Rybak Geotechnical, Inc.
Consulting Engineers
16022 Arminta Street, Ste. #7, Van Nuys, CA 91406 • (818) 785-0550 • www.rybakgeotechnical.com

Geotechnical Engineering Investigation
Proposed Parkpointe Encino Project
4929 Genesta Avenue and 17017 Ventura Boulevard
Encino, California

For
TRISTAR REALTY GROUP, LLC
Project No. 2908 February 24, 2017

Rybak Geotechnical, Inc. Consulting Engineers
16022 Arminta Street, Ste. #7, Van Nuys, CA 91406 • (818) 785-0550 • www.rybakgeotechnical.com

February 24, 2017
Project No. 2908

Tristar Realty Group, LLC
12502 Van Nuys Blvd., Suite 301
Pacoima, California 91331

Attn: Daniel Kashani

Dear Mr. Kashani:

We are pleased to submit our report on a geotechnical engineering investigation performed for the proposed Parkpointe Encino project to be located at 4929 Genesta Avenue and 17017 Ventura Blvd., in the Encino area of the City of Los Angeles, California. Our geotechnical engineering conclusions and recommendations are presented in this report.

Four wet-signed, wet-stamped copies of the report, along with a PDF on CD, have been transmitted herewith. It should be noted that no copies of this report have been submitted by us to any other individuals or agencies. Please review the report; if satisfactory, distribute the unbound copies to the applicable agencies and pay any required filing fee. To the best of our knowledge, two wet-signed/wet-stamped copies, along with a PDF copy on CD, should be submitted to the City of Los Angeles Department of Building and Safety, Grading Division. Should you have any questions regarding the report or this submittal, please do not hesitate to call.

Respectfully submitted,
RYBAK GEOTECHNICAL, INC.

RICHARD RYBAK
President

Table of Contents

APPENDIX

INTRODUCTION

This report presents the results of the geotechnical engineering investigation performed for the proposed Parkpointe Encino project to be located at 4929 Genesta Avenue and 17017 Ventura Boulevard, in the Encino area of the City of Los Angeles, California. The objectives of the investigation were to evaluate the soil conditions at the site and to provide geotechnical engineering recommendations for design and construction of the proposed development. This office should be kept informed of all design revisions so that the recommendations contained herein can be appropriately revised, if necessary.

The scope of our services included subsurface exploration and sampling, insitu percolation testing, laboratory testing, engineering analysis, review of pertinent geologic and geotechnical literature, and preparation of this report. Results of the subsurface exploration and laboratory testing are provided in the Appendix.

INTENT

It is the intent of this report to aid in the design and completion of the proposed project. Implementation of the "Conclusions and Recommendations" section of this report is intended to reduce certain geotechnical risks associated with construction of the project. The professional opinions and geotechnical advice contained in this report are subject to the general conditions described in the "Limitations" section of this report.

SITE DEVELOPMENT

The scope of the proposed development was provided by the client, Tristar Realty Group, LLC. This office was provided with two architectural plans, *Sheet A2.0 – Overall Site Plan* and *Sheet A2.1 – Composite Site Plan*, both dated January 24, 2017, and both prepared by Alan S. Bolvin, AIA, Architecture – Planning. In addition, we were provided with an *ALTA/ASCM Survey*, prepared by AJK Engineering and Surveying, dated February 24, 2016.

It is our understanding that the proposed project will include an at-grade, one-story-with-mezzanine commercial/retail building, to be located along the south side of the alley (APNs: 2258-013-020, 2258-013-021, and 2258-013-022). Along the north side of the alley, the project will include a four-story residential building over two levels of concrete parking garage (APNs: 2258-014-001, 2258-015-014, 2258-015-015, and 2258-015-016).

Plans are currently preliminary, however, it is anticipated that the lower garage level for the proposed residential structure will be on the order of 12 feet below the Genesta Avenue sidewalk along the east end, and about 15 feet below the Amestoy Street sidewalk along the west end. Grading is anticipated to consist of excavating up to 15 feet for the proposed residential building subterranean parking garage and removal and recompaction of unsuitable site soils for the proposed, at-grade commercial/retail building. A Geotechnical Map indicating the proposed project is included in the Appendix.

SITE DESCRIPTION

The subject site is located along the north side of Ventura Boulevard, between Amestoy Street to the west and Genesta Street to the east, in the Encino area of the City of Los Angeles. An alleyway bisects the property into a northern and southern portion. The southern portion will be occupied by the proposed at-grade commercial/retail structure, while the northern portion will be occupied by the residential development. At the time of exploration, the portion of the project located south of the alley was occupied by commercial buildings, and the portion north of the alley consisted of paved, at-grade parking lot. The subject property slopes up very gently from east to west, with a total topographic relief of about 8 feet. Vegetation on the subject site consists of sparse plantings in localized planters. Surface drainage on the subject site is by sheetflow towards the east.

EXPLORATION

Subsurface exploration at the subject site was performed on February 16, 17, and 22, 2017, and consisted of the excavation of eight borings to depths ranging between 1-1/2 and 66.5 feet below existing surface grade. The borings were excavated with the aid of a truck-mounted, hollow-stem auger drilling machine and hand-auger equipment.

The earth materials encountered during exploration were logged by the field engineer and classified by visual examination in accordance with the Unified Soil Classification System. Relatively undisturbed samples of the earth materials were obtained in the machine-drilled borings with the aid of a ring sampler driven with the impact of a 140 pound weight free falling over a height of 30 inches. Samples in the hand-auger boring were obtained using a thin wall sampler driven with a hand weight. The soil was retained in brass rings of 2.5-inch outside diameter and 1.0-inch height. Bulk samples of the soils were collected in the borings. Standard Penetration Test (SPT) soundings were also performed at representative depths in the machine-drilled borings.

Upon completion of sampling, each boring was backfilled with the spoils and tamped. The boring locations are indicated on the enclosed Geotechnical Map (Plate 1) and the soils encountered are shown on the exploration logs, Plates A-1 through A-14 in the Appendix.

LABORATORY TESTING

Representative samples of the site soils were transported to the laboratory and tested to determine pertinent geotechnical engineering properties. The results of the laboratory tests performed are summarized in the Appendix.

The dry density and moisture content were determined for the soils collected by the ring sampler. The dry unit weights and moisture contents are provided on the logs, Plates A-1 through A-14, and on Table 2 in the Appendix.

A total of four direct shear tests were performed with the purpose of establishing the shear strength of the soils. The tests were performed on relatively undisturbed samples of the natural soils. The method of testing was in conformance with ASTM D 3080. The shear tests were performed with a strain-controlled device. Samples were subjected to shearing forces under normal stresses of 1.0, 2.0, and 3.0 kips per square foot. The results of the direct shear tests are provided on Plates B-1 through B-4 in the Appendix.

Six consolidation tests were performed on relatively undisturbed samples of the site soils to determine the load-settlement characteristics of the soils at their field moisture content and in a saturated condition. The method of testing was in conformance with ASTM D2435. The results of the consolidation tests are provided on Plates C-1 through C-6 in the Appendix.

One swell test was performed on and undisturbed ring sample of the natural soils. The test was performed to assess swell potential during changes in moisture content. The sample was tested under a static normal stress of 60 pounds per square foot. The consolidation of the soil sample was measured under the field moisture condition. The swell of the soil sample was measured subsequent to sample saturation. The results of the swell test are provided on Table 3 in the Appendix.

Three determinations of percent passing the #200 sieve were performed on representative soil samples in accordance with ASTM Standard D-422. The results of the percent passing the #200 sieve testing are provided on Table 4 in the Appendix.

The Atterberg limits were determined for one sample in accordance with ASTM Standard D-4318. The results are provided on Table 5 in the Appendix.

PERCOLATION TESTING

Percolation testing was performed within boring B-4. The borehole infiltration test was utilized to determine the percolation rate. The sides of the boring were scratched to provide a clean interface for infiltration testing. All loose soils within the boring were removed. Subsequently, a 2-inch layer of fine gravel was placed at the bottom of the boring. Casing, consisting of perforated pipe wrapped in filter fabric, was placed in the boring and the annulus filled with coarse sand. Testing was commenced by adding water to a depth of 10 feet above the bottom of the casing. The first drop in water level was measured after 30 minutes. Testing was continued by now adding water to bring the water level in the boring to a depth of 10 feet above the casing tip.

Testing was performed over three additional 30 minute intervals for a duration of 2 hours. The following are the water level drops that occurred in boring B4 at each 30 minute interval:

30 MINUTE INTERVAL	B4 WATER LEVEL DROP (INCHES)
1st	12
2nd	12
3rd	12
4th	12

SUBSURFACE CONDITIONS

EARTH MATERIALS

Between zero and 6 feet of existing fill was encountered in the eight exploratory borings excavated on the subject site. The existing fill consists of sand, silty sand, and sandy silt, which are dark brown, moist to wet, loose to dense, and contain some rock and concrete fragments. The underlying natural soils consist of alluvial sediments deposited by stream and river action. The natural soils consist of sand, silty sand, sandy silt, clayey silt, and silty clay, which are light to dark brown, dark olive-brown, orange-brown, pale green-gray, pale tan, reddish-brown, slightly moist to very moist, medium-dense to dense and very firm to hard, and contain varying amounts of gravel.

GROUNDWATER

No groundwater was encountered during our exploration to the total depth of 66.5 feet. As part of this investigation, the referenced document entitled *Seismic Hazard Evaluation of the Canoga Park 7.5-Minute Quadrangle, Los Angeles County, California* was reviewed. Plate 1.2 from this document shows the historically highest ground water contours for this quadrangle, with the contours representing the depth to ground water in feet. According to Plate 1.2, the historically highest groundwater level in the immediate area of the subject project was at a depth of 30 feet below surface grade. Fluctuations in the level of groundwater may occur due to variations in rainfall, temperature, and other factors not evident at the time of the measurements reported herein. Fluctuations also may occur across the site. High groundwater levels can result in changed conditions.

FAULTING AND SEISMICITY CONSIDERATIONS

GENERAL

Based on criteria established by the California Geological Survey (CGS), faults may be categorized as active, potentially active or inactive. Active faults are those which show evidence of surface displacement within the last 11,000 years (Holocene-age). Potentially-active faults are those that show evidence of last displacement within the last 1.6 million years (Quaternary-age). Faults showing no evidence of displacement within the last 1.6 million years may be considered inactive for most purposes, except for some critical structures.

In 1972, the Alquist-Priolo Special Studies Zones Act (now known as the Alquist-Priolo Earthquake Fault Zoning Act) was passed into law. The Act defines "active" and "potentially active" faults utilizing the same aging criteria as that used by the CGS, above. However, the established policy is to zone only those potentially active faults that have a relatively high potential for ground rupture. Therefore, not all faults termed "potentially active" by the CGS are zoned under the Alquist-Priolo Act. The subject site is not located within any of the state's Alquist-Priolo Earthquake Fault Zones. In addition, no known, mapped active or potentially active faults traverse through the site.

Seismic sources other than faults with known surface expression are so-called "buried thrust faults". These faults are not exposed at the surface and are typically broadly defined based on the analysis of seismic wave recordings of several hundreds of small earthquakes in the southern California area.

Two major buried thrust faults in the Los Angeles area are the Elysian Park fold and thrust belt and the Torrance-Wilmington fold and thrust belt. It is postulated that the Elysian Park structure was responsible for the magnitude 5.9, October 1, 1987 Whittier Narrows earthquake. It is postulated that the Torrance-Wilmington structure was responsible for the magnitude 5.0, January 19, 1989 Malibu earthquake. It is believed that the magnitude 6.7, January 17, 1994 Northridge earthquake was caused by a blind section of the Oak Ridge system located beneath the San Fernando Valley.

Due to the buried nature of these thrust faults, their existence is usually not known until they produce an earthquake. The risk for surface rupture potential of these buried thrust faults is inferred to be low. However, the seismic risk of these buried structures in terms of recurrence and maximum potential magnitude, is not yet well established. Therefore, the potential for surface rupture on these surface-verging splays at magnitudes higher than 6.0 cannot be totally precluded.

SITE SEISMICITY

Being located in southern California, the subject site has been subject to ground shaking in the past on numerous occasions. The site, as all of the southern California area, is located in a seismically active region and will experience slight to intense ground shaking as the result of movement along various active faults in the region.

LIQUEFACTION EVALUATION

Liquefaction is a phenomena which involves the transformation of a granular material from a solid state into a liquid state as a result of increased pore-water pressures. Increase in pore-water pressures due to earthquake shaking is one cause of liquefaction. Liquefaction is most likely to occur in poorly consolidated, saturated fine-grained sands. The potential for liquefaction occurring in a soil deposit increases as the depth to groundwater decreases, the ground acceleration and duration of shaking increases, the relative density of the saturated soils decreases, and the amount of fines decreases. Complete liquefaction of a soil mass involves the near-total loss of soil strength. This typically occurs when the responsible earthquake is of such magnitude and duration that the pore water pressures are allowed to increase to the point where the effective grain-to-grain soil strength is completely diminished. However, the term "liquefaction" is also commonly utilized to describe the mechanism which results in limited deformation of the underlying soils. In this case, the pore-water pressures do not increase enough for the soil to lose all its strength, but only enough so that the soils undergo limited mobility.

The evaluation of liquefaction potential requires that the values of PGA and PGA_M be determined. PGA is the peak ground acceleration due to the Maximum Considered Earthquake – Geometric Mean (MCE_G). Once the PGA is determined, PGA_M can be determined by multiplying PGA by a site coefficient, F_{PGA}. Based on the USGS Design Maps Detailed Report, the PGA for the subject site is 0.715. Based on a F_{PGA} value of 1.0, the value of PGA_M is determined to be 0.715. A historic high depth to groundwater of 30 feet was used in the analyses. The liquefaction potential evaluation for the subject site was performed utilizing the computer program LIQUEFY2 (Blake, 1998). The results of the evaluation are included in the Appendix.

The liquefaction potential evaluation is performed for two different peak ground accelerations. For purposes of determining the settlement limits for the use of different foundation systems, a peak ground acceleration value equal to two-thirds of the PGA_M, or 0.48 g, will be utilized. The earthquake magnitude used in this evaluation was determined by deaggregation of the seismic source parameters utilizing the USGS 2008 Interactive Deaggregation web site. A 10% probability of exceedance in 50 years (475-year return period) was used. Alluvium conditions are assumed. The results of the deaggregation indicate an earthquake magnitude (M_W) of 6.61. The results of the enclosed liquefaction analyses indicate that the soils are not prone to liquefaction.

For purposes of preventing liquefaction-induced structure collapse, a peak ground acceleration value equal to PGA_M, or 0.715 g, was utilized. The earthquake magnitude used in this evaluation was 6.61, determined from the aforementioned USGS website and utilizing a 2% probability of exceedance in 50 years (2,475-year return period). The results of the enclosed liquefaction analyses indicate that the soils are not prone to liquefaction.

The following correction factors were utilized in the evaluation:

- Borehole diameter correction factor of 1.00.
- Sampler size correction factor of 1.20, since liners not used in sampler.
- N60 hammer correction factor of 1.30, since automatic-trip hammer used in sampling.

SEISMIC HAZARDS

FAULT RUPTURE

The results of geologic literature research indicate that the subject site is not underlain by any known, mapped active or potentially active fault deemed capable of rupturing the surface. The site is not located within any Alquist-Priolo Earthquake Fault Zone. The potential for fault rupture on this site is considered remote.

LANDSLIDING

The subject site ***is not*** located within a potential earthquake-induced landslide zone per the *Seismic Hazard Evaluation of the Canoga Park 7.5-Minute Quadrangle, Los Angeles County, California.*

DYNAMIC SETTLEMENTS

Seismically-induced settlement or compaction of dry or moist, cohesionless soils can also be a secondary effect of earthquake ground motion. Such settlements are typically most damaging when the settlements are differential in nature across the length of structures. Some seismically-induced settlement of the proposed structure should be expected as a result of strong ground shaking, however, due to the relatively dense and uniform nature of the soils, excessive differential settlements are not anticipated on this site.

CONCLUSIONS AND RECOMMENDATIONS

GENERAL

Development of the site is considered feasible from a geotechnical engineering standpoint, provided the following recommendations are implemented during design and construction. The design of the proposed project is still preliminary. This office should be kept informed of all design revisions so that the recommendations contained herein can be appropriately revised, if necessary.

Between zero and 3.5 feet of existing fill was encountered in borings B-1 through B-6. Borings B-7 and B-8 were excavated with hand-auger equipment within a relatively narrow planter located between the existing building and the sidewalk off of Ventura Boulevard. Boring B-7 was excavated to refusal on a cobble at 6 feet, while boring B-8 was excavated to refusal on concrete (footing?) at 1-1/2 feet. The existing fill was not penetrated in borings B-7 and B-8. Deeper fill may exist in other areas of the site that were not directly explored. Excavation for the proposed residential building subterranean parking levels is anticipated to penetrate through the existing fill and expose competent natural soils throughout the excavation bottom. The existing fill is not considered suitable for support of foundations, concrete flatwork, or additional structural fill. The existing fill will need to be removed and recompacted beneath the proposed at-grade commercial/retail building located south of the existing alley.

The proposed buildings may be supported on shallow, conventional spread footings embedded in either competent natural soils or properly compacted fill which has been placed atop competent, approved natural soils. All floor slabs should be supported on either competent natural soils or properly compacted fill which has been placed atop competent, approved natural soils.

Groundwater was not encountered during our exploration to the total depth of 66.5 feet. Excavation for the proposed subterranean parking levels is anticipated to extend to depths of approximately 17 feet below ground surface, including foundation construction. Based upon conditions encountered at the time of exploration, groundwater is ***not*** anticipated to be an issue during excavation. The need for temporary dewatering is ***not*** considered necessary on this project.

SEISMIC DESIGN CONSIDERATIONS

Although no known active faults traverse through the subject site, like most of Southern California, the subject site lies within a seismically active area. Earthquake resistant structural design is recommended. Designing structures to be earthquake-proof is generally considered to be impractical, especially for private projects, due to cost limitations. Significant damage to structures may be unavoidable during large earthquakes. The structural design of the proposed structures should be based on the latest version of the California Building Code (CBC). The following minimum seismic parameters should be used:

- Site Classification = Site Class D
- Mapped, short period Acceleration Parameter, $S_s = 2.061$
- Mapped Acceleration Parameter at 1.0 second period, $S_1 = 0.725$
- Site Coefficient, $F_a = 1.0$
- Site Coefficient, $F_v = 1.5$
- Mapped MCE Spectral Response Acceleration at Short Periods, $S_{ms} = 2.061$
- Mapped MCE Spectral Response Acceleration at 1 sec Period, $S_{m1} = 1.088$
- Design Earthquake Spectral Response Acceleration Parameter at Short Period, $S_{ds} = 1.374$
- Design Earthquake Spectral Response Acceleration Parameter at 1-sec Period, $S_{d1} = 0.725$
- Seismic Design Category = D

These minimum code values are intended to protect life and may not provide an acceptable level of protection against significant cosmetic damage and serious economic loss. A significantly higher than code lateral design parameter would be necessary to further reduce potential economic loss during a major seismic event. Structural engineers, however, often regard higher than code values as impractical for use in structural design. The structural engineer and project owner must decide what level of risk is acceptable and to assign appropriate seismic values for use in structural design. The risk of damage to the structure due to a large earthquake cannot be totally eliminated, and obtaining appropriate insurance as a mitigation measure is strongly recommended.

INFILTRATION FACILITY

The field-measured percolation rate needs to be reduced to accommodate both suitability assessment related considerations and design related considerations. The calculation worksheet is enclosed with this report. The combined safety factor calculated is 2.6. Prior to adjusting the field percolation rate by the aforementioned suitability assessment and design considerations, a County of Los Angeles reduction factor was applied to account for exfiltration occurring through the sides of the boring. The calculated design percolation rate is 0.3 inches per hour. This is less than the minimum allowable percolation rate of 0.5 inches. Therefore, percolation of water into the subsurface is not recommended. The use of above-ground sealed planters should be considered.

CONVENTIONAL FOUNDATIONS

Design – Proposed Residential Building

The foundations at the level of the lowest subterranean garage may consist of conventional spread footings embedded in the underlying natural alluvial soils. The footings should be embedded a minimum of 24 inches below lowest adjacent grade, and be embedded a minimum of 18 inches into the competent natural soils. Footings may need to be deepened if observations during construction indicate unsatisfactory soil conditions.

Continuous wall footings may be designed for an allowable bearing value of 4,000 pounds per square foot, and should be a minimum of 12 inches in width. Column footings may be designed for an allowable bearing value of 4,500 pounds per square foot, and should be a minimum of 24 inches in width.

A one-third increase in the above allowable bearing values may be utilized for wind or seismic loads. The recommended bearing values are net values, and the weight of the concrete in the footings may be taken as 50 pounds per cubic foot, and the weight of the soil backfill may be neglected when determining the downward loads.

All conventional, continuous wall footings should be reinforced with a minimum of four, #4 steel bars, two near the top and two near the bottom of the footings.

Design – Proposed Commercial/Retail Building

Conventional spread footings embedded in the underlying natural soils or on properly compacted fill which has been placed on competent, approved natural soils can be used for foundation support of the proposed at-grade commercial/retail building. The footings should be embedded a minimum of 24 inches below lowest adjacent grade, and be embedded a minimum of 18 inches into the competent natural soils or properly compacted fill. Footings may need to be deepened if observations during construction indicate unsatisfactory soil conditions.

Continuous wall footings may be designed for an allowable bearing value of 2,000 pounds per square foot, and should be a minimum of 12 inches in width. Column footings may be designed for an allowable bearing value of 2,500 pounds per square foot, and should be a minimum of 24 inches in width.

A one-third increase in the above allowable bearing values may be utilized for wind or seismic loads. The recommended bearing values are net values, and the weight of the concrete in the footings may be taken as 50 pounds per cubic foot, and the weight of the soil backfill may be neglected when determining the downward loads.

All conventional, continuous wall footings should be reinforced with a minimum of four, #4 steel bars, two near the top and two near the bottom of the footings.

Resistance to Lateral Loading

Resistance to lateral loading may be provided by soil friction and by the passive resistance of the natural soils or properly compacted fill. An allowable coefficient of friction of 0.4 may be used between footings and the supporting soils. An allowable passive resistance of the natural soils or properly compacted fill against footings may be assumed to be 300 pounds per cubic foot, to a maximum of 3,000 pounds per square foot. The passive resistance of the soils and the frictional resistance between the footings and the supporting soils may be combined without reduction in determining the total lateral resistance.

Settlement

A majority of the settlement of the new foundations is expected to occur upon initial loading. The settlement of the new foundations is not expected to exceed 1/2 inch. Differential settlements between adjacent columns is not expected to exceed 1/4 inch.

Installation

To provide proper foundation support, new footings should not surcharge any materials other than properly controlled fill or undisturbed, competent natural soils. Proper footing depth may be determined by extending an imaginary 1:1 plane downward from the bottom edges of the new footings. Proper footing depth is achieved when all of the following conditions are met:

- The bottom of footing is embedded the recommended depth below lowest adjacent grade and into the recommended bearing material.
- The imaginary plane is below any unsuitable soils or existing structures and only properly controlled fill or undisturbed natural soils are intersected by the extended plane.

All footing excavations should be cleaned of loose soil and debris and inspected and approved by the geotechnical engineer prior to the placement of reinforcing steel, concrete forms, or concrete. The geotechnical engineer should verify that the footing will penetrate into the recommended bearing material. Any required footing backfill should be mechanically compacted; flooding or jetting with water is not permitted.

FLOOR SLABS

Concrete floor slabs-on-grade which will not be impacted by vehicular traffic should be a minimum of 4 inches in thickness. Floor slabs which will be impacted by vehicular traffic should be a minimum of 5 inches in thickness. All floor slabs should be supported on undisturbed natural soils or on properly compacted fill. All floor slabs should be reinforced with a minimum of #4 steel bars placed 16 inches on center, each way. In order to decrease the potential for moisture to migrate through the floor slab, it is recommended that a fiber reinforcement additive be added to the concrete mix. This will improve the tensile strength of the concrete and reduce the likelihood of shrinkage cracks from developing. In addition, it is important that any shrinkage cracks that do develop in the floor slab be sealed prior to covering of the slab.

Floor slabs that would be critically affected by moisture, should be underlain by a vapor retardant which consists of a minimum 15-mil thick extruded polyolefin plastic (no recycled content or woven materials permitted). Permeance as tested before and after mandatory conditioning should be less than 0.01 perms and comply with the ASTM E1745 Class A requirements. Vapor retardant should be installed according to ASTM E1643, including proper perimeter seal. The concrete should be poured directly atop the vapor retardant. No sand should be placed atop the vapor retardant. The concrete mix should be designed so as to minimize possible curling of the slab. The concrete slabs should be allowed to cure properly before placing vinyl or other moisture-sensitive floor covering. Prior to placement of the vapor retardant or casting of concrete against soil, the soil subgrade should be wetted to eliminate any shrinkage cracks.

Construction activities and exposure to the environment can cause deterioration of prepared subgrades. Therefore, we recommend that our field representative observe the condition of the final subgrade earth materials immediately prior to slab-on-grade construction and, if necessary, perform further field density and moisture content tests to determine the suitability of the final prepared subgrade.

Concrete shrinks as it cures, resulting in shrinkage tension within the concrete mass. The development of tension results in cracks within the concrete since concrete is weak in tension. Therefore, the concrete should be placed using procedures to minimize the cracking within the slab. Shrinkage cracks can become excessive if water is added to the concrete above the allowable limit and proper finishing and curing practices are not followed. Concrete mixing, placement, finishing and curing should be performed per the American Concrete Institute Guide for Concrete Floor and Slab Construction (ACI 301.1R-89). Where shrinkage cracks would be unsightly, such as in the garage, concrete slabs on grade should be provided with tooled, crack control joints at 10 to 15-foot centers or as specified by the structural engineer.

Tile flooring can crack, reflecting cracks in the concrete slab below the tile. Therefore, the slab designer should consider additional steel reinforcement in concrete slabs on-grade that will directly support tile. The tile installer should consider installation methods that reduce possible cracking of the tile. A vinyl crack isolation membrane (approved by the Tile Council of America/Ceramic Tile Institute) is recommended between tile and concrete slabs-on-grade per the Portland Cement Association Specifications.

RETAINING WALLS

Restrained Design

It is anticipated that the subterranean parking basement retaining walls will be designed for a restrained condition (i.e. no movement allowed at the top of wall). These retaining walls are anticipated to be up to 15 feet in height. Retaining walls that will be properly drained, preventing the buildup of hydrostatic pressure, may be designed for an at-rest equivalent fluid pressure of 69 pounds per cubic foot.

Additional pressure should be added for a surcharge condition due to adjacent structures or vehicular traffic. For the purposes of this report, a retaining wall is considered “surcharged” when an imaginary plane projected upwards from the bottom of the back of the retaining wall at an angle of 45 degrees projects below vehicular traffic or existing structures/foundations. In addition to the recommended earth pressure, the upper ten feet of the basement walls adjacent to vehicular traffic should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pounds per square foot surcharge behind the walls due to traffic. If the traffic is kept back at least ten feet from the basement walls, the traffic surcharge may be neglected.

Seismic Loading on Retaining Walls

For retaining walls in excess of 6 feet in height, additional lateral seismic loading should be applied due to earthquake motions. This is according to the 2013 CBC. Using the Mononobe-Okabe method, and with a PGA_M value of 0.715, and assuming a $k_h = (1/2)(2/3)(PGA_M)$, or 0.24, a thrust of 11.7 $(H)^2$ pounds per one foot length of retaining wall should be applied at a point located 0.6(H) above the base of the retaining wall footing, where H is the height of the retaining wall in feet. For free-standing retaining walls, this seismic thrust should be added to the static active earth pressure. For retaining walls which are considered restrained from movement at the top and are designed for the recommended static, at-rest earth pressures, no additional seismic loading need be applied.

