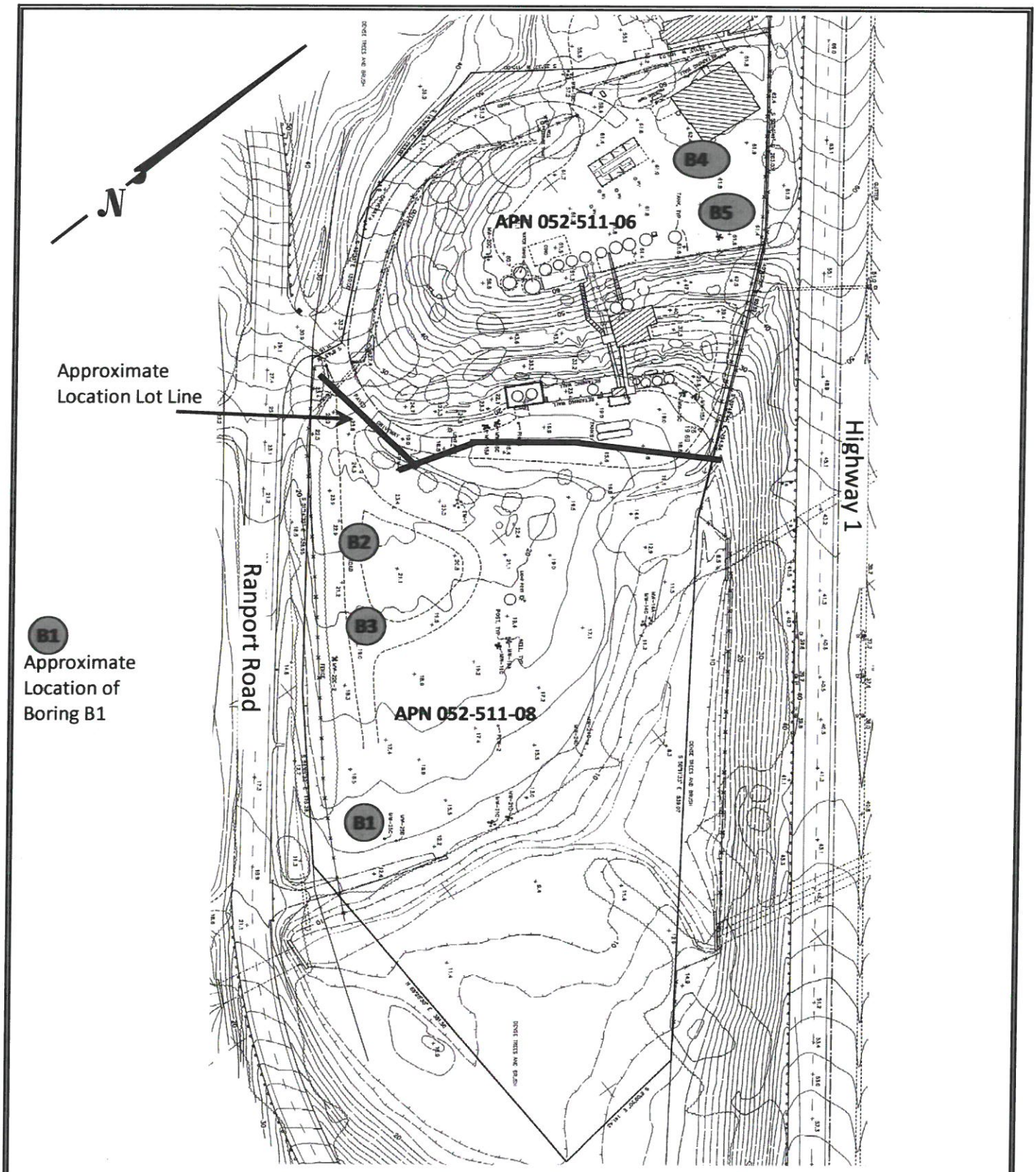
The soluble sulfate content was determined for samples considered representative of the on-soils likely to come in contact with concrete in accordance with test method California 417. The test results are presented in **Figure A-8**.

A-12. Particle Size Analysis

Particle size analyses were performed on samples considered representative of the on-site soils. The laboratory standard used was ASTM: D 422. The test results are presented in **Figures A-11 through A-13**.

A-13 Plasticity Index

The plasticity index was determined for a sample considered representative of the on-site soils in accordance with ASTM D4318. The test results are presented in **Figure A-4**.



Original Topographic Map by Mid Coast Engineers

Not to Scale

ROCK SOLID ENGINEERING, INC.	BORING LOCATION PLAN	FIGURE
	1400 Ranport Road, Watsonville	A-1

KEY TO LOGS

UNIFIED SOIL CLASSIFICATION SYSTEM

PRIMARY DIVISIONS			GROUP SYMBOL	SECONDARY DIVISIONS
COARSE GRAINED SOILS More than half of the material is larger than the No. 200 sieve	GRAVELS More than half of the coarse fraction is larger than the No. 4 sieve	CLEAN GRAVELS (Less than 5% fines)	GW	Well graded gravels, gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines
		GRAVEL WITH FINES	GM	Silty gravels, gravel-sand-silt mixtures, non-plastic fines
			GC	Clayey gravels, gravel-sand-clay mixtures, plastic fines
	SANDS More than half of the coarse fraction is smaller than the No. 4 sieve	CLEAN SANDS (Less than 5% fines)	SW	Well graded sands, gravelly sands, little or no fines
			SP	Poorly graded sands, gravelly sands, little or no fines
		SAND WITH FINES	SM	Silty sands, sand-silt mixtures, non-plastic fines
			SC	Clayey sands, sand-clay mixtures, plastic fines
FINE GRAINED SOILS More than half of the material is smaller than the No. 200 sieve	SILTS AND CLAYS Liquid limit less than 50		ML	Inorganic silts and very fine sands, silty or clayey fine sands or clayey silts with slight plasticity
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	Organic silts and organic silty clays of low plasticity
	SILTS AND CLAYS Liquid limit greater than 50		MH	Inorganic silts, micaceous or diatomacaceous fine sandy or silty soils, elastic silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS			Pt	Peat and other highly organic soils

GRAIN SIZE LIMITS

SILT AND CLAY	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

No. 200 No. 40 No. 10 No. 4 3/4 in. 3 in. 12 in.

US STANDARD SIEVE SIZE

RELATIVE DENSITY	
SAND AND GRAVEL	BLOWS/FT*
VERY LOOSE	0 - 4
LOOSE	4 - 10
MEDIUM DENSE	10 - 30
DENSE	30 - 50
VERY DENSE	OVER 50

CONSISTENCY	
SILT AND CLAY	BLOWS/FT*
VERY SOFT	0 - 2
SOFT	2 - 4
FIRM	4 - 8
STIFF	8 - 16
VERY STIFF	16 - 32
HARD	OVER 32

MOISTURE CONDITION
DRY
DAMP
MOIST
WET

* Number of blows of 140 pound hammer falling 30 inches to drive a 2 inch O.D. (1 3/8 inch I.D.) split spoon (ASTM D-1586).

