APPENDIX C – GEOTECHNICAL STUDIES

ENGEO "GEOTECHNICAL EXPLORATION UPDATE" (AUGUST 23, 2019)

PROPOSED CHICK-FIL-A RESTAURANT
NORTH LIVERMORE AVENUE
LIVERMORE, CALIFORNIA

GEOTECHNICAL EXPLORATION UPDATE

SUBMITTED TO
Mr. Mike Conn
MPVCA LLC
2420 Camino Ramon, Suite 215
San Ramon, CA 94583

PREPARED BY
ENGEO Incorporated

August 23, 2019

PROJECT NO.
14986.000.000

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GEOTECHNICAL
ENVIRONMENTAL
WATER RESOURCES
CONSTRUCTION SERVICES

Project No.
14986.000.000

August 23, 2019

Mr. Mike Conn
MPVCA LLC
2420 Camino Ramon, Suite 215
San Ramon, CA 94583

Subject: Proposed Chick-Fil-A Restaurant
North Livermore Avenue
Livermore, California

GEOTECHNICAL EXPLORATION UPDATE

Dear Mr. Conn:

ENGEO prepared this geotechnical exploration update to assume the role of Geotechnical Engineer-of-Record for the proposed Chick-Fil-A Restaurant project located at North Livermore Avenue in Livermore, California as outlined in our agreement dated February 4, 2019.

If you have any questions or comments regarding this report, please call and we will be glad to discuss them with you.

Sincerely,

ENGEO Incorporated

Alex Light

Andrew Firmin, GE

REGISTERED PROFESSIONAL ENGINEER
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STATE OF CALIFORNIA

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

ENGEO prepared this geotechnical report update to assume the role of Geotechnical Engineer-of-Record for the proposed Chick-Fil-A restaurant project on North Livermore Avenue in Livermore, California. We prepared this report update as outlined in our agreement dated February 4, 2019. MPVCA LLC authorized ENGEO to conduct the following scope of services:

- Review previous geotechnical reports for the project.
- Perform data analysis and develop conclusions and recommendations.
- Prepare this report update.

For our use, we received the following previous geotechnical reports for the project. These reports were the basis of our review.

1. Giles Engineering Associates; Geotechnical Engineering Exploration and Analysis, Livermore, California; April 24, 2017; Project No. 2G-1606012.

2. Giles Engineering Associates; Supplemental Geotechnical Engineering Exploration and Analysis, Livermore, California; May 3, 2018; Project No. 2G-1712002..

We also received a civil plan set prepared by Joseph C. Truxaw & Associates, dated May 14, 2018, that showed the proposed development.

This report was prepared for the exclusive use of our client and its consultants for design of this project. In the event that any changes are made in the character, design, or layout of the development, we must be contacted to review the conclusions and recommendations contained in this report to evaluate whether modifications are recommended. This document may not be reproduced in whole or in part by any means whatsoever, nor may it be quoted or excerpted without our express written consent.

1.2 PROJECT LOCATION AND DESCRIPTION

The subject property is located along North Livermore Avenue near the Interstate 580 Freeway,. The currently vacant property is bounded by North Livermore Avenue to the east, a commercial center to the south, and Arroyo Las Positas to the north and west. The creek is roughly 20 feet below adjacent terrain, with general bank gradients of approximately 1:1 (horizontal:vertical) or flatter. However, some portions of the bank are steeper, likely due to historic bank erosion. The attached Figure 1 and Figure 2 show a vicinity map and site plan, respectively.

We understand that the proposed development will consist of a one-story restaurant structure with associated drive lanes and parking areas, retaining walls, underground utilities, and stormwater quality features. The proposed restaurant building and associated improvements will be located as close as 10 to 15 feet from the existing top of creek bank. We anticipate site grading will include cuts and fills of up to approximately 2 feet.

1.3 SITE BACKGROUND

We reviewed readily available historic aerial photographs using www.historicaerials.com and Google Earth, which included various aerial photographs spanning from 1949 to 2018. We also reviewed in-house aerial photographs dated 1940 and 1965.

Based on our review, it appears that the majority of the site has remained relatively unchanged since 1940. Arroyo Las Positas, located adjacent to the northern boundary of the site, appears to be relatively stable during the years with available photos; a meander northwest of the site was realigned/straightened sometime between 1950 and 1958. This realignment appears to be within the incised channel and not due to erosion of the southern bank. Whether this adjustment was natural or due to grading activities is unknown; however, it does not appear to be related to the construction of Interstate 580 (then US 50). In the same general area, a small bridge or culvert was constructed across Arroyo Las Positas between 1960 and 1965. In conjunction with this structure, a small access roadway appears to have been cut into the south bank of Arroyo Las Positas, extending onto the site. This structure is no longer evident in the 1987 aerial photographs, and the roadway appears to have been filled in. Current topography shows a relatively gentle drainage swale that discharges into the creek in the general area.