Free-Standing Design

Drained retaining walls which are considered free-standing and not restrained from movement at the top, may be designed for a minimum equivalent fluid pressure 35 pounds per cubic foot. Additional pressure should be added for a surcharge condition due to sloping backslope, vehicular traffic or adjacent structures. This equivalent fluid pressure may be utilized provided the retaining walls are provided with proper subsurface and surface drainage, as recommended within this report.

Wall Drainage

Drained retaining walls should be provided with a drainage system extended at least two-thirds the height of the wall. At the base of the drain system, a subdrain covered with a minimum of 12 inches of gravel should be installed, and a compacted fill blanket or other seal placed at the surface. The clean bottom and subdrain pipe, behind the retaining wall, should be observed by a representative of this office, prior to placement of gravel or compacting backfill.

Alternatively, a plastic drainage composite such as Miradrain or equivalent may be installed in continuous, 4-foot wide columns along the entire back face of the wall, at 8 feet on center. The top of these drainage composite columns should terminate approximately 18 inches below the ground surface, where either hardscape of a minimum of 18 inches of relatively cohesive material should be placed as a cap. These vertical columns of drainage material would then be connected at the bottom of the wall to a collection panel or a one-cubic foot rock pocket drained by 4-inch subdrain pipe. Subdrainage pipes at the base of the retaining wall drainage system should outlet to an acceptable location via controlled drainage structures.

Waterproofing of the subterranean walls is recommended. The design and inspection of the waterproofing is not the responsibility of the geotechnical engineer. A waterproofing consultant should be retained in order to recommend a product or method which would provide protection to subterranean walls, floor slabs, and foundations.

Wall Backfill

Wall backfill should be mechanically compacted in loose layers not more than 8 inches in thickness, to the proper relative compaction. Flooding or jetting of the backfill is not permitted. Proper compaction of the backfill will be necessary to reduce settlements of the backfill and overlying flatwork.

Prior to backfilling, the excavation between retaining walls and the temporary excavations should be cleared of all loose materials, debris, construction materials, etc. The backfill should be free of rocks larger than 6 inches in any dimension. Proper compaction of the backfill will be necessary to reduce the potential settlement of the backfill. Some settlement of the backfill should be anticipated and any utilities and flatwork supported therein should be designed to accept differential settlement, particularly at the points of entry to the structures. Retaining wall backfill should be properly compacted, as recommended in the "Site Grading" section of this report. Compaction testing of the wall backfill every 2 vertical feet will be necessary during fill placement to verify percentage of compaction achieved during fill placement. The retaining wall backfill may be capped with proper surface drainage gutters to collect and transfer collected drainage to an appropriate location.

All retaining wall backfill should be placed in loose horizontal lifts not more than 8 inches in thickness, watered as necessary to achieve the proper moisture content, and mechanically compacted to at least 90 percent relative compaction. Flooding or jetting of the backfill is not permitted. Probing and testing should be performed by the project geotechnical engineer to verify proper compaction.

Where space limitations do not allow for conventional backfill compaction operations, the space should be backfilled with pea gravel. The pea gravel should be placed in lifts of no more than 2 feet in thickness and should be compacted with vibratory equipment. Ideally, the top two feet of backfill exposed to water infiltration should consist of compacted fill so that a relatively impervious condition is developed. Filter fabric should be placed atop the pea gravel prior to placement of the compacted fill.
Contractors should be informed that the use of heavy compaction equipment within close proximity to retaining walls could cause excessive wall movement and/or earth pressures in excess of design values.

The following observations should be performed by representatives of this firm during installation of subdrains, placement of wall backfill, and placement of any other backfill:

- Observe the area behind the walls prior to placement of gravel for drain support and subsequent to drain installation and prior to gravel backfilling.

- Perform visual observations to evaluate the suitability of on-site and import soils for backfill placement; collect and submit fill samples for required or recommended laboratory testing, where necessary.

- Perform field density and compaction testing to determine the percentage of compaction achieved during fill placement.

The governmental agencies having jurisdiction over the project should be notified prior to commencement of grading so that the necessary grading permits may be obtained and arrangements may be made for the required inspections.

SITE GRADING

Site Preparation and Compaction

It is anticipated that site grading for the proposed residential construction located north of the alley will include excavation of site soils for the proposed subterranean structure, foundations, and utility trenches, as well as placement of backfill for walls and trenches. For the proposed commercial/retail construction located south of the alley, it is anticipated that site grading will include removal and recompaction of existing unsuitable site soils for support of footings and floor slabs. All new fill should be placed on either undisturbed natural soils or compacted fill previously certified by a soils testing agency. All unsuitable deposits, such as any existing uncertified fill, disturbed natural soils, vegetation, and debris, should either be penetrated or removed and replaced by properly compacted fill soils for support of footings and concrete flatwork and additional fill. After excavating as required, the exposed subgrade soils should be carefully inspected by the geotechnical engineer or his representative to verify the removal of all unsuitable deposits. Subsequently, the exposed soils should be scarified to a depth of 6 inches, brought to near optimum moisture content, and compacted to a minimum of 90 percent of maximum dry density obtainable by the ASTM Designation D1557 method of compaction.

After compacting the exposed soils, all required fill should be placed in loose lifts not more than 8 inches in thickness, brought to near optimum moisture content, and properly compacted.

Very moist to wet soils were encountered in the area of the proposed commercial/retail building. The contractor should anticipate that soils to be replaced as compacted fill may require drying back to close to optimum moisture content. Alternatively, wet soils may be removed from the site and drier soils imported. Also, prior to placement of new fill on high moisture content soils, approved bottom stabilization techniques will be necessary.

Excavation Characteristics

The exploratory borings were excavated with the aid of a truck-mounted, hollow-stem auger drilling rig and hand-auger equipment. It is expected that conventional grading equipment can be utilized during construction.

Material for Fill

The on-site soils, less any debris or organic matter, may be used in compacted fills. Any required imported fill should be inspected by the geotechnical engineer prior to stockpiling onsite. Imported soils should consist of relatively granular soils with an Expansion Index of less than 20, and be free of vegetation, debris, and deleterious materials. Fill should be free of rocks larger than 6 inches in any dimension.

Utility Trench Backfill

Utility trenches should be properly backfilled in accordance with the requirements of the Green Book (latest edition). The pipe should be bedded with clean sands to a depth of at least 12 inches above the top of pipe. The use of gravel is not acceptable unless used in conjunction with filter fabric to prevent the gravel from having direct contact with the soil. The remainder of the trench should be properly backfilled with the on-site soils and mechanically compacted in lifts to the applicable minimum relative compaction. Jetting or flooding of backfill materials is not permitted.

Cesspool Abandonment

Any old cesspools encountered on the site during construction should be properly abandoned during grading operations. Cesspools should be completely cleaned out and filled to within 5 feet plus the depth of the future footings with a minimum two-sack slurry mix. The upper portion should be benched into the surrounding natural soils and the fill tested to the proper relative compaction. The entire footing footprint plus 5 feet out from the footing should be underlain by a uniform thickness of compacted fill.

Field Observation

The compaction of all required fill should be observed and tested by a representative of this firm. The observation and testing should include the following:

- Observe the exposed subgrade in areas to receive fill and in areas where excavation has resulted in the desired finished subgrade, observe proofrolling, and delineate areas requiring overexcavation.

- Perform visual observations to evaluate the suitability of on-site and import soils for fill placement; collect and submit soil samples for required or recommended laboratory testing, where necessary.

- Perform field density testing to determine the percentage of compaction achieved during fill placement. Verify proper moisture content during grading.

The governmental agencies having jurisdiction over the project should be notified prior to commencement of grading so that the necessary grading permits may be obtained and arrangements may be made for the required inspections.

EXPANSIVE SOILS

The surficial soils are considered to be Low in expansion potential. Special considerations with respect to expansive soils are not required.

TEMPORARY EXCAVATIONS

General

For the purposes of this report, a temporary excavation is considered "surcharged" when an imaginary plane projected upwards from the bottom of the excavation at an angle of 45 degrees projects below one of the following: 1) the public way, 2) a neighboring property, 3) vehicular traffic, or 4) existing structures/foundations. It is anticipated that temporary excavations up to 17 feet in height may be required for excavation and construction of the proposed subterranean level.

The temporary excavations are anticipated to expose existing fill and alluvial soils, which may be subject to caving where granular soils are exposed. Vertical excavations up to 5 feet in height may be attempted where not surcharged by adjacent traffic or structures. Vertical excavations greater than 5 feet will require sloping and/or shoring measures in order to provide a stable excavation. Where sufficient space is available, temporary unsurcharged excavations up to 12 feet high could be sloped back at a uniform 1:1 (H:V) slope gradient or flatter. A uniform slope does not have a vertical portion. Where space is limited, shoring or slot cut measures will be required. All surcharged temporary excavations should be shored or slot cut.

Where sloped embankments are utilized, the tops of the slopes should be barricaded to prevent vehicles and storage loads within 5 feet of the tops of the slopes. If the temporary construction embankments are to be maintained during the rainy season, berms are suggested along the tops of the slopes where necessary to prevent runoff water from entering the excavation and eroding the slope faces. All excavations should be stabilized within 30 days of initial excavation.

Slot Cuts

The following conditions are required for construction of slot cuts. Slot cuts are not allowed for vertical excavations in excess of 12 feet high. It is recommended that slot cut procedures be discussed at a pre-grading site meeting attended by at least the geotechnical engineer, the grading contractor, and the City grading inspector. The initial slope is to be excavated under direct observation provided by the geotechnical engineer or his representative. This initial cut is to be formed at a gradient of 1:1 (horizontal:vertical). The first series of slot cuts should be laid out along the temporary slope in a sequence identified as A-B-C, i.e. only the “A” slots would be excavated and the intervening two slots (B and C) would remain as buttresses. The removal and recompaction operations and/or footing excavations would be performed within the “A” slots. Subsequently, the “B” slots would be worked. Finally, upon completion of work within the “B” slots, the “C” slots would be excavated. The slots should be a maximum of 8 feet in width, and the intervening buttresses should be 16 feet in width.

Shoring

The following information on the design and installation of the shoring is preliminary. Review of the final shoring plans and specifications should be made by this office prior to bidding or negotiating with a shoring contractor.

A shoring system consisting of steel soldier beams is recommended for this project. The steel soldier beams may be placed in drilled holes and backfilled with concrete, or may be installed utilizing high frequency vibration. Where maximum excavation heights are less than 12 feet, the soldier piles are typically designed as cantilevers. Where excavations exceed 12 feet, or are surcharged, soldier piles may require lateral bracing utilizing drilled tie-back anchors to maintain an economical steel beam size and prevent excessive lateral deflection. The size of the steel beam and the acceptable shoring deflection should be determined by the project shoring engineer.

The design embedment of the shoring pile toes must be maintained during excavation activities. The toes of the perimeter shoring piles should be deepened to take into account any required excavations necessary for grading activities, foundations, and/or adjacent drainage systems.

Drilled, cast-in-place soldier piles should be placed no closer than 3 diameters on center. The minimum diameter of the piles is 18 inches. Structural concrete should be used for the soldier piles below the excavation; lean-mix concrete may be employed above that level. As an alternative, lean-mix concrete may be used throughout the pile where the reinforcing consists of a wide flange section. The slurry must be of sufficient strength to impart the lateral bearing pressure developed by the wide flange section to the soil. For design purposes, an allowable passive value for the soils below the bottom plane of excavation, may be assumed to be 500 pounds per square foot per foot of depth, to a maximum of 5,000 pounds per square foot. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed soils.

Casing may be required should caving be experienced in the granular soils. If casing is used, extreme care should be employed so that the pile is not pulled apart as the casing is withdrawn.

At no time should the distance between the surface of the concrete and the bottom of the casing be less than 5 feet.

It is recommended that the exposed soils be observed by the soils engineer to verify the cohesive nature of the soils and the areas where lagging may be omitted. Soldier piles should be designed for the full anticipated pressures. Due to arching in the soils, the pressure on the lagging will be less. It is recommended that the lagging be designed for the full design pressure but be limited to a maximum of 400 pounds per square foot.

Cantilever soldier piles with a level back slope may be designed for a minimum equivalent fluid pressure of 34 pounds per cubic foot. Additional pressure should be added for sloped conditions, adjacent traffic, and structural surcharge conditions.

Tied-back or raker-braced shoring, with a level backslope, may be designed for a uniform distribution of pressure of 20H pounds per square foot.

Because of the depth of the excavation, some means of monitoring the performance of the shoring system is recommended. The monitoring should include ground surface settlement and lateral deformation. Some movement of the shored embankments should be anticipated as a result of the relatively deep excavation. The shoring should be designed for a lateral movement not to exceed 1/2 inch. It is recommended that photographs of the existing improvements on the adjacent properties be made during construction to record any movements for use in the event of a dispute.

Tie-back Anchors

1. Design

Tie-back anchors may be used to resist lateral loads. Friction anchors are recommended. For design purposes, it may be assumed that the active wedge adjacent to the shoring is defined by a plane drawn 35 degrees with the vertical through the bottom plane of the excavation. Friction anchors should extend a minimum of 20 feet beyond the potentially active wedge and to greater lengths if necessary to develop the desired capacities. Only the frictional resistance developed beyond the imaginary plane would be effective in resisting lateral loads. The zone of earth material located between the imaginary plane and the face of the excavation will be termed the active wedge. The locations and depths of all offsite utilities should be thoroughly checked and incorporated into the drilling angle design for the tie-back anchors.

The capacities of the anchors should be determined by testing of the initial anchors, as outlined in a following section. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads. Anchors should be placed at least 6 feet on center to be considered isolated. For preliminary design purposes, it is estimated that drilled friction anchors constructed without utilizing post-grouting techniques will develop average skin frictions as follows: The design bond between gravity-filled tie-backs and the surrounding earth materials may be estimated to be 30 Ha pounds per square foot (factor of safety of 2.0 already included in the equation), where Ha is the depth of overburden, in feet, at the midpoint of the anchored portion of the tiebacks. The maximum resistance using this equation should not exceed 900 pounds per square foot. The design bond between pressure grout anchors and the surrounding earth materials may be estimated as being 2,000 pounds per square foot. The above values should be used only as a general guidance. The contractor must determine the appropriate value for bond based on his experience and knowledge with similar projects in similar geologic conditions. The contractor should be responsible for calculating the tie-back length since construction procedure will greatly influence the bond between the earth materials and the grout. For example, caving will loosen the earth materials and cause a significant reduction in the bond. In any event, the computed bond length should be verified by a proof-test program performed under the observation of our representative. Anchors should be placed at least 6 feet on center to be considered isolated.

The actual lengths of the anchors required may be longer than the lengths calculated using the above average skin friction values. Pull-out testing of the anchors should be performed to verify the actual length of anchors required.

2. Anchor Installation

Tied-back anchors may be installed between 20 and 40 degrees below the horizontal; however, occasionally alternative angles are necessary to avoid existing improvements and utilities. The locations and depths of all offsite utilities should be thoroughly checked prior to design and installation of the tie-back anchors. Anchor installation may be made difficult due to the presence of sand and gravel deposits. Caving of the anchor shafts should be anticipated and provisions should be implemented in order to minimize such caving. It is recommended that hollow-stem auger drilling equipment be used to install the anchors. The anchor shafts should be filled with concrete by pumping from the tip out, and the concrete should extend from the tip of the anchor to the active wedge. In order to minimize the chances of caving, it is recommended that the portion of the anchor shaft within the active wedge be backfilled with sand before testing the anchor. This portion of the shaft should be filled tightly and flush with the face of the excavation. The sand backfill should be placed by pumping; the sand may contain a small amount of cement to facilitate pumping.

3. Anchor Testing

All of the anchors should be tested to at least 150 percent of design load. The total deflection during this test should not exceed 12 inches. The rate of creep under the 150 percent test load should not exceed 0.1 inch over a 15 minute period in order for the anchor to be approved for the design loading.

At least ten percent of the anchors should be selected for "quick", 200 percent tests and three additional anchors be selected for 24-hour, 200 percent tests. The purpose of the 200 percent tests is to verify the friction value assumed in design. The anchors should be tested to develop twice the assumed friction value. These tests should be performed prior to installation of additional tie-backs. Where satisfactory tests are not achieved on the initial anchors, the anchor diameter and/or length should be increased until satisfactory test results are obtained.

The total deflection during the 24-hour 200 percent test should not exceed 12 inches. During the 24-hour tests, the anchor deflection should not exceed 0.75 inches measured after the 200 percent test load is applied.

For the "quick" 200 percent tests, the 200 percent test load should be maintained for 30 minutes. The total deflection of the anchor during the 200 percent quick tests should not exceed 12 inches; the deflection after the 200 percent load has been applied should not exceed 0.25 inch during the 30-minute period.

After a satisfactory test, each anchor should be locked-off at the design load. This should be verified by rechecking the load in the anchor. The load should be within 10 percent of the design load. The installation and testing of the anchors should be observed by a representative of this firm.

SITE SURFACE DRAINAGE

Proper surface drainage is critical to the future performance of the project. Uncontrolled infiltration of irrigation excess and storm runoff into the soils can adversely affect the performance of the planned improvements. Saturation of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change in the original designed engineering properties. Proper drainage should be maintained at all times.

All site drainage should be collected and controlled in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with applicable standards. In addition, drainage should not be allowed to flow uncontrolled over any descending slope. Discharge from downspouts, roof drains and scuppers are not recommended onto unprotected soils within 5 feet of the building perimeter. Planters which are located adjacent to foundations should be sealed to prevent moisture intrusion into the soils providing foundation support. Landscape irrigation is not recommended within 5 feet of the building perimeter footings except when enclosed in protected planters.

Positive surface gradients should be provided adjacent to all structures so as to direct surface water run-off and roof drainage away from foundations and slabs toward suitable discharge facilities.

PLAN REVIEW

This report was prepared on the basis of exploration, laboratory testing, and analysis of the existing site conditions. Formal plans ready for submittal to the Department of Building and Safety must be reviewed by Rybak Geotechnical, Inc. prior to such submittal. Any change in the scope of the project may require additional geotechnical work.

SITE OBSERVATION

It is recommended that all foundation excavations be observed by the geotechnical engineer's representative prior to placing forms, steel, or concrete. All excavations should be cleaned of loose soil and debris prior to inspection. The excavations should be approved by this office prior to the placement of reinforcing steel, concrete forms, or concrete. The geotechnical engineer should verify that the footings will be embedded into the recommended bearing material. All bottom excavations to receive compacted fill should be inspected by the geotechnical engineer prior to placing any controlled compacted fill. Any fill that is placed must be approved, tested, and verified if used for engineered purposes. Should the observations reveal any unforeseen hazard, additional recommendations may be necessary.

Please advise Rybak Geotechnical, Inc. at least 24 hours prior to any required site visit. All approved plans, permits, and geotechnical reports must be at the jobsite and available during inspections.

CONSTRUCTION SITE MAINTENANCE

The construction contractor or general contractor shall supervise and direct the work and they shall be solely responsible for all construction means, methods, techniques, sequences and procedures. Also, they shall be solely and completely responsible for conditions on the job site, including safety of all persons and property during the performance of work. Periodic or continuous observations by the geotechnical consultant will not, nor are intended to, include review of the adequacy of the contractor's safety measures in, on or near the construction site.

It is the responsibility of the contractor to maintain a safe construction site. When excavations exist on a site, the area should be fenced and warning signs posted. Earth material generated by foundation and subgrade excavations should be either removed from the site or properly placed as a controlled compacted fill. Earth material must not be spilled over any descending slope.

Workers should not be allowed to enter any unshored trench excavations over 5 feet deep. All shoring, bracing and excavation or confined space entry should be in accordance with current requirements of CAL/OSHA, the Industrial Accident Commission of the State of California, and all other public agencies having jurisdiction.

Please call this office with any questions. This report and the exploration are subject to the following "Limitations" section. Please read the "Limitations" carefully, as it limits our liability.

LIMITATIONS

This report was prepared in accordance with generally accepted geotechnical engineering principles and practice available at this time and with the degree of care and skill ordinarily exercised under similar circumstances by geotechnical engineers practicing in this area. No other representation, either expressed or implied, is made as to the conclusions and professional advice included in this report.

This report was prepared exclusively for the sole use and benefit of the clients, Tristar Realty Group, LLC, and their authorized agents, and is not transferrable. This report is intended for use with regard to the specific project discussed herein. The conclusions and recommendations contained in this report are based on the data relating only to the project and location discussed herein. Any changes in design or locations of structures from those outlined in this report should be provided to us so that we may review our conclusions and recommendations and make any necessary modifications. The subsurface conditions described in this report have been projected from borings on the site as indicated, and are believed representative of the project area.

However, soil, geologic and groundwater conditions can vary significantly between exploratory excavations and with passage of time. As with most projects, conditions revealed during construction may be at variance with the findings herein. If this occurs, the changed conditions must be evaluated by the geotechnical consultant, and additional recommendations provided, as warranted.

The recommendations contained within this report were developed with the assumption that the necessary geotechnical observations and testing will be performed during construction by a representative of this firm. If construction phase services are performed by others, they must accept full responsibility for all geotechnical aspects of the project, including this report.

Respectfully submitted,
RYBAK GEOTECHNICAL, INC.

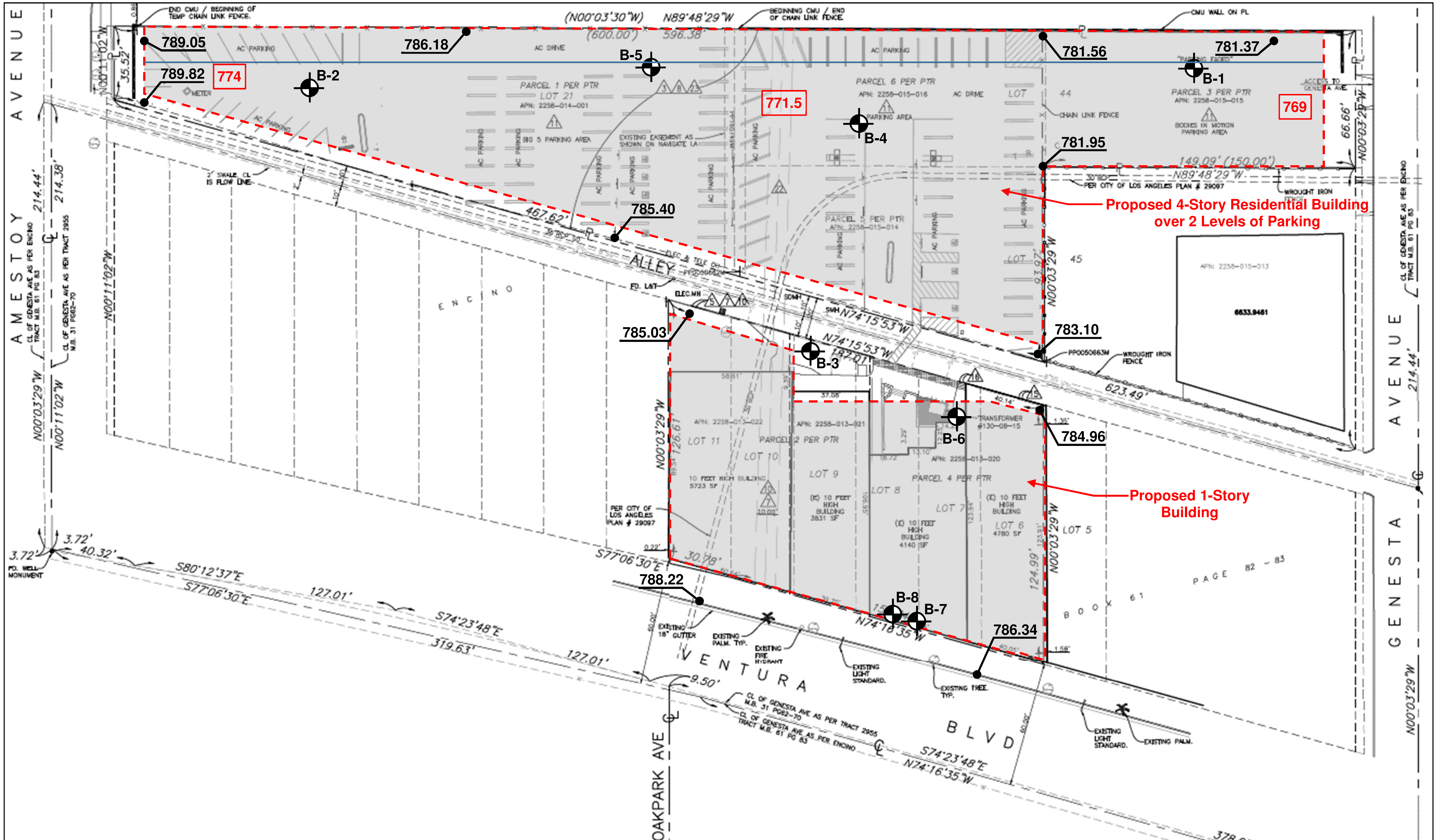
REGISTERED PROFESSIONAL ENGINEER
RICHARD RYBAK
Number 2131
Exp. 3/31/17
GEOTECHNICAL
STATE OF CALIFORNIA

RICHARD RYBAK
G.E. 2131

GEOTCHNICAL MAP

Parkpointe Encino

4929 Genesta Avenue and 17017 Ventura Blvd., Encino, CA



LEGEND:

B-8 Approximate Location of Boring

781.95 Existing Spot Elevations

769 Approximate Proposed Finish Floor Elevation

NORTH

Approximate Scale 1"=50'

Project: TRISTAR REALTY GROUP, LLC – 4929 Genesta Avenue and 17017 Ventura Blvd., Encino

Project No: 2908

Date: February 24, 2017

Rybak Geotechnical, Inc.

Consulting Engineers

Plate 1

RYBAK GEOTECHNICAL, INC.

BORING NO. 1

Client: TRISTAR (VENTURA)   Exploration Date: February 16, 2017
Project No.: 2908

Sample Depth, (ft.)	Blows per foot	Moisture Content (%)	Dry Density (lbs/cu.ft.)	Depth (ft.)	Soil Type	Description Surface Conditions: Relativley level asphalt (4" A.C.)
				0		
				1		NATURAL SOIL: Silty Sand, dark olive-brown, moist, medium-dense, occasional rock fragments, pinhole porosity
				2		
				3		dense
				4		
5.0	20	10.2	104.3	5		
				6		
				7		
				8		brown, dense
				9		
10.0	40	7.4	119.4	10		
				11		
				12		
				13	ML	Sandy, Clayey Silt, brown, moist, stiff, some rock fragments in matrix
				14		
15.0	82	12.1	118.0	15		
				16		
				17		
				18		
				19		gravelly, sandy silt, light brown, moist, very firm
20.0	79	8.6	119.9	20		
				21		
				22		
				23		
				24		clayey, sandy silt, orange-brown, moist, stiff
25.0	48	11.8	SPT	25		
				26		
27.0	72	11.2	114.4	27		
				28		
				29		
30.0	24	13.1	SPT	30		

RYBAK GEOTECHNICAL, INC.

BORING NO. 1 (Continued)

Client: TRISTAR (VENTURA) Exploration Date: February 16, 2017
Project No. 2908

Sample Depth, (ft.)	Blows per foot	Moisture Content (%)	Dry Density (lbs/cu.ft.)	Depth (ft.)	Soil Type	Description Surface Conditions:
30.0	24	13.1	SPT	30		
				31		
				32		
				33	SM	Gravelly, Silty Sand, brown, slightly moist, dense
				34		
35.0	38	3.4	SPT	35		
				36		
				37	ML	Sandy, Clayey Silt, pale green-gray, brown, orange-brown, moist, stiff
				38		
				39		
40.0	35	31.4	SPT	40		
				41		
				42		
				43		
				44		
45.0	30	30.6	SPT	45		
				46		
				47		
				48		
				49		gray, orange-brown, stiff to hard, some rock fragments
50.0	33	33.5	SPT	50		
				51		
				52		
				53		
				54		
55.0	47	48.6	SPT	55		
				56		
				57		
				58		
				59		
60.0	55	47.0	SPT	60		

RYBAK GEOTECHNICAL, INC.

