REPORT OF

GEOTECHNICAL INVESTIGATION

PROPOSED 3-STORY OFFICE BUILDING OVER ONE LEVEL PARKING

TEMESCAL CANYON ROAD

APN: 2821-210-011-4

CORONA, CALIFORNIA

FOR

MR. JOHN SOLDAT

PROJECT NO. 22-449

MARCH 4, 2023

GTC, Inc.

GeoTech Consultants, Inc.

1545 N. Verdugo Road No. 7

Glendale, California 91208

Tel: (310) 945-7288 Behnamgeotech@gmail.com

March 4, 2023 22-449

John Soldat
1902 Fullerton Ave.,
Corona, CA 92881

Subject: Geotechnical Investigation
Proposed 3-Story Office Building over One Level of Parking
Temescal Canyon Road,
APN: 282-121-011-04
Corona, CA

INTRODUCTION

Dear Mr. Soldat;

Pursuant to your request, a Limited Geotechnical Investigation has been performed at the proposed site of new construction in Glendale, California. During the course of this investigation, the engineering properties of the subsurface materials were evaluated in order to provide recommendations for development of the site, including earthwork, seismic design, retaining walls, excavations, shoring, and foundation design. The investigation included subsurface exploration, soil sampling, laboratory testing, engineering evaluation and analysis, consultation and preparation of this report. During the course of this investigation, the project plans provided by the client were used as reference. The plans were electronically submitted to our office.

GeoTech Consultants, Inc. (GTC) has prepared this soils investigation report for the proposed project by drilling 4 deep borings at 52, 17, 17, and 12 feet, conducting laboratory tests and engineering analysis and calculations independent of all previous work. All of the presented engineering values and analysis in this report are based on our samples derived from the borings and laboratory tests conducted in our soils lab.

Engineering for the proposed project should not begin until approval of the geotechnical investigation is granted by the local building official.

LIST OF ATTACHMENTS TO THIS REPORT

The attached Appendix I, describes the method of field exploration. Appendix II describes the laboratory testing procedures.

Plate No. 1 shows the Site Location.

The enclosed site plan, drawings No. 1, shows the approximate location of the exploratory borings and off-site properties in relation to the site boundaries and the proposed building.

Figure Nos. I-1 and I-2 presents summaries of the materials encountered at the location of our borings. Figure Nos. I-3 presents summaries of the materials encountered at the location of our boring drilled for the purpose of percolation test. Figure No. I-4 presents the Uniform Soil Classification System Chart; a guide to the Log of Exploratory Borings.

Figure Nos. II-1 and II-2 present the results of direct shear and consolidation tests performed on selected undisturbed soil samples.

Figure No.1 presents the soil bearing capacity.

ASCE Design Maps Summary Report & Seismic Parameters.

Table 1: Unrestrained and Restrained Drained Retaining walls, represents the result of active, at-rest,and seismic lateral pressure calculations on restrained and cantilever walls.

Table 3: Shoring Design, is the result of the computer printout calculations that follow the tables.

Following the Tables 1 & 2 are computer printouts, the result of our lateral pressure calculations on the shoring system.

Liquefaction and Dynamic Settlements Evaluation

It should be noted that the presented recommendations in this report are based on our understanding of the depth of excavation, structural setback and assumed loading data. This office should be notified if the actual loading and excavation depths are different from those used during this investigation.

PROJECT CONSIDERATIONS

It is our understanding that the proposed project will consist of construction of a new office building. The building will be a 4-story structure, approximately 60 feet in height, at same approximate elevation as Temescal Canyon Road (no subterranean levels are proposed). The approximate location of the proposed building is shown on the enclosed Site Plan; Drawing No. 1.

Structural loading data was not available during the course of preparation of this report. For the purpose of this report, it is assumed that concentrated loads will be on the order of 450 kips, combined dead plus frequently applied live loads. Perimeter and interior wall footings of the structure are expected to exert loads of on the order of 3 kips per lineal foot.

SITE CONDITIONS

SURFACE CONDITIONS

The site of the proposed project covers one vacant lot and is located along the west side of Temescal Canyon Road that have been assigned as APN 282-121-011-04. The site is irregular in shape, covering an area of about 50,000 square feet. See the enclosed Site Plan; Drawing No. 1.

There is an elevation difference of approximately 23 feet between the Temescal Canyon Road (East side) and West side of the property (with higher elevation).

PURPOSE AND SCOPE OF SERVICES

The purpose of this geotechnical investigation was to evaluate subsurface soil conditions at the site of the proposed improvements; and to provide geotechnical recommendations pertaining to earthwork and foundation aspects of the project. The scope of services performed for this geotechnical investigation consisted of subsurface exploration, laboratory testing, engineering analysis of field and laboratory data, and preparation of this report.

Environmental services such as evaluation and chemical analysis of the soil and groundwater for hazardous material were not included in our scope of services.

SUBSURFACE INVESTIGATION

The site was explored on December 7, 2022 by drilling four exploratory borings. The borings were extended to a maximum depth of 52 feet below existing grades with the aid of a truck-mounted drilling machine using 8-inchdiameter hollow-stem augers. The approximate locations of the borings are shown on the enclosed Drawing No.1. Continuous logs of the subsurface conditions, as encountered in the excavated borings, were recorded in the field and are presented on the Log of Exploratory borings.

LABORATORY TESTING

The laboratory tests were conducted on representative samples in order to determine certain physical properties of the subsurface materials. Field moisture content, in-situ density, shear strength, consolidation were determined from these tests.

- ASTM D422
  Standard Test Method for Particle-Size Analysis of Soils;
- ASTM D2216
  Standard Test Method for Determination of Water (Moisture) Content of Soil by Microwave Oven Heating;
- ASTM D3080
  Standard Test Method for Direct Shear Test of Soils under Consolidated Drained Conditions;
- ASTM D2435
  Standard Test Methods for One-Dimensional Consolidation Properties of Soils Using Incremental Loading;

The laboratory test results are presented on Figure Nos. II-1 and II-2, within Appendix II.

ENGINEERING ANALYSIS

The results of our field and laboratory investigations were evaluated. Based on the results of the laboratory testing, engineering analyses were performed in order to formulate recommendations for design and construction of foundations.

SUBSURFACE CONDITIONS

SOIL PROFILE

Correlation of the subsoil between the borings was considered to be good. Generally, the site, to the depths explored, was found to be covered with surficial fill underlain by natural alluvium deposites of silty sand in slightly moist, and dense condition. Thickness of the surficial fill was found to be about three feet (3) at the location of our borings. The existing surficial fill were found to be generally porous in-place and compressible. At their present state, such soils are considered to be inadequate for foundations and grade slabs support. The materials found below the planned foundation levels were found to be generally very dense, silty sand and sandy soils.

GROUNDWATER

During the course of our investigation, no groundwater was encountered in our exploratory borings drilled to maximum depth of 52 feet below existing ground surface. According to the Riverside Western Municipal Water District office, groundwater records from nearby wells indicate that the groundwater level is more than 100 feet below the ground surface.

SEISMIC CONSIDERATIONS

EVALUATION OF LIQUEFACTION POTENTIAL

Liquefaction is a phenomenon in which saturated silty to cohesionless soils below the groundwater table are subject to a temporary loss of strength due to the buildup of excess pore pressure during cyclic loading conditions such as those induced by an earthquake. Liquefaction related effects include loss of bearing strength, amplified ground oscillations, lateral spreading, and flow failures.

The project site has not yet been mapped for earthquake-induced hazard zones under the Seismic Hazards Mapping Act by CGS.

Groundwater was not encountered during exploration, which were excavated to a maximum depth of 52 feet below the existing site grade. According to the Riverside Western Municipal Water District office, groundwater records from nearby wells indicate that the groundwater level is more than 100 feet below the ground surface.

Based on the dense nature of the underlying soils, and the depth to the groundwater level, the potential for liquefaction occurring at the site is considered to be remote.

The site is located within a State of California Liquefaction Seismic Hazard Zone. During the course of our investigation, no groundwater was found in our boring No.1 drilled to depth of 52 feet below existing ground surface.

Seismically-induced settlement or compaction of dry or moist, cohesionless soils can be an effect related to 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 structures should be expected as a result of strong ground-shaking, however, due to the uniform nature of the underlying geologic materials, excessive differential settlements are not expected to occur.

Seismically-induced settlement calculations were performed using the SPT blowcounts collected from the exploratory borings, and the method by Tokimatsu and Seed (1987). The results indicate that seismically induced dry settlement on the site, subsequent to the recommended grading, will be on the order of 0.24 inches. This is considered to be negligible and well within the tolerance of a well-designed structure.

LANDSLIDING

The subject site is not mapped within an Earthquake Induced Landslide Zone by the State of California or City. No reported occurrences of landslides are known to have recently affected the site. Therefore, the potential for landslides is consider to be very low at the site.

TSUNAMIS

The subject site is not located within a Tsunami Hazard Zone delineated by the State of California. Due to the distance from the Pacific Ocean of (25) miles, and the elevation of the site at approximately (900) feet above sea level, the potential for tsunami inundation is very low.

SEISMIC PARAMETERS

Seismic design parameters based on the new code have been provided. In accordance with the ASCE 7-16, corresponding to CBC 2020, the project site can be classified as site "D". The mapped spectral accelerations of S_s= 2.289 and S_1= 0.911 can be used for this project. These parameters correspond to site coefficient values of F_a=1.0 and F_V= N/A respectively (see the seismic design parameters and note below).

S_{MS}= $F_a(S_s)$= 1.0 (2.289) = 2.289	S_{M1}= $F_V(S_1)$= N/A (see note below)
S_{DS}= ⅔(S_{MS})= ⅔ (2.289) = 1.526	S_{D1}= ⅔(S_{M1})= N/A (see note below)

Note: Since the seismic factor, S_1 is greater than 0.2 site-specific ground-motion hazard analysis may be required. The project structural engineer shall determine if an exemption can be applied in accordance with ASCE 7-16 Section 11.4.8. If an exemption applies, a long period coefficient (F_V) of 1.7 may be utilized for calculation of seismic parameters. A copy of the detailed ASCE out-put is included with this report.

EVALUATION AND RECOMMENDATIONS

GENERAL

Based on the geotechnical engineering data derived from this investigation, the site is considered to be suitable for the proposed development. Our findings conclude that the site of the proposed work will be safe against hazards from landslide, settlement or slippage. The proposed work will have no effect on the geotechnical stability of the area outside of the proposed work. The opinions, conclusions and recommendations presented herein are based on our field and office studies, the properties of the soils encountered in our borings, and the results of our laboratory testing program.

Geotechnical recommendations for temporary excavations, foundations, lateral design, grade slabs, subsurface walls, and observations during construction are presented in the remaining portions of this report.

EXPANSIVE POTENTIAL

Based on depth of the proposed subterranean levels, the proposed structure would not be prone to the effect of expansive soils.

SLOPE STABILITY

The property has less than 25 feet of overall elevation change at a gradient of approximately 11:1 or gentler (H:V) gradient. A slope stability analysis is not required for the property per Riverside County Department of Building and Safety.

TEMPORARY EXCAVATION

Unsupported/open Cuts: Where space limitations permit, unshored temporary excavation slopes could be used. Based upon the engineering characteristics of the site upper soils, it is our opinion that temporary excavation slopes in accordance with the following table should be used:

Maximum Depth of Cut (Ft)	Maximum Slope Ratio (Horizontal:Vertical)
0-4	Vertical
>4	1:1

In order to retard the chances of erosion, open cut slopes should be covered with plastic sheeting during the rainy periods. All temporary cuts should be stabilized within 4 week from the initial date of excavation.

Water should not be allowed to flow over the top of the excavation in an uncontrolled manner. No surcharge should be allowed within a 45-degree line drawn from the bottom of the excavation. Excavation surfaces should be kept moist but not saturated to retard raveling and sloughing during construction.

SITE PREPARATION

Debris from demolition and underground utility lines to be abandoned should be removed from the building area. All excavations resulting from removal of existing obstructions should be backfilled with soil compacted to at least 90 percent of the maximum density as determined by ASTM: D-1557. If any cesspools or seepage pits are encountered during shoring, they should be backfilled with vibrated gravel or slurry mix to 5 feet below finish grade. The upper 5 feet should be backfilled with soil compacted by mechanical means.

FILL PLACEMENT

Fill soils, if any, should be cleaned of deleterious debris, placed in 6 to 8 inch lifts, brought to about optimum moisture content and compacted to at least 90 percent of the maximum density for granular soils. The placement of the fill should be performed under our observation and testing.

GENERAL GUIDELINES FOR SITE GRADING

Site grading for the proposed project is expected to include excavation in order to create the finish grades, and backfilling behind the retaining walls. All grading operations should be performed in compliance with all applicable codes and the minimum specifications outlined below. Observation and testing will be necessary during these phases of the project to allow our office to provide certification of finished project.

SITE PREPARATION AND EXCAVATION

- Any existing structures designated for removal are to include the existing foundation system. All construction debris are to be stripped and wasted from the site.

- Any trees or shrubs designated for removal should be cut down and all stumps and roots should be removed. All major vegetation, organic soil and debris material are to be stripped and wasted from the site.
- The approximate horizontal and vertical extent of these excavations are to be verified by the project geotechnical consultant in the field.

FILL PLACEMENT

It is recommended that all fill be placed under engineering observation and in accordance with the following guidelines.

1. Compacted fill is to be placed over horizontal benches excavated into dense competent existing material.
2. Compacted fill may be placed to design grades using onsite inorganic soils or approved import.
3. Soil proposed for use as structural fill should be inorganic, free from deleterious materials, and contain no more than 15 percent by weight of rocks larger than four (4) inches (largest dimension).
4. We expect that materials excavated onsite will be suitable for use as compacted fill provided they do not contain appreciable quantities of organic debris.
5. Where in-place moisture content exceeds optimum values, the materials may need to be spread and dried, or mixed with dryer material. Final determination will be provided in the field by the project geotechnical consultants at the time the excavations take place.
6. Excavated material containing excessive organic debris will not be suitable for use in the compacted fill. Materials deemed unsuitable should be wasted offsite or as designated by the project architect or geotechnical consultant.
7. The approved material should be placed in layers, each not exceeding eight (8) inches in thickness (before compaction), water conditions to about two percent above optimum moisture content and compacted to a minimum 90 percent relative compaction based on ASTM Test D1557.
8. Fill compaction tests should be performed during placement of the future fills to verify acceptable compaction and moisture content. At a minimum, one test

should be performed within each 12 to 24 inches (vertical depth) or 500 cubic yards of fill (whichever is less). More frequent testing may be required by the geotechnical consultant.