In the southeastern portion of the property, a former residence is visible in the 1940 aerial photograph. The residence appears to include a main (larger) building at the eastern edge of the site, private access roadways, several ancillary structures throughout the residence, and trees and other landscaped areas. The residence appears to have largely been demolished between 1993 and 2002, although what appears to be a slab for the former structure at the eastern edge of the property is visible in the 2002 photographs. This feature was not observed in the 2003 photographs.

Additionally, we reviewed readily available historic topographic maps from 1906 through 2015, using www.historicaerials.com. The 1906 topographic map shows a former roadway in the vicinity of the site, connecting present day North Livermore Avenue and Portola Avenue. The former roadway appears to have been demolished in the vicinity of the site by the 1939 aerial photograph and the 1941 topographic map. Based on combined review of historic aerial photographs and topographic maps, it appears the former roadway did not cross the site; rather it likely connected to North Livermore Avenue to the south of the site.

2.0 FINDINGS

Giles Engineering Associates (Giles) previously performed a geotechnical exploration report for the subject project dated April 24, 2017. Giles performed a subsurface field exploration consisting of eight borings extending up to 51½ feet below existing grade, and presented a summary of site conditions, laboratory testing results, discussion of geologic and seismic hazards, and its conclusions and recommendations for the project in the report. This report is attached as Appendix A. Please refer to the attached report for additional information.

Giles performed additional subsurface field exploration to further evaluate slope stability as part of its referenced supplemental report dated May 3, 2018. Giles performed additional field exploration, including two borings extending up to 31½ feet below existing grade, and presented a summary of subsurface conditions, laboratory testing, and supplemental conclusions and recommendations for the project in the report. This report is attached as Appendix B. Please refer to the attached report for this information.

3.0 CONCLUSIONS

We reviewed the referenced previous geotechnical reports by Giles, and prepared updated conclusions and recommendations for the proposed development in the subsequent sections. These updated conclusions and recommendations should supersede Section 7 of the 2017 geotechnical exploration, and Section 5 of the 2018 supplemental geotechnical exploration. We are prepared to accept the role of Geotechnical Engineer-of-Record provided these recommendations are incorporated into project design and implemented during construction. From a geotechnical engineering viewpoint, in our opinion, the site is suitable for the proposed development.

3.1 SLOPE STABILITY AND CREEK BANK EROSION

In general, we concur with the slope stability methodology and analysis results presented in the previous geotechnical reports prepared by Giles. The analysis results identified acceptable factors of safety for both static and pseudo-static (seismic) conditions.

Based on recent project discussions, we understand the existing creek bank slope will not be cut back to create a maximum 1½:1 slope gradient as described in the Giles report dated May 3, 2018. Based on review of the civil plans, the proposed building and associated improvements will be located within the watercourse setback established by Alameda County Public Works Agency (2008). To address potential continued creek bank instability near the proposed building and improvements, as well as the county watercourse setback requirement, we recommend that a buried pier wall be constructed behind and parallel to portions of the existing creek bank. The buried pier wall will improve slope stability by providing additional lateral resistance through the buried concrete piers. We prepared a buried pier wall design dated May 8, 2019, including structural calculations, slope stability analysis, and construction drawings. We also prepared a detailed scour and geomorphology analysis of Arroyo Las Positas dated May 3, 2019, and revised August 23, 2019.

3.2 2016 CBC SEISMIC DESIGN PARAMETERS

The 2016 CBC utilizes design criteria set forth in the 2010 ASCE 7 Standard. Based on the subsurface conditions encountered, we characterized the site as Site Class C in accordance with the 2016 CBC. We provide the 2016 CBC seismic design parameters in Table 3.2-1 below, which include design spectral response acceleration parameters based on the mapped Risk-Targeted Maximum Considered Earthquake (MCER) spectral response acceleration parameters.

TABLE 3.2-1: 2016 CBC Seismic Design Parameters, Latitude: 37.699212 **Longitude:** -121.774189

PARAMETER	VALUE
Site Class	C
Mapped MCE_R Spectral Response Acceleration at Short Periods, S_S (g)	1.67
Mapped MCE_R Spectral Response Acceleration at 1-second Period, S_1 (g)	0.60
Site Coefficient, F_A	1.00
Site Coefficient, F_V	1.30
MCE_R Spectral Response Acceleration at Short Periods, S_{MS} (g)	1.67
MCE_R Spectral Response Acceleration at 1-second Period, S_{M1} (g)	0.78
Design Spectral Response Acceleration at Short Periods, S_{DS} (g)	1.12
Design Spectral Response Acceleration at 1-second Period, S_{D1} (g)	0.52

3.3 SOIL CORROSION POTENTIAL

As part of its previous studies, Giles obtained a representative soil sample and tested for determination of pH, resistivity, sulfate, and chloride. The results are summarized in the table below.