LOG OF EXPLORATORY BORING

Project No.:	14047	Boring:	B1
Project:	1400 Ranport Road	Location:	APN 08 South Edge of Proposed Steel Building
	Watsonville, California	Elevation:	
Date:	October 2, 2014	Method of Drilling:	Truck Mounted Drill Rig, 8in. Hollow Stem
Logged By:	DO		Auger, 140lb. Safety Hammer

2" DIA Sample; 2.5" DIA Sample; Bulk Sample; Terzaghi Split Spoon Sample; Static Water Table

Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows	Dry Density (pcf)	Moisture Content (%)	Wet Density (pcf)	Direct Shear c (psf)	Direct Shear φ°	Miscellaneous Laboratory Testing
	SM			**FILL**: Brown Silty SAND.							
	SC		X	Reddish Brown Clayey SAND. Dry, Medium Dense, Non-Plastic.	40	91.2	4.9	95.7			Consolidation Sulfate #200 Wash
	CL-SC			Blackish Brown with Orange Staining Sandy CLAY with Gravel. Dry, Plastic.							
			X	Blackish Brown with Orange Staining Sandy CLAY with Gravel. Dry, Stiff, Plastic. Organics Present.	12	68.3	53.5	104.8			#200 Wash
5			X	Material Consistent. Sand and Gravel Decrease with Depth. Moisture Increases with Depth.	4		58.1				
				Grey SAND. Wet, Non-Plastic.							
	CL-SC		X	Laminated Sandy CLAY and Clayey SAND with Some Organics and Gravel. Wet, Loose/Stiff, Plastic.	19						
	Pt		X	Peat. Wet.		47.9	152.3	120.8			
	CL-SC		X	Laminated Sandy CLAY and Clayey SAND with Some Organics and Gravel. Wet, Loose/Stiff, Plastic.	22		24.1				
10											
	Pt		X	Peat.	27	15.0	317.2	62.4			
15			X	Peat.	11		127.9				
			X	Peat.	12	21.6	229.5	71.1			
20				Boring Terminated @ 20 ft. Groundwater Not Encountered. Boring Backfilled With Cuttings.							
25											

ROCK SOLID ENGINEERING, INC.

FIGURE A-3

LOG OF EXPLORATORY BORING

Project No.:	14047	Boring:	B2
Project:	1400 Ranport Road	Location:	APN 08 NW Corner of Proposed Steel Building
	Watsonville, California	Elevation:	
Date:	October 2, 2014	Method of Drilling:	Truck Mounted Drill Rig, 8in. Hollow Stem Auger, 140lb. Safety Hammer
Logged By:	DO		

2" DIA Sample; 2.5" DIA Sample; Bulk Sample; Terzaghi Split Spoon Sample; Static Water Table

Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows	Dry Density (pcf)	Moisture Content (%)	Wet Density (pcf)	Direct Shear c (psf)	Direct Shear ϕ°	Miscellaneous Laboratory Testing
	SC			**FILL**: Grey to Orangish Brown Gravel and Silty SAND. Dry.							
	CL			Black with Grey Layers Organic Sandy CLAY. Moist, Very Stiff, Plastic.	25	98.0	20.1	117.7			Consolidation P.I.: 33 L.L.: 19
	CL-SC			Black Laminated Sandy CLAY and Clayey SAND. Some Organics and Gravel. Moist, Very Stiff, Plastic.	26	115.2	12.2	129.3			#200 Wash Sulfate
5				Material Consistent. Stiff.	13		20.7				
				Black CLAY.							
	SC			Grey Clayey SAND. Moist, Loose, Non-Plastic.	18	102.6	17.0	120.0			Sulfate
10											
	Pt			Peat with Some Silt. Wet, Non-Plastic. Perched Groundwater.	10		106.3				
15											
	SC			Grey Clayey SAND. Wet, Loose, Non-Plastic. Organics.	17	105.3	22.9	129.5			
20											
				Grey with Oxide Staining Clayey SAND. Saturated, Medium Dense, Medium Plastic.	14		23.4				Hydrometer
25											

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FIGURE A-4.1

LOG OF EXPLORATORY BORING

Project No.:	14047	Boring:	B2 Continued
Project:	1400 Ranport Road	Location:	APN 08 NW Corner of Proposed Steel Building
	Watsonville, California	Elevation:	
Date:	October 2, 2014	Method of Drilling:	Truck Mounted Drill Rig, 8in. Hollow Stem
Logged By:	DO		Auger, 140lb. Safety Hammer

2" DIA Sample | 2.5" DIA Sample | Bulk Sample | Terzaghi Split Spoon Sample | Static Water Table

Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows	Dry Density (pcf)	Moisture Content (%)	Wet Density (pcf)	Direct Shear c (psf)	Direct Shear ϕ°	Miscellaneous Laboratory Testing
30	SC		X	Grey Clayey SAND and Gravel. Moist, Very Stiff, Medium Plastic. Groundwater.	36	112.5	17.9	132.6			#200 Wash
35			X	Grey SAND with Some Fines. Saturated, Non-Plastic. Grey Clayey SAND Wet, Medium Dense, Medium Plastic. Grades to Clay.	11		23.4				Hydrometer #200 Wash
40	SP-SC		X	Grey Gravelly SAND with Trace Fines. Saturated, Medium Dense, Non-Plastic.	24	108.5	21.4	131.7	130	37	Hydrometer #200 Wash
45	SC		X	Grey Clayey SAND. Saturated, Dense, Non-Plastic.	38		24.5				#200 Wash
				Boring Terminated @ 50 ft. Perched Groundwater Was Encountered @ 15 and 30 ft. Boring Backfilled With Cuttings.							
50	SP-SC		X	Grey Gravelly SAND with Fines. Saturated, Very Dense, Non-Plastic.	100+		18.5				

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FIGURE A-4.2

LOG OF EXPLORATORY BORING

Project No.:	14047	Boring:	B3
Project:	1400 Ranport Road	Location:	APN 08 Middle of W Side of Proposed Bldg
	Watsonville, California	Elevation:	
Date:	October 2, 2014	Method of Drilling:	Truck Mounted Drill Rig, 8in. Hollow Stem
Logged By:	DO		Auger, 140lb. Safety Hammer

2" DIA Sample; 2.5" DIA Sample; Bulk Sample; Terzaghi Split Spoon Sample; Static Water Table

Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows	Dry Density (pcf)	Moisture Content (%)	Wet Density (pcf)	Direct Shear c (psf)	Direct Shear ϕ°	Miscellaneous Laboratory Testing
	SC			**FILL**: Orange Brown Clayey SAND. Wet, Non-Plastic.							
		X	X	Material Consistent.							
	SC-CL	X	X	Black Clay and SAND. Moist, Stiff/Loose, Plastic. Organic.	12	88.0	42.2	125.1			
			X	Black with Grey Laminated CLAY and SAND. Wet, Stiff/Loose, Plastic. Organics.	10		21.7				EI=14
5		X	X	Grey Brown Clayey SAND with Trace Gravel. Wet, Loose, Non-Plastic. Organics.	16	87.0	29.5	112.6			
10		X	X	Grey Brown CLAY and Clayey SAND. Saturated, Loose, Plastic.	10	97.8	25.9	123.2			
	Pt		X	Peat.	7		179.8				
15		X	X	Material Consistent.	14	15.1	309.2	61.8			
			X	Black CLAY with Sand. Saturated, Firm, Plastic. Organic.	5		67.3				
	CL		X	Black CLAY with Sand. Saturated, Firm, Plastic.	14		22.9				
20				Boring Terminated @ 19.5 ft. Groundwater Not Encountered. Boring Backfilled With Cuttings.							
25											