BORING NO. 1 (Continued)

Client: TRISTAR (VENTURA) Exploration Date: February 16, 2017
Project No. 2908

Sample Depth, (ft.)	Blows per foot	Moisture Content (%)	Dry Density (lbs/cu.ft.)	Depth (ft.)	Soil Type	Description Surface Conditions:
				60 - -		
				-		
				61 - -		
				-		
				62 - -		
				-		
				63 - -		
				-		
				64 - -		
				-		
65.0	45	60.1	SPT	65 - -		
				-		
				66 - -		
				-		
				67 - -		
				-		
				68 - -		
				-		Total depth: 66.5 feet
				69 - -		No groundwater
				-		No Fill
				70 - -		
				-		
				71 - -		
				-		
				72 - -		
				-		
				73 - -		
				-		
				74 - -		
				-		
				75 - -		
				-		
				76 - -		
				-		
				77 - -		
				-		
				78 - -		
				-		
				79 - -		
				-		
				80 - -		
				-		
				81 - -		
				-		
				82 - -		
				-		
				83 - -		
				-		
				84 - -		
				-		
				85 - -		
				-		
				86 - -		
				-		
				87 - -		
				-		
				88 - -		
				-		
				89 - -		
				-		
				90 - -		

RYBAK GEOTECHNICAL, INC.

BORING NO. 2

Client: TRISTAR (VENTURA) Exploration Date: February 16, 2017
Project No.: 2908

Sample Depth, (ft.)	Blows per foot	Moisture Content (%)	Dry Density (lbs/cu.ft.)	Depth (ft.)	Soil Type	Description Surface Conditions: Relativley level asphalt (4" A.C.)
				0		
						NATURAL SOIL: Sandy Silt, light brown, slightly moist, firm, occasional rock fragments
				1		
				2		
				3		
				4		
5.0	27	9.2	109.0	5		
				6		
				7		
				8		
				9		
10.0	85 for 10"	6.2	121.4	10		
				11		
				12		
						Sandy, Clayey Silt, brown, moist, stiff to hard, some rock fragments in matrix
				13		
				14		
15.0	50 for 5"	11.3	115.6	15		
				16		
				17		
						sandy silt, brown, moist, stiff, some rock fragments
				18		
				19		
20.0	80	13.0	109.5	20		
				21		
				22		orange-brown, moist, stiff, occasional rock fragments
				23		
				24		
25.0	21	10.9	SPT	25		
				26		
27.0	72	13.5	112.2	27		
				28		
				29	SM	Silty Sand, orange-brown, moist, fine-grained sand fraction
30.0	23	4.0	SPT	30		

RYBAK GEOTECHNICAL, INC.

BORING NO. 2 (Continued)

Client: TRISTAR (VENTURA) Exploration Date: February 16, 2017
Project No. 2908

Sample Depth, (ft.)	Blows per foot	Moisture Content (%)	Dry Density (lbs/cu.ft.)	Depth (ft.)	Soil Type	Description Surface Conditions:
				30		
				31		
				32		
				33		
				34		
35.0	21	16.0	SPT	35		
				36		
				37		
				38		
						gravelly
				39		
40.0	33	3.0	SPT	40		
				41		
				42		
					CL	Silty Clay, brown, moist, very firm
				43		
				44		
45.0	30	16.5	SPT	45		
				46		
				47		
				48		
					ML	Clayey Silt, pale tan, moist
				49		
50.0	39	61.3	SPT	50		
				51		
				52		
				53		
				54		
55.0	47	69.4	SPT	55		
				56		
				57		
				58		
				59		
60.0	55	61.8	SPT	60		

RYBAK GEOTECHNICAL, INC.

BORING NO. 2 (Continued)

Client: TRISTAR (VENTURA) Exploration Date: February 16, 2017
Project No. 2908

Sample Depth, (ft.)	Blows per foot	Moisture Content (%)	Dry Density (lbs/cu.ft.)	Depth (ft.)	Soil Type	Description Surface Conditions:
				60		
				61		
				62		
				63		
				64		
65.0	53	62.1	SPT	65		
				66		
				67		
				68		Total depth: 66.5 feet
				69		No groundwater No Fill
				70		
				71		
				72		
				73		
				74		
				75		
				76		
				77		
				78		
				79		
				80		
				81		
				82		
				83		
				84		
				85		
				86		
				87		
				88		
				89		
				90		

RYBAK GEOTECHNICAL, INC.

BORING NO. 3

Client: TRISTAR (VENTURA) Exploration Date: February 16, 2017
Project No.: 2908

Sample Depth, (ft.)	Blows per foot	Moisture Content (%)	Dry Density (lbs/cu.ft.)	Depth (ft.)	Soil Type	Description Surface Conditions: Relativley level sidewalk (6" concrete)
				0 - -		EXISTING FILL: Silty Sand, dark brown, moist, dense
				-		
				1 - -		
				-		
				2 - -		
				-	ML	NATURAL SOIL: Sandy Silt, brown, moist, firm, fine-grained sand fraction,
				3 - -		some rock fragments
				-		
				4 - -		
				-		
5.0	21	9.1	105.1	5 - -		
				-		
				6 - -		
				-		
				7 - -		
				-		
				8 - -		
				-		
				9 - -		
				-	CL	Sandy, Silty Clay, reddish-brown, moist, stiff
10.0	50 for 6"	14.2	117.4	10 - -		
				-		
				11 - -		
				-		
				12 - -		
				-		
				13 - -		
				-	SM	Gravelly, Silty Sand, brown, moist, dense
				14 - -		
				-		
15.0	36	9.6	118.2	15 - -		
				-		
				16 - -		
				-		
				17 - -	SP	Sand, orange-brown, moist, medium to coarse-grained, some gravel, friable
				-		
				18 - -		
				-		
				19 - -		
				-		
20.0	30	6.3	115.2	20 - -		
				-		
				21 - -		
				-		
				22 - -		
				-		
				23 - -		
				-		
				24 - -	SM	Silty Sand, brown, moist, some rock fragments
				-		
25.0	35	6.4	SPT	25 - -		
				-		
				26 - -		
				-		
27.0	50 for 6"	5.2	113.6	27 - -		
				-	SP	Sand, orange-brown, moist, medium to coarse-grained, some gravel, friable
				28 - -		
				-		
				29 - -		
				-		
30.0	68	6.9	SPT	30 - -		

RYBAK GEOTECHNICAL, INC.

BORING NO. 3 (Continued)

Client: TRISTAR (VENTURA) Exploration Date: February 16, 2017

Project No. 2908

Sample Depth, (ft.)	Blows per foot	Moisture Content (%)	Dry Density (lbs/cu.ft.)	Depth (ft.)	Soil Type	Description Surface Conditions:
				30		
				31		
				32	SM	Silty Sand, brown, moist to very moist, medium-grained, gravelly
				33		
				34		
35.0	34	21.8	SPT	35		
				36		
				37		
				38		
				39		
40.0	45	20.4	SPT	40		
				41		
				42		
				43		
				44		
45.0	34	5.6	SPT	45		
				46		
				47	SP-SM	Sand with silt, brown, moist, dense
				48		
				49		
50.0	58	8.0	SPT	50		
				51		
				52		
				53	ML	Clayey, Sandy Silt, dark brown, moist, firm, fine-grained sand fraction
				54		
55.0	63	25.0	SPT	55		
				56		
				57		
				58		clayey silt, dark brown, very moist, firm
				59		
60.0	68	27.3	SPT	60		

RYBAK GEOTECHNICAL, INC.

BORING NO. 3 (Continued)

Client: TRISTAR (VENTURA) Exploration Date: February 16, 2017
Project No. 2908

Sample Depth, (ft.)	Blows per foot	Moisture Content (%)	Dry Density (lbs/cu.ft.)	Depth (ft.)	Soil Type	Description Surface Conditions:
				60		
				61		
				62		
				63		clayey silt, pale green-gray, moist
				64		
65.0	56	37.4	SPT	65		
				66		
				67		
				68		Total depth: 66.5 feet
				69		No groundwater
						Fill to 2 feet
				70		
				71		
				72		
				73		
				74		
				75		
				76		
				77		
				78		
				79		
				80		
				81		
				82		
				83		
				84		
				85		
				86		
				87		
				88		
				89		
				90		

RYBAK GEOTECHNICAL, INC.

BORING NO. 4

Client: TRISTAR (VENTURA) Exploration Date: February 16, 2017
Project No.: 2908

Sample Depth, (ft.)	Blows per foot	Moisture Content (%)	Dry Density (lbs/cu.ft.)	Depth (ft.)	Soil Type	Description Surface Conditions: Relativley level sidewalk (6" concrete)
				0		EXISTING FILL: Silty Sand, dark brown, moist, dense
				1		
				2	ML	NATURAL SOIL: Sandy Silt, dark brown, moist, firm, clay binder
				3		
				4		
5.0	2	11.9	SPT	5		
				6		
				7		
				8		clayey silt, dark brown, moist, stiff, some rock fragments
				9		
10.0	30	13.0	SPT	10		
				11		
				12		
				13		
				14		clayey, sandy silt, reddish-brown, moist, stiff, some rock fragments
15.0	49	11.6	SPT	15		
				16		
				17		
				18		
				19		
20.0	44	15.1	SPT	20		
				21		
				22		clayey silt, brown, very moist, firm
				23		
				24		
25.0	17	12.3	SPT	25		
				26		
				27		
				28		Total depth: 26.5 feet No groundwater Fill to 1.5 feet
				29		
				30		

RYBAK GEOTECHNICAL, INC.

BORING NO. 5

Client: TRISTAR (VENTURA)　　Exploration Date: February 17, 2017

Project No.: 2908

Sample Depth, (ft.)	Blows per foot	Moisture Content (%)	Dry Density (lbs/cu.ft.)	Depth (ft.)	Soil Type	Description Surface Conditions: 4" Asphalt parking lot
				0		NATURAL SOIL: Clayey, Sandy Silt, dark brown, moist, medium-firm
				1		
				2		
				3		
				4		
						sandy, clayey silt, brown, moist, stiff, some rock fragments
5.0	14	10.4	SPT	5		
				6		
				7		
				8		
				9		
10.0	27	13.6	SPT	10		
				11		
				12		
				13		
				14		
						clayey, sandy silt
15.0	34	7.8	SPT	15		
				16		
				17		
				18		
				19	SM	Silty Sand, pale brown-gray, moist, some rock fragments
20.0	44	6	SPT	20		
				21		
				22		
				23		
						Total depth: 21.5 feet
				24		No groundwater
						No Fill
				25		
				26		
				27		
				28		
				29		
				30		

RYBAK GEOTECHNICAL, INC.

BORING NO. 6

Client: TRISTAR (VENTURA)   Exploration Date: February 22, 2017
Project No.: 2908

Sample Depth, (ft.)	Blows per foot	Moisture Content (%)	Dry Density (lbs/cu.ft.)	Depth (ft.)	Soil Type	Description Surface Conditions:
				0 - -		EXISTING FILL: Silty Sand, dark brown, very moist, some rock fragments and concrete fragments
				-		
				1 - -		
1.5	HS	13.8	102.8	-		
				2 - -		
				-		
				3 - -		
3.5	HS	19.1	103.3	-		
				4 - -	ML	NATURAL SOILS: Sandy Silt, dark brown, moist, some rock fragments
				-		
				5 - -		
5.5	HS	21.0	100.2	-		
				6 - -		
				-		
				7 - -		
				-		Total depth: 6 feet
				8 - -		No groundwater
				-		Fill to 3.5 feet
				9 - -		
				-		
				10 - -		
				-		
				11 - -		
				-		
				12 - -		
				-		
				13 - -		
				-		
				14 - -		
				-		
				15 - -		
				-		
				16 - -		
				-		
				17 - -		
				-		
				18 - -		
				-		
				19 - -		
				-		
				20 - -		
				-		
				21 - -		
				-		
				22 - -		
				-		
				23 - -		
				-		
				24 - -		
				-		
				25 - -		
				-		
				26 - -		
				-		
				27 - -		
				-		
				28 - -		
				-		
				29 - -		
				-		
				30 - -		

RYBAK GEOTECHNICAL, INC.

BORING NO. 7

Client: TRISTAR (VENTURA) Exploration Date: February 22, 2017
Project No.: 2908

Sample Depth, (ft.)	Blows per foot	Moisture Content (%)	Dry Density (lbs/cu.ft.)	Depth (ft.)	Soil Type	Description Surface Conditions:
				0		EXISTING FILL: Sandy Silt and Silty Sand, dark brown, wet, soft, soupy
				1		
				2		
				3		
				4		
4.5	HS	5.9	114.8			Gravelly Sand, dark brown, wet, medium to coarse-grained sand
				5		refusal at 6 feet on baseball-size cobble
				6		
				7		Total depth: 6 feet
				8		No groundwater
						Fill to total depth of 6 feet
				9		
				10		
				11		
				12		
				13		
				14		
				15		
				16		
				17		
				18		
				19		
				20		
				21		
				22		
				23		
				24		
				25		
				26		
				27		
				28		
				29		
				30		

RYBAK GEOTECHNICAL, INC.

BORING NO. 8

Client: TRISTAR (VENTURA) Exploration Date: February 22, 2017
Project No.: 2908

Sample Depth, (ft.)	Blows per foot	Moisture Content (%)	Dry Density (lbs/cu.ft.)	Depth (ft.)	Soil Type	Description Surface Conditions:
				0 - -		EXISTING FILL: Sandy Silt and Silty Sand, dark brown, wet, soft
				-		
				1 - -		refusal at 1-1/2 feet on concrete
1.5	HS	11.6	116.1	-		
				2 - -		
				-		
				3 - -		
				-		Total depth: 1.5 feet
				4 - -		No groundwater
				-		Fill to total depth of 1-1/2 feet
				5 - -		
				-		
				6 - -		
				-		
				7 - -		
				-		
				8 - -		
				-		
				9 - -		
				-		
				10 - -		
				-		
				11 - -		
				-		
				12 - -		
				-		
				13 - -		
				-		
				14 - -		
				-		
				15 - -		
				-		
				16 - -		
				-		
				17 - -		
				-		
				18 - -		
				-		
				19 - -		
				-		
				20 - -		
				-		
				21 - -		
				-		
				22 - -		
				-		
				23 - -		
				-		
				24 - -		
				-		
				25 - -		
				-		
				26 - -		
				-		
				27 - -		
				-		
				28 - -		
				-		
				29 - -		
				-		
				30 - -		

Table 1. Type and Quantity of Laboratory Test

Laboratory Test	Quantity	ASTM Standard
Dry Density	20	-----
Moisture Content	55	D-2216
Consolidation	6	D-2435
Direct Shear	4	D-3080
Swell Test	1	HUD 60 psf Method
Passing #200 Sieve	3	D-1140
Atterberg Limits	1	D-423,424

Table 2. Results of the Dry Density-Moisture Content Tests

Location	Depth ft.	Soil Description	Dry Density, pcf	Moisture Content, %
B1	5.0	Dark olive-brown, silty Sand	104.3	102.
B1	10.0	Brown, silty Sand	119.4	7.4
B1	15.0	Brown, sandy clayey Silt	118.0	12.1
B1	20.0	Light brown, gravelly sandy Silt	119.9	8.6
B1	25.0	Orange-brown, clayey sandy Silt	SPT	11.8
B1	27.0	Orange-brown, clayey sandy Silt	114.4	11.2
B1	30.0	Orange-brown, clayey sandy Silt	SPT	13.1
B1	35.0	Brown, gravelly silty Sand	SPT	3.4
B1	40.0	Pale green-gray/brown/orange-brown, sandy clayey Silt	SPT	31.4
B1	45.0	Pale green-gray/brown/orange-brown, sandy clayey Silt	SPT	30.6
B1	50.0	Gray/orange-brown, sandy clayey Silt	SPT	33.5
B1	55.0	Gray/orange-brown, sandy clayey Silt	SPT	48.6
B1	60.0	Gray/orange-brown, sandy clayey Silt	SPT	47.0
B1	65.0	Gray/orange-brown, sandy clayey Silt	SPT	60.1
B2	5.0	Light brown, sandy Silt	109.0	9.2
B2	10.0	Light brown, sandy Silt	121.4	6.2
B2	15.0	Brown, sandy clayey Silt	115.6	11.3
B2	20.0	Brown, sandy Silt	109.5	13.0
B2	25.0	Orange-brown, sandy Silt	SPT	10.9
B2	27.0	Orange-brown, sandy Silt	112.2	13.5

Table 2. Results of the Dry Density-Moisture Content Tests (Continued)

Location	Depth ft.	Soil Description	Dry Density, pcf	Moisture Content, %
B2	30.0	Orange-brown, silty Sand	SPT	4.0
B2	35.0	Orange-brown, silty Sand	SPT	16.0
B2	40.0	Orange-brown, gravelly silty Sand	SPT	3.0
B2	45.0	Brown, silty Clay	SPT	16.5
B2	50.0	Pale tan, clayey Silt	SPT	61.3
B2	55.0	Pale tan, clayey Silt	SPT	69.4
B2	60.0	Pale tan, clayey Silt	SPT	61.8
B2	65.0	Pale tan, clayey Silt	SPT	62.1
B3	5.0	Brown, sandy Silt	105.1	9.1
B3	10.0	Red-brown, sandy silty Clay	117.4	14.2
B3	15.0	Brown, gravelly silty Sand	118.2	9.6
B3	20.0	Orange-brown, Sand	115.2	6.3
B3	25.0	Brown, silty Sand	SPT	6.4
B3	27.0	Orange-brown, Sand	113.6	5.2
B3	30.0	Orange-brown, Sand	SPT	6.9
B3	35.0	Brown, silty Sand	SPT	21.8
B3	40.0	Brown, silty Sand	SPT	20.4
B3	45.0	Brown, silty Sand	SPT	5.6
B3	50.0	Brown, Sand with silt	SPT	8.0
B3	55.0	Dark brown, clayey sandy Silt	SPT	25.0
B3	60.0	Dark brown, clayey Silt	SPT	27.3
B3	65.0	Pale green-gray, clayey Silt	SPT	37.4
B4	5.0	Dark brown, sandy Silt	SPT	11.9
B4	10.0	Dark brown, clayey Silt	SPT	13.0
B4	15.0	Red-brown, clayey sandy Silt	SPT	11.6
B4	20.0	Red-brown, clayey sandy Silt	SPT	15.1
B4	25.0	Brown, clayey Silt	SPT	12.3
B5	5.0	Brown, sandy clayey Silt	SPT	10.4
B5	10.0	Brown, sandy clayey Silt	SPT	13.6
B5	15.0	Brown, clayey sandy Silt	SPT	7.8

Table 2. Results of the Dry Density-Moisture Content Tests (Continued)

Location	Depth ft.	Soil Description	Dry Density, pcf	Moisture Content, %
B5	20.0	Pale brown-gray, silty Sand	SPT	6.0
B6	1.5	Dark brown, silty Sand	102.8	13.8
B6	3.5	Dark brown, sandy Silt	103.3	19.1
B6	5.5	Dark brown, sandy Silt	100.2	21.0
B7	4.5	Dark brown, gravelly Sand	114.8	5.9
B8	1.5	Dark brown, sandy Silt and silty Sand	116.1	11.6

Table 3. Results of Swell Test

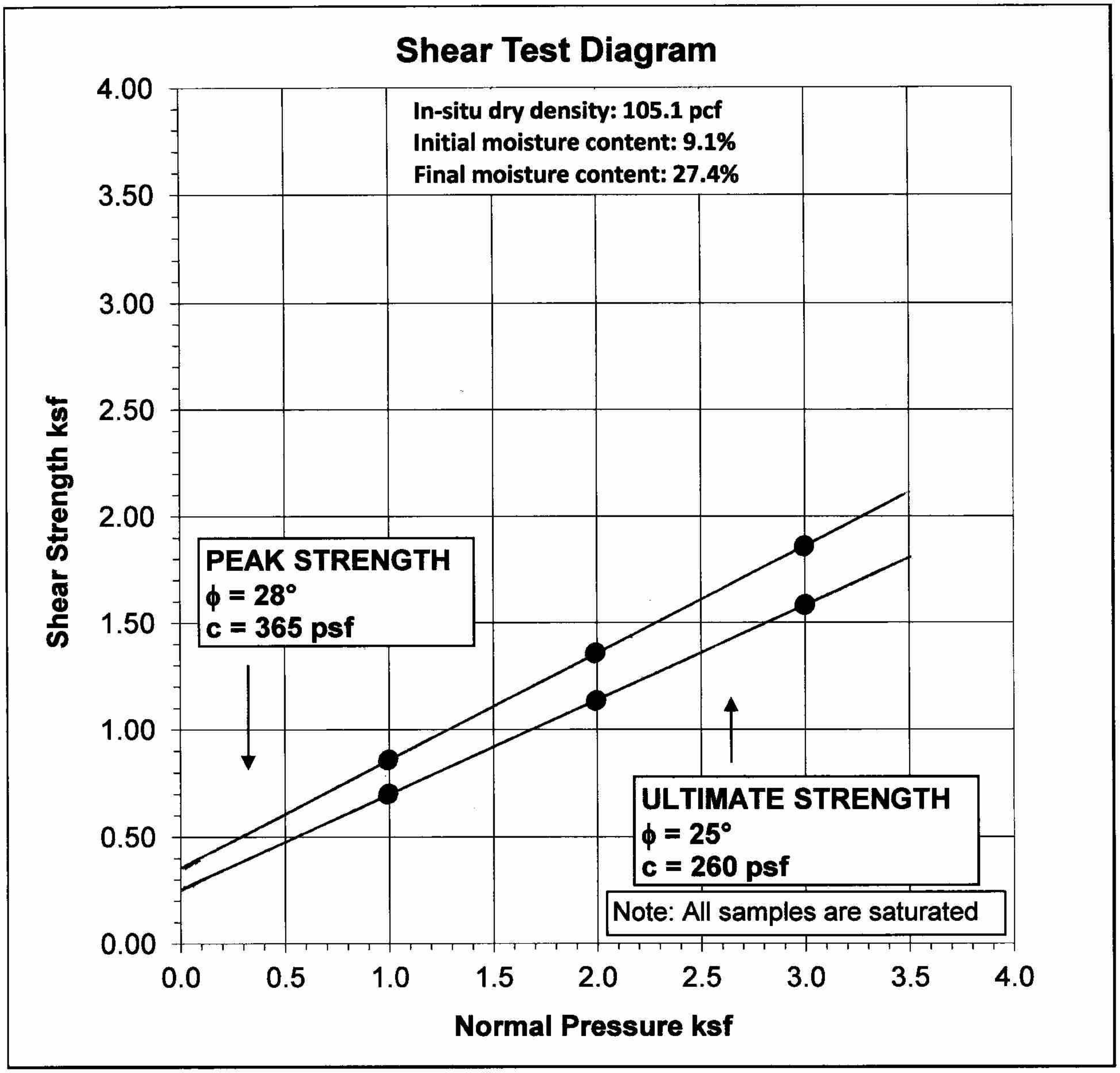
Location	Depth ft	Soil Descriptions	% Swell
B1	15.0	Brown, sandy clayey Silt	1.2 (Very Low)

Table 4. Results of Percentage Passing #200 Sieve Test

Location	Depth ft	Soil Description	% Passing #200 Sieve
B1	30.0	Orange-brown, clayey sandy Silt	62.0
B1	50.0	Gray/orange-brown, sandy clayey Silt	69.0
B2	35.0	Orange-brown, silty Sand	34.0

Table 5. Results of Atterberg Limits Tests

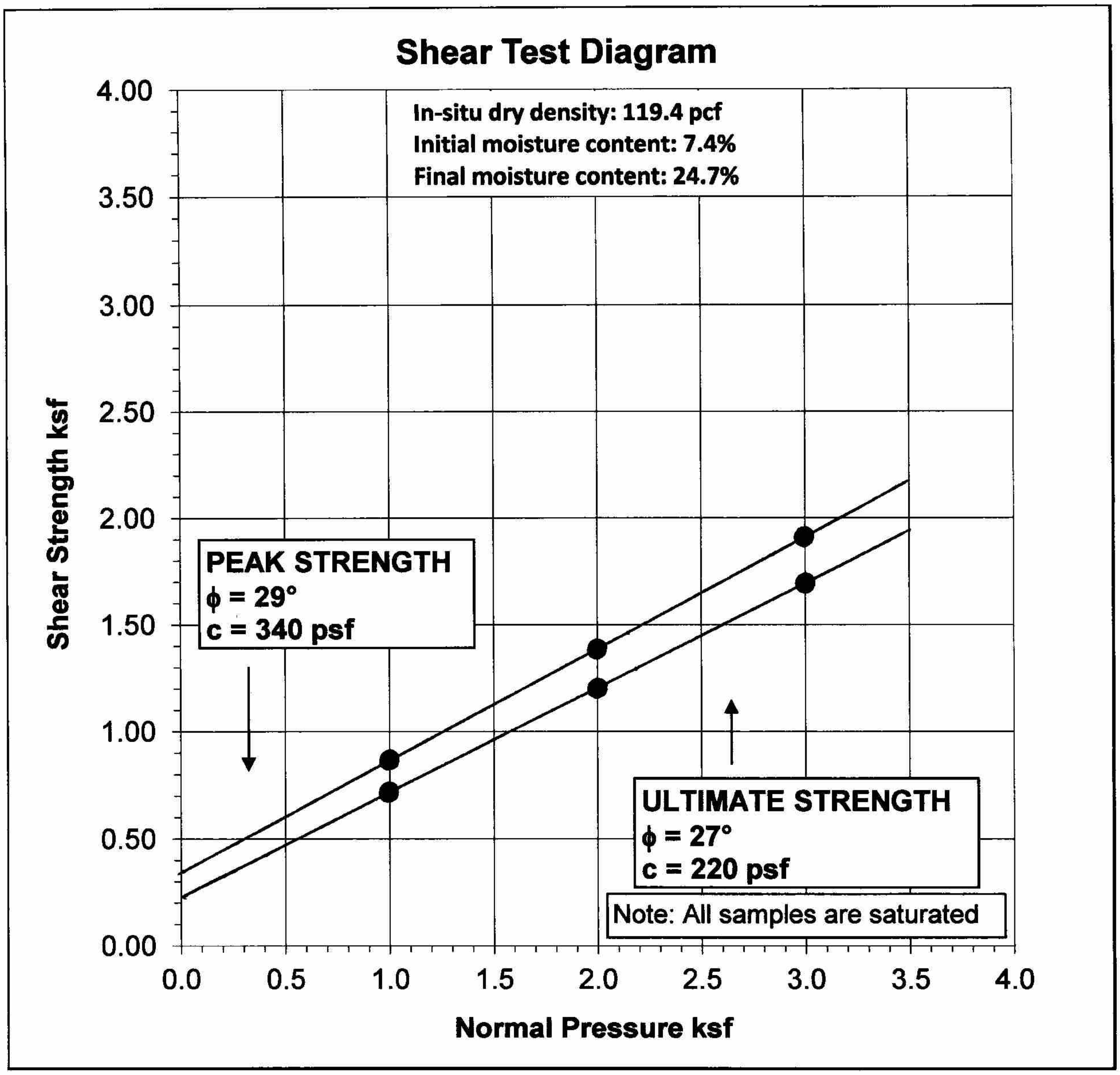
Location	Depth ft	Soil Description	Liquid Limit	Plastic Limit	Plasticity Index	U.S.C.S. Symbol
B2	45.0	Brown, silty Clay	48	25	23	CL



DATE: 2/21/2017	**SAMPLE: B3 - 5'**
CLIENT: TRISTAR (VENTURA)	**PROJECT NO. 2908**

RYBAK GEOTECHNICAL, INC.