9. If construction takes place during the winter months or unseasonable rainy periods, additional winterizing and erosion-control recommendations may be necessary.

FOUNDATIONS DESIGN

CONVENTIONAL

Conventional spread footing foundation systems properly compacted fill soils or competent native soils are expected to provide adequate support for the proposed building. Continuous foundation may be designed for a bearing capacity 2,400 pounds per square foot, and should be a minimum of 18 inches in width, 24 inches in depth below the lowest adjacent final grades. Column foundation may be designed for a bearing capacity of 2,600 pounds per square foot, and should be a minimum of 24 inches in width, 24 inches in depth below the lowest adjacent final grades. The bearing capacity may be may be increased at a rate of 150 pounds per square foot for each additional foot of footing depth and width, to a maximum value of 3,600 pounds per square foot.

The above given values are for the total of dead, plus frequently applied live loads. For short duration transient loading, such as wind or seismic forces, the given values may be increased by one-third.

EXPECTED SETTLEMENTS

Under the allowable maximum soil pressure, footings carrying the assumed maximum concentrated loads of 450 kips are expected to settle on the order of 3/4 of an inch. Continuous footings, with loads of about 3 kips per lineal foot are expected to settle on the order of 1/2 of an inch. Maximum differential settlements are expected to be on the order of 1/4 of an inch. The major portion of the settlements is expected to occur during construction.

LATERAL DESIGN

Lateral resistance at the base of footings in contact with native soils or properly compacted fill soils may be assumed to be the product of the dead load forces and a coefficient of friction of 0.30. Passive pressure on the face of footings may also be used to resist lateral forces. A passive pressure of zero at the ground surface and increasing at a rate of 300 pounds per square foot per foot of depth to a maximum value of 3,000 pounds per square foot may be used for footings poured against native and/or properly compacted fill soils.

RETAINING WALL DESIGN

CANTILEVER RETAINING WALLS

Retaining walls supporting a level back-slope may be designed utilizing a triangular distribution of pressure. Cantilever retaining walls may be designed for 37 pound per cubic foot for walls retaining up to 15 feet of earth for drained condition.

Retaining walls exceeding 6 feet in height shall be designed to resist the additional earth pressure caused by seismic ground shaking. An inverse triangular pressure distribution should be utilized for the additional seismic loads (13 pcf) as illustrated in the attachments to this report.

RETAINING WALL DRAINAGE

Proper subdrain should be installed behind the retaining walls. Subdrain for retaining walls normally consists of four-inch diameter perforated pipes, placed with perforation facing down. The pipe shall be encased in at least one-foot of gravel around the pipe. The gravel may consist of three-quarter inch to one inch crushed rocks.

RETAINING WALL BACKFILL

Where adequate space is available, granular fill should be placed and mechanically compacted in layers not more than 8 inches thick, behind the retaining walls (after the subdrain is installed) to a relative compaction of at least 90 percent. At least one field density tests should be taken for each 2 feet of the backfill. The degree of compaction of the wall backfill should be verified by the Soil Engineer.

Where space is limited, free-draining gravel should be placed behind the retaining walls. The gravel should then be capped with at least 18 inch thick site soils also compacted to a relative compaction of at least 90 percent. It should be noted that the backfill placed behind the retaining walls should be made after the concrete decking is cast. All grading surrounding the building should be such to ensure that water drains freely from the site and does not pond.

PAVEMENT RECOMMENDATION

In order to provide uniform support beneath the proposed pavement area, it is recommended that a minimum of 24 inches of the exposed subgrade beneath the pavement section be properly removed and recompacted to 90 percent of maximum density as determined by most recent version of ASTM D 1557. Subsequent to the recommended removal, the excavated subgrade shall be scarified to a minimum depth of 12 inches and recompacted to a minimum of 95 percent relative compaction prior to placement of compacted fill. The following pavement sections are recommended:

Pavement Area	Assumed Traffic Index	Asphalt Thickness (in)	Aggregate Base Thickness (in)
Parking / Driveways	5	3.0	5.0
Truck Access/Fire Lanes	7	4.0	8.0

A subgrade modulus of 120 pound per cubic inch may be assumed for design of concrete Paving. Concrete paving for passenger cars and moderate truck traffic shall be assumed be a minimum of 6 inches in thickness, and shall be underlain by 4 inches of aggregate base. Concrete paving for heavy truck traffic shall be a minimum 7½ inches in thickness, and shall be underlain by 6 inches of aggregate base. Concrete pavement should be reinforced with minimum of #3 steel bar on 24 inch centers each way.

DRAINAGE

Adequate site drainage is absolutely essential at the site and it should be provided. Roof drainage should be connected to an appropriate drainage system and carried away from the building and to the street. Yard drainage should be kept adequate to prevent ponding of water and saturation of the soils. Water should be directed to the street in an approved manner. Future performance of the building and appurtenances will be significantly influenced by the site drainage conditions. Planters and lawns adjacent to the building should be avoided. If planters are planned adjacent to the building, they should have the bottom and walls waterproofed and a drain installed to carry irrigation water away from the footing areas. Site drainage should be provided to divert roof and surface waters from the property through non-erodible drainage devices to the street. In no case should the surface waters be allowed to pond adjacent to building or behind the retaining walls. A minimum slope of two and five percent are recommended for paved and unpaved areas, respectively.

FLOOR SLAB ON GRADE

The slabs-on-grade thickness and reinforcement should reflect the anticipated use of the slab and should be designed by the Structural Engineer. The floor slabs-on- grade should be a minimum of 5 inches thick with minimum reinforcement consisting of #4 bars spaced maximum at 16 inches each way (#4 @ maximum 16" o.c. each way) placed slightly above the slab mid-height. Cracking of reinforced concrete is a relatively common occurrence. Some cracking of reinforced concrete, including slabs, can be anticipated. Irregularities in new slabs are also common. If cracking of slabs cannot be tolerated, heavily reinforced structural slabs are an option.

MOISTURE-SENSITIVE SPECIAL CONSIDERATIONS

GTC does not practice in the field of moisture vapor transmission evaluation and mitigation. Therefore it is recommended that a qualified consultant be engaged to evaluate the general and specific moisture vapor transmission paths and any impact on the proposed construction. The qualified consultant should provide recommendations for mitigation of potential adverse impacts of moisture vapor transmission on various components of the structure.

Where dampness would be objectionable, it is recommended that the floor slabs should be waterproofed. A qualified waterproofing consultant should be retained in order to recommend a product or method which would provide protection for concrete slabs-on-grade. All concrete slabs-on-grade should be supported on vapor retarder. The design of the slab and the installation of the vapor retarder should comply with the most recent revisions of ASTM E 1643 and ASTM E 1745.

Where a vapor retarder is used, a low-slump concrete should be used to minimize possible curling of the slabs. The barrier can be covered with a layer of trimmable, compactible, granular fill, where it is thought to be beneficial. See ACI 302.2R-32, Chapter 7 for information on the placement of vapor retarders and the use of a fill layer.

PERCOLATION TEST (SURFACE WATER INFILTRATION)

In order to establish a percolation rate for the site soils, we drilled additional boring No.5 for percolation test. The boring was drilled to depth of 15 feet. Casing (4-inches diameter perforated pipe) was placed within the boring to test the area from 10 to 15 feet and below of the existing grade. As per our borings No.1, 2, 3, and 4 the soils materials continue to be similar below 5 feet. After pre-soak, the boring was refilled with water and the absorption of the soils was measured. The result of the percolation test is attached in Percolation Test, Figure No. I.

The laboratory tests were conducted on representative samples in order to determine certain physical properties of the subsurface materials. Field moisture content, in-situ density, and sieve analysis were determined from these tests. The laboratory test results are presented on Figure Nos. I-1.

As per our test results, the site soil below the depths of 10-15 feet (where water infiltration would be allowed) has a percolation rates, on the order of 2.7 inches per hour. According to the Riverside County LID BMP Design Handbook, a safety factor of 3 should be applied to obtain the design infiltration rate.

It is the opinion of this firm that shallow infiltration basins and/or deep “dry well” type infiltration systems may be utilized for stormwater infiltration. The proposed infiltration systems shall maintain a minimum horizontal distance of 10 feet from any structures or foundations, or setback beyond a 1:1 (H:V) plane projected upward from the bottom of any subterranean structures, whichever is greater.

All infiltration devices should be provided with overflow protection. Once the device is full of water, additional water flowing to the device should be diverted to another acceptable disposal area, or disposed offsite in an acceptable manner.

All connections associated with stormwater infiltration devices should be sealed and water-tight. Water leaking into the subgrade soils can lead to loss of strength, piping, erosion, settlement and/or expansion of the effected earth materials.

Based on results of the site explorations, the onsite soils are primarily granular in nature. The soils are in the very low to low expansion range. The onsite soils should allow stormwater to percolate generally in a vertical manner. Therefore, the potential of creating a perched water condition is negligible. In addition, laboratory testing indicates that the onsite soils are not susceptible to significant hydroconsolidation.

Excavations proposed for the installation of stormwater facilities should comply with the "Temporary Excavations" sections of this (the referenced) reports well as CalOSHA Regulations where applicable.

PLAN REVIEW

Formal plans ready for submittal to the building department should be reviewed by GeoTech Consultants. Any changes in scope of the project may require additional work.

GEOTECHNICAL OBSERVATION

The building department requires that the geotechnical engineer of the record provides site observation during construction. Foundation excavations should be observed and approved by the geotechnical engineer prior to placing steel, forms, or concrete. The engineer should observe bottoms for fill, compaction of fill, temporary excavations, soldier piles, raker deadmen, the installation and stress testing of tieback anchors, and subdrains. All fill that is placed should be approved by the geotechnical engineer and the building department prior to use for support of structural footings and floor slabs.

The building department stamped plans, the permits, and the geotechnical reports should be at the site and available to our representative. The project consultant will perform the observation and post a notice (field memo) at the job site with the findings. This notice should be given to the agency inspector.

WORKMAN SAFETY-EXCAVATIONS

It is necessary for the contractor to provide adequate shoring and safety equipment as required by the State or Federal OSHA regulations. All regulations of the State or Federal OSHA should be followed before allowing workmen in a trench or other excavation. If excavations are to be made during the rainy season, particular care should be given to insure that berms or other devices will prevent surface water from flowing over the top of the excavations or ponding at the top of the excavations.

CLOSURE

The findings and recommendations presented in this report were based on the results of our field and laboratory investigations combined with professional engineering experience and judgment. The report was prepared in accordance with generally accepted engineering principles and practice. We make no other warranty, either express or implied.

It is noted that the conclusions and recommendations presented are based on exploration "window" borings and excavations which is in conformance with accepted engineering practice. Some variations of subsurface conditions are common between "windows" and major variations are possible.

-o0o-

The following Figures and Appendices are attached and complete this report:

Appendix I-Method of Field Exploration

Appendix II-Methods of Laboratory Testing

Site Location Map Plate No. 1

Seismic Hazard Zone Map   Plate No. 2

Historically Highest Groundwater Contour Map Plate No. 3

Seismic Hazard Map (Alluvium Condition) Plate No. 4

Site Plan Drawing No.1

Figure Nos. I-1 through I-2 Log of Borings & I-3 Guide to the log of borings

Figure Nos. II-1 and II-2 Direct Shear and Swell – Consolidation Tests

Summary of Calculations Fig. No. 1, Bearing Capacity Calculations

ASCE Design Maps Summary Report & Seismic Parameters.

Liquefaction Analysis.

Percolation Test Results

Respectfully Submitted

GeoTech Consultants, Inc.

Reviewed By:

Behnam Mahmoudkhani, M.Sc., P.E.
Civil Engineer
CE 88488

APPENDIX I
METHOD OF FIELD EXPLORATION

In order to define the subsurface conditions, four borings were made on the site. The approximate location of the drilled borings are shown on the enclosed Site Plan. Borings were extended to maximum depth of about 52 feet below the existing grades. Borings were drilled with an auger.

Continuous logs of the subsurface conditions, as encountered in the test borings, were recorded during the field work and are presented on Figure Nos. I-1 and I-2 within this Appendix. These figures also show the number and approximate depths of each of the recovered soil samples.

The drilling of the borings was supervised by our field engineer who logged the materials brought up from the borings. Undisturbed and bulk samples were collected at depths appropriate to the investigation. The undisturbed sampler utilized in our investigation included our 2.50 inch I.D. drive barrel lined with 1 inch brass rings. The sampler used in the exploratory borings was driven to a depth of 12 inches with a 140-pound hammer falling through a height of 30 inches. The number of blows to drive the sampler 12 inches is shown on the attached Logs of Borings.

APPENDIX II

LABORATORY TESTING PROCEDURES

Moisture Density

The moisture-density information provides a summary of soil consistency for each stratum and can also provide a correlation between soils found on this site and other nearby sites. The dry unit weight and field moisture content were determined for each undisturbed sample, and the results are shown on the log of exploratory borings.

Shear Tests

Shear tests were made with a direct shear machine at a constant rate of strain. The machine is designed to test the soil without completely removing the samples from the brass rings. A range of normal stresses were applied vertically, and the shear strength was progressively determined at each load in order to determine the internal angle of friction and the cohesion. The results of direct shear tests are presented on Figure No. II-1 within this Appendix.

Consolidation

The apparatus used for the consolidation tests is designed to receive the undisturbed brass ring of soil as it comes from the field. Loads were applied to the test specimen in several increments, and the resulting deformations were recorded at selected time intervals. Porous stones were placed in contact with the top and bottom of the specimen to permit the ready addition or release of water.

Undisturbed specimens were tested at the field and added water conditions. The test results are shown on Figure No. II-2 within this Appendix.