TABLE 3.3-1: Corrosivity Test Results

SAMPLE LOCATION	DEPTH	PH	RESISTIVITY (OHMS-CM)	CHLORIDE (MG/KG)	SULFATE (MG/KG)
B-1	1 to 5 feet	7.29	840	85	9

The 2016 CBC references the 2014 American Concrete Institute Manual, ACI 318-14, Section 19.3.1 for concrete durability requirements. ACI Table 19.3.1.1 provides the following exposure categories and classes, and Table 19.3.2.1 provides requirements for concrete in contact with soil based upon the exposure class.

TABLE 3.3-2: ACI Table 19.3.1.1: Exposure Categories and Classes

CATEGORY	SEVERITY	CLASS	CONDITION	
F Freezing and thawing	Not Applicable	F0	Concrete not exposed to freezing-and-thawing cycles	
	Moderate	F1	Concrete exposed to freezing-and-thawing cycles and occasional exposure to moisture	
	Severe	F2	Concrete exposed to freezing-and-thawing cycles and in continuous contact with moisture	
	Very Severe	F3	Concrete exposed to freezing-and-thawing cycles and in continuous contact with moisture and exposed to deicing chemicals	
			WATER- SOLUBLE SULFATE IN SOIL % BY WEIGHT*	**DISSOLVED SULFATE IN WATER MG/KG (PPM)****
S Sulfate	Not applicable	S0	$SO_4 < 0.10$	$SO_4 < 150$
	Moderate	S1	$0.10 \leq SO_4 < 0.20$	$150 \leq SO_4 \leq 1,500$ seawater
	Severe	S2	$0.20 \leq SO_4 \leq 2.00$	$1,500 \leq SO_4 \leq 10,000$
	Very severe	S3	$SO_4 > 2.00$	$SO_4 > 10,000$
			CONDITION	
P Requiring low permeability	Not applicable	P0	In contact with water where low permeability is not required.	
	Required	P1	In contact with water where low permeability is required.	
C Corrosion protection of reinforcement	Not applicable	C0	Concrete dry or protected from moisture	
	Moderate	C1	Concrete exposed to moisture but not to external sources of chlorides	
	Severe	C2	Concrete exposed to moisture and an external source of chlorides from deicing chemicals, salt, brackish water, seawater, or spray from these sources	

* Percent sulfate by mass in soil determined by ASTM C1580
**Concentration of dissolved sulfates in water in ppm determined by ASTM D516 or ASTM D4130

In accordance with the criteria presented in the above table, these soils are within the F0 freeze-thaw class, S0 sulfate exposure class, P0 exposure class and C1 corrosion class. Cement type, water-cement ratio, and concrete strength, are not specified for these ranges.

Considering a ‘Not Applicable’ sulfate exposure, there is no requirement for cement type or water-cement ratio; however, a minimum concrete compressive strength of 2,500 psi is specified by the building code. For this sulfate range, we recommend Type II cement and a concrete mix design for foundations and building slabs-on-grade that incorporates a maximum water-cement ratio of 0.50. It should be noted, however, that the structural engineering design requirements for concrete may result in more stringent concrete specifications.

Based on the resistivity measurements, the soils are considered corrosive to buried metal piping. Values tested for chloride do not pose a significant impact to metals or concrete.

If desired to investigate this further, we recommend a corrosion consultant be retained to evaluate if specific corrosion recommendations are advised for the project.

4.0 CONSTRUCTION MONITORING

As the project progresses, we recommend that we perform the following additional services:

1. Review the final grading and foundation plans and specifications prior to construction to evaluate whether our recommendations have been implemented, and to provide additional or modified recommendations, as needed. This will also allow us to determine whether any changes have occurred in the nature, design or location of the proposed improvements and will provide the opportunity to prepare a written response with updated recommendations.

2. Perform construction monitoring to check the validity of the assumptions we made to prepare this report. Earthwork operations should be performed under the observation of our representatives to confirm that the site is properly prepared, the selected fill materials are satisfactory, and that placement and compaction of the fills has been performed in accordance with our recommendations and the project specifications. Sufficient notification to us prior to earthwork is important.

If we are not retained to perform the services described above, then we are not responsible for any party’s interpretation of our report (and subsequent addenda, letters, and discussions).