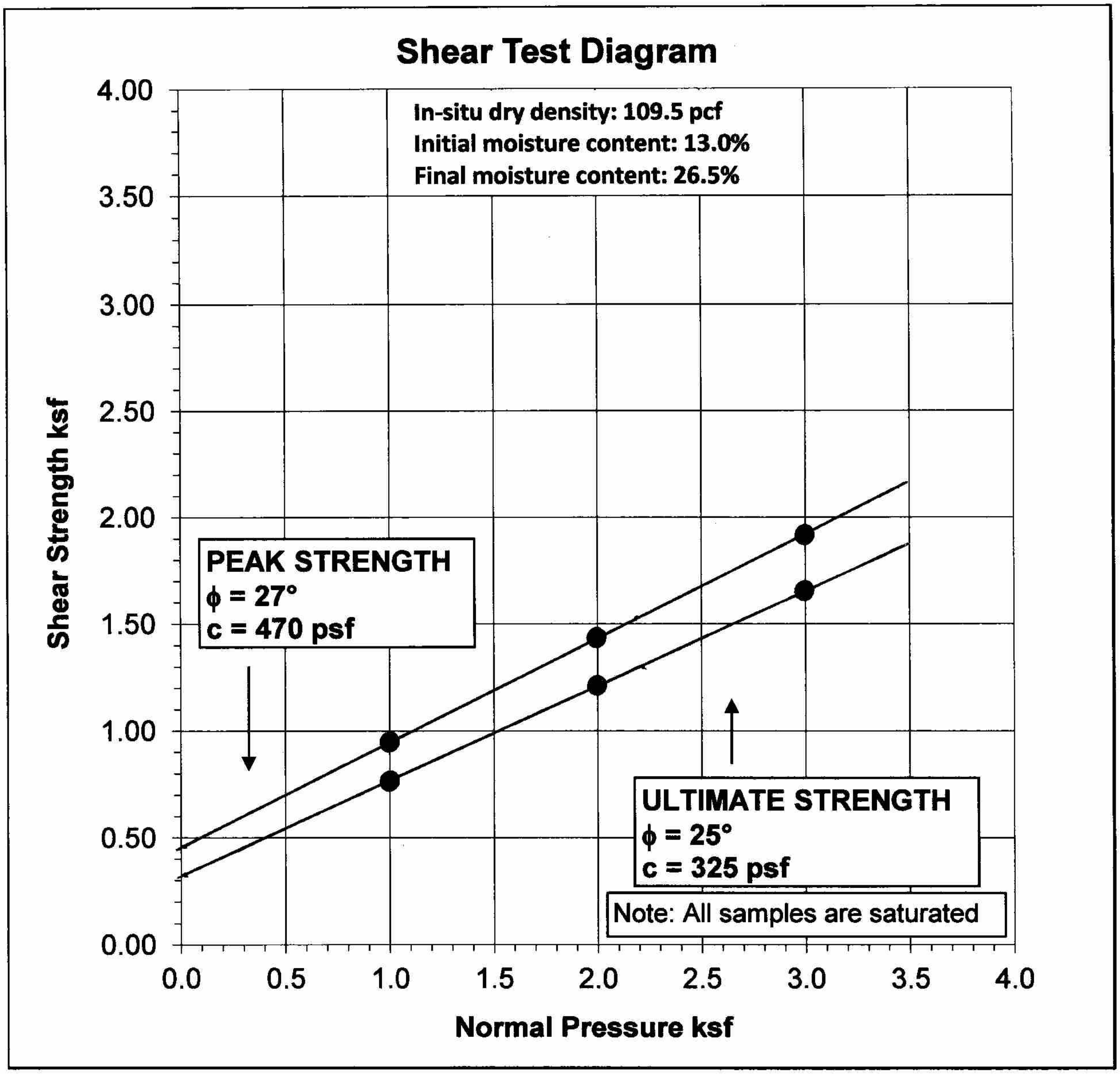
Plate B-1



DATE: 2/21/2017	**SAMPLE: B1 - 10'**
CLIENT: TRISTAR (VENTURA)	**PROJECT NO. 2908**

RYBAK GEOTECHNICAL, INC.

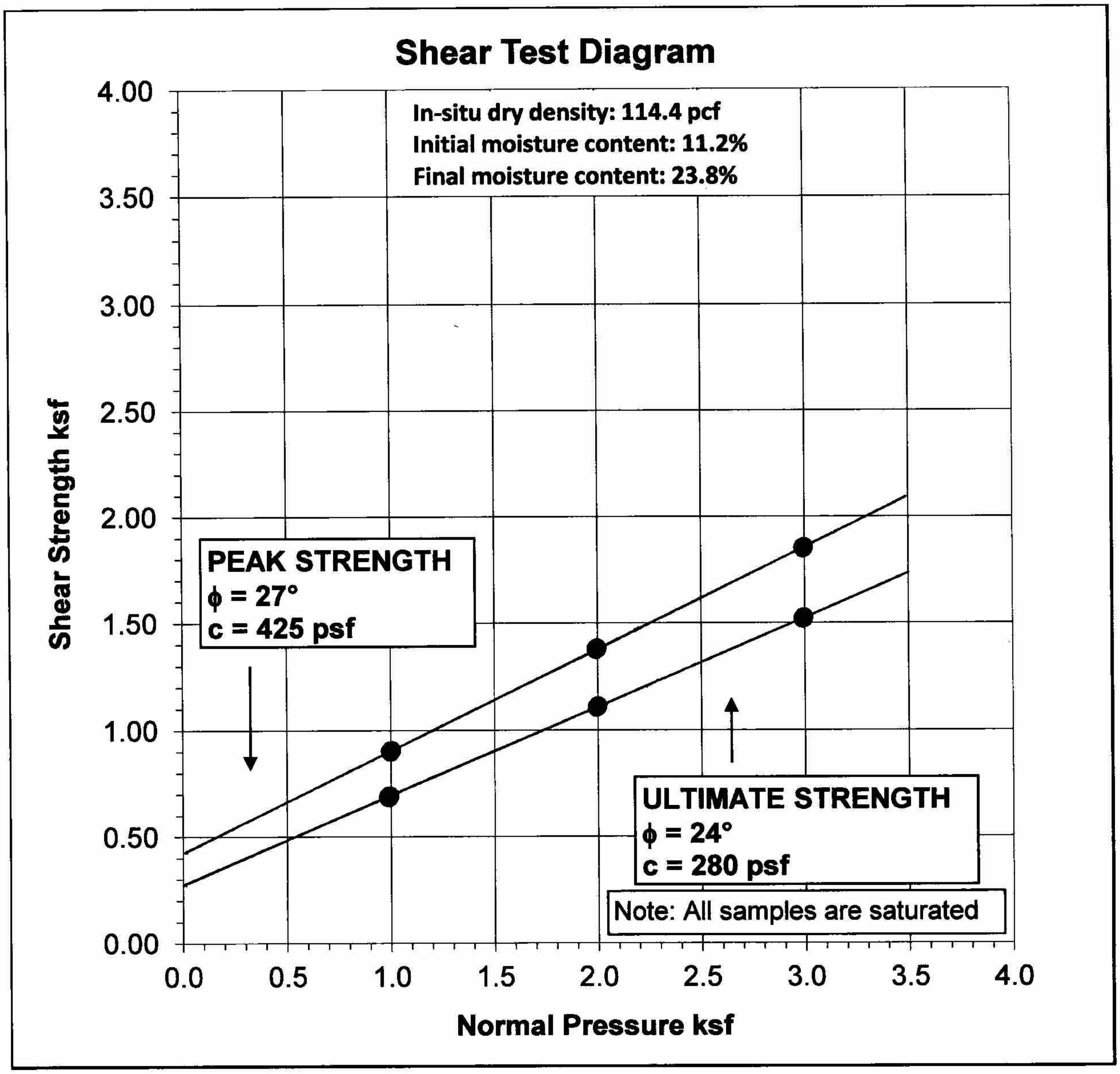
Plate B-2



DATE: 2/21/2017	SAMPLE: B2 - 20'
CLIENT: TRISTAR (VENTURA)	PROJECT NO. 2908

RYBAK GEOTECHNICAL, INC.

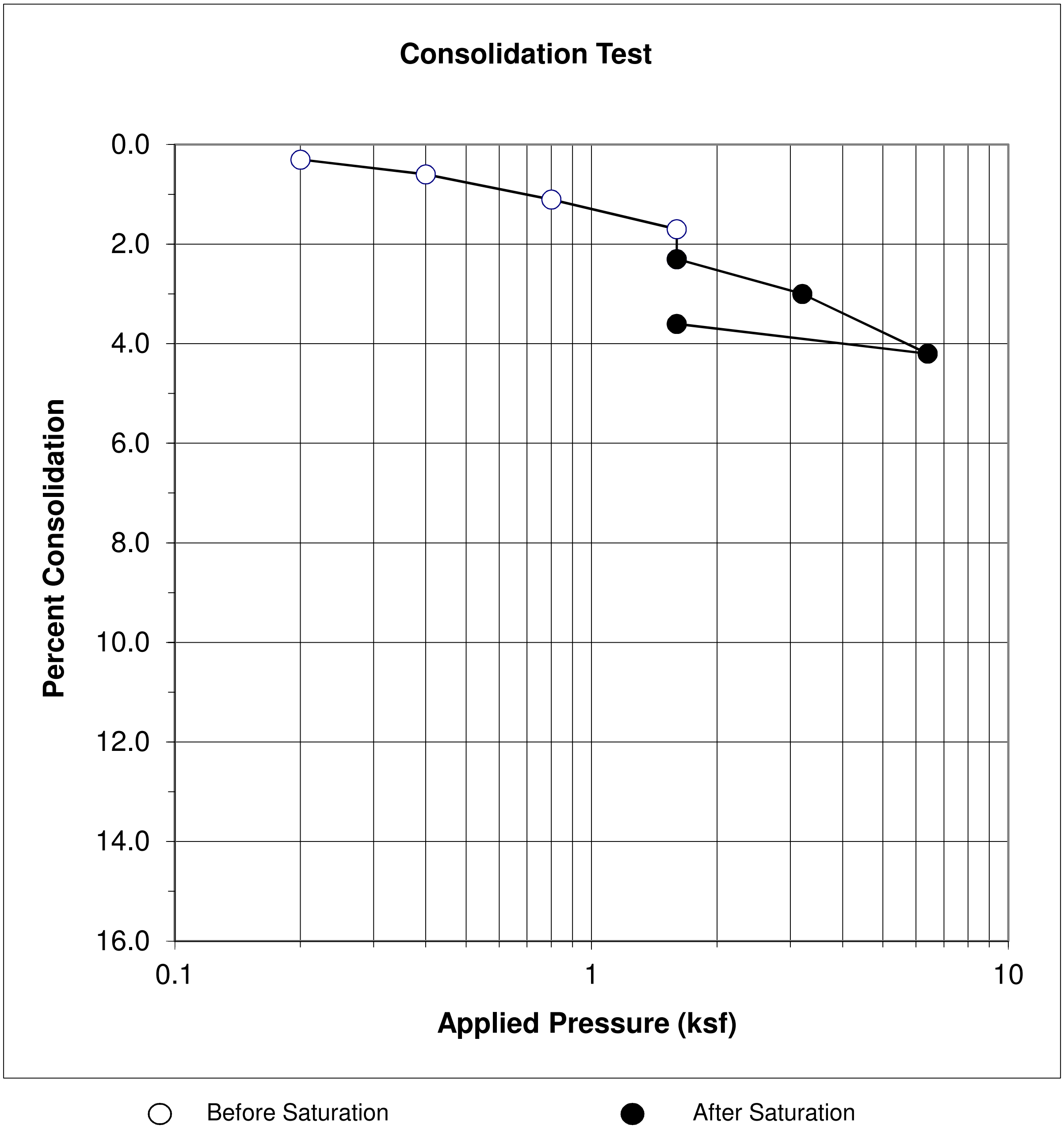
Plate B-3



DATE: 2/21/2017	SAMPLE: B1 - 27'
CLIENT: TRISTAR (VENTURA)	PROJECT NO. 2908

RYBAK GEOTECHNICAL, INC.

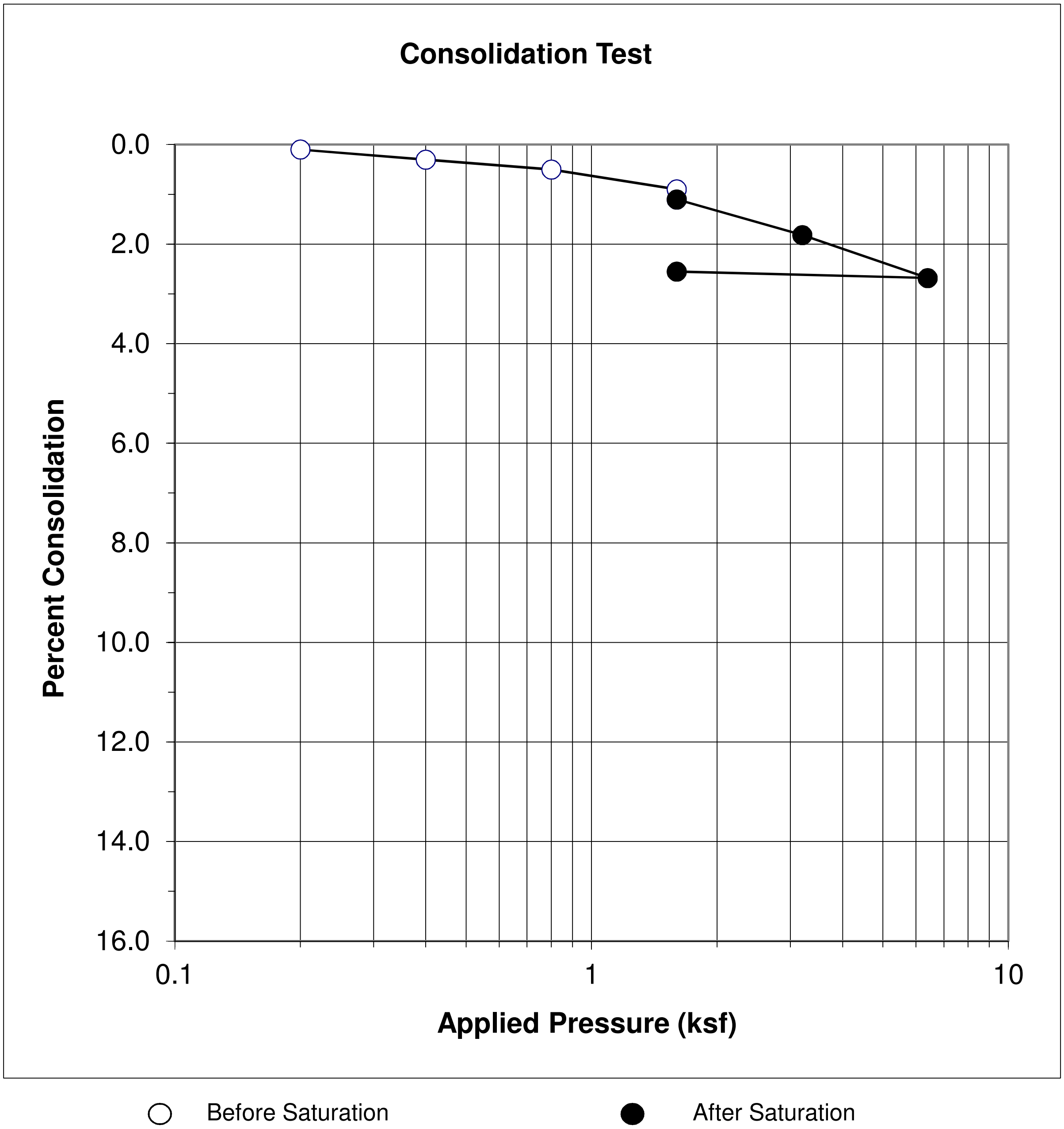
Plate B-4



DATE: 2/23/2017	SAMPLE: B6-3.5
CLIENT: TRISTAR (VENTURA)	PROJECT NO. 2908

RYBAK GEOTECHNICAL, INC.

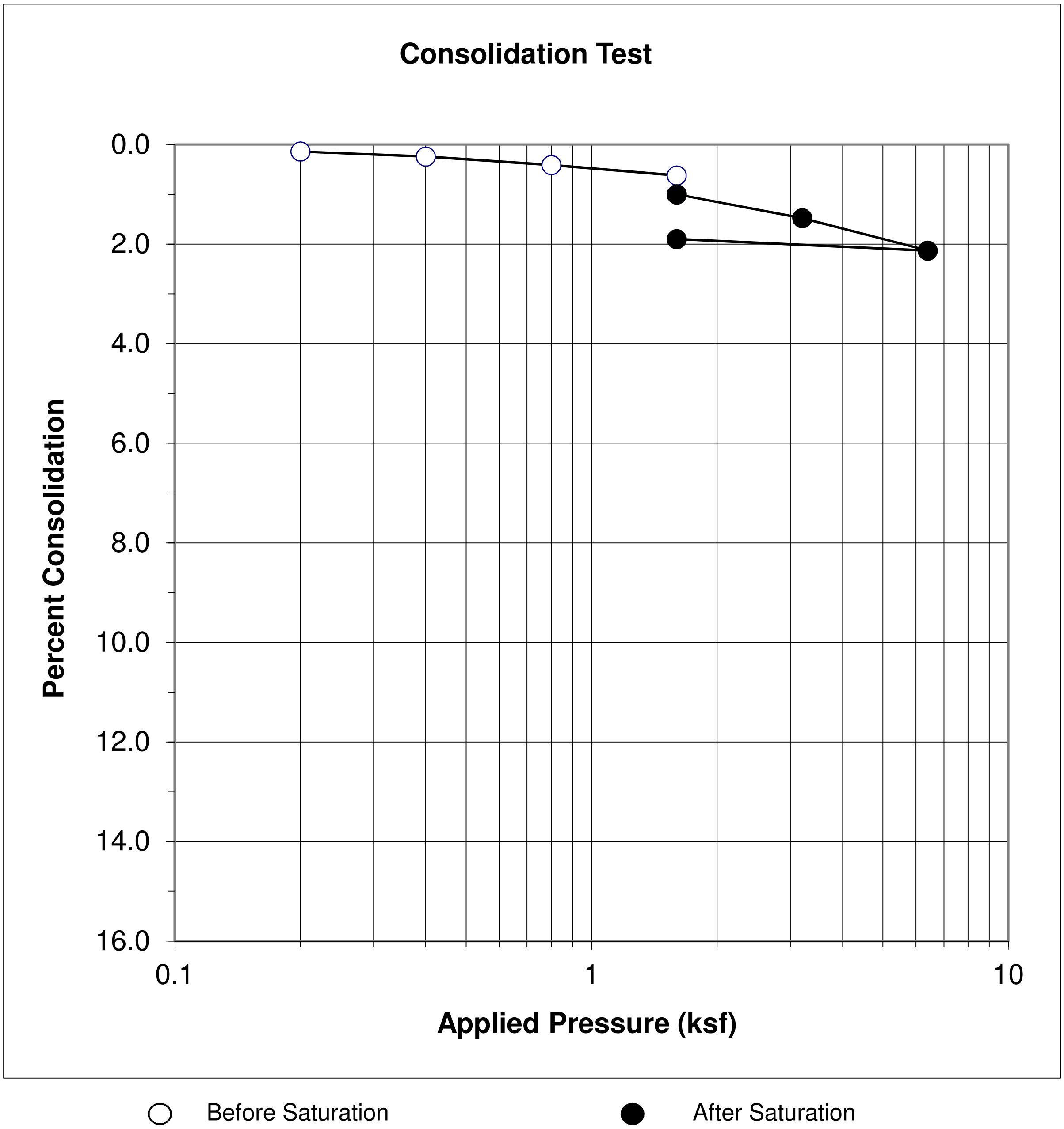
Plate C-1



DATE: 2/23/2017	**SAMPLE: B3-10'**
CLIENT: TRISTAR (VENTURA)	**PROJECT NO. 2908**

RYBAK GEOTECHNICAL, INC.

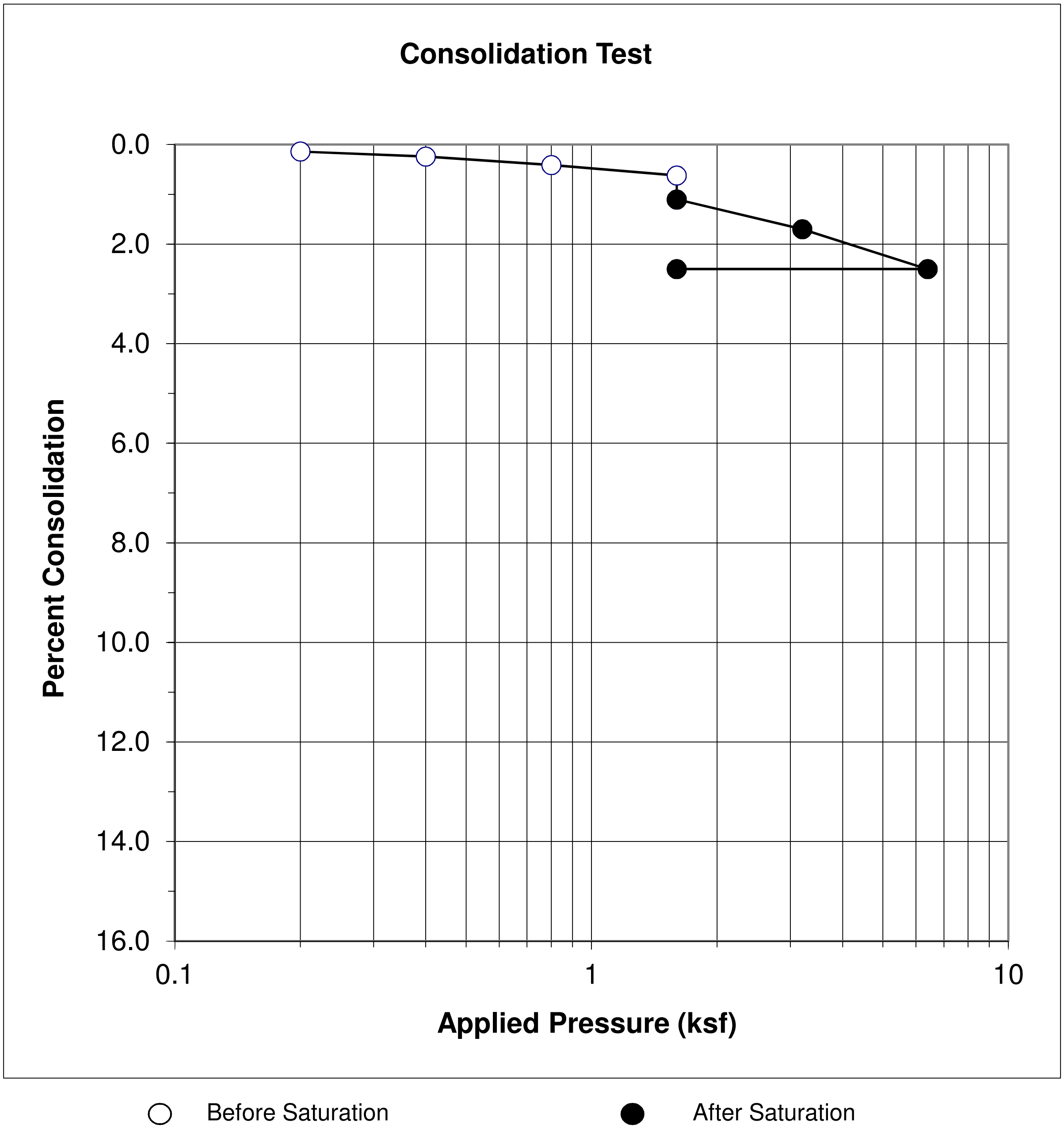
Plate C-2



DATE: 2/23/2017	**SAMPLE: B2-15'**
CLIENT: TRISTAR (VENTURA)	**PROJECT NO. 2908**

RYBAK GEOTECHNICAL, INC.

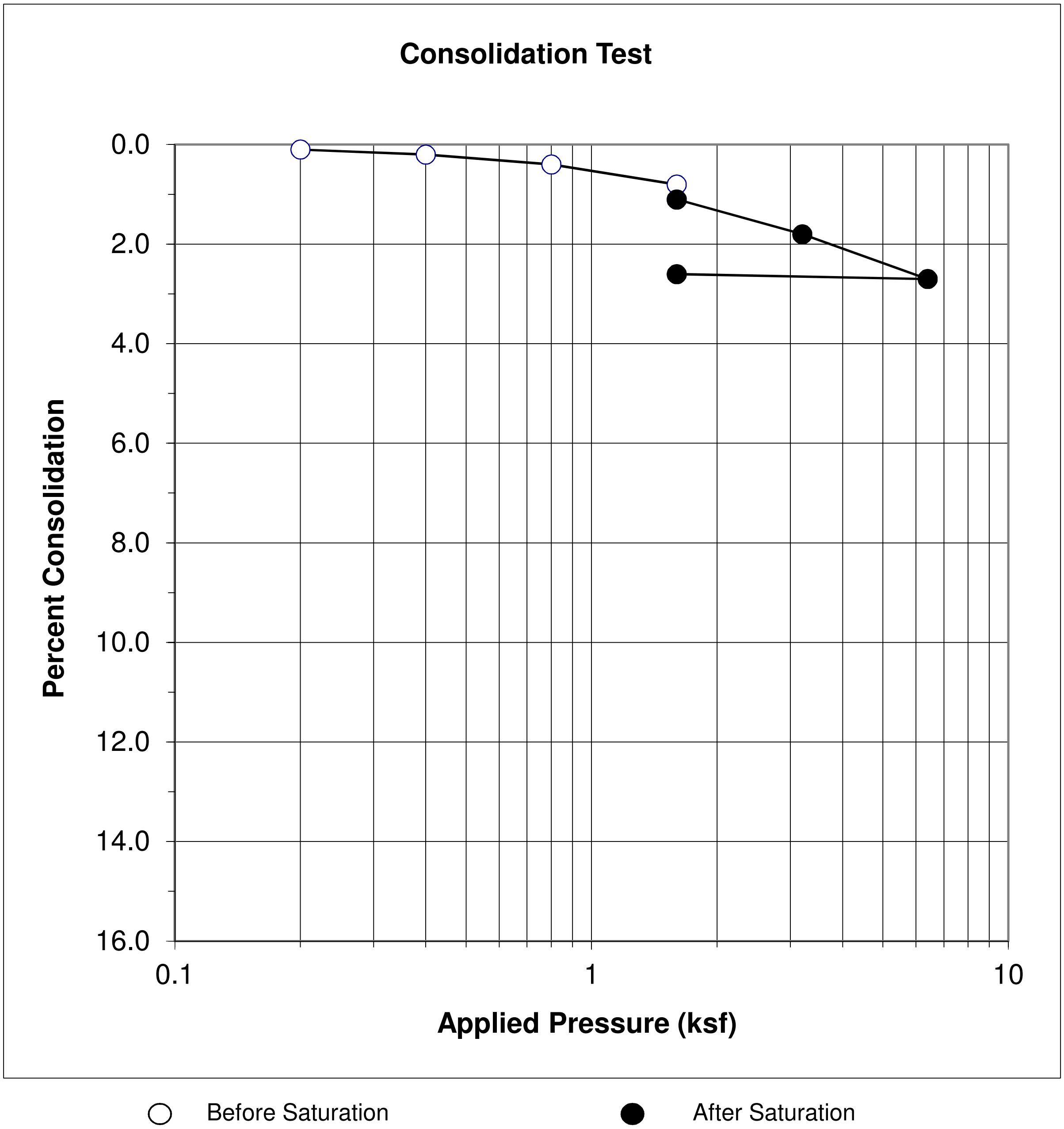
Plate C-3



DATE: 2/23/2017	SAMPLE: B3-27'
CLIENT: TRISTAR (VENTURA)	PROJECT NO. 2908

RYBAK GEOTECHNICAL, INC.