SITE LOCATION

JOB NAME : Temescal Canyon Rd (APN: 2821-210-11), Corona, CA		JOB No.	22-449
GeoTech Consultants, Inc.		PLATE No.	1

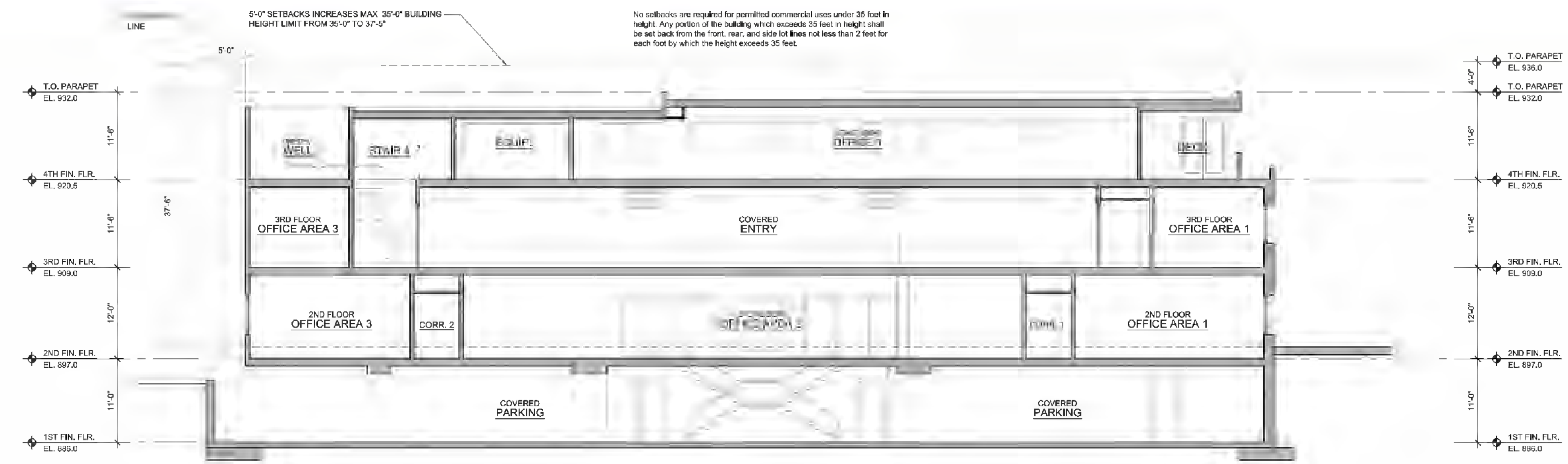
Scale: 1" = 30'

SITE PLAN

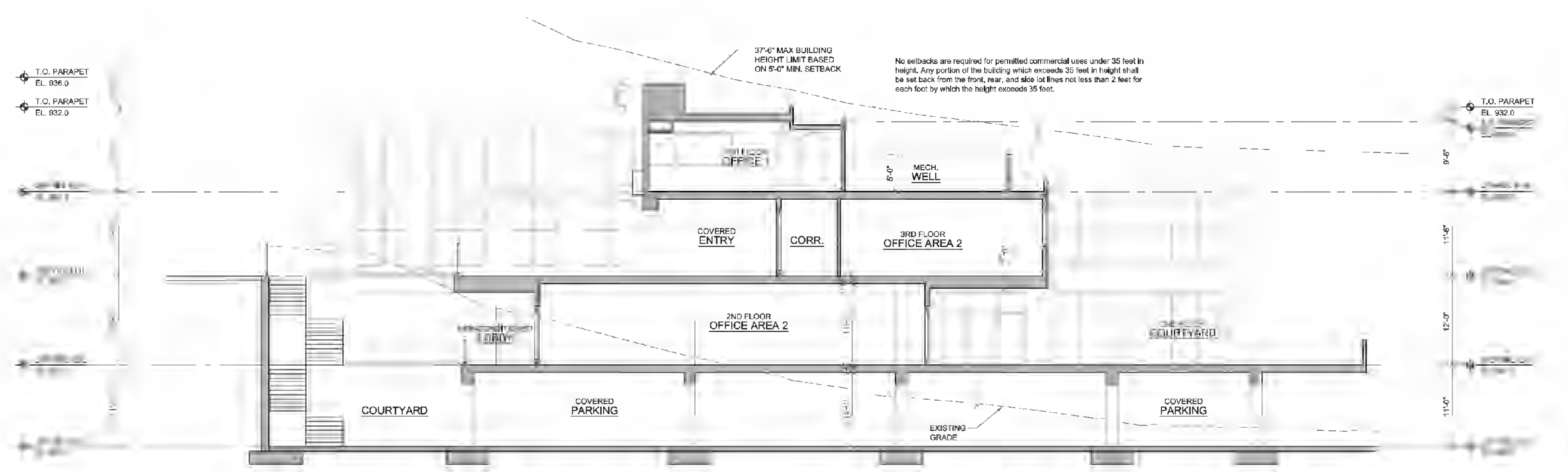
Proposed New 3-Story Office Building
Over Covered Parking on Vacant Land

Temescal Canyon Road. Corona, CA (APN: 2821-210-11)

FOR. Mr. John Soldat	DATE December 2022	PROJECT No. 22-449
GeoTech Consultants, Inc.		DRAWING No. 1



BUILDING SECTION - 2



BUILDING SECTION - 1

Scale: 1" = 20' (V=H)

CROSS SECTIONS

Proposed New 3-Story Office Building
Over Covered Parking on Vacant Land

Temescal Canyon Road. Corona, CA (APN: 2821-210-11)

FOR.	Mr. John Soldat	DATE	February 2023	PROJECT No.	22-449
GeoTech Consultants, Inc.				DRAWING No.	2

GTC
Geotech Consultants, Inc
1201 N. Pacific Ave Suite 201
Glendale, CA 91202
Telephone: (747) 215-6337
Email: Behnamgeotech@gmail.com

BORING NUMBER 1

CLIENT

PROJECT NUMBER 22-449

PROJECT LOCATION Temescal Canyon Road, Corona (APN 2821-210-11)

DATE 12/7/22

GROUND ELEVATION 895 ft **HOLE SIZE** 8 Inches

DRILLING CONTRACTOR Choice Drilling

GROUND WATER LEVELS:

DRILLING METHOD Hollow Stem Flight Auger

AT TIME OF DRILLING --- No Water

LOGGED BY Haybert Mahmoudi

AT END OF DRILLING --- No Water

CHECKED BY Behnam Mahmoudkhani

NOTES

DEPTH (ft)	SAMPLE TYPE NUMBER	BLOW COUNTS (N VALUE)	TESTS	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION
0	GB			Fill	1.0	SILTY SAND, (SM) dark brown, fine grained, dry to slightly moist, dense, with fine grained gravel, some roots, no odor (894.0)
				SM		SILTY SAND, (SM)
5	SPT	11-18-24 (42)	MC = 7% DD = 125 pcf Fines = 31%	SM	5.0	SILTY SAND, (SM) light brown, fine grained, moist, very dense, few coarse sand, few fine grained gravel, no odor (890.0)
10	SPT	10-20-31 (51)	MC = 12% DD = 109 pcf Fines = 71%	ML	10.0	CLAYEY SILT, (ML) brownish orange, moist, very stiff, with very fine grained sand, no odor (885.0)
15	SPT	35-50/3"	MC = 8% DD = 118 pcf Fines = 42%	SM	15.0	SILTY SAND, (SM) light tannish brown, fine grained, moist, very dense, few fine grained gravel, with clay, no odor (880.0)
20	SPT	14-38-46 (84)	MC = 11% DD = 120 pcf Fines = 39%	SM	20.0	SILTY SAND, (SM) dark brown, fine to coarse grained, moist, very dense, few fine grained gravel, no odor (875.0)
25	SPT	29-50/4"	MC = 12% DD = 116 pcf Fines = 46%	SM	25.0	SILTY SAND, (SM) light brown, very fine to fine grained, moist, very dense, with clay, no odor (870.0)

(Continued Next Page)

GTC

Geotech Consultants, Inc
1201 N. Pacific Ave Suite 201
Glendale, CA 91202
Telephone: (747) 215-6337
Email: Behnamgeotech@gmail.com

BORING NUMBER 1

CLIENT ______

PROJECT NUMBER 22-449

PROJECT LOCATION Temescal Canyon Road, Corona (APN 2821-210-11)

DEPTH (ft)	SAMPLE TYPE NUMBER	BLOW COUNTS (N VALUE)	TESTS	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION
				SM		SILTY SAND, (SM) light brown, very fine to fine grained, moist, very dense, with clay, no odor *(continued)*
30						30.0 — 865.0
	SPT	21-38-50/5"	MC = 13% DD = 105 pcf Fines = 82%	ML		CLAYEY SILT, (ML) greenish brown, moist, very stiff, with very fine grained sand, no odor
35						35.0 — 860.0
	SPT	33-50/5"	MC = 10% DD = 104 pcf Fines = 85%	ML		CLAYEY SILT, (ML) brownish green, moist, very stiff, with very fine grained sand, no odor
40						40.0 — 855.0
	SPT	28-34-40 (74)	MC = 13% DD = 104 pcf Fines = 86%	ML		CLAYEY SILT, (ML) No Change
45						45.0 — 850.0
	SPT	30-32-50/5"	MC = 10% DD = 105 pcf Fines = 80%	ML		CLAYEY SILT, (ML) No Change
50						50.0 — 845.0
	SPT	24-50/5"	MC = 12% DD = 103 pcf Fines = 79%	ML		CLAYEY SILT, (ML) No Change
						52.0 — 843.0

Bottom of borehole at 52.0 feet.

GTC Geotech Consultants, Inc
1201 N. Pacific Ave Suite 201
Glendale, CA 91202
Telephone: (747) 215-6337
Email: Behnamgeotech@gmail.com

BORING NUMBER 2

CLIENT ______

PROJECT NUMBER 22-449

PROJECT LOCATION Temescal Canyon Road, Corona (APN 2821-210-11)

DATE 12/7/22

GROUND ELEVATION 885 ft **HOLE SIZE** 8 Inches

DRILLING CONTRACTOR Choice Drilling

GROUND WATER LEVELS:

DRILLING METHOD Hollow Stem Flight Auger

AT TIME OF DRILLING --- No Water

LOGGED BY Haybert Mahmoudi

AT END OF DRILLING --- No Water

CHECKED BY Behnam Mahmoudkhani

NOTES ______

DEPTH (ft)	SAMPLE TYPE NUMBER	BLOW COUNTS (N VALUE)	TESTS	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	Elevation
0.0–2.0	GB			Fill		SILTY SAND, (SM) dark brown, fine grained, dry to slightly moist, dense, with fine grained gravel, some roots, no odor	883.0
2.0–5.0				ML		SANDY CLAYEY SILT, (ML)	880.0
5.0–10.0	MC	8-10-14 (24)	MC = 10% DD = 114 pcf	ML		SANDY CLAYEY SILT, (ML) dark brown, fine grained, moist, stiff, few coarse sand, few fine grained gravel, no odor	875.0
10.0–15.0	MC	29-50/5"	MC = 8% DD = 123 pcf	SM		SILTY SAND, (SM) brownish orange, fine grained, moist, very dense, few fine grained gravel, no odor	870.0
15.0–17.0	MC	25-40-50/4"	MC = 8% DD = 118 pcf	SM		SILTY SAND, (SM) brown, fine grained, moist, very dense, with clay, few fine grained gravel, no odor	868.0

Bottom of borehole at 17.0 feet.

GTC Geotech Consultants, Inc
1201 N. Pacific Ave Suite 201
Glendale, CA 91202
Telephone: (747) 215-6337
Email: Behnamgeotech@gmail.com

BORING NUMBER 3

CLIENT

PROJECT NUMBER 22-449

PROJECT LOCATION Temescal Canyon Road, Corona (APN 2821-210-11)

DATE 12/7/22

GROUND ELEVATION 915 ft **HOLE SIZE** 8 Inches

DRILLING CONTRACTOR Choice Drilling

GROUND WATER LEVELS:

DRILLING METHOD Hollow Stem Flight Auger

AT TIME OF DRILLING --- No Water

LOGGED BY Haybert Mahmoudi

AT END OF DRILLING --- No Water

CHECKED BY Behnam Mahmoudkhani

NOTES

DEPTH (ft)	SAMPLE TYPE NUMBER	BLOW COUNTS (N VALUE)	TESTS	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	Elevation
0.0	GB			Fill		SILTY SAND, (SM) dark brown, fine grained, dry to slightly moist, dense, with fine grained gravel, some roots, no odor	
3.0				SM		SILTY SAND, (SM)	912.0
5.0	MC	18-50/5"	MC = 5% DD = 115 pcf	SM		SILTY SAND, (SM) light brown, fine to coarse grained, slightly moist, very dense, with fine grained gravel, no odor	910.0
10.0	MC	34-50/5"	MC = 10% DD = 103 pcf	ML		CLAYEY SILT, (ML) light tannish brown, moist, very stiff, trace very fine grained sand, no odor	905.0
15.0	MC	27-50/5"	MC = 12% DD = 108 pcf	ML		SANDY CLAYEY SILT, (ML) light brown, very fine grained, moist, very stiff, no odor	900.0
17.0							898.0

Bottom of borehole at 17.0 feet.