5.0 EARTHWORK RECOMMENDATIONS

As used in this report, relative compaction refers to the in-place dry unit weight of soil expressed as a percentage of the maximum dry unit weight of the same soil, as determined by the ASTM D1557 laboratory compaction test procedure, latest edition. Compacted soil is not acceptable if it is unstable; it should exhibit only minimal flexing or pumping, as observed by an ENGEO representative. The term “moisture condition” refers to adjusting the moisture content of the soil by either drying if too wet or adding water if too dry.

We define “structural area” as any area sensitive to settlement of compacted soil. These areas include, but are not limited to building pads, sidewalks, pavement areas, and retaining walls.

5.1 GENERAL SITE CLEARING

Areas to be developed should be cleared of surface and subsurface deleterious materials, including existing building foundations and subsurface basements/root cellars, slabs, buried utility and irrigation lines, pavements, debris, and designated trees, shrubs, and associated roots.

Septic tanks, including all drain fields and other lines, if encountered, should be totally removed. Existing wells (if any) should be permitted for well destruction and properly destroyed in accordance with county and/or water agency guidelines.

Excavations extending below the planned finished site grades should be cleaned and backfilled with suitable material compacted to the recommendations presented below. ENGEO should be notified to observe and test backfilling.

Following clearing, the site should be stripped to remove surface organic materials from the ground surface to a depth of at least 2 to 3 inches below the surface. Strippings should be removed from the site or, if considered suitable by the landscape architect and owner, they may be used in landscape fill.

It may also be feasible to mulch organics in place, depending on the amount and type of vegetation present at the time of grading as well as the proposed mulching method. If desired, ENGEO can evaluate site vegetation at the time of grading to assess the feasibility of mulching organics in place.

5.2 EXISTING FILL AND SOFT OR LOOSE SOIL REMOVAL

The previous geotechnical reports did not identify existing fill at boring locations; however, we anticipate some existing fill and soft or loose soils may be present based on the history of the site. We recommend removing existing fill and soft or loose soils to competent native soil, as determined by ENGEO during grading.

5.3 EXPANSIVE SOIL MITIGATION

To reduce the risk of structural damage associated with the variably expansive soil conditions, we recommend constructing the upper 18 inches of the building pad with low-expansive fill. As an alternative to importing low-expansive fill for grading the building pad, it may be cost effective to lime treat the upper 18 inches of the finished building pad and to 10 feet laterally beyond.

Additional recommendations are presented in Section 6.

5.4 OVER-OPTIMUM SOIL MOISTURE CONDITIONS

The contractor should anticipate encountering excessively over-optimum (wet) soil moisture conditions during winter or spring grading, or during or following periods of rain. In addition, based on review of the previous borings and associated laboratory testing, wet soil conditions may be encountered at or near existing grade at some locations at the site. Wet soil can make proper compaction difficult or impossible.

Wet soil conditions can be mitigated by:

1. Frequent spreading and mixing during warm dry weather.
2. Mixing with drier materials.
3. Mixing with a lime, lime-flyash, or cement product; or
4. Stabilizing with aggregate, geotextile stabilization fabric, or both.

Options 3 and 4 should be evaluated by ENGEO prior to implementation.

5.5 ACCEPTABLE FILL

Onsite soil and rock material is suitable as fill provided it is processed to remove concentrations of organic material, debris, and particles greater than 8 inches in maximum dimension.

Imported fill materials should meet the above requirements and have a plasticity index (PI) less than 12, with at least 20 percent passing the No. 200 sieve. ENGEO should be contacted to sample and test proposed imported fill materials at least 5 days prior to delivery to the site.

5.6 FILL COMPACTION

5.6.1 Grading in Structural Areas

Native subgrade soils should be compacted prior to fill placement, following cutting operations, and in areas left at grade as follows.

1. Scarify to a depth of at least 8 inches.
2. Moisture condition soil to at least 4 percentage points over the optimum moisture content; **and**
3. Compact the soil to between 87 and 92 percent relative compaction. Compact the upper 6 inches of finish pavement subgrade to at least 90 percent relative compaction prior to aggregate base placement.

After the subgrade has been compacted, acceptable fill should be placed and compacted as follows:

1. Spread fill in loose lifts that do not exceed 8 inches.
2. Moisture condition lifts to at least 4 percentage points over the optimum moisture content; **and**
3. Compact fill to between 87 and 92 percent relative compaction; compact the upper 6 inches of fill in pavement areas to at least 90 percent relative compaction prior to aggregate base placement.