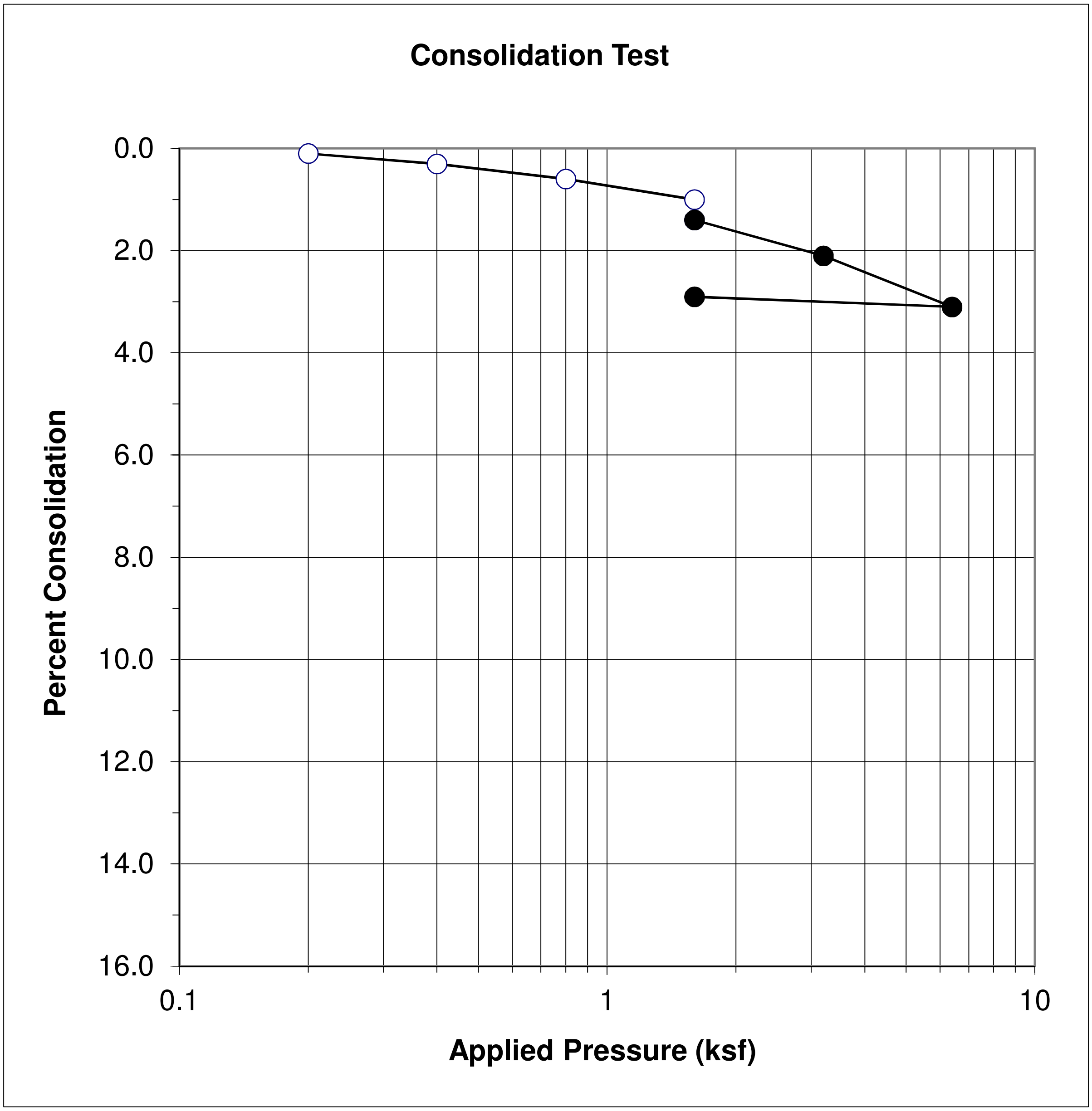
Plate C-4



DATE: 2/23/2017	SAMPLE: B3-35'
CLIENT: TRISTAR (VENTURA)	PROJECT NO. 2908

RYBAK GEOTECHNICAL, INC.

Plate C-5



○ Before Saturation ● After Saturation

DATE: 2/23/2017	SAMPLE: B1-45'
CLIENT: TRISTAR (VENTURA)	PROJECT NO. 2908

RYBAK GEOTECHNICAL, INC.

Plate C-6

USGS Design Maps Detailed Report

ASCE 7-10 Standard (34.1607°N, 118.5044°W)

Site Class D – "Stiff Soil", Risk Category I/II/III

Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

From Figure 22-1 [1] $S_S = 2.061$ g

From Figure 22-2 [2] $S_1 = 0.725$ g

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Chapter 20.

Table 20.3–1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ **or** $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index *PI* > 20, • Moisture content *w* ≥ 40%, and • Undrained shear strength $\bar{s}_u$ < 500 psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 11.4.3 — Site Coefficients and Risk–Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

Table 11.4–1: Site Coefficient F_a

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at Short Period				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	$\boxed{1.0}$
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_S

For Site Class = D and S_S = 2.061 g, F_a = 1.000

Table 11.4–2: Site Coefficient F_v

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at 1–s Period				
	$S_1 \le 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \ge 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	$\boxed{1.5}$
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_1

For Site Class = D and S_1 = 0.725 g, F_v = 1.500

Equation (11.4–1): $S_{MS} = F_aS_S = 1.000 \times 2.061 = 2.061$ g

Equation (11.4–2): $S_{M1} = F_vS_1 = 1.500 \times 0.725 = 1.088$ g

Section 11.4.4 — Design Spectral Acceleration Parameters

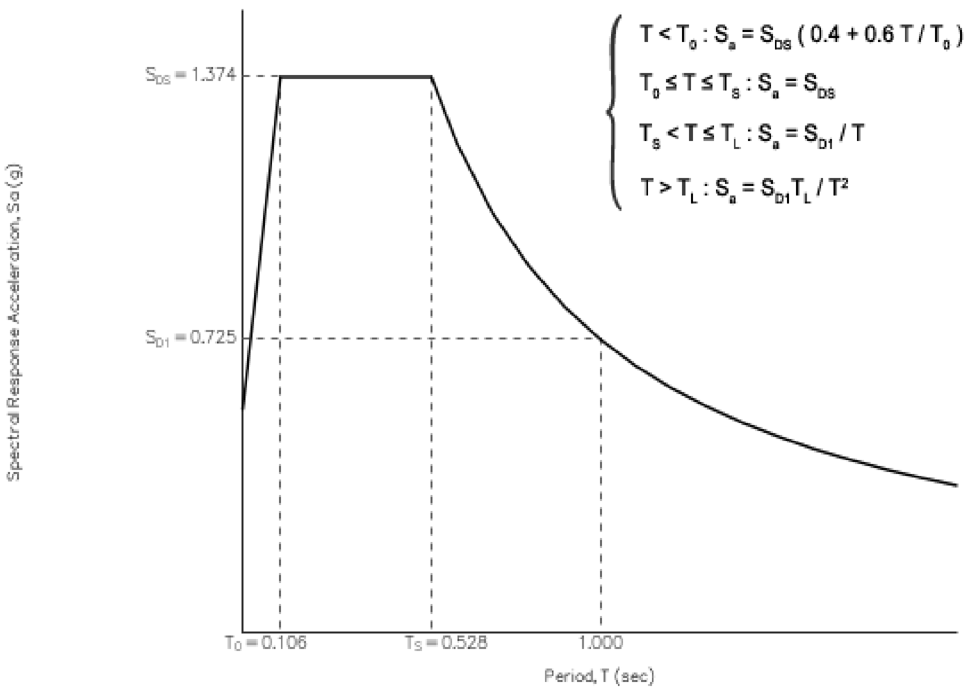
Equation (11.4–3): $S_{DS} = ⅔ S_{MS} = ⅔ \times 2.061 = 1.374$ g

Equation (11.4–4): $S_{D1} = ⅔ S_{M1} = ⅔ \times 1.088 = 0.725$ g

Section 11.4.5 — Design Response Spectrum

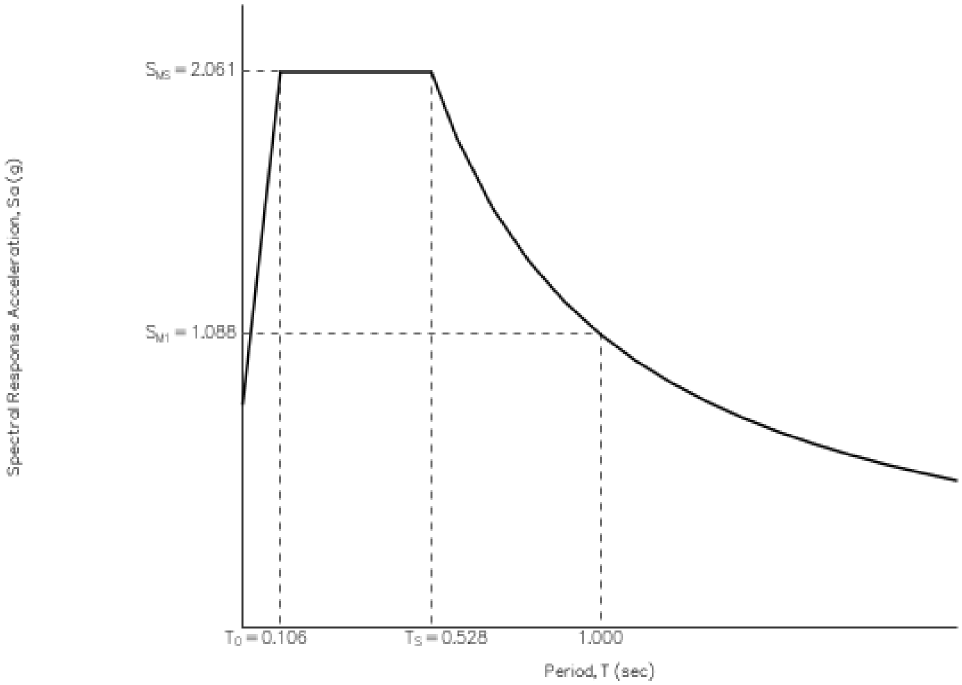
From Figure 22-12 [3] T_L = 8 seconds

Figure 11.4–1: Design Response Spectrum



Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum

The MCE_R Response Spectrum is determined by multiplying the design response spectrum above by 1.5.



Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7 [4] PGA = 0.715

Equation (11.8–1): $PGA_M = F_{PGA}PGA = 1.000 \times 0.715 = 0.715$ g

Table 11.8–1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of PGA

For Site Class = D and PGA = 0.715 g, F_{PGA} = 1.000

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17 [5] C_{RS} = 1.015

From Figure 22-18 [6] C_{R1} = 1.031

Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
S_{DS} < 0.167g	A	A	A
0.167g ≤ S_{DS} < 0.33g	B	B	C
0.33g ≤ S_{DS} < 0.50g	C	C	D
0.50g ≤ S_{DS}	D	D	D

For Risk Category = I and S_{DS} = 1.374 g, Seismic Design Category = D

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
S_{D1} < 0.067g	A	A	A
0.067g ≤ S_{D1} < 0.133g	B	B	C
0.133g ≤ S_{D1} < 0.20g	C	C	D
0.20g ≤ S_{D1}	D	D	D

For Risk Category = I and S_{D1} = 0.725 g, Seismic Design Category = D

Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ "the more severe design category in accordance with Table 11.6-1 or 11.6-2" = D

Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 22-1*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. *Figure 22-2*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. *Figure 22-12*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. *Figure 22-7*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. *Figure 22-17*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. *Figure 22-18*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf

PSH Deaggregation on NEHRP D soil

Tristar_Ventura 118.504° W, 34.161 N.

Peak Horiz. Ground Accel.>=0.4539 g

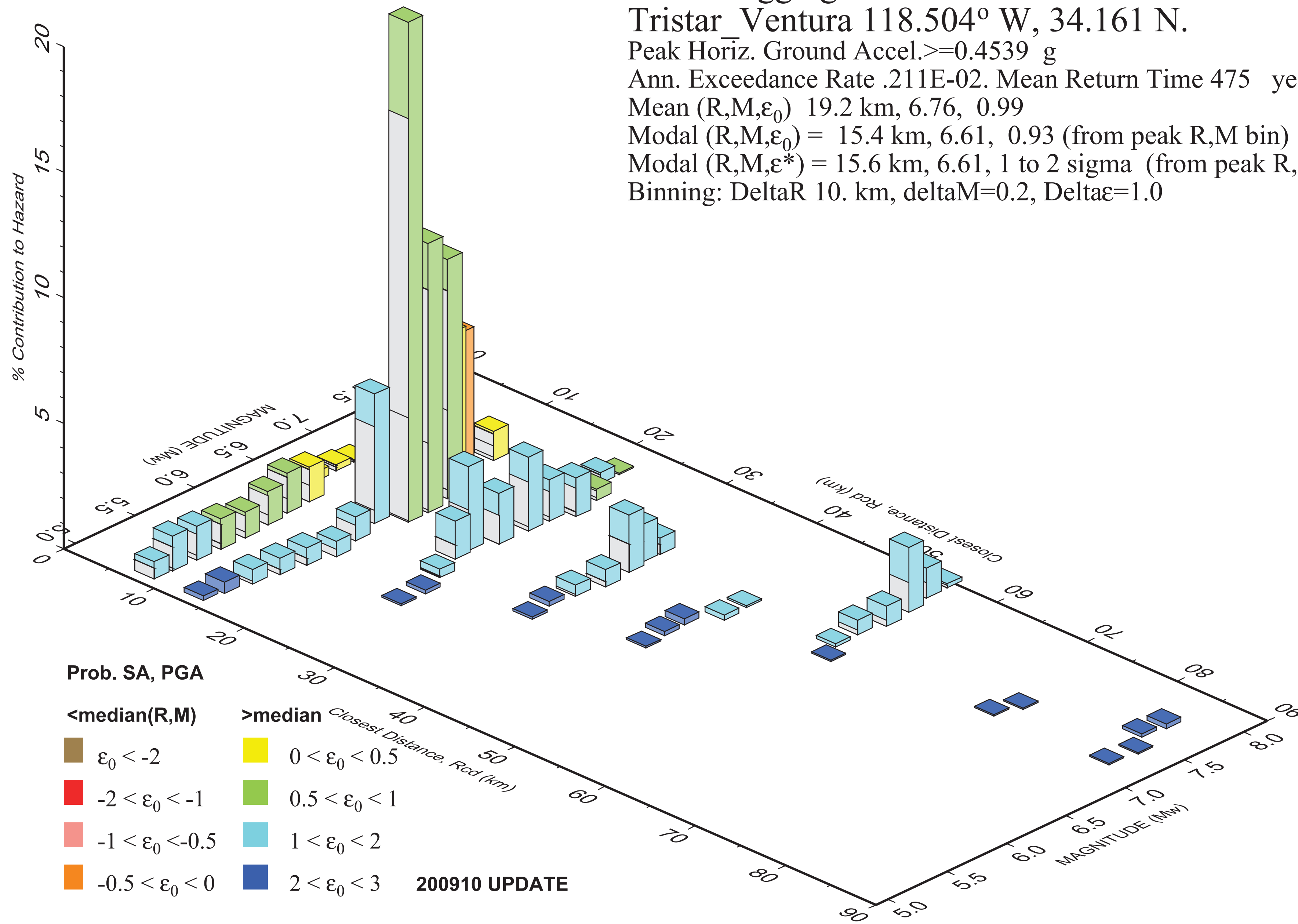
Ann. Exceedance Rate .211E-02. Mean Return Time 475 years

Mean (R,M,ε_0) 19.2 km, 6.76, 0.99

Modal (R,M,ε_0) = 15.4 km, 6.61, 0.93 (from peak R,M bin)

Modal (R,M,ε^*) = 15.6 km, 6.61, 1 to 2 sigma (from peak R,M,ε bin)

Binning: DeltaR 10. km, deltaM=0.2, Deltaε=1.0



GMT 2017 Feb 22 18:04:24 Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on soil with average vs= 250. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted

```
*****************************
*                           *
*     L I Q U E F Y 2       *
*                           *
*       Version 1.50        *
*                           *
*****************************
```

EMPIRICAL PREDICTION OF
EARTHQUAKE-INDUCED LIQUEFACTION POTENTIAL

JOB NUMBER: 2908   DATE: 02-22-2017

JOB NAME: Triv12

SOIL-PROFILE NAME: triv12.LDW

BORING GROUNDWATER DEPTH: 66.50 ft

CALCULATION GROUNDWATER DEPTH: 30.00 ft

DESIGN EARTHQUAKE MAGNITUDE: 6.61 Mw

SITE PEAK GROUND ACCELERATION: 0.715 g

BOREHOLE DIAMETER CORRECTION FACTOR: 1.00

SAMPLER SIZE CORRECTION FACTOR: 1.20

N60 HAMMER CORRECTION FACTOR: 1.30

MAGNITUDE SCALING FACTOR METHOD: Idriss (1998, in press)

    Magnitude Scaling Factor: 1.242

rd-CORRECTION METHOD: NCEER (1997)

FIELD SPT N-VALUES ARE CORRECTED FOR THE LENGTH OF THE DRIVE RODS.

    Rod Stick-Up Above Ground: 0.0 ft

CN NORMALIZATION FACTOR: 1.044 tsf

MINIMUM CN VALUE: 0.6

NCEER [1997] Method LIQUEFACTION ANALYSIS SUMMARY

File Name: triv12.OUT

SOIL NO.	CALC. DEPTH (ft)	TOTAL STRESS (tsf)	EFF. STRESS (tsf)	FIELD N (B/ft)	FC DELTA N1_60	C N	CORR. (N1)60 (B/ft)	LIQUE. RESIST RATIO	r d	INDUC. STRESS RATIO	LIQUE. SAFETY FACTOR
1	0.25	0.015	0.015	10	~	*	*	*	*	*	**
1	0.75	0.045	0.045	10	~	*	*	*	*	*	**
1	1.25	0.075	0.075	10	~	*	*	*	*	*	**
1	1.75	0.105	0.105	10	~	*	*	*	*	*	**
1	2.25	0.135	0.135	10	~	*	*	*	*	*	**
1	2.75	0.165	0.165	10	~	*	*	*	*	*	**
1	3.25	0.195	0.195	10	~	*	*	*	*	*	**
1	3.75	0.225	0.225	10	~	*	*	*	*	*	**
1	4.25	0.255	0.255	10	~	*	*	*	*	*	**
1	4.75	0.285	0.285	10	~	*	*	*	*	*	**
1	5.25	0.315	0.315	10	~	*	*	*	*	*	**
1	5.75	0.345	0.345	10	~	*	*	*	*	*	**
1	6.25	0.375	0.375	10	~	*	*	*	*	*	**
1	6.75	0.405	0.405	10	~	*	*	*	*	*	**
1	7.25	0.435	0.435	10	~	*	*	*	*	*	**
1	7.75	0.465	0.465	10	~	*	*	*	*	*	**
1	8.25	0.495	0.495	10	~	*	*	*	*	*	**
1	8.75	0.525	0.525	10	~	*	*	*	*	*	**
1	9.25	0.555	0.555	10	~	*	*	*	*	*	**
1	9.75	0.585	0.585	10	~	*	*	*	*	*	**
1	10.25	0.615	0.615	10	~	*	*	*	*	*	**
1	10.75	0.645	0.645	10	~	*	*	*	*	*	**
1	11.25	0.675	0.675	10	~	*	*	*	*	*	**
1	11.75	0.705	0.705	10	~	*	*	*	*	*	**
1	12.25	0.735	0.735	10	~	*	*	*	*	*	**
1	12.75	0.765	0.765	10	~	*	*	*	*	*	**
1	13.25	0.795	0.795	10	~	*	*	*	*	*	**
1	13.75	0.825	0.825	10	~	*	*	*	*	*	**
1	14.25	0.855	0.855	10	~	*	*	*	*	*	**
1	14.75	0.885	0.885	10	~	*	*	*	*	*	**
1	15.25	0.915	0.915	10	~	*	*	*	*	*	**
1	15.75	0.945	0.945	10	~	*	*	*	*	*	**
1	16.25	0.975	0.975	10	~	*	*	*	*	*	**
1	16.75	1.005	1.005	10	~	*	*	*	*	*	**
1	17.25	1.035	1.035	10	~	*	*	*	*	*	**
1	17.75	1.065	1.065	10	~	*	*	*	*	*	**
1	18.25	1.095	1.095	10	~	*	*	*	*	*	**
1	18.75	1.125	1.125	10	~	*	*	*	*	*	**
1	19.25	1.155	1.155	10	~	*	*	*	*	*	**
1	19.75	1.185	1.185	10	~	*	*	*	*	*	**
1	20.25	1.215	1.215	10	~	*	*	*	*	*	**
1	20.75	1.245	1.245	10	~	*	*	*	*	*	**
1	21.25	1.275	1.275	10	~	*	*	*	*	*	**

File Name: triv12.OUT

SOIL NO.	CALC. DEPTH (ft)	TOTAL STRESS (tsf)	EFF. STRESS (tsf)	FIELD N (B/ft)	FC DELTA N1_60	C N	CORR. (N1)60 (B/ft)	LIQUE. RESIST RATIO	r d	INDUC. STRESS RATIO	LIQUE. SAFETY FACTOR
1	21.75	1.305	1.305	10	~	*	*	*	*	*	**
1	22.25	1.335	1.335	10	~	*	*	*	*	*	**
1	22.75	1.365	1.365	10	~	*	*	*	*	*	**
1	23.25	1.395	1.395	10	~	*	*	*	*	*	**
1	23.75	1.425	1.425	10	~	*	*	*	*	*	**
1	24.25	1.455	1.455	10	~	*	*	*	*	*	**
1	24.75	1.485	1.485	10	~	*	*	*	*	*	**
1	25.25	1.515	1.515	10	~	*	*	*	*	*	**
1	25.75	1.545	1.545	10	~	*	*	*	*	*	**
1	26.25	1.575	1.575	10	~	*	*	*	*	*	**
1	26.75	1.605	1.605	10	~	*	*	*	*	*	**
1	27.25	1.635	1.635	10	~	*	*	*	*	*	**
1	27.75	1.665	1.665	10	~	*	*	*	*	*	**
1	28.25	1.695	1.695	10	~	*	*	*	*	*	**
1	28.75	1.725	1.725	10	~	*	*	*	*	*	**
1	29.25	1.755	1.755	10	~	*	*	*	*	*	**
1	29.75	1.785	1.785	10	~	*	*	*	*	*	**
2	30.25	1.815	1.807	24	10.57	0.758	39.0	Infin	0.928	0.433	NonLiq
2	30.75	1.845	1.822	24	10.57	0.758	39.0	Infin	0.924	0.435	NonLiq
2	31.25	1.875	1.836	24	10.57	0.758	39.0	Infin	0.920	0.437	NonLiq
2	31.75	1.905	1.850	24	10.57	0.758	39.0	Infin	0.916	0.438	NonLiq
3	32.25	1.935	1.865	38	0.05	0.703	41.7	Infin	0.912	0.440	NonLiq
3	32.75	1.965	1.879	38	0.05	0.703	41.7	Infin	0.907	0.441	NonLiq
3	33.25	1.995	1.894	38	0.05	0.703	41.7	Infin	0.903	0.442	NonLiq
3	33.75	2.025	1.908	38	0.05	0.703	41.7	Infin	0.899	0.444	NonLiq
3	34.25	2.055	1.922	38	0.05	0.703	41.7	Infin	0.895	0.445	NonLiq
3	34.75	2.085	1.937	38	0.05	0.703	41.7	Infin	0.891	0.446	NonLiq
3	35.25	2.115	1.951	38	0.05	0.703	41.7	Infin	0.887	0.447	NonLiq
3	35.75	2.145	1.966	38	0.05	0.703	41.7	Infin	0.883	0.448	NonLiq
3	36.25	2.175	1.980	38	0.05	0.703	41.7	Infin	0.879	0.449	NonLiq
4	36.75	2.205	1.994	35	0.05	0.657	35.9	Infin	0.875	0.450	NonLiq
4	37.25	2.235	2.009	35	0.05	0.657	35.9	Infin	0.871	0.450	NonLiq
4	37.75	2.265	2.023	35	0.05	0.657	35.9	Infin	0.867	0.451	NonLiq
4	38.25	2.295	2.038	35	0.05	0.657	35.9	Infin	0.863	0.452	NonLiq
4	38.75	2.325	2.052	35	0.05	0.657	35.9	Infin	0.859	0.452	NonLiq
4	39.25	2.355	2.066	35	0.05	0.657	35.9	Infin	0.855	0.453	NonLiq
4	39.75	2.385	2.081	35	0.05	0.657	35.9	Infin	0.851	0.453	NonLiq
4	40.25	2.415	2.095	35	0.05	0.657	35.9	Infin	0.846	0.453	NonLiq
4	40.75	2.445	2.110	35	0.05	0.657	35.9	Infin	0.842	0.454	NonLiq
4	41.25	2.475	2.124	35	0.05	0.657	35.9	Infin	0.838	0.454	NonLiq
4	41.75	2.505	2.138	35	0.05	0.657	35.9	Infin	0.834	0.454	NonLiq
4	42.25	2.535	2.153	35	0.05	0.657	35.9	Infin	0.830	0.454	NonLiq
5	42.75	2.565	2.167	30	10.70	0.620	39.7	Infin	0.826	0.454	NonLiq
5	43.25	2.595	2.182	30	10.70	0.620	39.7	Infin	0.822	0.454	NonLiq