GTC
Geotech Consultants, Inc
1201 N. Pacific Ave Suite 201
Glendale, CA 91202
Telephone: (747) 215-6337
Email: Behnamgeotech@gmail.com

BORING NUMBER 4

CLIENT

PROJECT NUMBER 22-449

PROJECT LOCATION Temescal Canyon Road, Corona (APN 2821-210-11)

DATE 12/7/22

GROUND ELEVATION 916 ft **HOLE SIZE** 8 Inches

DRILLING CONTRACTOR Choice Drilling

GROUND WATER LEVELS:

DRILLING METHOD Hollow Stem Flight Auger

AT TIME OF DRILLING --- No Water

LOGGED BY Haybert Mahmoudi

AT END OF DRILLING --- No Water

CHECKED BY Behnam Mahmoudkhani

NOTES

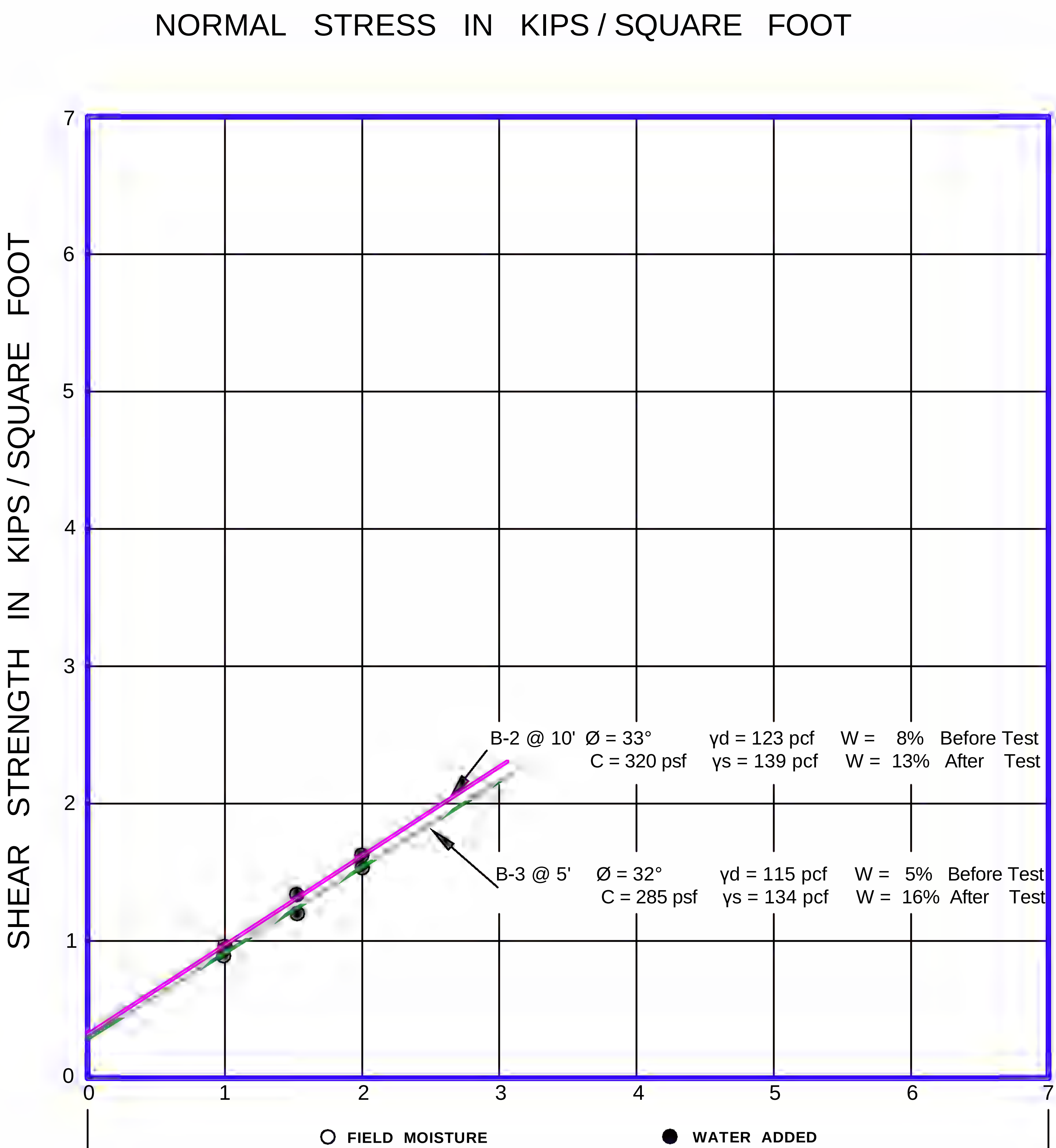
DEPTH (ft)	SAMPLE TYPE NUMBER	BLOW COUNTS (N VALUE)	TESTS	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION
0.0	GB			Fill		SILTY SAND, (SM) dark brown, fine grained, dry to slightly moist, dense, with fine grained gravel, some roots, no odor
2.5						
3.0				SM		SILTY SAND, (SM) (913.0)
5.0	MC	24-31-50/5"	MC = 9% DD = 116 pcf	SM		SILTY SAND, (SM) brown, fine grained, moist, very dense, with fine grained gravel, no odor (911.0)
7.5						
10.0	MC	37-50/5"	MC = 11% DD = 109 pcf	ML		CLAYEY SILT, (ML) light brown, moist, very stiff, trace very fine grained sand, no odor (906.0)
12.0						(904.0)

Bottom of borehole at 12.0 feet.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS		TYPICAL DESCRIPTIONS
			GRAPH	LETTER	
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS



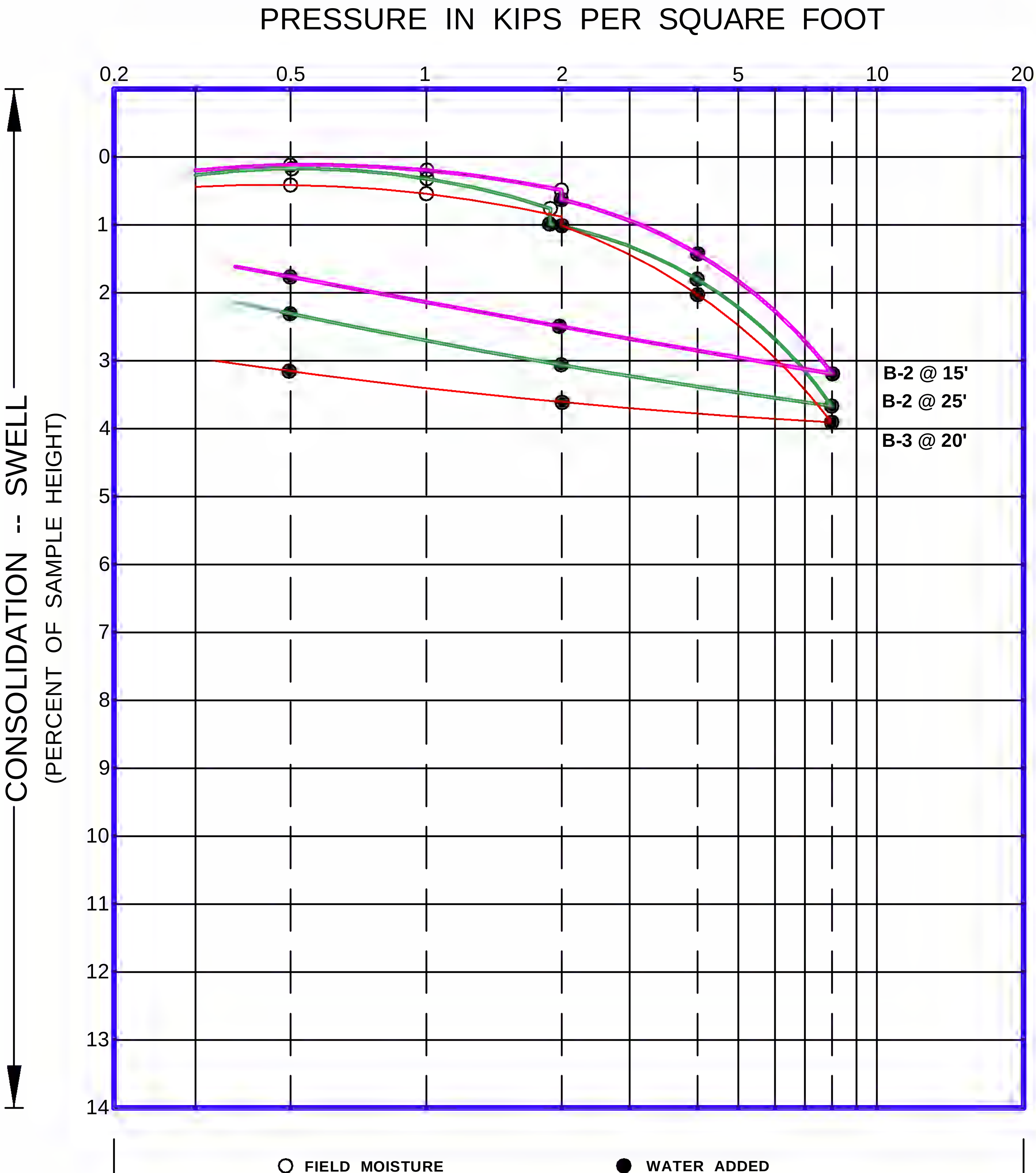
DIRECT SHEAR TESTS

JOB NAME: Temescal Canyon Road. Corona, CA (APN: 2821-210-11)

JOB No. 22-449

GeoTech Consultants, Inc.

FIGURE No. II - 1



SWELL - CONSOLIDATION TESTS

JOB NAME Temescal Canyon Road. Corona, CA (APN: 2821-210-11)

JOB No. 22-449

GeoTech Consultants, Inc.

FIGURE No. II - 2

Summary of Calculation

Bearing Capacity Calculations

Input

Soil Density	(γ)	121	pcf
Friction Angle	(φ)	32	
Cohesion	(c)	285	psf
Footing Width	(B)	1	ft
Footing Depth	(D)	2	ft
Effective Unit weight	(γ')		pcf
Factor of Safety	(FS)	4	

Continuous Footing

$q_{ult} = cNc + \gamma DNq + \gamma BN\gamma/2$

$q_{allow} = q_{ult}/FS$

$q_{allow} =$ 2400 psf

Square Footing

$q_{ult} = 1.3cNc + \gamma DNq + 0.4\gamma BN\gamma$

$q_{allow} = q_{ult}/FS$

$q_{allow} =$ 2600 psf

Increase per foot of Depth and Width

$q_{increase,}$ **depth** = 200 psf

Increase per foot of Width

$q_{increase,}$ **width** = 200 psf

$q\mathbf{max}_{allow} =$ 3800 psf

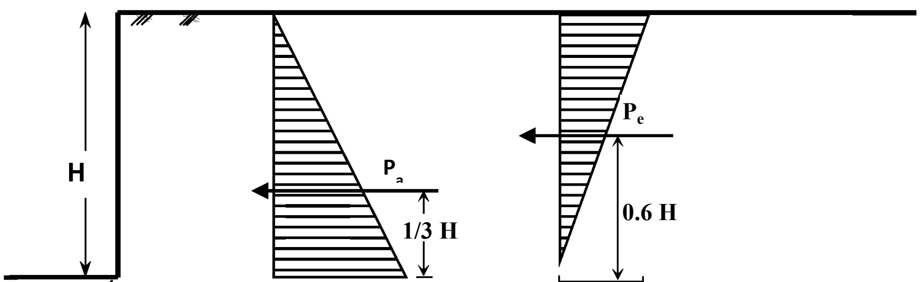
Job Name:	Temescal Canyon Road, Corona, CA.	Job No.	22-449
G.T.C		FIGURE No.	1

Table 1: NONRESTRAINED AND RESTRAINED DRAINED RETAINING WALLS

Wall Design Recommendations				
Retained Height & Back-slope Gradient (maximum)	Active Pressure Fluid Weight (pcf)	At-Rest Pressure Fluid Weight (pcf)	Restrained Design Earth Pressure (psf)*[1]	Seismically Induced Earth Pressure - Fluid Weight (pcf) *[2]
Up to 25 (ft.) Height & Level Back-Slope	$X_1 = 37$	-	$X_2 = 48 \times \mathbf{H}$	$Y = 13$

*[1] -Where H is the height of retained soil
*[2] - The seismically induced earth pressure should be applied as an inverted triangular pressure

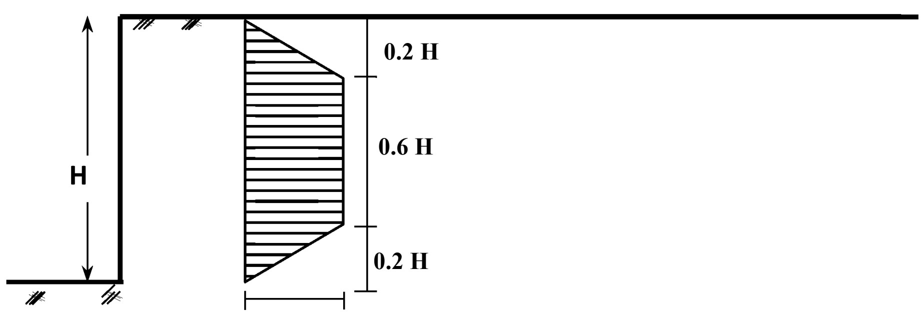
2. Cantilever Wall Design Based on Active Earth Pressure



X_1 pcf
Triangular Distribution of Active Earth Pressure

Y H (psf)
Triangular Inverse Distribution of Earth Pressure (Seismic)

1. Restrained Wall Design Based on At Rest Earth Pressure



X_2 H (psf)
Trapezoidal Distribution of Earth pressure

1. **Restrained Subterranean walls**, "walls for which horizontal movement is restricted at the top", shall be designed for an At-Rest lateral earth pressure (equivalent fluid weight) as illustrated in the above diagram of **Trapezoidal Distribution of Earth Pressure, X_2.H(psf).** **Our analysis of restrained and cantilevered retaining walls indicate that load combination of seismic plus static active is lower than the at-reat forces. Therefore, no additional loading due to seismic is required for restrained walls.**

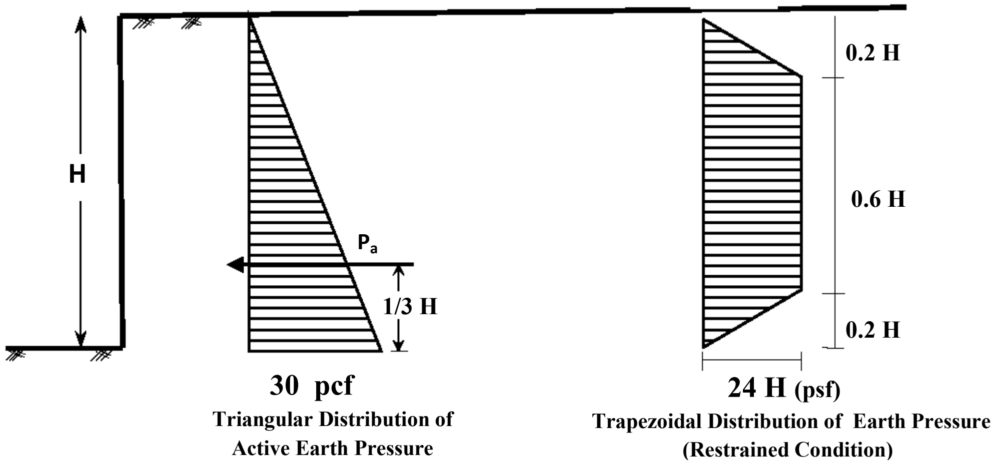
2. **Cantilevered retaining walls** higher than 6 feet shall be designed with the addition of seismic surcharge as illustrated on the above diagrams of Triangular Distribution of Active Earth Pressure and Triangular Inverse distribution of seismic pressure.