Where low-expansive fill (PI less than 12) material is used, the contractor should place and compact as follows:

1. Spread fill in loose lifts that do not exceed 8 inches.
2. Moisture condition soil to at least 2 percentage points over the optimum moisture content; **and**
3. Compact the soil to at least 90 percent relative compaction. Compact the upper 6-inches of building and finish pavement subgrade to at least 95 percent relative compaction.

Where lime treatment of the soil is used to mitigate expansive soil conditions, we recommend uniformly mixing the subgrade soil with 4 percent high-calcium lime by dry weight. The soil should be moisture conditioned to at least 3 percentage points above the optimum moisture content before mixing. The mixing should be performed in accordance with the current version of Caltrans Standard Specifications with the following exceptions:

1. Following mixing, the treated soils should be allowed to fully hydrate prior to compaction.

2. Following hydration, the treated soil should be compacted according to ASTM D-1557 to not less than 95 percent relative compaction at a moisture content at least 2 percentage points above the optimum to a non-yielding surface.

The pavement Caltrans Class 2 Aggregate Base section should be compacted to at least 95 percent relative compaction (ASTM D1557). Aggregate base should be compacted to or slightly above the optimum moisture content prior to compaction.

5.6.2 Underground Utility Backfill

5.6.2.1 General

The contractor is responsible for conducting trenching and shoring in accordance with CALOSHA requirements. Project consultants involved in utility design should specify pipe-bedding materials.

5.6.2.2 Structural Areas

Trench backfill should have a maximum particle size of 6 inches, should be placed in loose lifts not exceeding 12 inches, and should be moisture conditioned and compacted as follows:

1. For low-expansive fill (PI < 12), moisture condition trench backfill to or slightly above the optimum moisture content. For general fill, moisture condition trench backfill to 2 to 4 percent above the optimum moisture content. Moisture condition backfill outside the trench.
   and

2. For low-expansive fill (PI < 12), compact fill to a minimum of 90 percent relative compaction. For general fill, compact fill to between 87 and 92 percent relative compaction (90 percent minimum relative compaction at depths of 3 feet or more below finish grades)..

Where utility trenches cross perimeter building foundations, backfilling should be with native clay soil for pipe bedding and backfill for a distance of 2 feet on each side of the foundation. This will help prevent the normally granular bedding materials from acting as a conduit for water to enter beneath the building. As an alternative, a sand cement slurry (minimum 28-day compressive strength of 500 psi) may be used in place of native clay soil.

Jetting of backfill is not an acceptable means of compaction. We may allow thicker loose lift thicknesses based on acceptable density test results, where increased effort is applied to rocky fill, or for the first lift of fill over pipe bedding.

5.6.3 Landscape Fill

Fill should be processed, placed, and compacted in accordance with above sections, except that minimum compaction is 85 percent (ASTM D1557).

5.7 SLOPES

Final slope gradients should be constructed to 2:1 (horizontal:vertical) or flatter. The contractor is responsible to construct temporary construction slopes in accordance with CALOSHA requirements.

5.8 SITE DRAINAGE

5.8.1 Surface Drainage

The project civil engineer is responsible for designing surface drainage improvements. With regard to geotechnical engineering issues, we recommend that finish grades be sloped away from buildings and pavements to the maximum extent practical. The latest California Building Code Section 1804.4 specifies minimum slopes of 5 percent away from foundations. Where development conditions restrict meeting this slope requirement, we recommend that specific drainage requirements be developed. As a minimum, we recommend the following:

1. Discharge roof downspouts into closed conduits and direct away from foundations to appropriate drainage devices.

2. Do not allow water to pond near foundations, pavements, or exterior flatwork.

5.8.2 Subsurface Drainage

Based on our review and current grading concepts for the site, we do not anticipate that subdrainage systems will be recommended. We recommend that we review the site grading plans to further evaluate the need for subdrainage systems and observe the earthwork operations during site grading.

5.9 STORMWATER INFILTRATION

Due to the density of the site soils and fines content (percentage passing the No. 200 sieve), the near-surface site soils are expected to have a very low permeability value for stormwater infiltration in grassy swales or permeable pavers, unless subdrains are installed. Therefore, Best Management Practices should assume that little to no stormwater infiltration will occur at the site.

5.10 STORMWATER BIORETENTION AREAS

If bioretention areas are implemented, we recommend that, when practical, they be planned a minimum of 5 feet away from structural site improvements, such as buildings, streets, retaining walls, and sidewalks/driveways. When this is not practical, bioretention areas located within 5 feet of structural site improvements can either:

1. Be constructed with structural side walls capable of withstanding the loads from the adjacent improvements, or

2. Incorporate filter material compacted to between 85 and 90 percent relative compaction (ASTM D1557, latest edition) and a waterproofing system designed to reduce the potential for moisture transmission into the subgrade soil beneath the adjacent improvement.