File Name: triv12.OUT

SOIL NO.	CALC. DEPTH (ft)	TOTAL STRESS (tsf)	EFF. STRESS (tsf)	FIELD N (B/ft)	FC DELTA N1_60	C N	CORR. (N1)60 (B/ft)	LIQUE. RESIST RATIO	r d	INDUC. STRESS RATIO	LIQUE. SAFETY FACTOR
5	43.75	2.625	2.196	30	10.70	0.620	39.7	Infin	0.818	0.454	NonLiq
5	44.25	2.655	2.210	30	10.70	0.620	39.7	Infin	0.814	0.454	NonLiq
5	44.75	2.685	2.225	30	10.70	0.620	39.7	Infin	0.810	0.454	NonLiq
5	45.25	2.715	2.239	30	10.70	0.620	39.7	Infin	0.806	0.454	NonLiq
5	45.75	2.745	2.254	30	10.70	0.620	39.7	Infin	0.802	0.454	NonLiq
5	46.25	2.775	2.268	30	10.70	0.620	39.7	Infin	0.798	0.454	NonLiq
5	46.75	2.805	2.282	30	10.70	0.620	39.7	Infin	0.794	0.453	NonLiq
5	47.25	2.835	2.297	30	10.70	0.620	39.7	Infin	0.789	0.453	NonLiq
5	47.75	2.865	2.311	30	10.70	0.620	39.7	Infin	0.785	0.452	NonLiq
6	48.25	2.895	2.326	33	0.04	0.600	30.9	Infin	0.781	0.452	NonLiq
6	48.75	2.925	2.340	33	0.04	0.600	30.9	Infin	0.777	0.452	NonLiq
6	49.25	2.955	2.354	33	0.04	0.600	30.9	Infin	0.773	0.451	NonLiq
6	49.75	2.985	2.369	33	0.04	0.600	30.9	Infin	0.769	0.450	NonLiq
6	50.25	3.015	2.383	33	0.04	0.600	30.9	Infin	0.765	0.450	NonLiq
6	50.75	3.045	2.398	33	0.04	0.600	30.9	Infin	0.761	0.449	NonLiq
6	51.25	3.075	2.412	33	0.04	0.600	30.9	Infin	0.757	0.448	NonLiq
6	51.75	3.105	2.426	33	0.04	0.600	30.9	Infin	0.753	0.448	NonLiq
6	52.25	3.135	2.441	33	0.04	0.600	30.9	Infin	0.749	0.447	NonLiq
7	52.75	3.165	2.455	47	0.05	0.600	44.0	Infin	0.745	0.446	NonLiq
7	53.25	3.195	2.470	47	0.05	0.600	44.0	Infin	0.741	0.445	NonLiq
7	53.75	3.225	2.484	47	0.05	0.600	44.0	Infin	0.737	0.444	NonLiq
7	54.25	3.255	2.498	47	0.05	0.600	44.0	Infin	0.732	0.444	NonLiq
7	54.75	3.285	2.513	47	0.05	0.600	44.0	Infin	0.728	0.443	NonLiq
7	55.25	3.315	2.527	47	0.05	0.600	44.0	Infin	0.724	0.442	NonLiq
7	55.75	3.345	2.542	47	0.05	0.600	44.0	Infin	0.720	0.441	NonLiq
7	56.25	3.375	2.556	47	0.05	0.600	44.0	Infin	0.716	0.440	NonLiq
7	56.75	3.405	2.570	47	0.05	0.600	44.0	Infin	0.712	0.438	NonLiq
7	57.25	3.435	2.585	47	0.05	0.600	44.0	Infin	0.708	0.437	NonLiq
8	57.75	3.465	2.599	55	0.06	0.600	51.5	Infin	0.704	0.436	NonLiq
8	58.25	3.495	2.614	55	0.06	0.600	51.5	Infin	0.700	0.435	NonLiq
8	58.75	3.525	2.628	55	0.06	0.600	51.5	Infin	0.696	0.434	NonLiq
8	59.25	3.555	2.642	55	0.06	0.600	51.5	Infin	0.692	0.433	NonLiq
8	59.75	3.585	2.657	55	0.06	0.600	51.5	Infin	0.688	0.431	NonLiq
8	60.25	3.615	2.671	55	0.06	0.600	51.5	Infin	0.684	0.430	NonLiq
8	60.75	3.645	2.686	55	0.06	0.600	51.5	Infin	0.680	0.429	NonLiq
8	61.25	3.675	2.700	55	0.06	0.600	51.5	Infin	0.676	0.427	NonLiq
8	61.75	3.705	2.714	55	0.06	0.600	51.5	Infin	0.671	0.426	NonLiq
8	62.25	3.735	2.729	55	0.06	0.600	51.5	Infin	0.667	0.425	NonLiq
9	62.75	3.765	2.743	45	0.05	0.600	42.2	Infin	0.663	0.423	NonLiq
9	63.25	3.795	2.758	45	0.05	0.600	42.2	Infin	0.659	0.422	NonLiq
9	63.75	3.825	2.772	45	0.05	0.600	42.2	Infin	0.655	0.420	NonLiq
9	64.25	3.855	2.786	45	0.05	0.600	42.2	Infin	0.651	0.419	NonLiq
9	64.75	3.885	2.801	45	0.05	0.600	42.2	Infin	0.647	0.417	NonLiq
9	65.25	3.915	2.815	45	0.05	0.600	42.2	Infin	0.643	0.416	NonLiq

File Name: triv12.OUT

SOIL NO.	CALC. DEPTH (ft)	TOTAL STRESS (tsf)	EFF. STRESS (tsf)	FIELD N (B/ft)	FC DELTA N1_60	C N	CORR. (N1)60 (B/ft)	LIQUE. RESIST RATIO	r d	INDUC. STRESS RATIO	LIQUE. SAFETY FACTOR
9	65.75	3.945	2.830	45	0.05	0.600	42.2	Infin	0.639	0.414	NonLiq
9	66.25	3.975	2.844	45	0.05	0.600	42.2	Infin	0.635	0.412	NonLiq

```
****************************
*                          *
*      L I Q U E F Y 2     *
*                          *
*       Version 1.50       *
*                          *
****************************
```

EMPIRICAL PREDICTION OF
EARTHQUAKE-INDUCED LIQUEFACTION POTENTIAL

JOB NUMBER: 2908 DATE: 02-22-2017

JOB NAME: Triv110

SOIL-PROFILE NAME: triv110.LDW

BORING GROUNDWATER DEPTH: 66.50 ft

CALCULATION GROUNDWATER DEPTH: 30.00 ft

DESIGN EARTHQUAKE MAGNITUDE: 6.61 Mw

SITE PEAK GROUND ACCELERATION: 0.715 g

BOREHOLE DIAMETER CORRECTION FACTOR: 1.00

SAMPLER SIZE CORRECTION FACTOR: 1.20

N60 HAMMER CORRECTION FACTOR: 1.30

MAGNITUDE SCALING FACTOR METHOD: Idriss (1998, in press)

Magnitude Scaling Factor: 1.242

rd-CORRECTION METHOD: NCEER (1997)

FIELD SPT N-VALUES ARE CORRECTED FOR THE LENGTH OF THE DRIVE RODS.

Rod Stick-Up Above Ground: 0.0 ft

CN NORMALIZATION FACTOR: 1.044 tsf

MINIMUM CN VALUE: 0.6

```
-------------------      -----------------------------
NCEER [1997] Method      LIQUEFACTION ANALYSIS SUMMARY              PAGE  1
-------------------      -----------------------------

File Name:  triv110.OUT

------------------------------------------------------------------------------
    | CALC.| TOTAL| EFF. |FIELD | FC  |     | CORR.|LIQUE.|     |INDUC.|LIQUE.
SOIL| DEPTH|STRESS|STRESS|  N   |DELTA|  C  |(N1)60|RESIST|  r  |STRESS|SAFETY
 NO.| (ft) | (tsf)| (tsf)|(B/ft)|N1_60|   N |(B/ft)| RATIO|   d | RATIO|FACTOR
----+------+------+------+------+-----+-----+------+------+-----+------+------
  1 |  0.25| 0.015| 0.015|  10  |  ~  |  *  |    * |   *  |  *  |    * |  **
  1 |  0.75| 0.045| 0.045|  10  |  ~  |  *  |    * |   *  |  *  |    * |  **
  1 |  1.25| 0.075| 0.075|  10  |  ~  |  *  |    * |   *  |  *  |    * |  **
  1 |  1.75| 0.105| 0.105|  10  |  ~  |  *  |    * |   *  |  *  |    * |  **
  1 |  2.25| 0.135| 0.135|  10  |  ~  |  *  |    * |   *  |  *  |    * |  **
  1 |  2.75| 0.165| 0.165|  10  |  ~  |  *  |    * |   *  |  *  |    * |  **
  1 |  3.25| 0.195| 0.195|  10  |  ~  |  *  |    * |   *  |  *  |    * |  **
  1 |  3.75| 0.225| 0.225|  10  |  ~  |  *  |    * |   *  |  *  |    * |  **
  1 |  4.25| 0.255| 0.255|  10  |  ~  |  *  |    * |   *  |  *  |    * |  **
  1 |  4.75| 0.285| 0.285|  10  |  ~  |  *  |    * |   *  |  *  |    * |  **
  1 |  5.25| 0.315| 0.315|  10  |  ~  |  *  |    * |   *  |  *  |    * |  **
  1 |  5.75| 0.345| 0.345|  10  |  ~  |  *  |    * |   *  |  *  |    * |  **
  1 |  6.25| 0.375| 0.375|  10  |  ~  |  *  |    * |   *  |  *  |    * |  **
  1 |  6.75| 0.405| 0.405|  10  |  ~  |  *  |    * |   *  |  *  |    * |  **
  1 |  7.25| 0.435| 0.435|  10  |  ~  |  *  |    * |   *  |  *  |    * |  **
  1 |  7.75| 0.465| 0.465|  10  |  ~  |  *  |    * |   *  |  *  |    * |  **
  1 |  8.25| 0.495| 0.495|  10  |  ~  |  *  |    * |   *  |  *  |    * |  **
  1 |  8.75| 0.525| 0.525|  10  |  ~  |  *  |    * |   *  |  *  |    * |  **
  1 |  9.25| 0.555| 0.555|  10  |  ~  |  *  |    * |   *  |  *  |    * |  **
  1 |  9.75| 0.585| 0.585|  10  |  ~  |  *  |    * |   *  |  *  |    * |  **
  1 | 10.25| 0.615| 0.615|  10  |  ~  |  *  |    * |   *  |  *  |    * |  **
  1 | 10.75| 0.645| 0.645|  10  |  ~  |  *  |    * |   *  |  *  |    * |  **
  1 | 11.25| 0.675| 0.675|  10  |  ~  |  *  |    * |   *  |  *  |    * |  **
  1 | 11.75| 0.705| 0.705|  10  |  ~  |  *  |    * |   *  |  *  |    * |  **
  1 | 12.25| 0.735| 0.735|  10  |  ~  |  *  |    * |   *  |  *  |    * |  **
  1 | 12.75| 0.765| 0.765|  10  |  ~  |  *  |    * |   *  |  *  |    * |  **
  1 | 13.25| 0.795| 0.795|  10  |  ~  |  *  |    * |   *  |  *  |    * |  **
  1 | 13.75| 0.825| 0.825|  10  |  ~  |  *  |    * |   *  |  *  |    * |  **
  1 | 14.25| 0.855| 0.855|  10  |  ~  |  *  |    * |   *  |  *  |    * |  **
  1 | 14.75| 0.885| 0.885|  10  |  ~  |  *  |    * |   *  |  *  |    * |  **
  1 | 15.25| 0.915| 0.915|  10  |  ~  |  *  |    * |   *  |  *  |    * |  **
  1 | 15.75| 0.945| 0.945|  10  |  ~  |  *  |    * |   *  |  *  |    * |  **
  1 | 16.25| 0.975| 0.975|  10  |  ~  |  *  |    * |   *  |  *  |    * |  **
  1 | 16.75| 1.005| 1.005|  10  |  ~  |  *  |    * |   *  |  *  |    * |  **
  1 | 17.25| 1.035| 1.035|  10  |  ~  |  *  |    * |   *  |  *  |    * |  **
  1 | 17.75| 1.065| 1.065|  10  |  ~  |  *  |    * |   *  |  *  |    * |  **
  1 | 18.25| 1.095| 1.095|  10  |  ~  |  *  |    * |   *  |  *  |    * |  **
  1 | 18.75| 1.125| 1.125|  10  |  ~  |  *  |    * |   *  |  *  |    * |  **
  1 | 19.25| 1.155| 1.155|  10  |  ~  |  *  |    * |   *  |  *  |    * |  **
  1 | 19.75| 1.185| 1.185|  10  |  ~  |  *  |    * |   *  |  *  |    * |  **
  1 | 20.25| 1.215| 1.215|  10  |  ~  |  *  |    * |   *  |  *  |    * |  **
  1 | 20.75| 1.245| 1.245|  10  |  ~  |  *  |    * |   *  |  *  |    * |  **
  1 | 21.25| 1.275| 1.275|  10  |  ~  |  *  |    * |   *  |  *  |    * |  **
```

File Name: triv110.OUT

SOIL NO.	CALC. DEPTH (ft)	TOTAL STRESS (tsf)	EFF. STRESS (tsf)	FIELD N (B/ft)	FC DELTA N1_60	C N	CORR. (N1)60 (B/ft)	LIQUE. RESIST RATIO	r d	INDUC. STRESS RATIO	LIQUE. SAFETY FACTOR
1	21.75	1.305	1.305	10	~	*	*	*	*	*	**
1	22.25	1.335	1.335	10	~	*	*	*	*	*	**
1	22.75	1.365	1.365	10	~	*	*	*	*	*	**
1	23.25	1.395	1.395	10	~	*	*	*	*	*	**
1	23.75	1.425	1.425	10	~	*	*	*	*	*	**
1	24.25	1.455	1.455	10	~	*	*	*	*	*	**
1	24.75	1.485	1.485	10	~	*	*	*	*	*	**
1	25.25	1.515	1.515	10	~	*	*	*	*	*	**
1	25.75	1.545	1.545	10	~	*	*	*	*	*	**
1	26.25	1.575	1.575	10	~	*	*	*	*	*	**
1	26.75	1.605	1.605	10	~	*	*	*	*	*	**
1	27.25	1.635	1.635	10	~	*	*	*	*	*	**
1	27.75	1.665	1.665	10	~	*	*	*	*	*	**
1	28.25	1.695	1.695	10	~	*	*	*	*	*	**
1	28.75	1.725	1.725	10	~	*	*	*	*	*	**
1	29.25	1.755	1.755	10	~	*	*	*	*	*	**
1	29.75	1.785	1.785	10	~	*	*	*	*	*	**
2	30.25	1.815	1.807	24	10.57	0.758	39.0	Infin	0.928	0.433	NonLiq
2	30.75	1.845	1.822	24	10.57	0.758	39.0	Infin	0.924	0.435	NonLiq
2	31.25	1.875	1.836	24	10.57	0.758	39.0	Infin	0.920	0.437	NonLiq
2	31.75	1.905	1.850	24	10.57	0.758	39.0	Infin	0.916	0.438	NonLiq
3	32.25	1.935	1.865	38	0.05	0.703	41.7	Infin	0.912	0.440	NonLiq
3	32.75	1.965	1.879	38	0.05	0.703	41.7	Infin	0.907	0.441	NonLiq
3	33.25	1.995	1.894	38	0.05	0.703	41.7	Infin	0.903	0.442	NonLiq
3	33.75	2.025	1.908	38	0.05	0.703	41.7	Infin	0.899	0.444	NonLiq
3	34.25	2.055	1.922	38	0.05	0.703	41.7	Infin	0.895	0.445	NonLiq
3	34.75	2.085	1.937	38	0.05	0.703	41.7	Infin	0.891	0.446	NonLiq
3	35.25	2.115	1.951	38	0.05	0.703	41.7	Infin	0.887	0.447	NonLiq
3	35.75	2.145	1.966	38	0.05	0.703	41.7	Infin	0.883	0.448	NonLiq
3	36.25	2.175	1.980	38	0.05	0.703	41.7	Infin	0.879	0.449	NonLiq
4	36.75	2.205	1.994	35	0.05	0.657	35.9	Infin	0.875	0.450	NonLiq
4	37.25	2.235	2.009	35	0.05	0.657	35.9	Infin	0.871	0.450	NonLiq
4	37.75	2.265	2.023	35	0.05	0.657	35.9	Infin	0.867	0.451	NonLiq
4	38.25	2.295	2.038	35	0.05	0.657	35.9	Infin	0.863	0.452	NonLiq
4	38.75	2.325	2.052	35	0.05	0.657	35.9	Infin	0.859	0.452	NonLiq
4	39.25	2.355	2.066	35	0.05	0.657	35.9	Infin	0.855	0.453	NonLiq
4	39.75	2.385	2.081	35	0.05	0.657	35.9	Infin	0.851	0.453	NonLiq
4	40.25	2.415	2.095	35	0.05	0.657	35.9	Infin	0.846	0.453	NonLiq
4	40.75	2.445	2.110	35	0.05	0.657	35.9	Infin	0.842	0.454	NonLiq
4	41.25	2.475	2.124	35	0.05	0.657	35.9	Infin	0.838	0.454	NonLiq
4	41.75	2.505	2.138	35	0.05	0.657	35.9	Infin	0.834	0.454	NonLiq
4	42.25	2.535	2.153	35	0.05	0.657	35.9	Infin	0.830	0.454	NonLiq
5	42.75	2.565	2.167	30	10.70	0.620	39.7	Infin	0.826	0.454	NonLiq
5	43.25	2.595	2.182	30	10.70	0.620	39.7	Infin	0.822	0.454	NonLiq

File Name: triv110.OUT

SOIL NO.	CALC. DEPTH (ft)	TOTAL STRESS (tsf)	EFF. STRESS (tsf)	FIELD N (B/ft)	FC DELTA N1_60	C N	CORR. (N1)60 (B/ft)	LIQUE. RESIST RATIO	r d	INDUC. STRESS RATIO	LIQUE. SAFETY FACTOR
5	43.75	2.625	2.196	30	10.70	0.620	39.7	Infin	0.818	0.454	NonLiq
5	44.25	2.655	2.210	30	10.70	0.620	39.7	Infin	0.814	0.454	NonLiq
5	44.75	2.685	2.225	30	10.70	0.620	39.7	Infin	0.810	0.454	NonLiq
5	45.25	2.715	2.239	30	10.70	0.620	39.7	Infin	0.806	0.454	NonLiq
5	45.75	2.745	2.254	30	10.70	0.620	39.7	Infin	0.802	0.454	NonLiq
5	46.25	2.775	2.268	30	10.70	0.620	39.7	Infin	0.798	0.454	NonLiq
5	46.75	2.805	2.282	30	10.70	0.620	39.7	Infin	0.794	0.453	NonLiq
5	47.25	2.835	2.297	30	10.70	0.620	39.7	Infin	0.789	0.453	NonLiq
5	47.75	2.865	2.311	30	10.70	0.620	39.7	Infin	0.785	0.452	NonLiq
6	48.25	2.895	2.326	33	0.04	0.600	30.9	Infin	0.781	0.452	NonLiq
6	48.75	2.925	2.340	33	0.04	0.600	30.9	Infin	0.777	0.452	NonLiq
6	49.25	2.955	2.354	33	0.04	0.600	30.9	Infin	0.773	0.451	NonLiq
6	49.75	2.985	2.369	33	0.04	0.600	30.9	Infin	0.769	0.450	NonLiq
6	50.25	3.015	2.383	33	0.04	0.600	30.9	Infin	0.765	0.450	NonLiq
6	50.75	3.045	2.398	33	0.04	0.600	30.9	Infin	0.761	0.449	NonLiq
6	51.25	3.075	2.412	33	0.04	0.600	30.9	Infin	0.757	0.448	NonLiq
6	51.75	3.105	2.426	33	0.04	0.600	30.9	Infin	0.753	0.448	NonLiq
6	52.25	3.135	2.441	33	0.04	0.600	30.9	Infin	0.749	0.447	NonLiq
7	52.75	3.165	2.455	47	0.05	0.600	44.0	Infin	0.745	0.446	NonLiq
7	53.25	3.195	2.470	47	0.05	0.600	44.0	Infin	0.741	0.445	NonLiq
7	53.75	3.225	2.484	47	0.05	0.600	44.0	Infin	0.737	0.444	NonLiq
7	54.25	3.255	2.498	47	0.05	0.600	44.0	Infin	0.732	0.444	NonLiq
7	54.75	3.285	2.513	47	0.05	0.600	44.0	Infin	0.728	0.443	NonLiq
7	55.25	3.315	2.527	47	0.05	0.600	44.0	Infin	0.724	0.442	NonLiq
7	55.75	3.345	2.542	47	0.05	0.600	44.0	Infin	0.720	0.441	NonLiq
7	56.25	3.375	2.556	47	0.05	0.600	44.0	Infin	0.716	0.440	NonLiq
7	56.75	3.405	2.570	47	0.05	0.600	44.0	Infin	0.712	0.438	NonLiq
7	57.25	3.435	2.585	47	0.05	0.600	44.0	Infin	0.708	0.437	NonLiq
8	57.75	3.465	2.599	55	0.06	0.600	51.5	Infin	0.704	0.436	NonLiq
8	58.25	3.495	2.614	55	0.06	0.600	51.5	Infin	0.700	0.435	NonLiq
8	58.75	3.525	2.628	55	0.06	0.600	51.5	Infin	0.696	0.434	NonLiq
8	59.25	3.555	2.642	55	0.06	0.600	51.5	Infin	0.692	0.433	NonLiq
8	59.75	3.585	2.657	55	0.06	0.600	51.5	Infin	0.688	0.431	NonLiq
8	60.25	3.615	2.671	55	0.06	0.600	51.5	Infin	0.684	0.430	NonLiq
8	60.75	3.645	2.686	55	0.06	0.600	51.5	Infin	0.680	0.429	NonLiq
8	61.25	3.675	2.700	55	0.06	0.600	51.5	Infin	0.676	0.427	NonLiq
8	61.75	3.705	2.714	55	0.06	0.600	51.5	Infin	0.671	0.426	NonLiq
8	62.25	3.735	2.729	55	0.06	0.600	51.5	Infin	0.667	0.425	NonLiq
9	62.75	3.765	2.743	45	0.05	0.600	42.2	Infin	0.663	0.423	NonLiq
9	63.25	3.795	2.758	45	0.05	0.600	42.2	Infin	0.659	0.422	NonLiq
9	63.75	3.825	2.772	45	0.05	0.600	42.2	Infin	0.655	0.420	NonLiq
9	64.25	3.855	2.786	45	0.05	0.600	42.2	Infin	0.651	0.419	NonLiq
9	64.75	3.885	2.801	45	0.05	0.600	42.2	Infin	0.647	0.417	NonLiq
9	65.25	3.915	2.815	45	0.05	0.600	42.2	Infin	0.643	0.416	NonLiq

File Name: triv110.OUT

SOIL NO.	CALC. DEPTH (ft)	TOTAL STRESS (tsf)	EFF. STRESS (tsf)	FIELD N (B/ft)	FC DELTA N1_60	C N	CORR. (N1)60 (B/ft)	LIQUE. RESIST RATIO	r d	INDUC. STRESS RATIO	LIQUE. SAFETY FACTOR
9	65.75	3.945	2.830	45	0.05	0.600	42.2	Infin	0.639	0.414	NonLiq
9	66.25	3.975	2.844	45	0.05	0.600	42.2	Infin	0.635	0.412	NonLiq

```
*****************************
*                           *
*     L I Q U E F Y 2       *
*                           *
*      Version 1.50         *
*                           *
*****************************
```

EMPIRICAL PREDICTION OF
EARTHQUAKE-INDUCED LIQUEFACTION POTENTIAL

JOB NUMBER: 2908   DATE: 02-22-2017

JOB NAME: Triv22

SOIL-PROFILE NAME: triv22.LDW

BORING GROUNDWATER DEPTH: 66.50 ft

CALCULATION GROUNDWATER DEPTH: 30.00 ft

DESIGN EARTHQUAKE MAGNITUDE: 6.61 Mw

SITE PEAK GROUND ACCELERATION: 0.715 g

BOREHOLE DIAMETER CORRECTION FACTOR: 1.00

SAMPLER SIZE CORRECTION FACTOR: 1.20

N60 HAMMER CORRECTION FACTOR: 1.30

MAGNITUDE SCALING FACTOR METHOD: Idriss (1998, in press)

    Magnitude Scaling Factor: 1.242

rd-CORRECTION METHOD: NCEER (1997)

FIELD SPT N-VALUES ARE CORRECTED FOR THE LENGTH OF THE DRIVE RODS.

    Rod Stick-Up Above Ground: 0.0 ft

CN NORMALIZATION FACTOR: 1.044 tsf

MINIMUM CN VALUE: 0.6

File Name: triv22.OUT

SOIL NO.	CALC. DEPTH (ft)	TOTAL STRESS (tsf)	EFF. STRESS (tsf)	FIELD N (B/ft)	FC DELTA N1_60	C N	CORR. (N1)60 (B/ft)	LIQUE. RESIST RATIO	r d	INDUC. STRESS RATIO	LIQUE. SAFETY FACTOR
1	0.25	0.015	0.015	10	~	*	*	*	*	*	**
1	0.75	0.045	0.045	10	~	*	*	*	*	*	**
1	1.25	0.075	0.075	10	~	*	*	*	*	*	**
1	1.75	0.105	0.105	10	~	*	*	*	*	*	**
1	2.25	0.135	0.135	10	~	*	*	*	*	*	**
1	2.75	0.165	0.165	10	~	*	*	*	*	*	**
1	3.25	0.195	0.195	10	~	*	*	*	*	*	**
1	3.75	0.225	0.225	10	~	*	*	*	*	*	**
1	4.25	0.255	0.255	10	~	*	*	*	*	*	**
1	4.75	0.285	0.285	10	~	*	*	*	*	*	**
1	5.25	0.315	0.315	10	~	*	*	*	*	*	**
1	5.75	0.345	0.345	10	~	*	*	*	*	*	**
1	6.25	0.375	0.375	10	~	*	*	*	*	*	**
1	6.75	0.405	0.405	10	~	*	*	*	*	*	**
1	7.25	0.435	0.435	10	~	*	*	*	*	*	**
1	7.75	0.465	0.465	10	~	*	*	*	*	*	**
1	8.25	0.495	0.495	10	~	*	*	*	*	*	**
1	8.75	0.525	0.525	10	~	*	*	*	*	*	**
1	9.25	0.555	0.555	10	~	*	*	*	*	*	**
1	9.75	0.585	0.585	10	~	*	*	*	*	*	**
1	10.25	0.615	0.615	10	~	*	*	*	*	*	**
1	10.75	0.645	0.645	10	~	*	*	*	*	*	**
1	11.25	0.675	0.675	10	~	*	*	*	*	*	**
1	11.75	0.705	0.705	10	~	*	*	*	*	*	**
1	12.25	0.735	0.735	10	~	*	*	*	*	*	**
1	12.75	0.765	0.765	10	~	*	*	*	*	*	**
1	13.25	0.795	0.795	10	~	*	*	*	*	*	**
1	13.75	0.825	0.825	10	~	*	*	*	*	*	**
1	14.25	0.855	0.855	10	~	*	*	*	*	*	**
1	14.75	0.885	0.885	10	~	*	*	*	*	*	**
1	15.25	0.915	0.915	10	~	*	*	*	*	*	**
1	15.75	0.945	0.945	10	~	*	*	*	*	*	**
1	16.25	0.975	0.975	10	~	*	*	*	*	*	**
1	16.75	1.005	1.005	10	~	*	*	*	*	*	**
1	17.25	1.035	1.035	10	~	*	*	*	*	*	**
1	17.75	1.065	1.065	10	~	*	*	*	*	*	**
1	18.25	1.095	1.095	10	~	*	*	*	*	*	**
1	18.75	1.125	1.125	10	~	*	*	*	*	*	**
1	19.25	1.155	1.155	10	~	*	*	*	*	*	**
1	19.75	1.185	1.185	10	~	*	*	*	*	*	**
1	20.25	1.215	1.215	10	~	*	*	*	*	*	**
1	20.75	1.245	1.245	10	~	*	*	*	*	*	**
1	21.25	1.275	1.275	10	~	*	*	*	*	*	**