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FIGURE A-5

LOG OF EXPLORATORY BORING

Project No.:	14047	Boring:	B4
Project:	1400 Ranport Road	Location:	APN 06 NE Croner of Prop. Near (E) Structure
	Watsonville, California	Elevation:	
Date:	October 2, 2014	Method of Drilling:	Truck Mounted Drill Rig, 8in. Hollow Stem
Logged By:	DO		Auger, 140lb. Safety Hammer

2" DIA Sample; 2.5" DIA Sample; Bulk Sample; Terzaghi Split Spoon Sample; Static Water Table

Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows	Dry Density (pcf)	Moisture Content (%)	Wet Density (pcf)	Direct Shear c (psf)	Direct Shear ϕ°	Miscellaneous Laboratory Testing
	SC			**FILL**: Brown and Grey Gravel Over Orange Clayey SAND. Moist, Non-Plastic.							
	CL		X	**NATIVE**: Brown CLAY with Sand. Moist, Stiff, Medium Plastic.	20	114.8	14.1	131.0			#200 Wash
			X	Material Consistent.	11		16.3				Sulfate
5	SM-ML		X	Light Orangish Brown Silty SAND/Sandy SILT. Dry, Loose, Non-Plastic.	14	107.3	14.4	122.8			#200 Wash Sulfate
10			X	Mottled Brown and Orangish Brown Sandy SILT. Moist, Very Stiff, Medium Plastic.	31	114.0	16.7	133.0			
15	CL		X	Brown with Oxide Staining Sandy CLAY. Moist, Very Stiff, Medium Plastic.	15		17.2				
	SC		X	Light Brown Clayey SAND. Moist, Non-Plastic.	17		20.7				
	CL			Grey CLAY. Moist, Very Stiff, Plastic.							
20				Boring Terminated @ 20 ft. Groundwater Not Encountered. Boring Backfilled With Cuttings.							
25											

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FIGURE A-6

LOG OF EXPLORATORY BORING

Project No.:	14047	Boring:	B5
Project:	1400 Ranport Road	Location:	NE Corner of Site S Corner of Bldng Pad
	Watsonville, California	Elevation:	
Date:	October 2, 2014	Method of Drilling:	Truck Mounted Drill Rig, 8in. Hollow Stem
Logged By:	DO		Auger, 140lb. Safety Hammer

2" DIA Sample | 2.5" DIA Sample | Bulk Sample | Terzaghi Split Spoon Sample | Static Water Table

Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows	Dry Density (pcf)	Moisture Content (%)	Wet Density (pcf)	Direct Shear c (psf)	Direct Shear ϕ°	Miscellaneous Laboratory Testing
	SC			**FILL**: Orange Brown Clayey Gravelly SAND.							
		X	X	Orangish Brown SAND. Moist, Medium Dense, Non-Plastic.	37	118.6	6.3	126.1			Sulfate
			X	Material Consistent. Thin Brown Sandy CLAY Layer in Center of Sample.	9		16.8				
5	CL	X	X	**NATIVE**: Brown Sandy CLAY. Wet, Stiff, Plastic.	12	104.9	20.4	126.2			
			X	Orangish Brown Sandy CLAY. Moist, Very Stiff, Plastic.	18		18.6				
10				Boring Terminated @ 10 ft. Groundwater Not Encountered. Boring Backfilled With Cuttings.							
15											
20											
25											

ROCK SOLID ENGINEERING, INC.

FIGURE A-7

SUMMARY OF LABORATORY TEST RESULTS

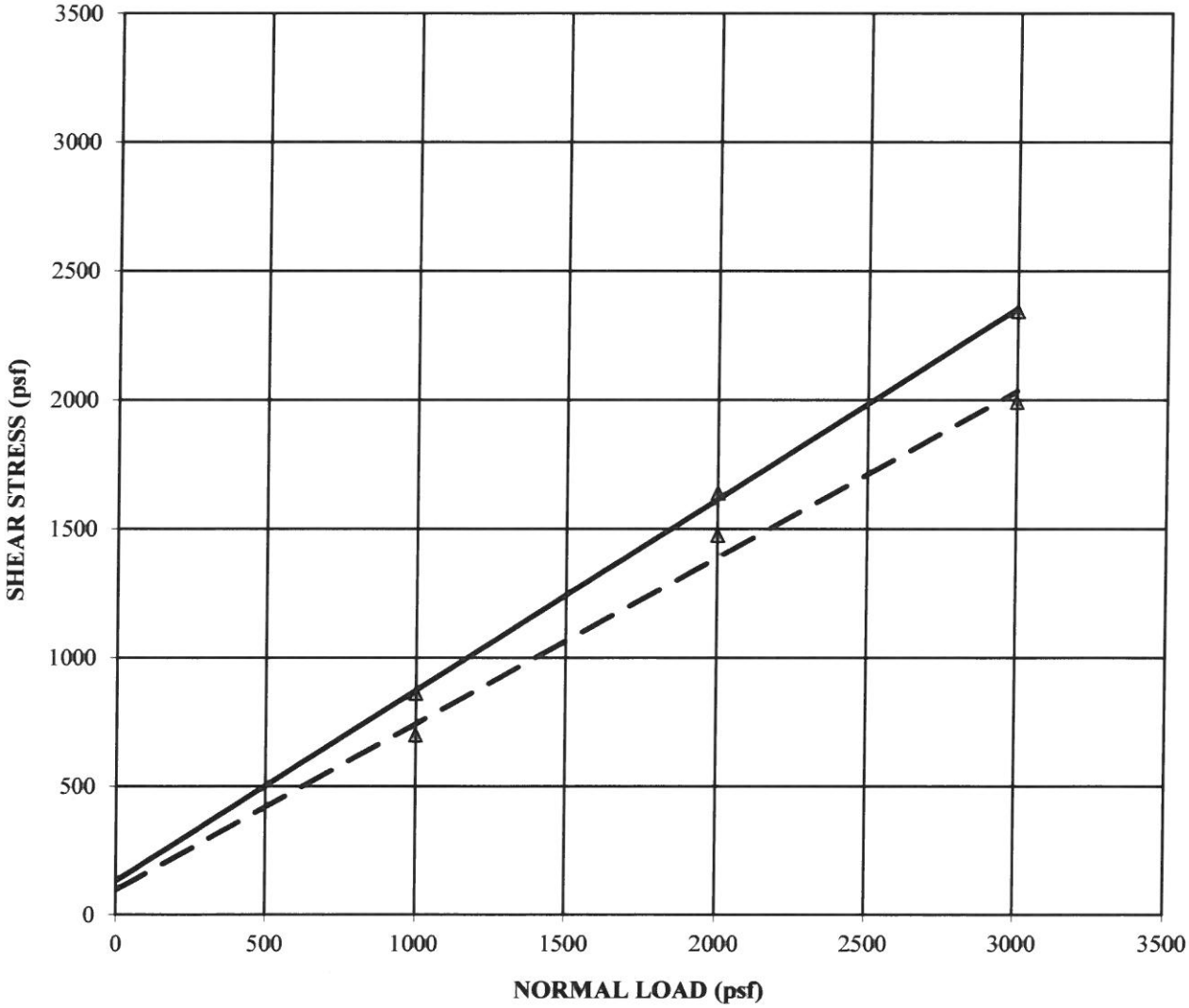
BORING	DEPTH	SOIL TYPE	IN-SITU			DIRECT SHEAR		GRAIN SIZE (%)				EXPANSION INDEX	SOLUBLE SULFATES (ppm)
			DRY DENSITY (pcf)	MOISTURE CONTENT (%)	WET DENSITY (pcf)	COHESION (psf) (PEAK)	FRICTION ANGLE (PEAK)	GRAVEL	SAND	SILT	CLAY		
B1	1.0	SC	91.2	4.9	95.7								29
B1	3.5	CL-SC	68.3	53.5	104.8					50			
B1	5.0	CL-SC		58.1						24			
B1	7.0	Pt	47.9	152.3	120.8								
B1	8.5	CL-SC		24.1									
B1	13.5	Pt	15.0	317.2	62.4								
B1	15.0	Pt		127.9									
B1	18.5	Pt	21.6	229.5	71.1								
B2	1.0	CL	98.0	20.1	117.7								
B2	3.5	CL-SC	115.2	12.2	129.3					31			59
B2	5.0	CL-SC		20.7									
B2	8.5	SC	102.6	17.0	120.0								26
B2	13.5	Pt		106.3									
B2	18.5	SC	105.3	22.9	129.5								40
B2	23.5	SC		23.4									
B2	28.5	SC	112.5	17.9	132.6					30			
B2	33.5	SC		23.4						36			
B2	38.5	SP-SC	108.5	21.4	131.7	130	37	6	87	7			
B2	43.5	SC		24.5						42			
B2	48.5	SP-SC		18.5				16	78	6			
B3	1.0	SC-CL	88.0	42.2	125.1								