Table 2: Shoring Design

Shoring Lateral Pressures Recommendations		
Surface Slope of Retained Material Horizontal to Vertical	Static Equivalent Fluid Weight (pcf)	Restrained Condition Design Earth Pressure (psf)*
Level up to 25 ft.	X = 30	24

* -Where H is the retained height of the excavation soil

Shoring Design Based on Active Earth Pressure



Cantilevered soldier pile should be designed to resist an active earth pressure. The active earth pressure condition assumes that a triangular pressure distribution is utilized in the shoring design. If the soldier piles are not allowed to deflect, they shall be designed for the Restrained Condition. Soldier piles designed for the restrained condition should utilize a trapezoidal pressure distribution.

Earth pressure on structure analysis

Input data

Project

Task : Lateral Earth Pressure Permanent Condition (At-Rest) Up to 25 ft. Back-Slope
Descript. : Temescal Canyon Road
Author : Behnam M. Khani
Date : 2/27/2023

Settings

USA - Safety factor-GeoTech (Parameters Reduce) (2)

Excavations

Active earth pressure calculation : Mazindrani (Rankin)
Passive earth pressure calculation : Mazindrani (Rankin)
Earthquake analysis : Mononobe-Okabe
Shape of earth wedge : Calculate as skew
Verification methodology : Limit states (LSD)

Reduction coeff. of soil parameters Permanent design situation			
Reduction coeff. of internal friction :	$\gamma_{m\varphi}$ =	1.50	[–]
Reduction coeff. of cohesion :	γ_{mc} =	1.50	[–]
Reduction coeff. of Poisson's ratio :	$\gamma_{m\nu}$ =	1.00	[–]
Coefficient of unit weight behind construction :	$\gamma_{m\gamma}$ =	1.00	[–]
Coefficient of unit weight in front of constr. :	$\gamma_{m\gamma}$ =	1.00	[–]

Basic soil parameters

No.	Name	Pattern	φ_{ef} [°]	c_{ef} [psf]	γ [pcf]	γ_{su} [pcf]	δ [°]
1	Silty Sand		32.00	285.0	121.00	71.50	0.00

All soils are considered as cohesionless for at rest pressure analysis.

Soil parameters

Silty Sand
Unit weight : γ = 121.0 pcf
Stress-state : effective
Angle of internal friction : φ_{ef} = 32.00 °
Cohesion of soil : c_{ef} = 285.0 psf
Angle of friction struc.-soil : δ = 0.00 °
Soil : cohesionless
Saturated unit weight : γ_{sat} = 134.0 pcf

Geological profile and assigned soils

No.	Layer [ft]	Assigned soil	Pattern
1	30.00	Silty Sand	

No.	Layer [ft]	Assigned soil	Pattern
2	-	Silty Sand	

Terrain profile

Terrain behind the structure is flat.

Water influence

Ground water table is located below the structure.

Settings of the stage of construction

Design situation : permanent

Analysis No. 1

Pressure at rest behind the structure - partial results

Layer No.	Thickness [ft]	α [°]	φ_d [°]	c_d [psf]	γ [pcf]	K_r	Comment
1	25.00	0.00	21.33	190.0	121.00	0.636	

Pressure at rest distribution behind the structure (without surcharge)

Layer No.	Start [ft] End [ft]	σ_z [psf]	σ_w [psf]	Pressure [psf]	Hor. comp. [psf]	Vert. comp. [psf]
1	0.00	0.0	0.0	0.0	0.0	0.0
	25.00	3025.0	0.0	1924.5	1924.5	0.0

Overall pressure acting on the structure

Point No.	Depth [ft]	Hor. comp. [psf]	Vert. comp. [psf]
1	0.00	0.0	0.0
2	25.00	1924.5	0.0

Resultant forces

Total horizontal pressure acting on construction	**=**	**24056.57**	**lbf/ft**
Application point of horiz. comp. lies in depth	**=**	**16.67**	**ft**
Total vertical pressure acting on construction	**=**	**0.00**	**lbf/ft**
Dist. of vertical comp. from top of constr.	**=**	**0.00**	**ft**

Earth pressure on structure analysis

Input data

Project

Task : Lateral Earth Pressure Permanent Condition (Seismic) Up to 25 ft. Back-Slope
Descript. : Temescal Canyon Road
Author : Behnam M. Khani
Date : 2/27/2023

Settings

USA - Safety factor-GeoTech (Parameters Reduce) (2)

Excavations

Active earth pressure calculation : Mazindrani (Rankin)
Passive earth pressure calculation : Mazindrani (Rankin)
Earthquake analysis : Mononobe-Okabe
Shape of earth wedge : Calculate as skew
Verification methodology : Limit states (LSD)

Reduction coeff. of soil parameters Seismic design situation			
Reduction coeff. of internal friction :	$\gamma_{m\varphi}$ =	1.00	[–]
Reduction coeff. of cohesion :	γ_{mc} =	1.00	[–]
Reduction coeff. of Poisson's ratio :	$\gamma_{m\nu}$ =	1.00	[–]
Coefficient of unit weight behind construction :	$\gamma_{m\gamma}$ =	1.00	[–]
Coefficient of unit weight in front of constr. :	$\gamma_{m\gamma}$ =	1.00	[–]

Basic soil parameters

No.	Name	Pattern	φ_{ef} [°]	c_{ef} [psf]	γ [pcf]	γ_{su} [pcf]	δ [°]
1	Silty Sand		32.00	285.0	121.00	71.50	0.00

All soils are considered as cohesionless for at rest pressure analysis.

Soil parameters

Silty Sand
Unit weight : γ = 121.0 pcf
Stress-state : effective
Angle of internal friction : φ_{ef} = 32.00 °
Cohesion of soil : c_{ef} = 285.0 psf
Angle of friction struc.-soil : δ = 0.00 °
Soil : cohesionless
Saturated unit weight : γ_{sat} = 134.0 pcf

Geological profile and assigned soils

No.	Layer [ft]	Assigned soil	Pattern
1	30.00	Silty Sand	

No.	Layer [ft]	Assigned soil	Pattern
2	-	Silty Sand	

Terrain profile

Terrain behind the structure is flat.

Water influence

Ground water table is located below the structure.

Earthquake

Horizontal seismic coefficient k_h = 0.3500
Vertical seismic coefficient k_v = 0.0000
Coeff. to compute point of application k.H = 0.60
Water below the GWT is restricted.

Settings of the stage of construction

Design situation : seismic

Analysis No. 1

Active pressure behind the structure - partial results

Layer No.	Thickness [ft]	α [°]	φ_d [°]	c_d [psf]	γ [pcf]	δ_d [°]	K_a	Comment
1	8.50	0.00	32.00	285.0	121.00	0.00	0.000	
2	16.50	0.00	32.00	285.0	121.00	0.00	0.203	

Active pressure distribution behind the structure (without surcharge)

Layer No.	Start [ft] End [ft]	σ_z [psf]	σ_w [psf]	Pressure [psf]	Hor. comp. [psf]	Vert. comp. [psf]
1	0.00	0.0	0.0	0.0	0.0	0.0
	8.50	1028.3	0.0	0.0	0.0	0.0
2	8.50	1028.3	0.0	0.0	0.0	0.0
	25.00	3025.0	0.0	613.5	613.5	0.0

Earthquake effects (active earth pressure) - partial results

Layer No.	Thickness [ft]	φ_d [°]	β [°]	ψ [°]	K_a	K_{ae}	K_{ae}-K_a	Comment
1	8.50	32.00	0.00	19.29	0.307	0.585	0.278	
2	16.50	32.00	0.00	19.29	0.307	0.585	0.278	

Earthquake effects (active earth pressure)

Layer No.	Start [ft] End [ft]	σ_z [psf]	σ_D [psf]	Pressure [psf]	Hor. comp. [psf]	Vertical comp. [psf]
1	0.00	0.0	3025.0	839.7	839.7	0.0
	8.50	1028.3	1996.7	554.2	554.2	0.0
2	8.50	1028.3	1996.7	554.2	554.2	0.0
	25.00	3025.0	0.0	0.0	0.0	0.0

Overall pressure acting on the structure

Point No.	Depth [ft]	Hor. comp. [psf]	Vert. comp. [psf]
1	0.00	671.7	0.0
2	8.50	500.5	0.0
3	25.00	781.4	0.0

Resultant forces

Total horizontal pressure acting on construction = 15557.56 lbf/ft
Application point of horiz. comp. lies in depth = 13.09 ft
Total vertical pressure acting on construction = 0.00 lbf/ft
Dist. of vertical comp. from top of constr. = 0.00 ft

Earth pressure on structure analysis

Input data

Project

Task : Lateral Earth Pressure Permanent Condition (Active) Up to 25 ft. Back-Slope
Descript. : Temescal Canyon Road
Author : Behnam M. Khani
Date : 2/27/2023

Settings

USA - Safety factor-GeoTech (Parameters Reduce) (2)

Excavations

Active earth pressure calculation : Mazindrani (Rankin)
Passive earth pressure calculation : Mazindrani (Rankin)
Earthquake analysis : Mononobe-Okabe
Shape of earth wedge : Calculate as skew
Verification methodology : Limit states (LSD)

Reduction coeff. of soil parameters Permanent design situation			
Reduction coeff. of internal friction :	$\gamma_{m\varphi}$ =	1.50	[–]
Reduction coeff. of cohesion :	γ_{mc} =	1.50	[–]
Reduction coeff. of Poisson's ratio :	$\gamma_{m\nu}$ =	1.00	[–]
Coefficient of unit weight behind construction :	$\gamma_{m\gamma}$ =	1.00	[–]
Coefficient of unit weight in front of constr. :	$\gamma_{m\gamma}$ =	1.00	[–]

Basic soil parameters

No.	Name	Pattern	φ_{ef} [°]	c_{ef} [psf]	γ [pcf]	γ_{su} [pcf]	δ [°]
1	Silty Sand		32.00	285.0	121.00	71.50	0.00

All soils are considered as cohesionless for at rest pressure analysis.

Soil parameters

Silty Sand
Unit weight : γ = 121.0 pcf
Stress-state : effective
Angle of internal friction : φ_{ef} = 32.00 °
Cohesion of soil : c_{ef} = 285.0 psf
Angle of friction struc.-soil : δ = 0.00 °
Soil : cohesionless
Saturated unit weight : γ_{sat} = 134.0 pcf

Geological profile and assigned soils

No.	Layer [ft]	Assigned soil	Pattern
1	30.00	Silty Sand	

No.	Layer [ft]	Assigned soil	Pattern
2	-	Silty Sand	

Terrain profile

Terrain behind the structure is flat.

Water influence

Ground water table is located below the structure.

Settings of the stage of construction

Design situation : permanent

Analysis No. 1

Active pressure behind the structure - partial results

Layer No.	Thickness [ft]	α [°]	φ_d [°]	c_d [psf]	γ [pcf]	δ_d [°]	K_a	Comment
1	4.60	0.00	21.33	190.0	121.00	0.00	0.000	
2	20.40	0.00	21.33	190.0	121.00	0.00	0.381	

Active pressure distribution behind the structure (without surcharge)

Layer No.	Start [ft] / End [ft]	σ_z [psf]	σ_w [psf]	Pressure [psf]	Hor. comp. [psf]	Vert. comp. [psf]
1	0.00	0.0	0.0	0.0	0.0	0.0
	4.60	556.4	0.0	0.0	0.0	0.0
2	4.60	556.4	0.0	0.0	0.0	0.0
	25.00	3025.0	0.0	1151.6	1151.6	0.0

Overall pressure acting on the structure

Point No.	Depth [ft]	Hor. comp. [psf]	Vert. comp. [psf]
1	0.00	0.0	0.0
2	4.60	0.0	0.0
3	25.00	1151.6	0.0

Resultant forces

Total horizontal pressure acting on construction	**=**	**11747.59**	**lbf/ft**
Application point of horiz. comp. lies in depth	**=**	**18.20**	**ft**
Total vertical pressure acting on construction	**=**	**0.00**	**lbf/ft**
Dist. of vertical comp. from top of constr.	**=**	**0.00**	**ft**

Earth pressure on structure analysis

Input data

Project

Task : Lateral Earth Pressure Temporary Condition (Active) Up to 25 ft. Back-Slope
Descript. : Temescal Canyon Road
Author : Behnam M. Khani
Date : 2/27/2023

Settings

USA - Safety factor-GeoTech (Parameters Reduce) (2)

Excavations

Active earth pressure calculation : Mazindrani (Rankin)
Passive earth pressure calculation : Mazindrani (Rankin)
Earthquake analysis : Mononobe-Okabe
Shape of earth wedge : Calculate as skew
Verification methodology : Limit states (LSD)

Reduction coeff. of soil parameters Transient design situation			
Reduction coeff. of internal friction :	$\gamma_{m\varphi}$ =	1.25	[–]
Reduction coeff. of cohesion :	γ_{mc} =	1.25	[–]
Reduction coeff. of Poisson's ratio :	$\gamma_{m\nu}$ =	1.00	[–]
Coefficient of unit weight behind construction :	$\gamma_{m\gamma}$ =	1.00	[–]
Coefficient of unit weight in front of constr. :	$\gamma_{m\gamma}$ =	1.00	[–]

Basic soil parameters

No.	Name	Pattern	φ_{ef} [°]	c_{ef} [psf]	γ [pcf]	γ_{su} [pcf]	δ [°]
1	Silty Sand		32.00	285.0	121.00	71.50	0.00

All soils are considered as cohesionless for at rest pressure analysis.