File Name: triv22.OUT

SOIL NO.	CALC. DEPTH (ft)	TOTAL STRESS (tsf)	EFF. STRESS (tsf)	FIELD N (B/ft)	FC DELTA N1_60	C N	CORR. (N1)60 (B/ft)	LIQUE. RESIST RATIO	r d	INDUC. STRESS RATIO	LIQUE. SAFETY FACTOR
1	21.75	1.305	1.305	10	~	*	*	*	*	*	**
1	22.25	1.335	1.335	10	~	*	*	*	*	*	**
1	22.75	1.365	1.365	10	~	*	*	*	*	*	**
1	23.25	1.395	1.395	10	~	*	*	*	*	*	**
1	23.75	1.425	1.425	10	~	*	*	*	*	*	**
1	24.25	1.455	1.455	10	~	*	*	*	*	*	**
1	24.75	1.485	1.485	10	~	*	*	*	*	*	**
1	25.25	1.515	1.515	10	~	*	*	*	*	*	**
1	25.75	1.545	1.545	10	~	*	*	*	*	*	**
1	26.25	1.575	1.575	10	~	*	*	*	*	*	**
1	26.75	1.605	1.605	10	~	*	*	*	*	*	**
1	27.25	1.635	1.635	10	~	*	*	*	*	*	**
1	27.75	1.665	1.665	10	~	*	*	*	*	*	**
1	28.25	1.695	1.695	10	~	*	*	*	*	*	**
1	28.75	1.725	1.725	10	~	*	*	*	*	*	**
1	29.25	1.755	1.755	10	~	*	*	*	*	*	**
1	29.75	1.785	1.785	10	~	*	*	*	*	*	**
2	30.25	1.815	1.807	21	9.26	0.703	32.3	Infin	0.928	0.433	NonLiq
2	30.75	1.845	1.822	21	9.26	0.703	32.3	Infin	0.924	0.435	NonLiq
2	31.25	1.875	1.836	21	9.26	0.703	32.3	Infin	0.920	0.437	NonLiq
2	31.75	1.905	1.850	21	9.26	0.703	32.3	Infin	0.916	0.438	NonLiq
2	32.25	1.935	1.865	21	9.26	0.703	32.3	Infin	0.912	0.440	NonLiq
2	32.75	1.965	1.879	21	9.26	0.703	32.3	Infin	0.907	0.441	NonLiq
2	33.25	1.995	1.894	21	9.26	0.703	32.3	Infin	0.903	0.442	NonLiq
2	33.75	2.025	1.908	21	9.26	0.703	32.3	Infin	0.899	0.444	NonLiq
2	34.25	2.055	1.922	21	9.26	0.703	32.3	Infin	0.895	0.445	NonLiq
2	34.75	2.085	1.937	21	9.26	0.703	32.3	Infin	0.891	0.446	NonLiq
2	35.25	2.115	1.951	21	9.26	0.703	32.3	Infin	0.887	0.447	NonLiq
2	35.75	2.145	1.966	21	9.26	0.703	32.3	Infin	0.883	0.448	NonLiq
2	36.25	2.175	1.980	21	9.26	0.703	32.3	Infin	0.879	0.449	NonLiq
2	36.75	2.205	1.994	21	9.26	0.703	32.3	Infin	0.875	0.450	NonLiq
2	37.25	2.235	2.009	21	9.26	0.703	32.3	Infin	0.871	0.450	NonLiq
2	37.75	2.265	2.023	21	9.26	0.703	32.3	Infin	0.867	0.451	NonLiq
3	38.25	2.295	2.038	33	0.04	0.657	33.9	Infin	0.863	0.452	NonLiq
3	38.75	2.325	2.052	33	0.04	0.657	33.9	Infin	0.859	0.452	NonLiq
3	39.25	2.355	2.066	33	0.04	0.657	33.9	Infin	0.855	0.453	NonLiq
3	39.75	2.385	2.081	33	0.04	0.657	33.9	Infin	0.851	0.453	NonLiq
3	40.25	2.415	2.095	33	0.04	0.657	33.9	Infin	0.846	0.453	NonLiq
3	40.75	2.445	2.110	33	0.04	0.657	33.9	Infin	0.842	0.454	NonLiq
3	41.25	2.475	2.124	33	0.04	0.657	33.9	Infin	0.838	0.454	NonLiq
3	41.75	2.505	2.138	33	0.04	0.657	33.9	Infin	0.834	0.454	NonLiq
4	42.25	2.535	2.153	30	~	~	~	~	~	~	~~
4	42.75	2.565	2.167	30	~	~	~	~	~	~	~~
4	43.25	2.595	2.182	30	~	~	~	~	~	~	~~

File Name: triv22.OUT

SOIL NO.	CALC. DEPTH (ft)	TOTAL STRESS (tsf)	EFF. STRESS (tsf)	FIELD N (B/ft)	FC DELTA N1_60	C N	CORR. (N1)60 (B/ft)	LIQUE. RESIST RATIO	r d	INDUC. STRESS RATIO	LIQUE. SAFETY FACTOR
4	43.75	2.625	2.196	30	~	~	~	~	~	~	~~
4	44.25	2.655	2.210	30	~	~	~	~	~	~	~~
4	44.75	2.685	2.225	30	~	~	~	~	~	~	~~
4	45.25	2.715	2.239	30	~	~	~	~	~	~	~~
4	45.75	2.745	2.254	30	~	~	~	~	~	~	~~
4	46.25	2.775	2.268	30	~	~	~	~	~	~	~~
4	46.75	2.805	2.282	30	~	~	~	~	~	~	~~
4	47.25	2.835	2.297	30	~	~	~	~	~	~	~~
4	47.75	2.865	2.311	30	~	~	~	~	~	~	~~
5	48.25	2.895	2.326	39	0.05	0.600	36.5	Infin	0.781	0.452	NonLiq
5	48.75	2.925	2.340	39	0.05	0.600	36.5	Infin	0.777	0.452	NonLiq
5	49.25	2.955	2.354	39	0.05	0.600	36.5	Infin	0.773	0.451	NonLiq
5	49.75	2.985	2.369	39	0.05	0.600	36.5	Infin	0.769	0.450	NonLiq
5	50.25	3.015	2.383	39	0.05	0.600	36.5	Infin	0.765	0.450	NonLiq
5	50.75	3.045	2.398	39	0.05	0.600	36.5	Infin	0.761	0.449	NonLiq
5	51.25	3.075	2.412	39	0.05	0.600	36.5	Infin	0.757	0.448	NonLiq
5	51.75	3.105	2.426	39	0.05	0.600	36.5	Infin	0.753	0.448	NonLiq
5	52.25	3.135	2.441	39	0.05	0.600	36.5	Infin	0.749	0.447	NonLiq
5	52.75	3.165	2.455	39	0.05	0.600	36.5	Infin	0.745	0.446	NonLiq
5	53.25	3.195	2.470	39	0.05	0.600	36.5	Infin	0.741	0.445	NonLiq
5	53.75	3.225	2.484	39	0.05	0.600	36.5	Infin	0.737	0.444	NonLiq
5	54.25	3.255	2.498	39	0.05	0.600	36.5	Infin	0.732	0.444	NonLiq
5	54.75	3.285	2.513	39	0.05	0.600	36.5	Infin	0.728	0.443	NonLiq
5	55.25	3.315	2.527	39	0.05	0.600	36.5	Infin	0.724	0.442	NonLiq
5	55.75	3.345	2.542	39	0.05	0.600	36.5	Infin	0.720	0.441	NonLiq
5	56.25	3.375	2.556	39	0.05	0.600	36.5	Infin	0.716	0.440	NonLiq
5	56.75	3.405	2.570	39	0.05	0.600	36.5	Infin	0.712	0.438	NonLiq
5	57.25	3.435	2.585	39	0.05	0.600	36.5	Infin	0.708	0.437	NonLiq
5	57.75	3.465	2.599	39	0.05	0.600	36.5	Infin	0.704	0.436	NonLiq
5	58.25	3.495	2.614	39	0.05	0.600	36.5	Infin	0.700	0.435	NonLiq
5	58.75	3.525	2.628	39	0.05	0.600	36.5	Infin	0.696	0.434	NonLiq
5	59.25	3.555	2.642	39	0.05	0.600	36.5	Infin	0.692	0.433	NonLiq
5	59.75	3.585	2.657	39	0.05	0.600	36.5	Infin	0.688	0.431	NonLiq
5	60.25	3.615	2.671	39	0.05	0.600	36.5	Infin	0.684	0.430	NonLiq
5	60.75	3.645	2.686	39	0.05	0.600	36.5	Infin	0.680	0.429	NonLiq
5	61.25	3.675	2.700	39	0.05	0.600	36.5	Infin	0.676	0.427	NonLiq
5	61.75	3.705	2.714	39	0.05	0.600	36.5	Infin	0.671	0.426	NonLiq
5	62.25	3.735	2.729	39	0.05	0.600	36.5	Infin	0.667	0.425	NonLiq
5	62.75	3.765	2.743	39	0.05	0.600	36.5	Infin	0.663	0.423	NonLiq
5	63.25	3.795	2.758	39	0.05	0.600	36.5	Infin	0.659	0.422	NonLiq
5	63.75	3.825	2.772	39	0.05	0.600	36.5	Infin	0.655	0.420	NonLiq
5	64.25	3.855	2.786	39	0.05	0.600	36.5	Infin	0.651	0.419	NonLiq
5	64.75	3.885	2.801	39	0.05	0.600	36.5	Infin	0.647	0.417	NonLiq
5	65.25	3.915	2.815	39	0.05	0.600	36.5	Infin	0.643	0.416	NonLiq

File Name: triv22.OUT

SOIL NO.	CALC. DEPTH (ft)	TOTAL STRESS (tsf)	EFF. STRESS (tsf)	FIELD N (B/ft)	FC DELTA N1_60	C N	CORR. (N1)60 (B/ft)	LIQUE. RESIST RATIO	r d	INDUC. STRESS RATIO	LIQUE. SAFETY FACTOR
5	65.75	3.945	2.830	39	0.05	0.600	36.5	Infin	0.639	0.414	NonLiq
5	66.25	3.975	2.844	39	0.05	0.600	36.5	Infin	0.635	0.412	NonLiq

```
*****************************
*                           *
*      L I Q U E F Y 2      *
*                           *
*       Version 1.50        *
*                           *
*****************************
```

EMPIRICAL PREDICTION OF
EARTHQUAKE-INDUCED LIQUEFACTION POTENTIAL

JOB NUMBER: 2908 DATE: 02-22-2017

JOB NAME: Triv210

SOIL-PROFILE NAME: triv210.LDW

BORING GROUNDWATER DEPTH: 66.50 ft

CALCULATION GROUNDWATER DEPTH: 30.00 ft

DESIGN EARTHQUAKE MAGNITUDE: 6.61 Mw

SITE PEAK GROUND ACCELERATION: 0.480 g

BOREHOLE DIAMETER CORRECTION FACTOR: 1.00

SAMPLER SIZE CORRECTION FACTOR: 1.20

N60 HAMMER CORRECTION FACTOR: 1.30

MAGNITUDE SCALING FACTOR METHOD: Idriss (1998, in press)

Magnitude Scaling Factor: 1.242

rd-CORRECTION METHOD: NCEER (1997)

FIELD SPT N-VALUES ARE CORRECTED FOR THE LENGTH OF THE DRIVE RODS.

Rod Stick-Up Above Ground: 0.0 ft

CN NORMALIZATION FACTOR: 1.044 tsf

MINIMUM CN VALUE: 0.6

NCEER [1997] Method   LIQUEFACTION ANALYSIS SUMMARY

File Name: triv210.OUT

SOIL NO.	CALC. DEPTH (ft)	TOTAL STRESS (tsf)	EFF. STRESS (tsf)	FIELD N (B/ft)	FC DELTA N1_60	C N	CORR. (N1)60 (B/ft)	LIQUE. RESIST RATIO	r d	INDUC. STRESS RATIO	LIQUE. SAFETY FACTOR
1	0.25	0.015	0.015	10	~	*	*	*	*	*	**
1	0.75	0.045	0.045	10	~	*	*	*	*	*	**
1	1.25	0.075	0.075	10	~	*	*	*	*	*	**
1	1.75	0.105	0.105	10	~	*	*	*	*	*	**
1	2.25	0.135	0.135	10	~	*	*	*	*	*	**
1	2.75	0.165	0.165	10	~	*	*	*	*	*	**
1	3.25	0.195	0.195	10	~	*	*	*	*	*	**
1	3.75	0.225	0.225	10	~	*	*	*	*	*	**
1	4.25	0.255	0.255	10	~	*	*	*	*	*	**
1	4.75	0.285	0.285	10	~	*	*	*	*	*	**
1	5.25	0.315	0.315	10	~	*	*	*	*	*	**
1	5.75	0.345	0.345	10	~	*	*	*	*	*	**
1	6.25	0.375	0.375	10	~	*	*	*	*	*	**
1	6.75	0.405	0.405	10	~	*	*	*	*	*	**
1	7.25	0.435	0.435	10	~	*	*	*	*	*	**
1	7.75	0.465	0.465	10	~	*	*	*	*	*	**
1	8.25	0.495	0.495	10	~	*	*	*	*	*	**
1	8.75	0.525	0.525	10	~	*	*	*	*	*	**
1	9.25	0.555	0.555	10	~	*	*	*	*	*	**
1	9.75	0.585	0.585	10	~	*	*	*	*	*	**
1	10.25	0.615	0.615	10	~	*	*	*	*	*	**
1	10.75	0.645	0.645	10	~	*	*	*	*	*	**
1	11.25	0.675	0.675	10	~	*	*	*	*	*	**
1	11.75	0.705	0.705	10	~	*	*	*	*	*	**
1	12.25	0.735	0.735	10	~	*	*	*	*	*	**
1	12.75	0.765	0.765	10	~	*	*	*	*	*	**
1	13.25	0.795	0.795	10	~	*	*	*	*	*	**
1	13.75	0.825	0.825	10	~	*	*	*	*	*	**
1	14.25	0.855	0.855	10	~	*	*	*	*	*	**
1	14.75	0.885	0.885	10	~	*	*	*	*	*	**
1	15.25	0.915	0.915	10	~	*	*	*	*	*	**
1	15.75	0.945	0.945	10	~	*	*	*	*	*	**
1	16.25	0.975	0.975	10	~	*	*	*	*	*	**
1	16.75	1.005	1.005	10	~	*	*	*	*	*	**
1	17.25	1.035	1.035	10	~	*	*	*	*	*	**
1	17.75	1.065	1.065	10	~	*	*	*	*	*	**
1	18.25	1.095	1.095	10	~	*	*	*	*	*	**
1	18.75	1.125	1.125	10	~	*	*	*	*	*	**
1	19.25	1.155	1.155	10	~	*	*	*	*	*	**
1	19.75	1.185	1.185	10	~	*	*	*	*	*	**
1	20.25	1.215	1.215	10	~	*	*	*	*	*	**
1	20.75	1.245	1.245	10	~	*	*	*	*	*	**
1	21.25	1.275	1.275	10	~	*	*	*	*	*	**

File Name: triv210.OUT

SOIL NO.	CALC. DEPTH (ft)	TOTAL STRESS (tsf)	EFF. STRESS (tsf)	FIELD N (B/ft)	FC DELTA N1_60	C N	CORR. (N1)60 (B/ft)	LIQUE. RESIST RATIO	r d	INDUC. STRESS RATIO	LIQUE. SAFETY FACTOR
1	21.75	1.305	1.305	10	~	*	*	*	*	*	**
1	22.25	1.335	1.335	10	~	*	*	*	*	*	**
1	22.75	1.365	1.365	10	~	*	*	*	*	*	**
1	23.25	1.395	1.395	10	~	*	*	*	*	*	**
1	23.75	1.425	1.425	10	~	*	*	*	*	*	**
1	24.25	1.455	1.455	10	~	*	*	*	*	*	**
1	24.75	1.485	1.485	10	~	*	*	*	*	*	**
1	25.25	1.515	1.515	10	~	*	*	*	*	*	**
1	25.75	1.545	1.545	10	~	*	*	*	*	*	**
1	26.25	1.575	1.575	10	~	*	*	*	*	*	**
1	26.75	1.605	1.605	10	~	*	*	*	*	*	**
1	27.25	1.635	1.635	10	~	*	*	*	*	*	**
1	27.75	1.665	1.665	10	~	*	*	*	*	*	**
1	28.25	1.695	1.695	10	~	*	*	*	*	*	**
1	28.75	1.725	1.725	10	~	*	*	*	*	*	**
1	29.25	1.755	1.755	10	~	*	*	*	*	*	**
1	29.75	1.785	1.785	10	~	*	*	*	*	*	**
2	30.25	1.815	1.807	21	9.26	0.703	32.3	Infin	0.928	0.291	NonLiq
2	30.75	1.845	1.822	21	9.26	0.703	32.3	Infin	0.924	0.292	NonLiq
2	31.25	1.875	1.836	21	9.26	0.703	32.3	Infin	0.920	0.293	NonLiq
2	31.75	1.905	1.850	21	9.26	0.703	32.3	Infin	0.916	0.294	NonLiq
2	32.25	1.935	1.865	21	9.26	0.703	32.3	Infin	0.912	0.295	NonLiq
2	32.75	1.965	1.879	21	9.26	0.703	32.3	Infin	0.907	0.296	NonLiq
2	33.25	1.995	1.894	21	9.26	0.703	32.3	Infin	0.903	0.297	NonLiq
2	33.75	2.025	1.908	21	9.26	0.703	32.3	Infin	0.899	0.298	NonLiq
2	34.25	2.055	1.922	21	9.26	0.703	32.3	Infin	0.895	0.299	NonLiq
2	34.75	2.085	1.937	21	9.26	0.703	32.3	Infin	0.891	0.299	NonLiq
2	35.25	2.115	1.951	21	9.26	0.703	32.3	Infin	0.887	0.300	NonLiq
2	35.75	2.145	1.966	21	9.26	0.703	32.3	Infin	0.883	0.301	NonLiq
2	36.25	2.175	1.980	21	9.26	0.703	32.3	Infin	0.879	0.301	NonLiq
2	36.75	2.205	1.994	21	9.26	0.703	32.3	Infin	0.875	0.302	NonLiq
2	37.25	2.235	2.009	21	9.26	0.703	32.3	Infin	0.871	0.302	NonLiq
2	37.75	2.265	2.023	21	9.26	0.703	32.3	Infin	0.867	0.303	NonLiq
3	38.25	2.295	2.038	33	0.04	0.657	33.9	Infin	0.863	0.303	NonLiq
3	38.75	2.325	2.052	33	0.04	0.657	33.9	Infin	0.859	0.304	NonLiq
3	39.25	2.355	2.066	33	0.04	0.657	33.9	Infin	0.855	0.304	NonLiq
3	39.75	2.385	2.081	33	0.04	0.657	33.9	Infin	0.851	0.304	NonLiq
3	40.25	2.415	2.095	33	0.04	0.657	33.9	Infin	0.846	0.304	NonLiq
3	40.75	2.445	2.110	33	0.04	0.657	33.9	Infin	0.842	0.305	NonLiq
3	41.25	2.475	2.124	33	0.04	0.657	33.9	Infin	0.838	0.305	NonLiq
3	41.75	2.505	2.138	33	0.04	0.657	33.9	Infin	0.834	0.305	NonLiq
4	42.25	2.535	2.153	30	~	~	~	~	~	~	~~
4	42.75	2.565	2.167	30	~	~	~	~	~	~	~~
4	43.25	2.595	2.182	30	~	~	~	~	~	~	~~

File Name: triv210.OUT

SOIL NO.	CALC. DEPTH (ft)	TOTAL STRESS (tsf)	EFF. STRESS (tsf)	FIELD N (B/ft)	FC DELTA N1_60	C N	CORR. (N1)60 (B/ft)	LIQUE. RESIST RATIO	r d	INDUC. STRESS RATIO	LIQUE. SAFETY FACTOR
4	43.75	2.625	2.196	30	~	~	~	~	~	~	~~
4	44.25	2.655	2.210	30	~	~	~	~	~	~	~~
4	44.75	2.685	2.225	30	~	~	~	~	~	~	~~
4	45.25	2.715	2.239	30	~	~	~	~	~	~	~~
4	45.75	2.745	2.254	30	~	~	~	~	~	~	~~
4	46.25	2.775	2.268	30	~	~	~	~	~	~	~~
4	46.75	2.805	2.282	30	~	~	~	~	~	~	~~
4	47.25	2.835	2.297	30	~	~	~	~	~	~	~~
4	47.75	2.865	2.311	30	~	~	~	~	~	~	~~
5	48.25	2.895	2.326	39	0.05	0.600	36.5	Infin	0.781	0.303	NonLiq
5	48.75	2.925	2.340	39	0.05	0.600	36.5	Infin	0.777	0.303	NonLiq
5	49.25	2.955	2.354	39	0.05	0.600	36.5	Infin	0.773	0.303	NonLiq
5	49.75	2.985	2.369	39	0.05	0.600	36.5	Infin	0.769	0.302	NonLiq
5	50.25	3.015	2.383	39	0.05	0.600	36.5	Infin	0.765	0.302	NonLiq
5	50.75	3.045	2.398	39	0.05	0.600	36.5	Infin	0.761	0.302	NonLiq
5	51.25	3.075	2.412	39	0.05	0.600	36.5	Infin	0.757	0.301	NonLiq
5	51.75	3.105	2.426	39	0.05	0.600	36.5	Infin	0.753	0.301	NonLiq
5	52.25	3.135	2.441	39	0.05	0.600	36.5	Infin	0.749	0.300	NonLiq
5	52.75	3.165	2.455	39	0.05	0.600	36.5	Infin	0.745	0.300	NonLiq
5	53.25	3.195	2.470	39	0.05	0.600	36.5	Infin	0.741	0.299	NonLiq
5	53.75	3.225	2.484	39	0.05	0.600	36.5	Infin	0.737	0.298	NonLiq
5	54.25	3.255	2.498	39	0.05	0.600	36.5	Infin	0.732	0.298	NonLiq
5	54.75	3.285	2.513	39	0.05	0.600	36.5	Infin	0.728	0.297	NonLiq
5	55.25	3.315	2.527	39	0.05	0.600	36.5	Infin	0.724	0.296	NonLiq
5	55.75	3.345	2.542	39	0.05	0.600	36.5	Infin	0.720	0.296	NonLiq
5	56.25	3.375	2.556	39	0.05	0.600	36.5	Infin	0.716	0.295	NonLiq
5	56.75	3.405	2.570	39	0.05	0.600	36.5	Infin	0.712	0.294	NonLiq
5	57.25	3.435	2.585	39	0.05	0.600	36.5	Infin	0.708	0.294	NonLiq
5	57.75	3.465	2.599	39	0.05	0.600	36.5	Infin	0.704	0.293	NonLiq
5	58.25	3.495	2.614	39	0.05	0.600	36.5	Infin	0.700	0.292	NonLiq
5	58.75	3.525	2.628	39	0.05	0.600	36.5	Infin	0.696	0.291	NonLiq
5	59.25	3.555	2.642	39	0.05	0.600	36.5	Infin	0.692	0.290	NonLiq
5	59.75	3.585	2.657	39	0.05	0.600	36.5	Infin	0.688	0.290	NonLiq
5	60.25	3.615	2.671	39	0.05	0.600	36.5	Infin	0.684	0.289	NonLiq
5	60.75	3.645	2.686	39	0.05	0.600	36.5	Infin	0.680	0.288	NonLiq
5	61.25	3.675	2.700	39	0.05	0.600	36.5	Infin	0.676	0.287	NonLiq
5	61.75	3.705	2.714	39	0.05	0.600	36.5	Infin	0.671	0.286	NonLiq
5	62.25	3.735	2.729	39	0.05	0.600	36.5	Infin	0.667	0.285	NonLiq
5	62.75	3.765	2.743	39	0.05	0.600	36.5	Infin	0.663	0.284	NonLiq
5	63.25	3.795	2.758	39	0.05	0.600	36.5	Infin	0.659	0.283	NonLiq
5	63.75	3.825	2.772	39	0.05	0.600	36.5	Infin	0.655	0.282	NonLiq
5	64.25	3.855	2.786	39	0.05	0.600	36.5	Infin	0.651	0.281	NonLiq
5	64.75	3.885	2.801	39	0.05	0.600	36.5	Infin	0.647	0.280	NonLiq
5	65.25	3.915	2.815	39	0.05	0.600	36.5	Infin	0.643	0.279	NonLiq

File Name: triv210.OUT

SOIL NO.	CALC. DEPTH (ft)	TOTAL STRESS (tsf)	EFF. STRESS (tsf)	FIELD N (B/ft)	FC DELTA N1_60	C N	CORR. (N1)60 (B/ft)	LIQUE. RESIST RATIO	r d	INDUC. STRESS RATIO	LIQUE. SAFETY FACTOR
5	65.75	3.945	2.830	39	0.05	0.600	36.5	Infin	0.639	0.278	NonLiq
5	66.25	3.975	2.844	39	0.05	0.600	36.5	Infin	0.635	0.277	NonLiq

```
*****************************
*                           *
*      L I Q U E F Y 2      *
*                           *
*       Version 1.50        *
*                           *
*****************************
```

EMPIRICAL PREDICTION OF
EARTHQUAKE-INDUCED LIQUEFACTION POTENTIAL

JOB NUMBER: 2908   DATE: 02-22-2017

JOB NAME: Triv32

SOIL-PROFILE NAME: triv32.LDW

BORING GROUNDWATER DEPTH: 66.50 ft

CALCULATION GROUNDWATER DEPTH: 30.00 ft

DESIGN EARTHQUAKE MAGNITUDE: 6.61 Mw

SITE PEAK GROUND ACCELERATION: 0.715 g

BOREHOLE DIAMETER CORRECTION FACTOR: 1.00

SAMPLER SIZE CORRECTION FACTOR: 1.20

N60 HAMMER CORRECTION FACTOR: 1.30

MAGNITUDE SCALING FACTOR METHOD: Idriss (1998, in press)

    Magnitude Scaling Factor: 1.242

rd-CORRECTION METHOD: NCEER (1997)

FIELD SPT N-VALUES ARE CORRECTED FOR THE LENGTH OF THE DRIVE RODS.