SUMMARY OF LABORATORY TEST RESULTS

BORING	DEPTH	SOIL TYPE	IN-SITU			DIRECT SHEAR		GRAIN SIZE (%)				EXPANSION INDEX	SOLUBLE SULFATES (ppm)
			DRY DENSITY (pcf)	MOISTURE CONTENT (%)	WET DENSITY (pcf)	COHESION (psf) (PEAK)	FRICTION ANGLE (PEAK)	GRAVEL	SAND	SILT	CLAY		
B3	2.5	SC-CL		21.7								14	
B3	5.0	SC-CL	87.0	29.5	112.6								
B3	10.0	SC-CL	97.8	25.9	123.2								
B3	11.5	Pt		179.8									
B3	15.0	Pt	15.1	309.2	61.8								
B3	16.5	Pt		67.3									
B3	18.0	CL		22.9									
B4	1.0	CL	114.8	14.1	131.0					52			
B4	2.5	CL		16.3									17
B4	5.0	SM-ML	107.3	14.4	122.8					50			12
B4	10.0	SM-ML	114.0	16.7	133.0								
B4	15.0	CL		17.2									
B4	18.5	SC		20.7									
B5	1.0	SC	118.6	6.3	126.1								200
B5	2.5	SC		16.8									
B5	5.0	CL	104.9	20.4	126.2								
B5	8.5	CL		18.6									

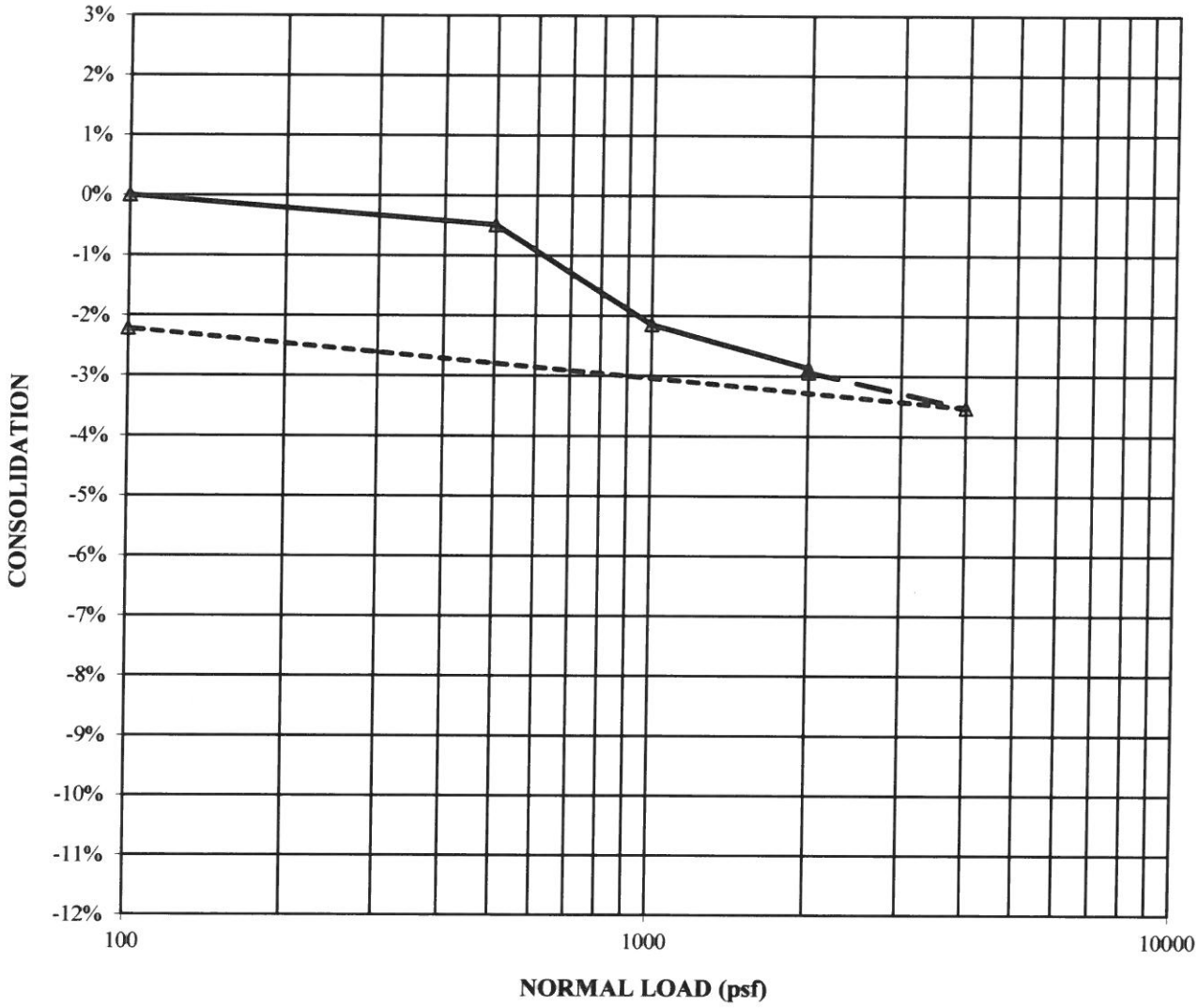
			COHESION (psf)	FRICTION ANGLE
BORING:	B2			
DEPTH (ft):	38.5			
SOIL TYPE (USCS):	SP-SC	PEAK	130	37
		RESIDUAL	90	33