Soil parameters

Silty Sand
Unit weight : γ = 121.0 pcf
Stress-state : effective
Angle of internal friction : φ_{ef} = 32.00 °
Cohesion of soil : c_{ef} = 285.0 psf
Angle of friction struc.-soil : δ = 0.00 °
Soil : cohesionless
Saturated unit weight : γ_{sat} = 134.0 pcf

Geological profile and assigned soils

No.	Layer [ft]	Assigned soil	Pattern
1	30.00	Silty Sand	

No.	Layer [ft]	Assigned soil	Pattern
2	-	Silty Sand	

Terrain profile

Terrain behind the structure is flat.

Water influence

Ground water table is located below the structure.

Settings of the stage of construction

Design situation : transient

Analysis No. 1

Active pressure behind the structure - partial results

Layer No.	Thickness [ft]	α [°]	φ_d [°]	c_d [psf]	γ [pcf]	δ_d [°]	K_a	Comment
1	5.98	0.00	25.60	228.0	121.00	0.00	0.000	
2	19.02	0.00	25.60	228.0	121.00	0.00	0.302	

Active pressure distribution behind the structure (without surcharge)

Layer No.	Start [ft] End [ft]	σ_z [psf]	σ_w [psf]	Pressure [psf]	Hor. comp. [psf]	Vert. comp. [psf]
1	0.00	0.0	0.0	0.0	0.0	0.0
	5.98	724.1	0.0	0.0	0.0	0.0
2	5.98	724.1	0.0	0.0	0.0	0.0
	25.00	3025.0	0.0	912.4	912.4	0.0

Overall pressure acting on the structure

Point No.	Depth [ft]	Hor. comp. [psf]	Vert. comp. [psf]
1	0.00	0.0	0.0
2	5.98	0.0	0.0
3	25.00	912.4	0.0

Resultant forces

Total horizontal pressure acting on construction = 8675.37 lbf/ft
Application point of horiz. comp. lies in depth = 18.66 ft
Total vertical pressure acting on construction = 0.00 lbf/ft
Dist. of vertical comp. from top of constr. = 0.00 ft

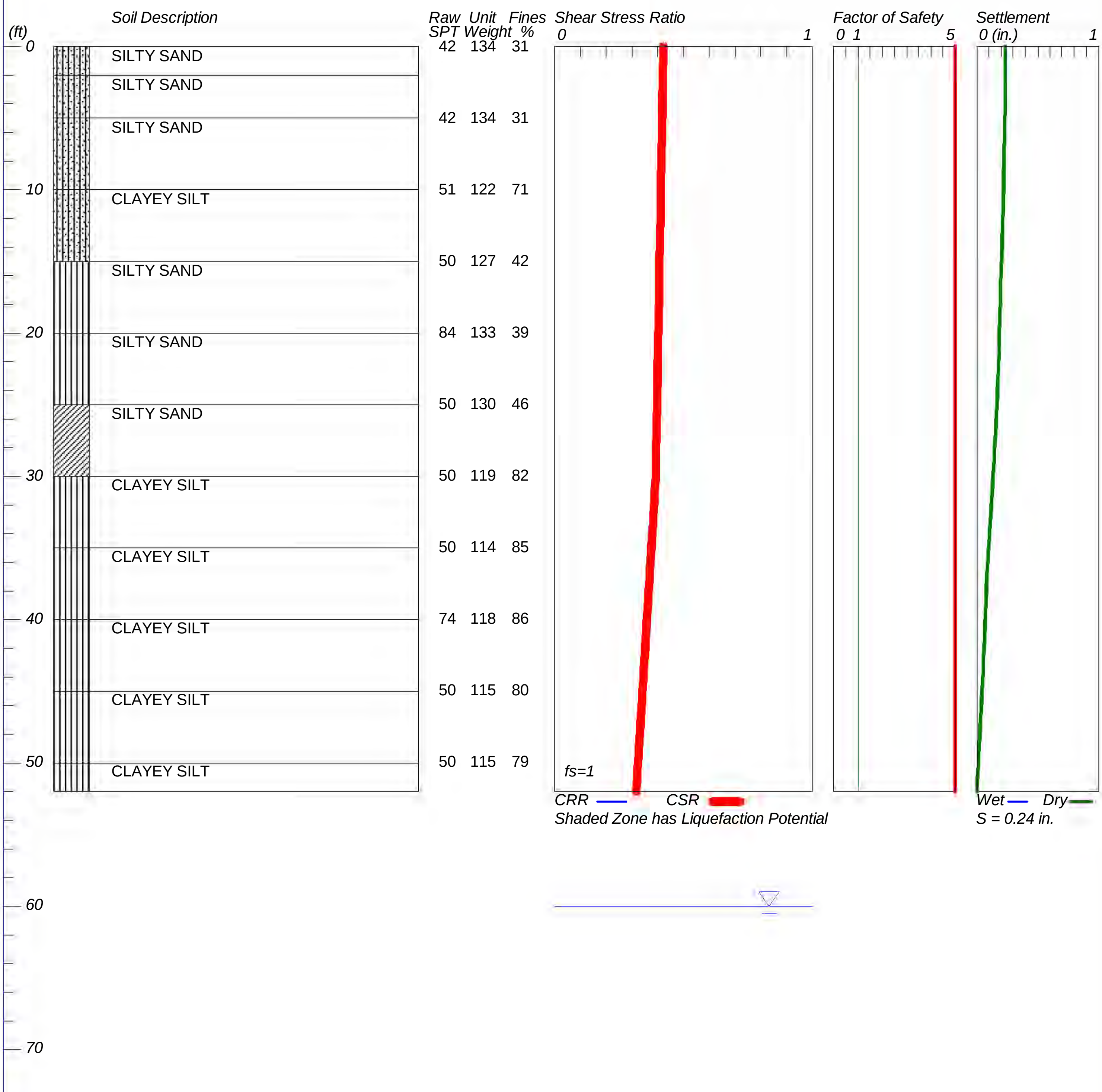
LIQUEFACTION EVALUATION

Temescal Canyon Rd. Los Angeles

Hole No.=B-1 Water Depth=60 ft Surface Elev.=895

Magnitude=6.7
Acceleration=0.65g

Soil Description	Raw SPT	Unit Weight	Fines %
SILTY SAND	42	134	31
SILTY SAND			
SILTY SAND	42	134	31
CLAYEY SILT	51	122	71
SILTY SAND	50	127	42
SILTY SAND	84	133	39
SILTY SAND	50	130	46
CLAYEY SILT	50	119	82
CLAYEY SILT	50	114	85
CLAYEY SILT	74	118	86
CLAYEY SILT	50	115	80
CLAYEY SILT	50	115	79



****

LIQUEFACTION ANALYSIS CALCULATION SHEET

****

Licensed to , GeoTech Consultants, Inc. 3/4/2023 4:02:22 PM

Input File Name: \\MYCLOUD-GPS9AC\Public\BEHNAM\2022\Temescal Canyon Rd (APN 2821-210-11)\PDF\New folder\Temescal Canyon Road.liq
Title: Temescal Canyon Rd. Los Angeles
Subtitle: GeoTech Consultants (Method Tokimatsu/Seed)

Input Data:
Surface Elev.=895
Hole No.=B-1
Depth of Hole=52.0 ft
Water Table during Earthquake= 60.0 ft
Water Table during In-Situ Testing= 60.0 ft
Max. Acceleration=0.65 g
Earthquake Magnitude=6.7

Earthquake Magnitude=6.7
2. Settlement Analysis Method: Tokimatsu / Seed
3. Fines Correction for Liquefaction: Idriss/Seed (SPT only)
4. Fine Correction for Settlement: During Liquefaction*
5. Settlement Calculation in: All zones*
6. Hammer Energy Ratio, Ce=1.2
7. Borehole Diameter, Cb=1
8. Sampeling Method, Cs=1
fs=1, Plot one CSR (fs=1)
10. Use Curve Smoothing: Yes*
* Recommended Options

In-Situ Test Data:

Depth ft	SPT	Gamma pcf	Fines %
0.0	42.0	134.0	31.0
5.0	42.0	134.0	31.0
10.0	51.0	122.0	71.0
15.0	50.0	127.0	42.0
20.0	84.0	133.0	39.0
25.0	50.0	130.0	46.0
30.0	50.0	119.0	82.0

35.0	50.0	114.0	85.0
40.0	74.0	118.0	86.0
45.0	50.0	115.0	80.0
50.0	50.0	115.0	79.0

Output Results:

Calculation segment, dz=0.050 ft

User defined Print Interval, dp=5.00 ft

CSR Calculation:

Depth ft	gamma pcf	sigma tsf	gamma' pcf	sigma' tsf	rd	CSR	fs (user)	CSRfs w/fs
0.00	134.0	0.000	134.0	0.000	1.00	0.42	1.0	0.42
5.00	134.0	0.335	134.0	0.335	0.99	0.42	1.0	0.42
10.00	122.0	0.655	122.0	0.655	0.98	0.41	1.0	0.41
15.00	127.0	0.966	127.0	0.966	0.97	0.41	1.0	0.41
20.00	133.0	1.291	133.0	1.291	0.95	0.40	1.0	0.40
25.00	130.0	1.620	130.0	1.620	0.94	0.40	1.0	0.40
30.00	119.0	1.931	119.0	1.931	0.93	0.39	1.0	0.39
35.00	114.0	2.223	114.0	2.223	0.89	0.38	1.0	0.38
40.00	118.0	2.513	118.0	2.513	0.85	0.36	1.0	0.36
45.00	115.0	2.804	115.0	2.804	0.81	0.34	1.0	0.34
50.00	115.0	3.092	115.0	3.092	0.77	0.32	1.0	0.32

CSR is based on water table at 60.0 during earthquake

CRR Calculation from SPT or BPT data:

Depth ft	SPT	Cebs	Cr	sigma' tsf	Cn	(N1)60	Fines %	d(N1)60	(N1)60f	CRR7.5
0.00	42.00	1.20	0.75	0.000	1.70	64.26	31.00	15.22	79.48	2.00
5.00	42.00	1.20	0.75	0.335	1.70	64.26	31.00	15.22	79.48	2.00
10.00	51.00	1.20	0.85	0.655	1.24	64.27	71.00	17.85	82.12	2.00
15.00	50.00	1.20	0.95	0.966	1.02	57.98	42.00	16.60	74.58	2.00
20.00	84.00	1.20	0.95	1.291	0.88	84.27	39.00	21.85	106.12	2.00
25.00	50.00	1.20	0.95	1.620	0.79	44.78	46.00	13.96	58.74	2.00
30.00	50.00	1.20	1.00	1.931	0.72	43.17	82.00	13.63	56.81	2.00
35.00	50.00	1.20	1.00	2.223	0.67	40.24	85.00	13.05	53.29	2.00
40.00	74.00	1.20	1.00	2.513	0.63	56.02	86.00	16.20	72.22	2.00
45.00	50.00	1.20	1.00	2.804	0.60	35.83	80.00	12.17	48.00	2.00
50.00	50.00	1.20	1.00	3.092	0.57	34.12	79.00	11.82	45.95	2.00

CRR is based on water table at 60.0 during In-Situ Testing

Factor of Safety, - Earthquake Magnitude= 6.7:

Depth ft	sigC' tsf	CRR7.5 tsf	Ksigma	CRRv	MSF	CRRm	CSRfs w/fs	F.S. CRRm/CSRfs
0.00	0.00	2.00	1.00	2.00	1.33	2.67	0.42	5.00
5.00	0.22	2.00	1.00	2.00	1.33	2.67	0.42	5.00
10.00	0.43	2.00	1.00	2.00	1.33	2.67	0.41	5.00
15.00	0.63	2.00	1.00	2.00	1.33	2.67	0.41	5.00
20.00	0.84	2.00	1.00	2.00	1.33	2.67	0.40	5.00
25.00	1.05	2.00	1.00	2.00	1.33	2.66	0.40	5.00
30.00	1.26	2.00	0.97	1.93	1.33	2.58	0.39	5.00
35.00	1.44	2.00	0.94	1.88	1.33	2.51	0.38	5.00
40.00	1.63	2.00	0.92	1.83	1.33	2.44	0.36	5.00
45.00	1.82	2.00	0.89	1.78	1.33	2.38	0.34	5.00
50.00	2.01	2.00	0.87	1.74	1.33	2.32	0.32	5.00

* F.S.<1: Liquefaction Potential Zone. (If above water table: F.S.=5)
(F.S. is limited to 5, CRR is limited to 2, CSR is limited to 2)

CPT convert to SPT for Settlement Analysis:
Fines Correction for Settlement Analysis:

Depth ft	Ic	qc/N60	qc1 tsf	(N1)60	Fines %	d(N1)60	(N1)60s
0.00	-	-	-	79.48	31.0	0.00	79.48
5.00	-	-	-	79.48	31.0	0.00	79.48
10.00	-	-	-	82.12	71.0	0.00	82.12
15.00	-	-	-	74.58	42.0	0.00	74.58
20.00	-	-	-	100.00	39.0	0.00	100.00
25.00	-	-	-	58.74	46.0	0.00	58.74
30.00	-	-	-	56.81	82.0	0.00	56.81
35.00	-	-	-	53.29	85.0	0.00	53.29
40.00	-	-	-	72.22	86.0	0.00	72.22
45.00	-	-	-	48.00	80.0	0.00	48.00
50.00	-	-	-	45.95	79.0	0.00	45.95

(N1)60s has been fines corrected in liquefaction analysis, therefore d(N1)60=0.
Fines=NoLiq means the soils are not liquefiable.

Settlement of Saturated Sands:
Settlement Analysis Method: Tokimatsu / Seed

Depth	CSRfs	F.S.	Fines	(N1)60s	Dr	ec	dsz	dsp	S
ft	w/fs		%		%	%	in.	in.	in.