In addition, one of the following options should be followed.

1. We recommend that bioretention design incorporate a waterproofing system lining the bioswale excavation and a subdrain, or other storm drain system, to collect and convey water to an approved outlet. The waterproofing system should cover the bioretention area excavation in such a manner as to reduce the potential for moisture transmission beneath the adjacent improvements.

2. Alternatively, and with some risk of movement of adjacent improvements, if infiltration is desired, we recommend the perimeter of the bioretention areas be lined with an HDPE tree root barrier that extends at least 1 foot below the bottom of the bioretention areas/infiltration trenches.

Site improvements located adjacent to bioretention areas that are underlain by base rock, sand, or other imported granular materials, should be designed with a deepened edge that extends to the bottom of the imported material underlying the improvement.

Where adjacent site improvements include buildings greater than three stories, streets steeper than 3 percent, or design elements subject to lateral loads (such as from impact or traffic patterns), additional design considerations may be recommended. If the surface of the bioretention area is depressed, the slope gradient should follow the slope guidelines described in earlier section(s) of this document. In addition, although not recommended, if trees are to be planted within bioretention areas, HDPE Tree Boxes that extend below the bottom of the bioretention system should be installed to reduce potential impact to subdrain systems that may be part of the bioretention area design. For this condition, the waterproofing system should be connected to the HPDE Tree Box with a waterproof seal.

Given the nature of bioretention systems and possible proximity to improvements, we recommend ENGEO be retained to review design plans and provide testing and observation services during the installation of linings, compaction of the filter material, and connection of designed drains.

It should be noted that the contractor is responsible for conducting all excavation and shoring in a manner that does not cause damage to adjacent improvements during construction and future maintenance of the bioretention areas. As with any excavation adjacent to improvements, the contractor should reduce the exposure time such that the improvements are not detrimentally impacted.

5.11 LANDSCAPING CONSIDERATION

As the near-surface soils are moderately to highly expansive, we recommend greatly restricting the amount of surface water infiltration near structures, pavements, flatwork, and slabs-on-grade. This may be accomplished by:

- Selecting landscaping that requires little or no watering, especially within 3 feet of structures, slabs-on-grade, or pavements.
- Using low precipitation sprinkler heads.
- Regulating the amount of water distributed to lawn or planter areas by installing automated moisture sensors on the sprinkler system.
- Providing surface grades to drain rainfall or landscape watering to appropriate collection systems and away from structures, slabs-on-grade, or pavements.
- Preventing water from draining toward or ponding near building foundations, slabs-on-grade, or pavements.
- Avoiding open planting areas within 3 feet of the building perimeter.

We recommend that these items be incorporated into the landscaping plans.

6.0 FOUNDATION RECOMMENDATIONS

We developed foundation recommendations by reviewing previous field exploration and laboratory test results, and performing additional engineering analysis.

6.1 CONVENTIONAL FOOTINGS WITH SLABS-ON-GRADE

The proposed restaurant building can be supported on continuous perimeter and isolated interior spread footings bearing in competent native soil or compacted fill with slab-on-grade floors.

We recommend constructing the upper 18 inches of the building pad with low-expansive fill. The low-expansive fill should have a PI of less than 12. As an alternative to importing low-expansive fill for grading the building pad, it may be cost effective to lime treat the upper 18 inches of the finished building pad and to 10 feet laterally beyond.

6.1.1 Footing Dimensions and Allowable Bearing Capacity

Provide minimum footing dimensions as follows in the Table 6.1.1-1 below.

TABLE 6.1.1-1: Minimum Footing Dimensions

FOOTING TYPE	*MINIMUM DEPTH (INCHES)	MINIMUM WIDTH (INCHES)
Continuous	18	16
Isolated	18	24

* below lowest adjacent pad grade

We recommend perimeter footings (continuous and isolated) be embedded to a minimum depth of 24 inches to reduce potential moisture fluctuation below the building.

Minimum footing depths as recommended above are taken from lowest adjacent pad grade. The cold joint between the exterior footing and slab-on-grade should be located at least 4 inches above adjacent exterior grade.

Foundations recommended above may be designed for a maximum allowable bearing pressure of 3,000 pounds per square foot (psf) for dead-plus-live loads. This bearing capacity may be increased by one-third for the short-term effects of wind or seismic loading.

The maximum allowable bearing pressure is a net value; the weight of the footing may be neglected for design purposes. Footings located adjacent to utility trenches should have their bearing surfaces below an imaginary 1:1 (horizontal:vertical) plane projected upward from the bottom edge of the trench to the footing.