    Rod Stick-Up Above Ground: 0.0 ft

CN NORMALIZATION FACTOR: 1.044 tsf

MINIMUM CN VALUE: 0.6

NCEER [1997] Method   LIQUEFACTION ANALYSIS SUMMARY

File Name: triv32.OUT

SOIL NO.	CALC. DEPTH (ft)	TOTAL STRESS (tsf)	EFF. STRESS (tsf)	FIELD N (B/ft)	FC DELTA N1_60	C N	CORR. (N1)60 (B/ft)	LIQUE. RESIST RATIO	r d	INDUC. STRESS RATIO	LIQUE. SAFETY FACTOR
1	0.25	0.015	0.015	10	~	*	*	*	*	*	**
1	0.75	0.045	0.045	10	~	*	*	*	*	*	**
1	1.25	0.075	0.075	10	~	*	*	*	*	*	**
1	1.75	0.105	0.105	10	~	*	*	*	*	*	**
1	2.25	0.135	0.135	10	~	*	*	*	*	*	**
1	2.75	0.165	0.165	10	~	*	*	*	*	*	**
1	3.25	0.195	0.195	10	~	*	*	*	*	*	**
1	3.75	0.225	0.225	10	~	*	*	*	*	*	**
1	4.25	0.255	0.255	10	~	*	*	*	*	*	**
1	4.75	0.285	0.285	10	~	*	*	*	*	*	**
1	5.25	0.315	0.315	10	~	*	*	*	*	*	**
1	5.75	0.345	0.345	10	~	*	*	*	*	*	**
1	6.25	0.375	0.375	10	~	*	*	*	*	*	**
1	6.75	0.405	0.405	10	~	*	*	*	*	*	**
1	7.25	0.435	0.435	10	~	*	*	*	*	*	**
1	7.75	0.465	0.465	10	~	*	*	*	*	*	**
1	8.25	0.495	0.495	10	~	*	*	*	*	*	**
1	8.75	0.525	0.525	10	~	*	*	*	*	*	**
1	9.25	0.555	0.555	10	~	*	*	*	*	*	**
1	9.75	0.585	0.585	10	~	*	*	*	*	*	**
1	10.25	0.615	0.615	10	~	*	*	*	*	*	**
1	10.75	0.645	0.645	10	~	*	*	*	*	*	**
1	11.25	0.675	0.675	10	~	*	*	*	*	*	**
1	11.75	0.705	0.705	10	~	*	*	*	*	*	**
1	12.25	0.735	0.735	10	~	*	*	*	*	*	**
1	12.75	0.765	0.765	10	~	*	*	*	*	*	**
1	13.25	0.795	0.795	10	~	*	*	*	*	*	**
1	13.75	0.825	0.825	10	~	*	*	*	*	*	**
1	14.25	0.855	0.855	10	~	*	*	*	*	*	**
1	14.75	0.885	0.885	10	~	*	*	*	*	*	**
1	15.25	0.915	0.915	10	~	*	*	*	*	*	**
1	15.75	0.945	0.945	10	~	*	*	*	*	*	**
1	16.25	0.975	0.975	10	~	*	*	*	*	*	**
1	16.75	1.005	1.005	10	~	*	*	*	*	*	**
1	17.25	1.035	1.035	10	~	*	*	*	*	*	**
1	17.75	1.065	1.065	10	~	*	*	*	*	*	**
1	18.25	1.095	1.095	10	~	*	*	*	*	*	**
1	18.75	1.125	1.125	10	~	*	*	*	*	*	**
1	19.25	1.155	1.155	10	~	*	*	*	*	*	**
1	19.75	1.185	1.185	10	~	*	*	*	*	*	**
1	20.25	1.215	1.215	10	~	*	*	*	*	*	**
1	20.75	1.245	1.245	10	~	*	*	*	*	*	**
1	21.25	1.275	1.275	10	~	*	*	*	*	*	**

File Name: triv32.OUT

SOIL NO.	CALC. DEPTH (ft)	TOTAL STRESS (tsf)	EFF. STRESS (tsf)	FIELD N (B/ft)	FC DELTA N1_60	C N	CORR. (N1)60 (B/ft)	LIQUE. RESIST RATIO	r d	INDUC. STRESS RATIO	LIQUE. SAFETY FACTOR
1	21.75	1.305	1.305	10	~	*	*	*	*	*	**
1	22.25	1.335	1.335	10	~	*	*	*	*	*	**
1	22.75	1.365	1.365	10	~	*	*	*	*	*	**
1	23.25	1.395	1.395	10	~	*	*	*	*	*	**
1	23.75	1.425	1.425	10	~	*	*	*	*	*	**
1	24.25	1.455	1.455	10	~	*	*	*	*	*	**
1	24.75	1.485	1.485	10	~	*	*	*	*	*	**
1	25.25	1.515	1.515	10	~	*	*	*	*	*	**
1	25.75	1.545	1.545	10	~	*	*	*	*	*	**
1	26.25	1.575	1.575	10	~	*	*	*	*	*	**
1	26.75	1.605	1.605	10	~	*	*	*	*	*	**
1	27.25	1.635	1.635	10	~	*	*	*	*	*	**
1	27.75	1.665	1.665	10	~	*	*	*	*	*	**
1	28.25	1.695	1.695	10	~	*	*	*	*	*	**
1	28.75	1.725	1.725	10	~	*	*	*	*	*	**
1	29.25	1.755	1.755	10	~	*	*	*	*	*	**
1	29.75	1.785	1.785	10	~	*	*	*	*	*	**
2	30.25	1.815	1.807	68	0.10	0.758	80.6	Infin	0.928	0.433	NonLiq
2	30.75	1.845	1.822	68	0.10	0.758	80.6	Infin	0.924	0.435	NonLiq
2	31.25	1.875	1.836	68	0.10	0.758	80.6	Infin	0.920	0.437	NonLiq
2	31.75	1.905	1.850	68	0.10	0.758	80.6	Infin	0.916	0.438	NonLiq
3	32.25	1.935	1.865	34	0.05	0.703	37.3	Infin	0.912	0.440	NonLiq
3	32.75	1.965	1.879	34	0.05	0.703	37.3	Infin	0.907	0.441	NonLiq
3	33.25	1.995	1.894	34	0.05	0.703	37.3	Infin	0.903	0.442	NonLiq
3	33.75	2.025	1.908	34	0.05	0.703	37.3	Infin	0.899	0.444	NonLiq
3	34.25	2.055	1.922	34	0.05	0.703	37.3	Infin	0.895	0.445	NonLiq
3	34.75	2.085	1.937	34	0.05	0.703	37.3	Infin	0.891	0.446	NonLiq
3	35.25	2.115	1.951	34	0.05	0.703	37.3	Infin	0.887	0.447	NonLiq
3	35.75	2.145	1.966	34	0.05	0.703	37.3	Infin	0.883	0.448	NonLiq
3	36.25	2.175	1.980	34	0.05	0.703	37.3	Infin	0.879	0.449	NonLiq
3	36.75	2.205	1.994	34	0.05	0.703	37.3	Infin	0.875	0.450	NonLiq
3	37.25	2.235	2.009	34	0.05	0.703	37.3	Infin	0.871	0.450	NonLiq
4	37.75	2.265	2.023	45	0.06	0.657	46.2	Infin	0.867	0.451	NonLiq
4	38.25	2.295	2.038	45	0.06	0.657	46.2	Infin	0.863	0.452	NonLiq
4	38.75	2.325	2.052	45	0.06	0.657	46.2	Infin	0.859	0.452	NonLiq
4	39.25	2.355	2.066	45	0.06	0.657	46.2	Infin	0.855	0.453	NonLiq
4	39.75	2.385	2.081	45	0.06	0.657	46.2	Infin	0.851	0.453	NonLiq
4	40.25	2.415	2.095	45	0.06	0.657	46.2	Infin	0.846	0.453	NonLiq
4	40.75	2.445	2.110	45	0.06	0.657	46.2	Infin	0.842	0.454	NonLiq
4	41.25	2.475	2.124	45	0.06	0.657	46.2	Infin	0.838	0.454	NonLiq
4	41.75	2.505	2.138	45	0.06	0.657	46.2	Infin	0.834	0.454	NonLiq
4	42.25	2.535	2.153	45	0.06	0.657	46.2	Infin	0.830	0.454	NonLiq
5	42.75	2.565	2.167	34	0.04	0.620	32.9	Infin	0.826	0.454	NonLiq
5	43.25	2.595	2.182	34	0.04	0.620	32.9	Infin	0.822	0.454	NonLiq

File Name: triv32.OUT

SOIL NO.	CALC. DEPTH (ft)	TOTAL STRESS (tsf)	EFF. STRESS (tsf)	FIELD N (B/ft)	FC DELTA N1_60	C N	CORR. (N1)60 (B/ft)	LIQUE. RESIST RATIO	r d	INDUC. STRESS RATIO	LIQUE. SAFETY FACTOR
5	43.75	2.625	2.196	34	0.04	0.620	32.9	Infin	0.818	0.454	NonLiq
5	44.25	2.655	2.210	34	0.04	0.620	32.9	Infin	0.814	0.454	NonLiq
5	44.75	2.685	2.225	34	0.04	0.620	32.9	Infin	0.810	0.454	NonLiq
5	45.25	2.715	2.239	34	0.04	0.620	32.9	Infin	0.806	0.454	NonLiq
5	45.75	2.745	2.254	34	0.04	0.620	32.9	Infin	0.802	0.454	NonLiq
5	46.25	2.775	2.268	34	0.04	0.620	32.9	Infin	0.798	0.454	NonLiq
6	46.75	2.805	2.282	58	0.07	0.600	54.4	Infin	0.794	0.453	NonLiq
6	47.25	2.835	2.297	58	0.07	0.600	54.4	Infin	0.789	0.453	NonLiq
6	47.75	2.865	2.311	58	0.07	0.600	54.4	Infin	0.785	0.452	NonLiq
6	48.25	2.895	2.326	58	0.07	0.600	54.4	Infin	0.781	0.452	NonLiq
6	48.75	2.925	2.340	58	0.07	0.600	54.4	Infin	0.777	0.452	NonLiq
6	49.25	2.955	2.354	58	0.07	0.600	54.4	Infin	0.773	0.451	NonLiq
6	49.75	2.985	2.369	58	0.07	0.600	54.4	Infin	0.769	0.450	NonLiq
6	50.25	3.015	2.383	58	0.07	0.600	54.4	Infin	0.765	0.450	NonLiq
6	50.75	3.045	2.398	58	0.07	0.600	54.4	Infin	0.761	0.449	NonLiq
6	51.25	3.075	2.412	58	0.07	0.600	54.4	Infin	0.757	0.448	NonLiq
6	51.75	3.105	2.426	58	0.07	0.600	54.4	Infin	0.753	0.448	NonLiq
6	52.25	3.135	2.441	58	0.07	0.600	54.4	Infin	0.749	0.447	NonLiq
6	52.75	3.165	2.455	58	0.07	0.600	54.4	Infin	0.745	0.446	NonLiq
7	53.25	3.195	2.470	56	0.06	0.600	52.5	Infin	0.741	0.445	NonLiq
7	53.75	3.225	2.484	56	0.06	0.600	52.5	Infin	0.737	0.444	NonLiq
7	54.25	3.255	2.498	56	0.06	0.600	52.5	Infin	0.732	0.444	NonLiq
7	54.75	3.285	2.513	56	0.06	0.600	52.5	Infin	0.728	0.443	NonLiq
7	55.25	3.315	2.527	56	0.06	0.600	52.5	Infin	0.724	0.442	NonLiq
7	55.75	3.345	2.542	56	0.06	0.600	52.5	Infin	0.720	0.441	NonLiq
7	56.25	3.375	2.556	56	0.06	0.600	52.5	Infin	0.716	0.440	NonLiq
7	56.75	3.405	2.570	56	0.06	0.600	52.5	Infin	0.712	0.438	NonLiq
7	57.25	3.435	2.585	56	0.06	0.600	52.5	Infin	0.708	0.437	NonLiq
7	57.75	3.465	2.599	56	0.06	0.600	52.5	Infin	0.704	0.436	NonLiq
7	58.25	3.495	2.614	56	0.06	0.600	52.5	Infin	0.700	0.435	NonLiq
7	58.75	3.525	2.628	56	0.06	0.600	52.5	Infin	0.696	0.434	NonLiq
7	59.25	3.555	2.642	56	0.06	0.600	52.5	Infin	0.692	0.433	NonLiq
7	59.75	3.585	2.657	56	0.06	0.600	52.5	Infin	0.688	0.431	NonLiq
7	60.25	3.615	2.671	56	0.06	0.600	52.5	Infin	0.684	0.430	NonLiq
7	60.75	3.645	2.686	56	0.06	0.600	52.5	Infin	0.680	0.429	NonLiq
7	61.25	3.675	2.700	56	0.06	0.600	52.5	Infin	0.676	0.427	NonLiq
7	61.75	3.705	2.714	56	0.06	0.600	52.5	Infin	0.671	0.426	NonLiq
7	62.25	3.735	2.729	56	0.06	0.600	52.5	Infin	0.667	0.425	NonLiq
7	62.75	3.765	2.743	56	0.06	0.600	52.5	Infin	0.663	0.423	NonLiq
7	63.25	3.795	2.758	56	0.06	0.600	52.5	Infin	0.659	0.422	NonLiq
7	63.75	3.825	2.772	56	0.06	0.600	52.5	Infin	0.655	0.420	NonLiq
7	64.25	3.855	2.786	56	0.06	0.600	52.5	Infin	0.651	0.419	NonLiq
7	64.75	3.885	2.801	56	0.06	0.600	52.5	Infin	0.647	0.417	NonLiq
7	65.25	3.915	2.815	56	0.06	0.600	52.5	Infin	0.643	0.416	NonLiq

SOIL NO.	CALC. DEPTH (ft)	TOTAL STRESS (tsf)	EFF. STRESS (tsf)	FIELD N (B/ft)	FC DELTA N1_60	C N	CORR. (N1)60 (B/ft)	LIQUE. RESIST RATIO	r d	INDUC. STRESS RATIO	LIQUE. SAFETY FACTOR
7	65.75	3.945	2.830	56	0.06	0.600	52.5	Infin	0.639	0.414	NonLiq
7	66.25	3.975	2.844	56	0.06	0.600	52.5	Infin	0.635	0.412	NonLiq

```
*****************************
*                           *
*     L I Q U E F Y 2       *
*                           *
*       Version 1.50        *
*                           *
*****************************
```

EMPIRICAL PREDICTION OF
EARTHQUAKE-INDUCED LIQUEFACTION POTENTIAL

JOB NUMBER: 2908   DATE: 02-22-2017

JOB NAME: Triv310

SOIL-PROFILE NAME: triv310.LDW

BORING GROUNDWATER DEPTH: 66.50 ft

CALCULATION GROUNDWATER DEPTH: 30.00 ft

DESIGN EARTHQUAKE MAGNITUDE: 6.61 Mw

SITE PEAK GROUND ACCELERATION: 0.480 g

BOREHOLE DIAMETER CORRECTION FACTOR: 1.00

SAMPLER SIZE CORRECTION FACTOR: 1.20

N60 HAMMER CORRECTION FACTOR: 1.30

MAGNITUDE SCALING FACTOR METHOD: Idriss (1998, in press)

    Magnitude Scaling Factor: 1.242

rd-CORRECTION METHOD: NCEER (1997)

FIELD SPT N-VALUES ARE CORRECTED FOR THE LENGTH OF THE DRIVE RODS.

    Rod Stick-Up Above Ground: 0.0 ft

CN NORMALIZATION FACTOR: 1.044 tsf

MINIMUM CN VALUE: 0.6

NCEER [1997] Method LIQUEFACTION ANALYSIS SUMMARY

File Name: triv310.OUT

SOIL NO.	CALC. DEPTH (ft)	TOTAL STRESS (tsf)	EFF. STRESS (tsf)	FIELD N (B/ft)	FC DELTA N1_60	C N	CORR. (N1)60 (B/ft)	LIQUE. RESIST RATIO	r d	INDUC. STRESS RATIO	LIQUE. SAFETY FACTOR
1	0.25	0.015	0.015	10	~	*	*	*	*	*	**
1	0.75	0.045	0.045	10	~	*	*	*	*	*	**
1	1.25	0.075	0.075	10	~	*	*	*	*	*	**
1	1.75	0.105	0.105	10	~	*	*	*	*	*	**
1	2.25	0.135	0.135	10	~	*	*	*	*	*	**
1	2.75	0.165	0.165	10	~	*	*	*	*	*	**
1	3.25	0.195	0.195	10	~	*	*	*	*	*	**
1	3.75	0.225	0.225	10	~	*	*	*	*	*	**
1	4.25	0.255	0.255	10	~	*	*	*	*	*	**
1	4.75	0.285	0.285	10	~	*	*	*	*	*	**
1	5.25	0.315	0.315	10	~	*	*	*	*	*	**
1	5.75	0.345	0.345	10	~	*	*	*	*	*	**
1	6.25	0.375	0.375	10	~	*	*	*	*	*	**
1	6.75	0.405	0.405	10	~	*	*	*	*	*	**
1	7.25	0.435	0.435	10	~	*	*	*	*	*	**
1	7.75	0.465	0.465	10	~	*	*	*	*	*	**
1	8.25	0.495	0.495	10	~	*	*	*	*	*	**
1	8.75	0.525	0.525	10	~	*	*	*	*	*	**
1	9.25	0.555	0.555	10	~	*	*	*	*	*	**
1	9.75	0.585	0.585	10	~	*	*	*	*	*	**
1	10.25	0.615	0.615	10	~	*	*	*	*	*	**
1	10.75	0.645	0.645	10	~	*	*	*	*	*	**
1	11.25	0.675	0.675	10	~	*	*	*	*	*	**
1	11.75	0.705	0.705	10	~	*	*	*	*	*	**
1	12.25	0.735	0.735	10	~	*	*	*	*	*	**
1	12.75	0.765	0.765	10	~	*	*	*	*	*	**
1	13.25	0.795	0.795	10	~	*	*	*	*	*	**
1	13.75	0.825	0.825	10	~	*	*	*	*	*	**
1	14.25	0.855	0.855	10	~	*	*	*	*	*	**
1	14.75	0.885	0.885	10	~	*	*	*	*	*	**
1	15.25	0.915	0.915	10	~	*	*	*	*	*	**
1	15.75	0.945	0.945	10	~	*	*	*	*	*	**
1	16.25	0.975	0.975	10	~	*	*	*	*	*	**
1	16.75	1.005	1.005	10	~	*	*	*	*	*	**
1	17.25	1.035	1.035	10	~	*	*	*	*	*	**
1	17.75	1.065	1.065	10	~	*	*	*	*	*	**
1	18.25	1.095	1.095	10	~	*	*	*	*	*	**
1	18.75	1.125	1.125	10	~	*	*	*	*	*	**
1	19.25	1.155	1.155	10	~	*	*	*	*	*	**
1	19.75	1.185	1.185	10	~	*	*	*	*	*	**
1	20.25	1.215	1.215	10	~	*	*	*	*	*	**
1	20.75	1.245	1.245	10	~	*	*	*	*	*	**
1	21.25	1.275	1.275	10	~	*	*	*	*	*	**

File Name: triv310.OUT

SOIL NO.	CALC. DEPTH (ft)	TOTAL STRESS (tsf)	EFF. STRESS (tsf)	FIELD N (B/ft)	FC DELTA N1_60	C N	CORR. (N1)60 (B/ft)	LIQUE. RESIST RATIO	r d	INDUC. STRESS RATIO	LIQUE. SAFETY FACTOR
1	21.75	1.305	1.305	10	~	*	*	*	*	*	**
1	22.25	1.335	1.335	10	~	*	*	*	*	*	**
1	22.75	1.365	1.365	10	~	*	*	*	*	*	**
1	23.25	1.395	1.395	10	~	*	*	*	*	*	**
1	23.75	1.425	1.425	10	~	*	*	*	*	*	**
1	24.25	1.455	1.455	10	~	*	*	*	*	*	**
1	24.75	1.485	1.485	10	~	*	*	*	*	*	**
1	25.25	1.515	1.515	10	~	*	*	*	*	*	**
1	25.75	1.545	1.545	10	~	*	*	*	*	*	**
1	26.25	1.575	1.575	10	~	*	*	*	*	*	**
1	26.75	1.605	1.605	10	~	*	*	*	*	*	**
1	27.25	1.635	1.635	10	~	*	*	*	*	*	**
1	27.75	1.665	1.665	10	~	*	*	*	*	*	**
1	28.25	1.695	1.695	10	~	*	*	*	*	*	**
1	28.75	1.725	1.725	10	~	*	*	*	*	*	**
1	29.25	1.755	1.755	10	~	*	*	*	*	*	**
1	29.75	1.785	1.785	10	~	*	*	*	*	*	**
2	30.25	1.815	1.807	68	0.10	0.758	80.6	Infin	0.928	0.291	NonLiq
2	30.75	1.845	1.822	68	0.10	0.758	80.6	Infin	0.924	0.292	NonLiq
2	31.25	1.875	1.836	68	0.10	0.758	80.6	Infin	0.920	0.293	NonLiq
2	31.75	1.905	1.850	68	0.10	0.758	80.6	Infin	0.916	0.294	NonLiq
3	32.25	1.935	1.865	34	0.05	0.703	37.3	Infin	0.912	0.295	NonLiq
3	32.75	1.965	1.879	34	0.05	0.703	37.3	Infin	0.907	0.296	NonLiq
3	33.25	1.995	1.894	34	0.05	0.703	37.3	Infin	0.903	0.297	NonLiq
3	33.75	2.025	1.908	34	0.05	0.703	37.3	Infin	0.899	0.298	NonLiq
3	34.25	2.055	1.922	34	0.05	0.703	37.3	Infin	0.895	0.299	NonLiq
3	34.75	2.085	1.937	34	0.05	0.703	37.3	Infin	0.891	0.299	NonLiq
3	35.25	2.115	1.951	34	0.05	0.703	37.3	Infin	0.887	0.300	NonLiq
3	35.75	2.145	1.966	34	0.05	0.703	37.3	Infin	0.883	0.301	NonLiq
3	36.25	2.175	1.980	34	0.05	0.703	37.3	Infin	0.879	0.301	NonLiq
3	36.75	2.205	1.994	34	0.05	0.703	37.3	Infin	0.875	0.302	NonLiq
3	37.25	2.235	2.009	34	0.05	0.703	37.3	Infin	0.871	0.302	NonLiq
4	37.75	2.265	2.023	45	0.06	0.657	46.2	Infin	0.867	0.303	NonLiq
4	38.25	2.295	2.038	45	0.06	0.657	46.2	Infin	0.863	0.303	NonLiq
4	38.75	2.325	2.052	45	0.06	0.657	46.2	Infin	0.859	0.304	NonLiq
4	39.25	2.355	2.066	45	0.06	0.657	46.2	Infin	0.855	0.304	NonLiq
4	39.75	2.385	2.081	45	0.06	0.657	46.2	Infin	0.851	0.304	NonLiq
4	40.25	2.415	2.095	45	0.06	0.657	46.2	Infin	0.846	0.304	NonLiq
4	40.75	2.445	2.110	45	0.06	0.657	46.2	Infin	0.842	0.305	NonLiq
4	41.25	2.475	2.124	45	0.06	0.657	46.2	Infin	0.838	0.305	NonLiq
4	41.75	2.505	2.138	45	0.06	0.657	46.2	Infin	0.834	0.305	NonLiq
4	42.25	2.535	2.153	45	0.06	0.657	46.2	Infin	0.830	0.305	NonLiq
5	42.75	2.565	2.167	34	0.04	0.620	32.9	Infin	0.826	0.305	NonLiq
5	43.25	2.595	2.182	34	0.04	0.620	32.9	Infin	0.822	0.305	NonLiq

File Name: triv310.OUT

SOIL NO.	CALC. DEPTH (ft)	TOTAL STRESS (tsf)	EFF. STRESS (tsf)	FIELD N (B/ft)	FC DELTA N1_60	C N	CORR. (N1)60 (B/ft)	LIQUE. RESIST RATIO	r d	INDUC. STRESS RATIO	LIQUE. SAFETY FACTOR
5	43.75	2.625	2.196	34	0.04	0.620	32.9	Infin	0.818	0.305	NonLiq
5	44.25	2.655	2.210	34	0.04	0.620	32.9	Infin	0.814	0.305	NonLiq
5	44.75	2.685	2.225	34	0.04	0.620	32.9	Infin	0.810	0.305	NonLiq
5	45.25	2.715	2.239	34	0.04	0.620	32.9	Infin	0.806	0.305	NonLiq
5	45.75	2.745	2.254	34	0.04	0.620	32.9	Infin	0.802	0.305	NonLiq
5	46.25	2.775	2.268	34	0.04	0.620	32.9	Infin	0.798	0.304	NonLiq
6	46.75	2.805	2.282	58	0.07	0.600	54.4	Infin	0.794	0.304	NonLiq
6	47.25	2.835	2.297	58	0.07	0.600	54.4	Infin	0.789	0.304	NonLiq
6	47.75	2.865	2.311	58	0.07	0.600	54.4	Infin	0.785	0.304	NonLiq
6	48.25	2.895	2.326	58	0.07	0.600	54.4	Infin	0.781	0.303	NonLiq
6	48.75	2.925	2.340	58	0.07	0.600	54.4	Infin	0.777	0.303	NonLiq
6	49.25	2.955	2.354	58	0.07	0.600	54.4	Infin	0.773	0.303	NonLiq
6	49.75	2.985	2.369	58	0.07	0.600	54.4	Infin	0.769	0.302	NonLiq
6	50.25	3.015	2.383	58	0.07	0.600	54.4	Infin	0.765	0.302	NonLiq
6	50.75	3.045	2.398	58	0.07	0.600	54.4	Infin	0.761	0.302	NonLiq
6	51.25	3.075	2.412	58	0.07	0.600	54.4	Infin	0.757	0.301	NonLiq
6	51.75	3.105	2.426	58	0.07	0.600	54.4	Infin	0.753	0.301	NonLiq
6	52.25	3.135	2.441	58	0.07	0.600	54.4	Infin	0.749	0.300	NonLiq
6	52.75	3.165	2.455	58	0.07	0.600	54.4	Infin	0.745	0.300	NonLiq
7	53.25	3.195	2.470	56	0.06	0.600	52.5	Infin	0.741	0.299	NonLiq
7	53.75	3.225	2.484	56	0.06	0.600	52.5	Infin	0.737	0.298	NonLiq
7	54.25	3.255	2.498	56	0.06	0.600	52.5	Infin	0.732	0.298	NonLiq
7	54.75	3.285	2.513	56	0.06	0.600	52.5	Infin	0.728	0.297	NonLiq
7	55.25	3.315	2.527	56	0.06	0.600	52.5	Infin	0.724	0.296	NonLiq
7	55.75	3.345	2.542	56	0.06	0.600	52.5	Infin	0.720	0.296	NonLiq
7	56.25	3.375	2.556	56	0.06	0.600	52.5	Infin	0.716	0.295	NonLiq
7	56.75	3.405	2.570	56	0.06	0.600	52.5	Infin	0.712	0.294	NonLiq
7	57.25	3.435	2.585	56	0.06	0.600	52.5	Infin	0.708	0.294	NonLiq
7	57.75	3.465	2.599	56	0.06	0.600	52.5	Infin	0.704	0.293	NonLiq
7	58.25	3.495	2.614	56	0.06	0.600	52.5	Infin	0.700	0.292	NonLiq
7	58.75	3.525	2.628	56	0.06	0.600	52.5	Infin	0.696	0.291	NonLiq
7	59.25	3.555	2.642	56	0.06	0.600	52.5	Infin	0.692	0.290	NonLiq
7	59.75	3.585	2.657	56	0.06	0.600	52.5	Infin	0.688	0.290	NonLiq
7	60.25	3.615	2.671	56	0.06	0.600	52.5	Infin	0.684	0.289	NonLiq
7	60.75	3.645	2.686	56	0.06	0.600	52.5	Infin	0.680	0.288	NonLiq
7	61.25	3.675	2.700	56	0.06	0.600	52.5	Infin	0.676	0.287	NonLiq
7	61.75	3.705	2.714	56	0.06	0.600	52.5	Infin	0.671	0.286	NonLiq
7	62.25	3.735	2.729	56	0.06	0.600	52.5	Infin	0.667	0.285	NonLiq
7	62.75	3.765	2.743	56	0.06	0.600	52.5	Infin	0.663	0.284	NonLiq
7	63.25	3.795	2.758	56	0.06	0.600	52.5	Infin	0.659	0.283	NonLiq
7	63.75	3.825	2.772	56	0.06	0.600	52.5	Infin	0.655	0.282	NonLiq
7	64.25	3.855	2.786	56	0.06	0.600	52.5	Infin	0.651	0.281	NonLiq
7	64.75	3.885	2.801	56	0.06	0.600	52.5	Infin	0.647	0.280	NonLiq
7	65.25	3.915	2.815	56	0.06	0.600	52.5	Infin	0.643	0.279	NonLiq

SOIL NO.	CALC. DEPTH (ft)	TOTAL STRESS (tsf)	EFF. STRESS (tsf)	FIELD N (B/ft)	FC DELTA N1_60	C N	CORR. (N1)60 (B/ft)	LIQUE. RESIST RATIO	r d	INDUC. STRESS RATIO	LIQUE. SAFETY FACTOR
7	65.75	3.945	2.830	56	0.06	0.600	52.5	Infin	0.639	0.278	NonLiq
7	66.25	3.975	2.844	56	0.06	0.600	52.5	Infin	0.635	0.277	NonLiq

Boring 1

66.5

30.0	10.0	0	120.0	0	0.100	15.25
32.0	24.0	1	120.0	62	0.100	30.25
36.5	38.0	1	120.0	0	0.100	35.25
42.5	35.0	1	120.0	0	0.100	40.25
48.0	30.0	1	120.0	69	0.100	45.25
52.5	33.0	1	120.0	0	0.100	50.25
57.5	47.0	1	120.0	0	0.100	55.25
62.5	55.0	1	120.0	0	0.100	60.25
66.5	45.0	1	120.0	0	0.100	66.25

Boring 2

66.5

30.0	10.0	0	120.0	0	0.100	15.25
38.0	21.0	1	120.0	34	0.100	35.25
42.0	33.0	1	120.0	0	0.100	40.25
48.0	30.0	0	120.0	0	0.100	45.25
66.5	39.0	1	120.0	0	0.100	66.25

Boring 3

66.5

30.0	10.0	0	120.0	0	0.100	15.25
32.0	68.0	1	120.0	0	0.100	30.25
37.5	34.0	1	120.0	0	0.100	35.25
42.5	45.0	1	120.0	0	0.100	40.25
46.5	34.0	1	120.0	0	0.100	45.25
53.0	58.0	1	120.0	0	0.100	50.25
66.5	56.0	1	120.0	0	0.100	66.25