Settlement of Saturated Sands=0.000 in.
qc1 and (N1)60 is after fines correction in liquefaction analysis
dsz is per each segment, dz=0.05 ft
dsp is per each print interval, dp=5.00 ft
S is cumulated settlement at this depth

Settlement of Dry Sands:

Depth	sigma'	sigC'	(N1)60s	CSRfs	Gmax	g*Ge/Gm	g_eff	ec7.5	Cec	ec	dsz	dsp	S
ft	tsf	tsf		w/fs	tsf			%		%	in.	in.	in.
51.95	3.20	2.08	45.23	0.32	2296.4	4.4E-4	0.0972	0.0307	0.84	0.0258	3.09E-4	0.000	0.000
50.00	3.09	2.01	45.95	0.32	2267.8	4.4E-4	0.0967	0.0306	0.84	0.0256	3.08E-4	0.012	0.012
45.00	2.80	1.82	48.00	0.34	2191.3	4.4E-4	0.0943	0.0298	0.84	0.0250	3.00E-4	0.030	0.043
40.00	2.51	1.63	72.22	0.36	2376.7	3.8E-4	0.0706	0.0223	0.84	0.0187	2.25E-4	0.026	0.069
35.00	2.22	1.44	53.29	0.38	2020.2	4.1E-4	0.1228	0.0388	0.84	0.0326	3.91E-4	0.028	0.096
30.00	1.93	1.26	56.81	0.39	1923.7	3.9E-4	0.1086	0.0343	0.84	0.0288	3.45E-4	0.037	0.133
25.00	1.62	1.05	58.74	0.40	1781.5	3.6E-4	0.0876	0.0277	0.84	0.0232	2.79E-4	0.031	0.165
20.00	1.29	0.84	100.00	0.40	1898.8	2.7E-4	0.0486	0.0154	0.84	0.0129	1.55E-4	0.020	0.185
15.00	0.97	0.63	74.58	0.41	1489.8	2.6E-4	0.0606	0.0192	0.84	0.0161	1.93E-4	0.018	0.202
10.00	0.66	0.43	82.12	0.41	1266.7	2.1E-4	0.0373	0.0118	0.84	0.0099	1.19E-4	0.016	0.218
5.00	0.34	0.22	79.48	0.42	895.9	1.6E-4	0.0279	0.0088	0.84	0.0074	8.89E-5	0.011	0.229
0.00	0.00	0.00	79.48	0.42	4.9	8.6E-7	0.0010	0.0003	0.84	0.0003	3.23E-6	0.006	0.235

Settlement of Dry Sands=0.235 in.
dsz is per each segment, dz=0.05 ft
dsp is per each print interval, dp=5.00 ft
S is cumulated settlement at this depth

Total Settlement of Saturated and Dry Sands=0.235 in.
Differential Settlement=0.118 to 0.155 in.

Units	Depth = ft, Stress or Pressure = tsf (atm), Unit Weight = pcf, Settlement = in.
SPT	Field data from Standard Penetration Test (SPT)
BPT	Field data from Becker Penetration Test (BPT)
qc	Field data from Cone Penetration Test (CPT)
fc	Friction from CPT testing
gamma	Total unit weight of soil
gamma'	Effective unit weight of soil
Fines	Fines content [%]
D50	Mean grain size

Dr	Relative Density
sigma	Total vertical stress [tsf]
sigma'	Effective vertical stress [tsf]
sigC'	Effective confining pressure [tsf]
rd	Stress reduction coefficient
CSR	Cyclic stress ratio induced by earthquake
fs	User request factor of safety, apply to CSR
w/fs	With user request factor of safety inside
CSRfs	CSR with User request factor of safety
CRR7.5	Cyclic resistance ratio (M=7.5)
Ksigma	Overburden stress correction factor for CRR7.5
CRRv	CRR after overburden stress correction, CRRv=CRR7.5 * Ksigma
MSF	Magnitude scaling factor for CRR (M=7.5)
CRRm	After magnitude scaling correction CRRm=CRRv * MSF
F.S.	Factor of Safety against liquefaction F.S.=CRRm/CSRfs
F.S*	User inputed Factor of Safety
Cebs	Energy Ratio, Borehole Dia., and Sample Method Corrections
Cr	Rod Length Corrections
Cn	Overburden Pressure Correction
(N1)60	SPT after corrections, (N1)60=SPT * Cr * Cn * Cebs
d(N1)60	Fines correction of SPT
(N1)60f	(N1)60 after fines corrections, (N1)60f=(N1)60 + d(N1)60
Cq	Overburden stress correction factor
qc1	CPT after Overburden stress correction
dqc1	Fines correction of CPT
qc1f	CPT after Fines and Overburden correction, qc1f=qc1 + dqc1
qc1n	CPT after normalization in Robertson's method
Kc	Fine correction factor in Robertson's Method
qc1f	CPT after Fines correction in Robertson's Method
Ic	Soil type index in Suzuki's and Robertson's Methods
(N1)60s	(N1)60 after seattlement fines corrections
ec	Volumetric strain for saturated sands
dz	Calculation segment, dz=0.050 ft
dsz	Settlement in each segment, dz
dp	User defined print interval
dsp	Settlement in each print interval, dp
Gmax	Shear Modulus at low strain
g_eff	gamma_eff, Effective shear Strain
g*Ge/Gm	gamma_eff * G_eff/G_max, Strain-modulus ratio
ec7.5	Volumetric Strain for magnitude=7.5
Cec	Magnitude correction factor for any magnitude
ec	Volumetric strain for dry sands, ec=Cec * ec7.5
NoLiq	No-Liquefy Soils

References:

NCEER Workshop on Evaluation of Liquefaction Resistance of Soils. Youd, T.L., and Idriss, I.M., eds., Technical Report NCEER 97-0022.

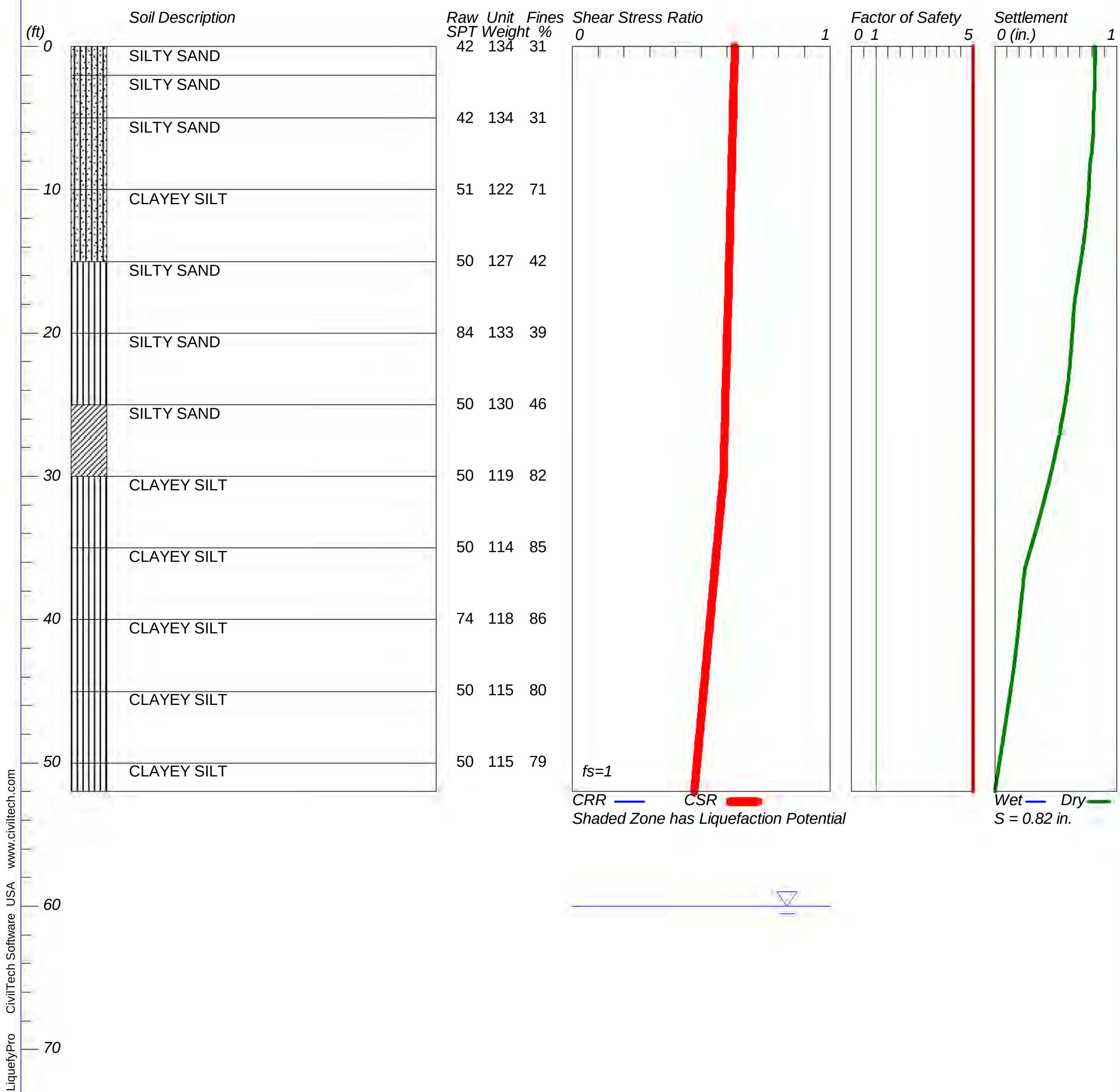
SP117. Southern California Earthquake Center. Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California. University of Southern California. March 1999.

LIQUEFACTION EVALUATION

Temescal Canyon Rd. Los Angeles

Hole No.=B-1 Water Depth=60 ft Surface Elev.=895

Magnitude=6.9
Acceleration=0.97g



****

LIQUEFACTION ANALYSIS CALCULATION SHEET

****

Licensed to , GeoTech Consultants, Inc. 3/4/2023 4:04:33 PM

Input File Name: \\MYCLOUD-GPS9AC\Public\BEHNAM\2022\Temescal Canyon Rd (APN 2821-210-11)\PDF\New folder\Temescal Canyon Road.liq
Title: Temescal Canyon Rd. Los Angeles
Subtitle: GeoTech Consultants (Method Tokimatsu/Seed)

Input Data:
Surface Elev.=895
Hole No.=B-1
Depth of Hole=52.0 ft
Water Table during Earthquake= 60.0 ft
Water Table during In-Situ Testing= 60.0 ft
Max. Acceleration=0.97 g
Earthquake Magnitude=6.9

Earthquake Magnitude=6.9
2. Settlement Analysis Method: Tokimatsu / Seed
3. Fines Correction for Liquefaction: Idriss/Seed (SPT only)
4. Fine Correction for Settlement: During Liquefaction*
5. Settlement Calculation in: All zones*
6. Hammer Energy Ratio, Ce=1.2
7. Borehole Diameter, Cb=1
8. Sampeling Method, Cs=1
fs=1, Plot one CSR (fs=1)
10. Use Curve Smoothing: Yes*
* Recommended Options

In-Situ Test Data:

Depth ft	SPT	Gamma pcf	Fines %
0.0	42.0	134.0	31.0
5.0	42.0	134.0	31.0
10.0	51.0	122.0	71.0
15.0	50.0	127.0	42.0
20.0	84.0	133.0	39.0
25.0	50.0	130.0	46.0
30.0	50.0	119.0	82.0

35.0	50.0	114.0	85.0
40.0	74.0	118.0	86.0
45.0	50.0	115.0	80.0
50.0	50.0	115.0	79.0

Output Results:

Calculation segment, dz=0.050 ft
User defined Print Interval, dp=5.00 ft

CSR Calculation:

Depth ft	gamma pcf	sigma tsf	gamma' pcf	sigma' tsf	rd	CSR	fs (user)	CSRfs w/fs
0.00	134.0	0.000	134.0	0.000	1.00	0.63	1.0	0.63
5.00	134.0	0.335	134.0	0.335	0.99	0.62	1.0	0.62
10.00	122.0	0.655	122.0	0.655	0.98	0.62	1.0	0.62
15.00	127.0	0.966	127.0	0.966	0.97	0.61	1.0	0.61
20.00	133.0	1.291	133.0	1.291	0.95	0.60	1.0	0.60
25.00	130.0	1.620	130.0	1.620	0.94	0.59	1.0	0.59
30.00	119.0	1.931	119.0	1.931	0.93	0.59	1.0	0.59
35.00	114.0	2.223	114.0	2.223	0.89	0.56	1.0	0.56
40.00	118.0	2.513	118.0	2.513	0.85	0.53	1.0	0.53
45.00	115.0	2.804	115.0	2.804	0.81	0.51	1.0	0.51
50.00	115.0	3.092	115.0	3.092	0.77	0.48	1.0	0.48

CSR is based on water table at 60.0 during earthquake

CRR Calculation from SPT or BPT data:

Depth ft	SPT	Cebs	Cr	sigma' tsf	Cn	(N1)60	Fines %	d(N1)60	(N1)60f	CRR7.5
0.00	42.00	1.20	0.75	0.000	1.70	64.26	31.00	15.22	79.48	2.00
5.00	42.00	1.20	0.75	0.335	1.70	64.26	31.00	15.22	79.48	2.00
10.00	51.00	1.20	0.85	0.655	1.24	64.27	71.00	17.85	82.12	2.00
15.00	50.00	1.20	0.95	0.966	1.02	57.98	42.00	16.60	74.58	2.00
20.00	84.00	1.20	0.95	1.291	0.88	84.27	39.00	21.85	106.12	2.00
25.00	50.00	1.20	0.95	1.620	0.79	44.78	46.00	13.96	58.74	2.00
30.00	50.00	1.20	1.00	1.931	0.72	43.17	82.00	13.63	56.81	2.00
35.00	50.00	1.20	1.00	2.223	0.67	40.24	85.00	13.05	53.29	2.00
40.00	74.00	1.20	1.00	2.513	0.63	56.02	86.00	16.20	72.22	2.00
45.00	50.00	1.20	1.00	2.804	0.60	35.83	80.00	12.17	48.00	2.00
50.00	50.00	1.20	1.00	3.092	0.57	34.12	79.00	11.82	45.95	2.00

CRR is based on water table at 60.0 during In-Situ Testing

Factor of Safety, - Earthquake Magnitude= 6.9:

Depth ft	sigC' tsf	CRR7.5 tsf	Ksigma	CRRv	MSF	CRRm	CSRfs w/fs	F.S. CRRm/CSRfs
0.00	0.00	2.00	1.00	2.00	1.24	2.48	0.63	5.00
5.00	0.22	2.00	1.00	2.00	1.24	2.48	0.62	5.00
10.00	0.43	2.00	1.00	2.00	1.24	2.48	0.62	5.00
15.00	0.63	2.00	1.00	2.00	1.24	2.48	0.61	5.00
20.00	0.84	2.00	1.00	2.00	1.24	2.48	0.60	5.00
25.00	1.05	2.00	1.00	2.00	1.24	2.47	0.59	5.00
30.00	1.26	2.00	0.97	1.93	1.24	2.39	0.59	5.00
35.00	1.44	2.00	0.94	1.88	1.24	2.33	0.56	5.00
40.00	1.63	2.00	0.92	1.83	1.24	2.27	0.53	5.00
45.00	1.82	2.00	0.89	1.78	1.24	2.21	0.51	5.00
50.00	2.01	2.00	0.87	1.74	1.24	2.15	0.48	5.00

* F.S.<1: Liquefaction Potential Zone. (If above water table: F.S.=5)
(F.S. is limited to 5, CRR is limited to 2, CSR is limited to 2)

CPT convert to SPT for Settlement Analysis:
Fines Correction for Settlement Analysis:

Depth ft	Ic	qc/N60	qc1 tsf	(N1)60	Fines %	d(N1)60	(N1)60s
0.00	-	-	-	79.48	31.0	0.00	79.48
5.00	-	-	-	79.48	31.0	0.00	79.48
10.00	-	-	-	82.12	71.0	0.00	82.12
15.00	-	-	-	74.58	42.0	0.00	74.58
20.00	-	-	-	100.00	39.0	0.00	100.00
25.00	-	-	-	58.74	46.0	0.00	58.74
30.00	-	-	-	56.81	82.0	0.00	56.81
35.00	-	-	-	53.29	85.0	0.00	53.29
40.00	-	-	-	72.22	86.0	0.00	72.22
45.00	-	-	-	48.00	80.0	0.00	48.00
50.00	-	-	-	45.95	79.0	0.00	45.95

(N1)60s has been fines corrected in liquefaction analysis, therefore d(N1)60=0.
Fines=NoLiq means the soils are not liquefiable.