6.1.2 Waterstop

If a two-pour system is used for footings and slab, the cold joint between the exterior footing and slab-on-grade should be located at least 4 inches above adjacent finish exterior grade. If this is not done, then we recommend the addition of a waterstop between the two pours to reduce moisture penetration through the cold joint and migration under the slab. Use of a monolithic pour would eliminate the need for the waterstop.

6.1.3 Reinforcement

The structural engineer should design footing reinforcement to support the intended structural loads without excessive settlement. Continuous footings should be reinforced with top and bottom steel to provide structural continuity and to permit spanning of local irregularities. At a minimum, continuous footings should be designed to structurally span a clear distance of 5 feet.

6.1.4 Foundation Lateral Resistance

Lateral loads may be resisted by friction along the base and by passive pressure along the sides of foundations. The passive pressure is based on an equivalent fluid weight in pounds per cubic foot (pcf). We recommend the following allowable values for design:

- Passive Lateral Pressure: 250 pcf
- Coefficient of Friction: 0.30

The above allowable values include a factor of safety of 1.5. Increase the above values by one-third for the short-term effects of wind or seismic loading.

Passive lateral pressure should not be used for footings on or above slopes.

6.1.5 Settlement

Provided our report recommendations are followed and given the proposed construction, we estimate total and differential foundation settlements to be less than approximately 1 and ½ inch, respectively.

7.0 SLABS-ON-GRADE

7.1 LOW-EXPANSIVE FILL

We recommend constructing the upper 18 inches of the building pad with low-expansive fill. The low-expansive fill should have a PI of less than 12. As an alternative to importing low-expansive fill for grading the building pad, it may be cost effective to lime treat the upper 18 inches of the finished building pad and to 10 feet laterally beyond.

7.2 MINIMUM DESIGN SECTION

To reduce the effects of expansive soil on interior slabs, we recommend the following:

1. Provide a minimum concrete thickness of 5 inches.

2. Reinforce slabs with No. 3 rebar on 18-inch centers, each way, placed within the middle third of the slab.

3. Pre-saturate the upper 12 inches of slab subgrade to a moisture content of at least optimum moisture content.

The structural engineer should provide final design thickness and additional reinforcement, if necessary, for the intended structural loads.

7.3 SLAB MOISTURE VAPOR REDUCTION

When buildings are constructed with concrete slab-on-grade, water vapor from beneath the slab will migrate through the slab and into the building. This water vapor can be reduced but not stopped. Vapor transmission can negatively affect floor coverings and lead to increased moisture within a building. When water vapor migrating through the slab would be undesirable, we recommend the following to reduce, but not stop, water vapor transmission upward through the slab-on-grade.

1. Construct a moisture retarder system directly beneath the slab on-grade that consists of the following:

    a. Vapor retarder membrane sealed at all seams and pipe penetrations and connected to all footings. Vapor retarders shall conform to Class A vapor retarder in accordance with ASTM E 1745, latest edition, “Standard Specification for Plastic Water Vapor Retarders used in Contact with Soil or Granular Fill under Concrete Slabs”. The vapor retarder should be **underlain by**

    b. 4 inches of clean gravel with 100 percent passing the ¾-inch sieve and less than 5 percent passing the No. 4 Sieve.

2. Use a concrete water-cement ratio for slabs-on-grade of no more than 0.50.

3. Provide inspection and testing during concrete placement to check that the proper concrete and water cement ratio are used.

4. Moist cure slabs for a minimum of 3 days or use other equivalent curing specified by the structural engineer.

The structural engineer should be consulted as to the use of a layer of clean sand or pea gravel (less than 5 percent passing the U.S. Standard No. 200 Sieve) placed on top of the vapor retarder membrane to assist in concrete curing.

7.4 SUBGRADE MODULUS FOR STRUCTURAL SLAB DESIGN

Provided the site earthwork is conducted in accordance with the recommendations of this report, a subgrade modulus of 150 psi/in can be used for structural slab design.

7.5 EXTERIOR FLATWORK

Exterior flatwork includes items such as concrete sidewalks, steps, and outdoor courtyards exposed to foot traffic only. A minimum section of 4 inches of concrete over 4 inches of aggregate base should be provided. The aggregate base should be compacted to at least 90 percent relative compaction (ASTM D1557). Flatwork edges should be thickened to at least 6 inches to help control moisture variations in the subgrade. Wire mesh or rebar should be placed within the middle third of the slab to help control the width and offset of cracks. Control and construction joints should be constructed in accordance with current Portland Cement Association Guidelines.