TEST SAMPLE TYPE:	FIELD MOISTURE:	21.4%
IN-SITU (SATURATED)	SATURATED MOIST:	23.3%



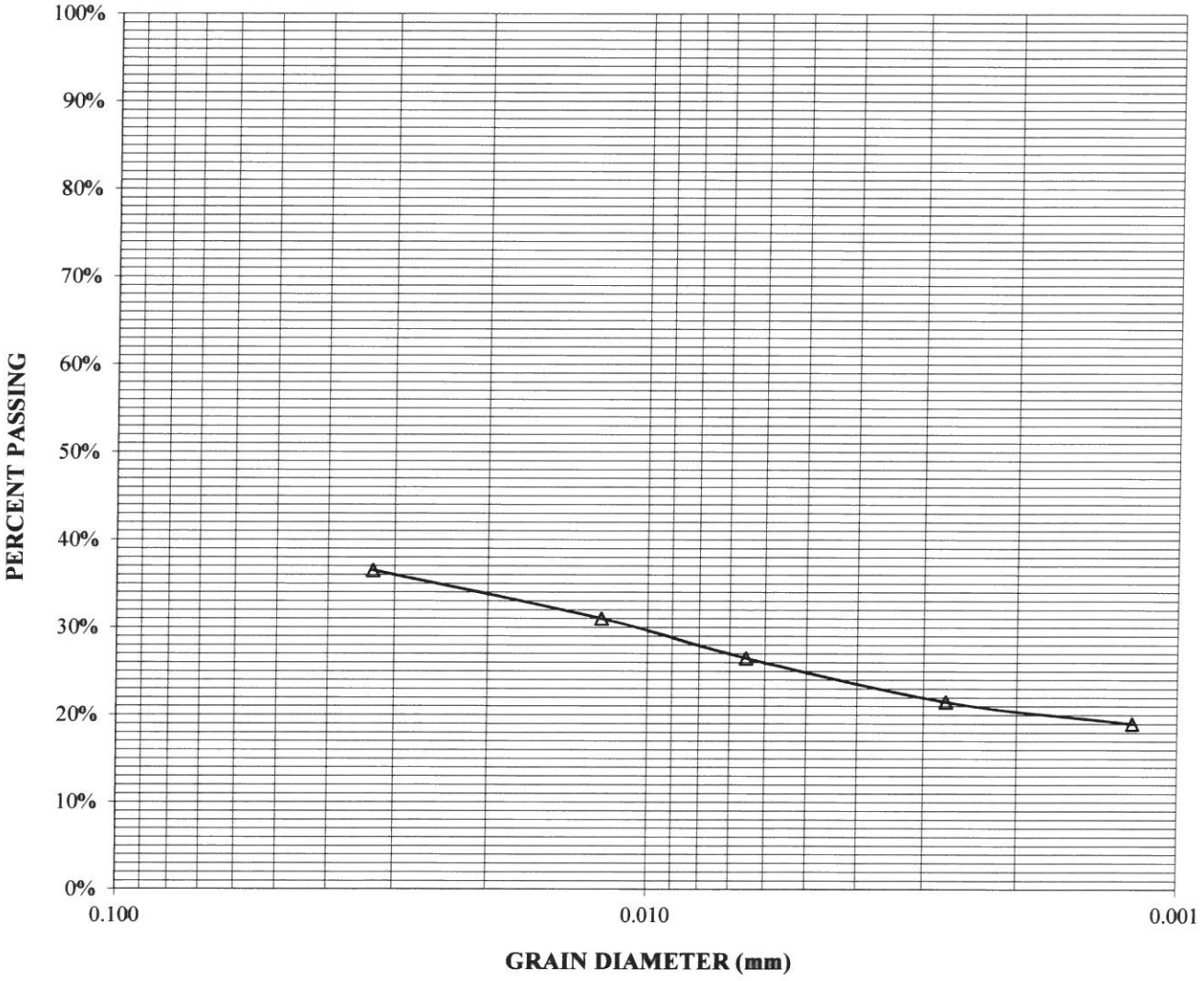
ROCK SOLID ENGINEERING, INC.	**DIRECT SHEAR TEST RESULTS**	FIGURE
	1400 Ranport Road, Watsonville	A-9

BORING:	B2	FIELD MOISTURE	
DEPTH (ft):	1.0	SATURATED	
SOIL TYPE (USCS):	CL	REBOUND	
SEATING WEIGHT:	220 psf	FIELD MOISTURE:	20.1%
		SATURATED MOIST:	24.0%



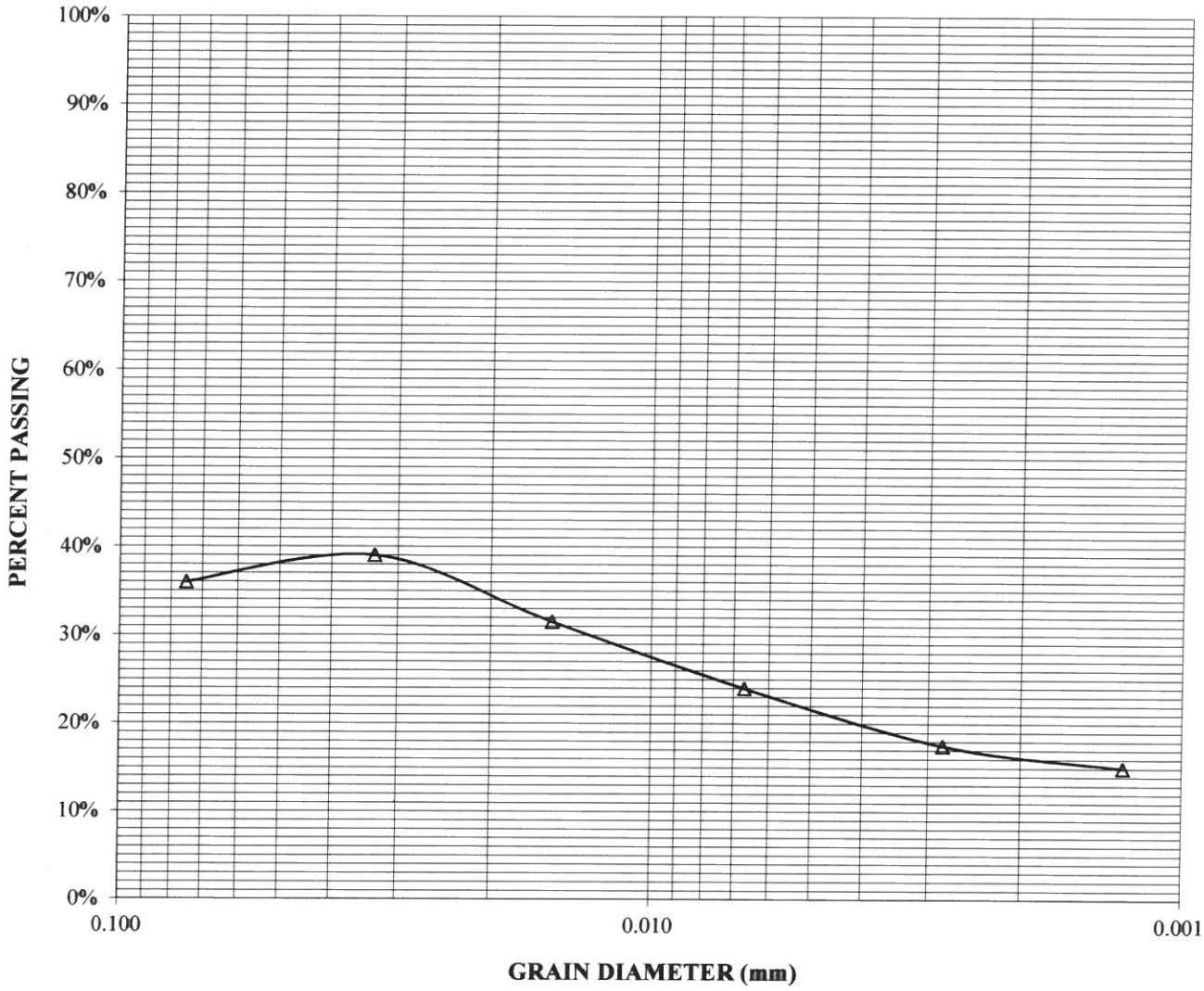
ROCK SOLID ENGINEERING, INC.	**CONSOLIDATION TEST RESULTS**	FIGURE
	1400 Ranport Road, Watsonville	A-10