Settlement of Saturated Sands:
Settlement Analysis Method: Tokimatsu / Seed

Depth ft	CSRfs w/fs	F.S.	Fines %	(N1)60s	Dr %	ec %	dsz in.	dsp in.	S in.

Settlement of Saturated Sands=0.000 in.
qc1 and (N1)60 is after fines correction in liquefaction analysis
dsz is per each segment, dz=0.05 ft
dsp is per each print interval, dp=5.00 ft
S is cumulated settlement at this depth

Settlement of Dry Sands:

Depth ft	sigma' tsf	sigC' tsf	(N1)60s	CSRfs w/fs	Gmax tsf	g*Ge/Gm	g_eff	ec7.5 %	Cec	ec %	dsz in.	dsp in.	S in.
51.95	3.20	2.08	45.23	0.47	2296.4	6.6E-4	0.2733	0.0864	0.90	0.0778	9.33E-4	0.001	0.001
50.00	3.09	2.01	45.95	0.48	2267.8	6.6E-4	0.2715	0.0859	0.90	0.0773	9.27E-4	0.036	0.037
45.00	2.80	1.82	48.00	0.51	2191.3	6.5E-4	0.2622	0.0829	0.90	0.0746	8.95E-4	0.091	0.129
40.00	2.51	1.63	72.22	0.53	2376.7	5.7E-4	0.1755	0.0555	0.90	0.0499	5.99E-4	0.073	0.201
35.00	2.22	1.44	53.29	0.56	2020.2	6.2E-4	0.5159	0.1632	0.90	0.1468	1.76E-3	0.095	0.296
30.00	1.93	1.26	56.81	0.59	1923.7	5.9E-4	0.4156	0.1314	0.90	0.1182	1.42E-3	0.159	0.455
25.00	1.62	1.05	58.74	0.59	1781.5	5.4E-4	0.2902	0.0918	0.90	0.0826	9.91E-4	0.120	0.575
20.00	1.29	0.84	100.00	0.60	1898.8	4.1E-4	0.1192	0.0377	0.90	0.0339	4.07E-4	0.061	0.637
15.00	0.97	0.63	74.58	0.61	1489.8	3.9E-4	0.2636	0.0833	0.90	0.0750	9.00E-4	0.069	0.706
10.00	0.66	0.43	82.12	0.62	1266.7	3.2E-4	0.1075	0.0340	0.90	0.0306	3.67E-4	0.065	0.770
5.00	0.34	0.22	79.48	0.62	895.9	2.3E-4	0.0553	0.0175	0.90	0.0157	1.89E-4	0.041	0.812
0.00	0.00	0.00	79.48	0.63	4.9	1.3E-6	0.0010	0.0003	0.90	0.0003	3.47E-6	0.011	0.823

Settlement of Dry Sands=0.823 in.
dsz is per each segment, dz=0.05 ft
dsp is per each print interval, dp=5.00 ft
S is cumulated settlement at this depth

Total Settlement of Saturated and Dry Sands=0.823 in.
Differential Settlement=0.411 to 0.543 in.

Units	Depth = ft, Stress or Pressure = tsf (atm), Unit Weight = pcf, Settlement = in.
SPT	Field data from Standard Penetration Test (SPT)
BPT	Field data from Becker Penetration Test (BPT)
qc	Field data from Cone Penetration Test (CPT)
fc	Friction from CPT testing
gamma	Total unit weight of soil
gamma'	Effective unit weight of soil
Fines	Fines content [%]
D50	Mean grain size

Dr	Relative Density
sigma	Total vertical stress [tsf]
sigma'	Effective vertical stress [tsf]
sigC'	Effective confining pressure [tsf]
rd	Stress reduction coefficient
CSR	Cyclic stress ratio induced by earthquake
fs	User request factor of safety, apply to CSR
w/fs	With user request factor of safety inside
CSRfs	CSR with User request factor of safety
CRR7.5	Cyclic resistance ratio (M=7.5)
Ksigma	Overburden stress correction factor for CRR7.5
CRRv	CRR after overburden stress correction, CRRv=CRR7.5 * Ksigma
MSF	Magnitude scaling factor for CRR (M=7.5)
CRRm	After magnitude scaling correction CRRm=CRRv * MSF
F.S.	Factor of Safety against liquefaction F.S.=CRRm/CSRfs
F.S*	User inputed Factor of Safety
Cebs	Energy Ratio, Borehole Dia., and Sample Method Corrections
Cr	Rod Length Corrections
Cn	Overburden Pressure Correction
(N1)60	SPT after corrections, (N1)60=SPT * Cr * Cn * Cebs
d(N1)60	Fines correction of SPT
(N1)60f	(N1)60 after fines corrections, (N1)60f=(N1)60 + d(N1)60
Cq	Overburden stress correction factor
qc1	CPT after Overburden stress correction
dqc1	Fines correction of CPT
qc1f	CPT after Fines and Overburden correction, qc1f=qc1 + dqc1
qc1n	CPT after normalization in Robertson's method
Kc	Fine correction factor in Robertson's Method
qc1f	CPT after Fines correction in Robertson's Method
Ic	Soil type index in Suzuki's and Robertson's Methods
(N1)60s	(N1)60 after seattlement fines corrections
ec	Volumetric strain for saturated sands
dz	Calculation segment, dz=0.050 ft
dsz	Settlement in each segment, dz
dp	User defined print interval
dsp	Settlement in each print interval, dp
Gmax	Shear Modulus at low strain
g_eff	gamma_eff, Effective shear Strain
g*Ge/Gm	gamma_eff * G_eff/G_max, Strain-modulus ratio
ec7.5	Volumetric Strain for magnitude=7.5
Cec	Magnitude correction factor for any magnitude
ec	Volumetric strain for dry sands, ec=Cec * ec7.5
NoLiq	No-Liquefy Soils

References:

NCEER Workshop on Evaluation of Liquefaction Resistance of Soils. Youd, T.L., and Idriss, I.M., eds., Technical Report NCEER 97-0022.

SP117. Southern California Earthquake Center. Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California. University of Southern California. March 1999.

GeoTech Services

Percolation Test

1. General Percolation Test Information

Diameter: 8 in

Date prepared and soaked: 1/23/2023

Is pre-soak required* ? YES

soak start time: 8:0000 AM

soak end time: 11:00 AM

2. Percolation Test Data

Test hole: **#5**

Starting time: 11:30:00 AM

Date reading taken: 1/23/2023

Soil texture description

Depth (ft)	Soil Texture
5	Sandy Clayey Silt
10	Silty Sand
15	Silty Sand

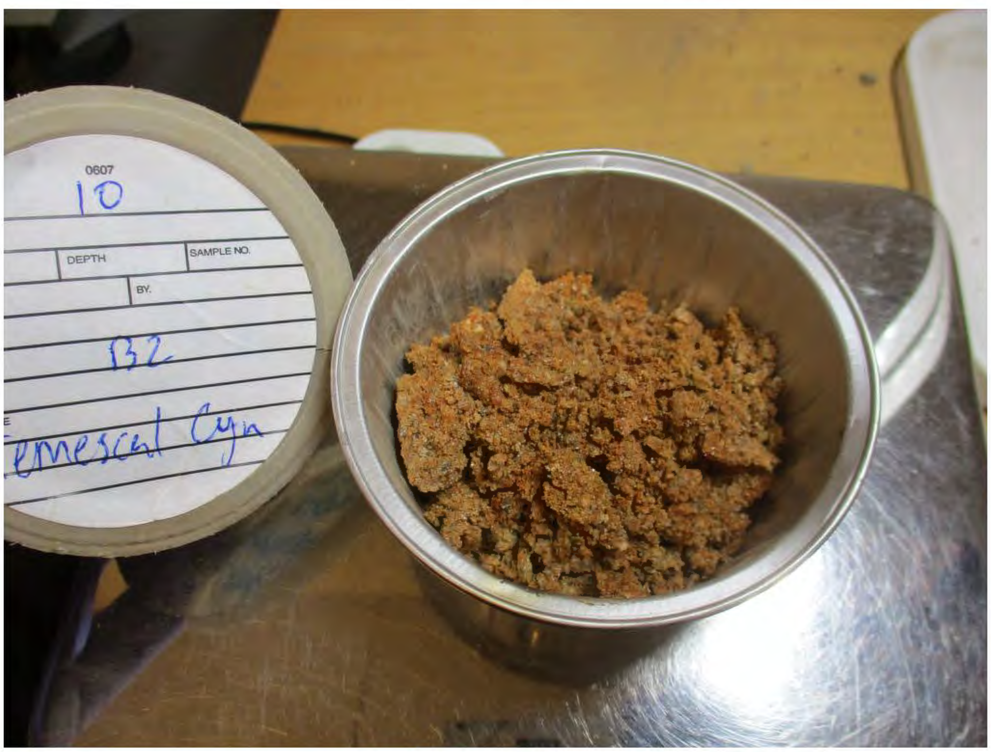
Reading	Start time	End Time	Start	End	Perc rate	%	Pass
1	11:00:00	11:10:00	100	99.4	16.67	N/A	N/A
2	11:10:00	11:20:00	99.4	98.82	17.24	N/A	N/A
3	11:20:00	11:30:00	98.82	98.31	19.61	15.0	NO
4	11:30:00	11:50:00	98.31	97.39	21.74	20.7	NO
7	11:50:00	12:10:00	97.39	96.5	22.22	0.0	YES

Percolation Rate for Test Hole #1

22.2 mpi (min/in)

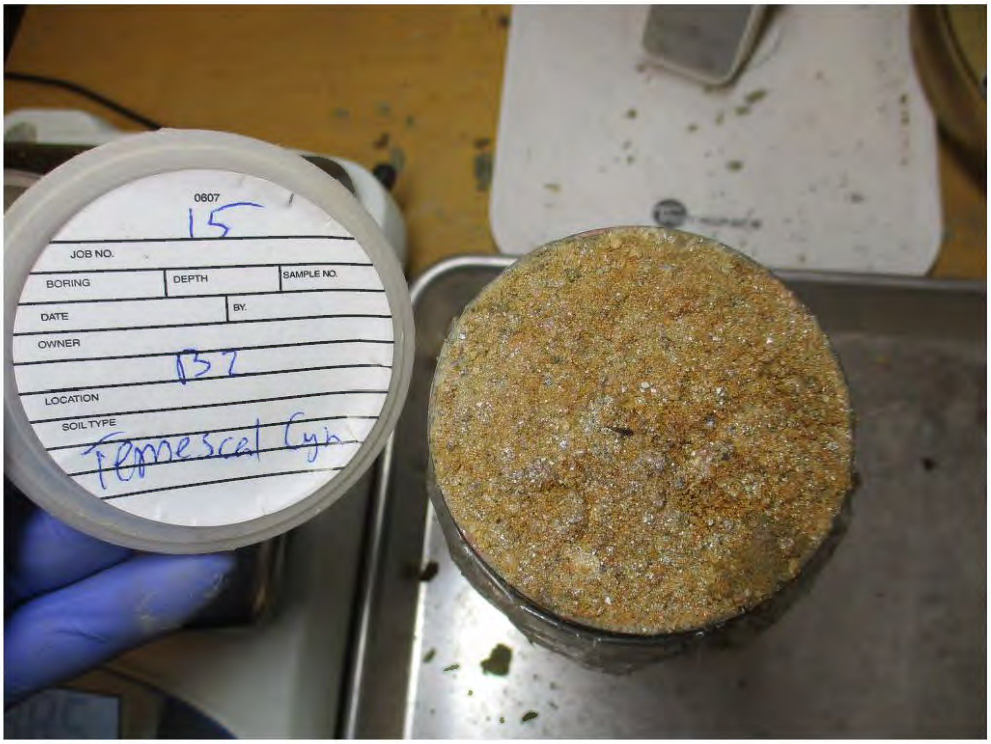
2.7 (in/hr)

JOB NAME :	Temescal Canyon Road, Corona, CA	**JOB NO.**	22-449
GeoTech Consultants, Inc.		**FIGURE NO.**	I



JOB NAME:	Temescal Canyon Road, Corona, CA	**Job No.** 22-449

GeoTech Consultants, Inc.



JOB NAME: Temescal Canyon Road, Corona, CA **Job No.** 22-449

GeoTech Consultants, Inc.