BORING:	B2	PERCENT	PERCENT
DEPTH (ft):	23.5	PASSING No. 200	CLAY FRACTION <.002mm
SOIL TYPE (USCS):	SC		20%



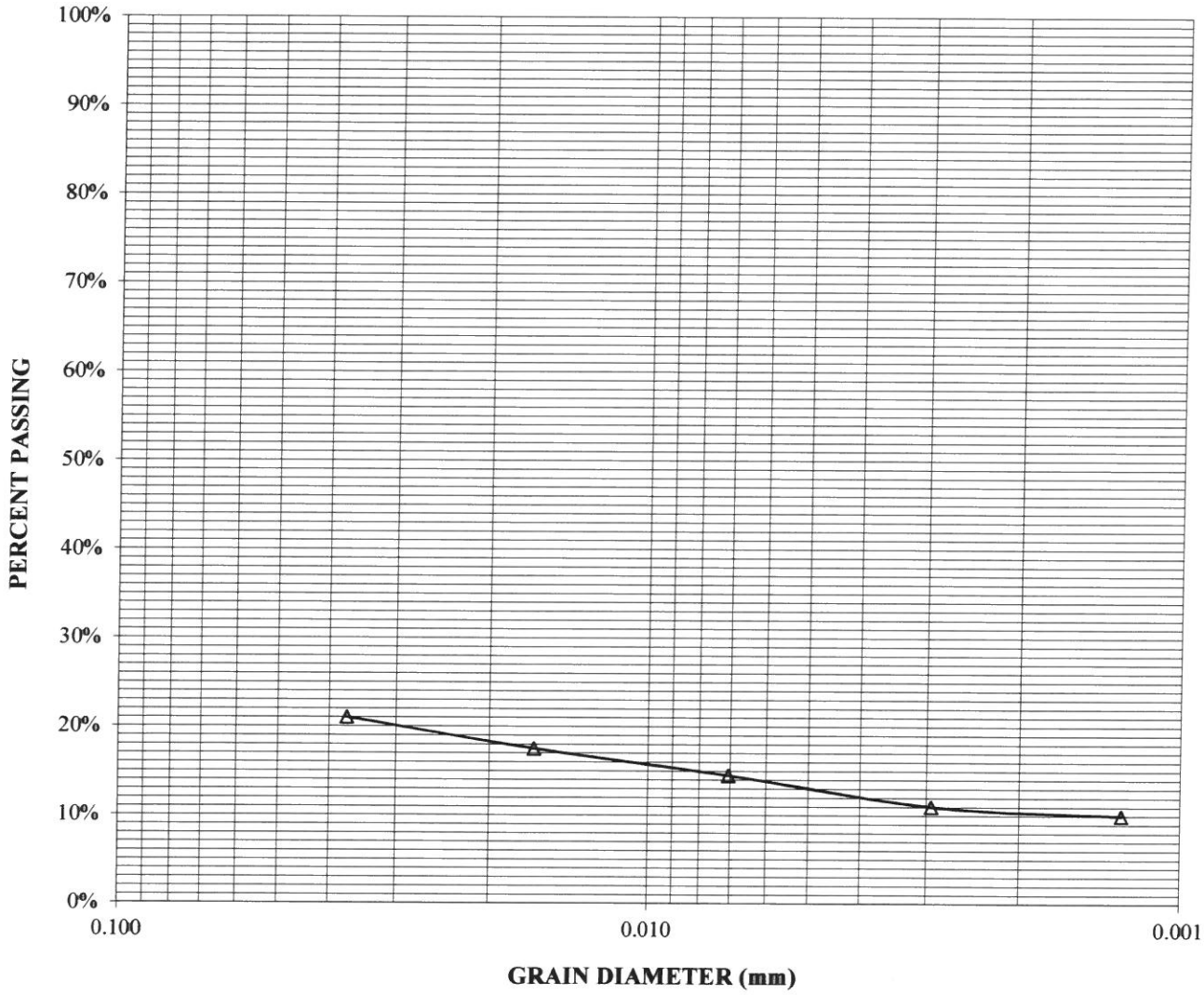
ROCK SOLID ENGINEERING, INC.	GRAIN SIZE DISTRIBUTION	FIGURE
	1400 Ranport Road, Watsonville	A-11

BORING:	B2	PERCENT	PERCENT
DEPTH (ft):	33.5	PASSING No. 200	CLAY FRACTION <.002mm
SOIL TYPE (USCS):	SC	36%	16%



ROCK SOLID ENGINEERING, INC.	GRAIN SIZE DISTRIBUTION	FIGURE
	1400 Ranport Road, Watsonville	A-12

BORING:	B2	PERCENT	PERCENT
DEPTH (ft):	38.5	PASSING No. 4	CLAY FRACTION<.002mm
SOIL TYPE (USCS):	SP-SC	87.0%	10%



ROCK SOLID ENGINEERING, INC.	GRAIN SIZE DISTRIBUTION	FIGURE
	1400 Ranport Road, Watsonville	A-13

APPENDIX B

LIQUEFACTION ANALYSIS

- Methodology — Page B-1
- Calculations — Pages B-2 through B-5

METHODOLOGY

B-1. Our quantitative liquefaction analysis was performed on the observed soil configuration which is considered representative of the conditions at the subject site.

B-2. The analysis uses empirical predictions of earthquake-induced liquefaction potential and is based on the published methods used by Seed and others (Reference 9). This analysis is based on a comparison of the in-situ cyclic stress ration (CSR) with the CSR from historical data collected in areas which experienced liquefaction for a given magnitude earthquake and soil configuration.

B-3. The design seismic event was assumed to occur along the San Andreas Fault with a corresponding magnitude of M=7.9. Our analysis was performed assuming a peak ground acceleration (PGA_M) of 0.67g in accordance with ASCE 7-10, Section 11.8.3.

B-4. Grain size distribution, in-situ water content, and density were determined for samples considered representative of the potentially liquefiable soils encountered. The results of our laboratory testing are presented in Appendix A.

B-5. Material properties chosen for our analysis are conservatively based on laboratory test results and our experience in the vicinity.

B-6. Our calculations of the analyzed soil configuration are presented in tabular form in 1 foot increments below.

LIQUEFACTION ANALYSIS

Project No.:	14047
Project:	1400 Ranport Road, Watsonville, CA
Date:	October 24, 2014
Run By:	YW

Drilling Information	
Boring Diameter (in.):	8.0
C_E Value Based on Hammer Type:	0.6
Sampler Type:	Non-Standard SPT w/Liners Removed

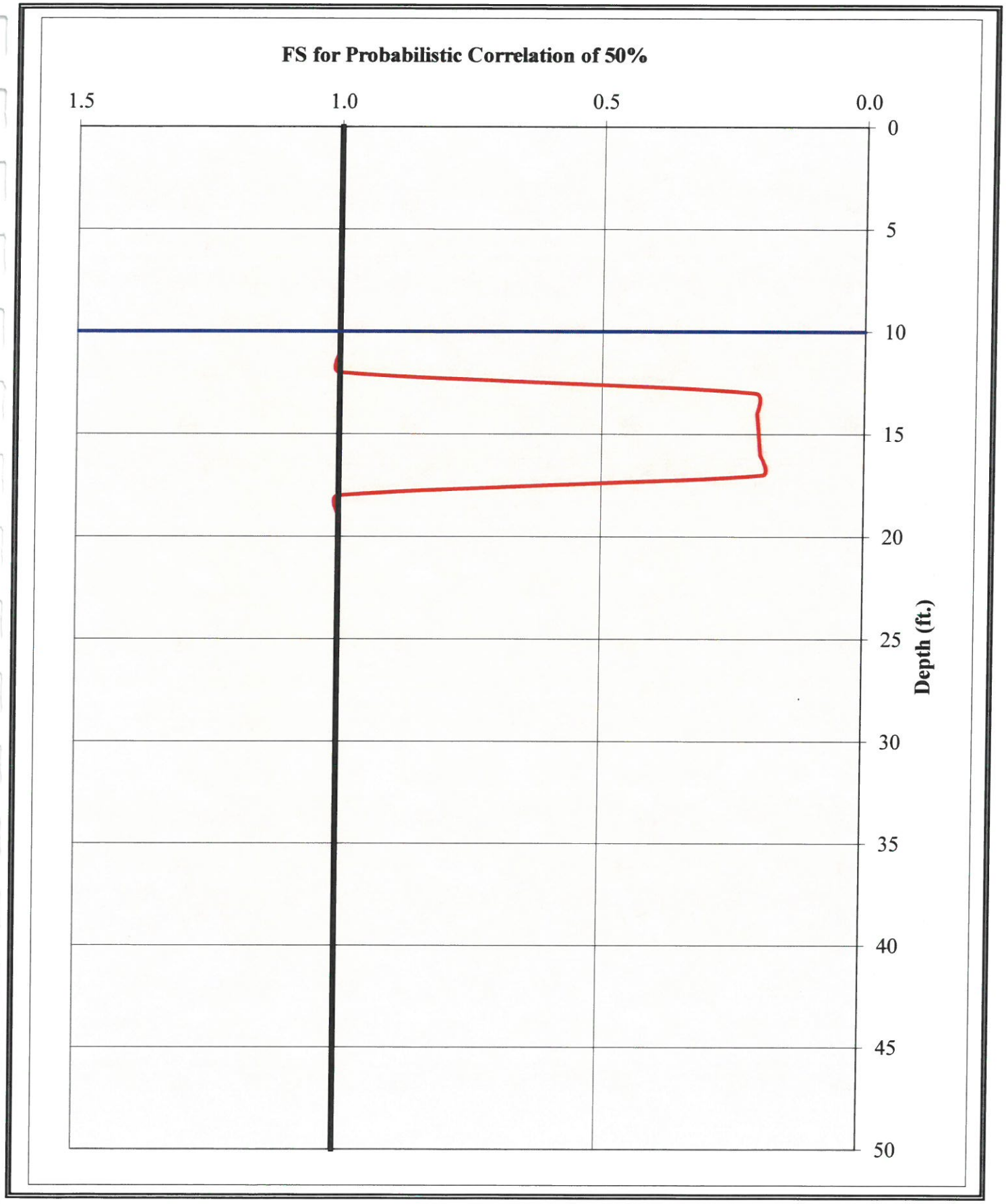
Site Information	
Design Fault:	San Andreas
Design Magnitude:	7.9
Latitude:	N36.9744
Longitude:	W121.95444
Design Pga (g):	0.67
Average Shear Wave Velocity (fps):	1000
Design Groundwater Depth (ft.)	10.0

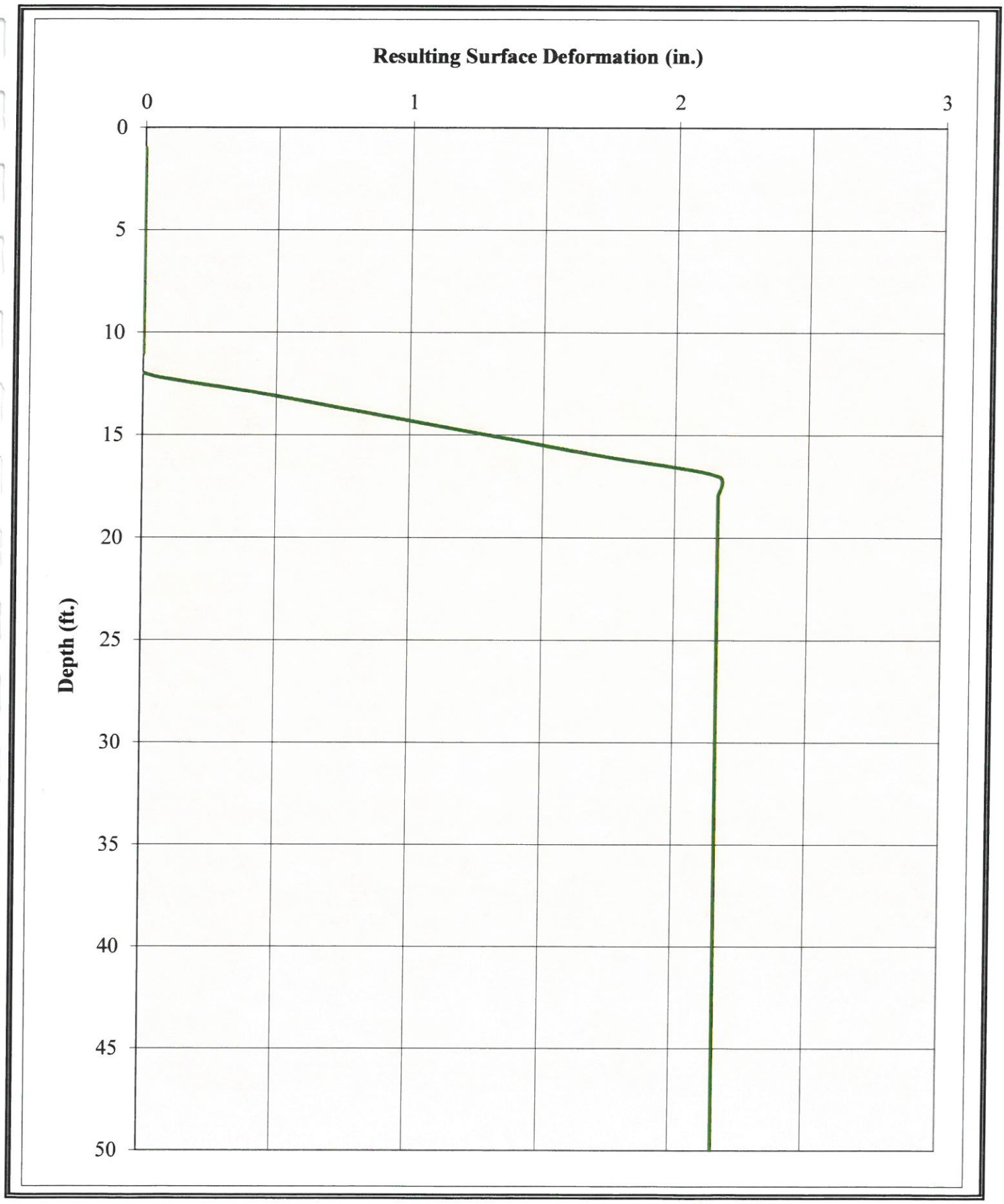
Calculations

DEPTH (ft.)	SOIL TYPE	TOTAL STRESS (psf)	EFFECTIVE STRESS (psf)	$(N_1)_{60CS}$	CSR*	SUSCEPTIBLE	CSR* NEEDED FOR LIQUEFACTION	FACTOR OF SAFETY	CUMULATIVE RECONSOLIDATION (in.)
1	CL	118	118	14.4	0.30	NO	N/A	N/A	N/A
2	CL	235	235	10.6	0.31	NO	N/A	N/A	N/A
3	CL	353	353	9.0	0.32	NO	N/A	N/A	N/A
4	CL	471	471	8.1	0.33	NO	N/A	N/A	N/A
5	CL	589	589	7.4	0.34	NO	N/A	N/A	N/A
6	CL	706	706	7.0	0.35	NO	N/A	N/A	N/A
7	CL	824	824	6.6	0.36	NO	N/A	N/A	N/A
8	CL	942	942	6.4	0.37	NO	N/A	N/A	N/A
9	CL	1059	1059	6.2	0.38	NO	N/A	N/A	N/A
10	SC	1179	1179	32.8	0.39	NO	N/A	N/A	N/A
11	SC	1299	1237	32.8	0.41	NO	N/A	N/A	N/A
12	SC	1419	1295	32.4	0.43	NO	N/A	N/A	N/A
13	OL	1482	1295	11.0	0.45	YES	0.09	0.21	0.45
14	OL	1544	1295	11.2	0.46	YES	0.09	0.21	0.87
15	OL	1607	1295	11.3	0.47	YES	0.10	0.20	1.29

Calculations (continued)

DEPTH (ft.)	SOIL TYPE	TOTAL STRESS (psf)	EFFECTIVE STRESS (psf)	$(N_1)_{60CS}$	CSR*	SUSCEPTIBLE	CSR* NEEDED FOR LIQUEFACTION	FACTOR OF SAFETY	CUMULATIVE RECONSOLIDATION (in.)
16	OL	1669	1295	11.4	0.49	YES	0.10	0.20	1.71
17	OL	1731	1295	11.6	0.50	YES	0.10	0.19	2.16
18	SC	1861	1362	15.6	0.52	NO	N/A	N/A	N/A
19	SC	1990	1429	15.4	0.53	NO	N/A	N/A	N/A
20	SC	2120	1496	15.2	0.54	NO	N/A	N/A	N/A
21	SC	2249	1563	15.1	0.54	NO	N/A	N/A	N/A
22	SC	2379	1630	14.8	0.55	NO	N/A	N/A	N/A
23	SC	2508	1697	14.6	0.56	NO	N/A	N/A	N/A
24	SC	2638	1764	14.5	0.56	NO	N/A	N/A	N/A
25	SC	2767	1831	14.3	0.56	NO	N/A	N/A	N/A
26	SC	2897	1898	14.1	0.56	NO	N/A	N/A	N/A
27	SC	3026	1966	13.9	0.56	NO	N/A	N/A	N/A
28	SP	3159	2036	19.1	0.56	NO	N/A	N/A	N/A
29	SP	3292	2106	18.9	0.56	NO	N/A	N/A	N/A
30	SP	3424	2176	18.6	0.56	NO	N/A	N/A	N/A
31	SP	3557	2246	18.3	0.56	NO	N/A	N/A	N/A
32	SP	3689	2317	18.1	0.55	NO	N/A	N/A	N/A
33	SP	3822	2387	17.8	0.55	NO	N/A	N/A	N/A
34	SP	3955	2457	17.6	0.55	NO	N/A	N/A	N/A
35	SP	4087	2527	17.3	0.55	NO	N/A	N/A	N/A
36	SP	4220	2597	17.1	0.55	NO	N/A	N/A	N/A
37	SP	4352	2668	16.9	0.55	NO	N/A	N/A	N/A
38	SP	4484	2737	10.0	0.55	NO	N/A	N/A	N/A
39	SP	4616	2806	9.9	0.55	NO	N/A	N/A	N/A
40	SP	4747	2875	9.8	0.54	NO	N/A	N/A	N/A
41	SP	4879	2945	9.7	0.55	NO	N/A	N/A	N/A
42	SP	5011	3014	9.6	0.55	NO	N/A	N/A	N/A
43	SP	5143	3083	8.6	0.54	NO	N/A	N/A	N/A
44	SP	5274	3153	8.6	0.54	NO	N/A	N/A	N/A
45	SP	5406	3222	8.5	0.54	NO	N/A	N/A	N/A
46	SP	5538	3291	8.4	0.54	NO	N/A	N/A	N/A
47	SP	5669	3361	8.3	0.54	NO	N/A	N/A	N/A
48	SP	5801	3430	8.2	0.54	NO	N/A	N/A	N/A
49	SP	5933	3499	8.2	0.54	NO	N/A	N/A	N/A
50	SP	6064	3568	8.1	0.53	NO	N/A	N/A	N/A


FS for Probabilistic Correlation of 50%
1.5
1.0
0.5
0.0
0
5
10
15
20
25
30
35
40
45
50
Depth (ft.)


Resulting Surface Deformation (in.)
0
1
2
3
Depth (ft.)
0
5
10
15
20
25
30
35
40
45
50