7.6 TRENCH BACKFILL

All trenches below building slabs-on-grade, and to 5 feet laterally beyond any edge, should be backfilled and compacted in accordance with the above Earthwork Recommendations section.

8.0 RETAINING WALLS

8.1 LATERAL SOIL PRESSURES

Proposed retaining walls should be designed to resist lateral earth pressures from adjoining natural materials and/or backfill and from any surcharge loads. Provided that adequate drainage is included as recommended below, walls restrained from movement at the top should be designed to resist an equivalent fluid pressure of 70 pounds per cubic foot (pcf). In addition, restrained walls should be designed to resist an additional uniform pressure equivalent to one-half of any surcharge loads applied at the surface.

Unrestrained retaining walls should be designed with adequate drainage to resist an equivalent fluid pressure of 50 pcf plus one-third of any surcharge loads.

The above lateral earth pressures assume level backfill conditions and sufficient drainage behind the walls to prevent any build-up of hydrostatic pressures from surface water infiltration and/or a rise in the groundwater level. If adequate drainage is not provided, we recommend that an additional equivalent fluid pressure of 40 pcf be added to the values recommended above for both restrained and unrestrained walls. Damp-proofing of the walls should be included in areas where wall moisture would be problematic.

8.2 RETAINING WALL DRAINAGE

Graded rock drains or geosynthetic drainage composites should be provided behind the retaining walls to reduce hydrostatic lateral forces. For rock drain construction, we recommend two types of rock drain alternatives:

1. A minimum 12-inch-thick layer of Class 2 Permeable Filter Material (Caltrans Specification 68-2.02F) placed directly behind the wall, or

2. A minimum 12-inch-thick layer of washed, crushed rock with 100 percent passing the ¾-inch sieve and less than 5 percent passing the No. 4 sieve. Envelop rock in a minimum 6-ounce, nonwoven geotextile filter fabric.

For both types of rock drains:

1. Place the rock drain directly behind the walls of the structure.

2. Extend rock drains from the wall base to within 12 inches of the top of the wall.

3. Place a minimum of 4-inch-diameter perforated pipe (glued joints and end caps) at the base of the wall, inside the rock drain and fabric, with perforations placed down.

4. Place pipe at a gradient at least 1 percent to direct water away from the wall by gravity to a drainage facility.

ENGEO should review and approve geosynthetic composite drainage systems prior to use.

8.3 BACKFILL

Backfill behind retaining walls should be placed and compacted in accordance with Section 5.6. Light compaction equipment should be used within 5 feet of the wall face. If heavy compaction equipment is used, the walls should be temporarily braced to avoid excessive wall movement.

8.4 FOUNDATIONS

Retaining walls may be supported on continuous footings embedded to a depth of at least 24 inches below lowest adjacent grade, and designed in accordance with recommendations presented in Section 6.1.

9.0 PAVEMENT DESIGN

9.1 FLEXIBLE PAVEMENTS

We developed the pavement recommendations based on the Caltrans Highway Design Manual design method, as this method is the preferred method for pavement design in California. Based on the soil conditions observed, we have assumed a Resistance Value (R-value) of 5 for a clayey subgrade and Traffic Indices (TI) provided by the Civil Engineer utilizing the methods contained in Chapter 630 of the Caltrans Highway Design Manual (including the asphalt factor of safety).

TABLE 9.1-1: Recommended Asphalt Concrete Pavement Sections

TRAFFIC INDEX	SECTION	
	ASPHALT CONCRETE (INCHES)	CLASS 2 AGGREGATE BASE (INCHES)
5	3	10
6	3.5	13
7	4	16

The civil engineer should determine the appropriate traffic indices based on the estimated traffic loads and frequencies.

9.2 RIGID PAVEMENTS

We developed rigid pavement sections according to the methodology presented in American Concrete Institute (ACI) report 330R, “Guide for the Design and Construction of Concrete Parking Lots.” We used Section 613.3 of the 2012 edition of the Caltrans Highway Design Manual to convert Traffic Indexes (TIs) to Average Daily Truck Traffic volumes (ADTTs) for use with the ACI method.

The rigid pavement sections are presented as jointed plain concrete pavement (JPCP) over Class 2 Aggregate Base (AB) developed assuming a 20-year lifetime. The sections required for the TIs presented over a soil subgrade with a Modulus of Subgrade Reaction of 75 psi/in and a minimum 28-day concrete compressive strength of 3,500 psi are presented in Table 9.2-1. Provide minimum control joint spacing in accordance with Portland Cement Association guidelines. The concrete pavement sections should be laterally restrained with concrete curbs or shoulders.

TABLE 9.2-1: Recommended Concrete Pavement Sections

TI	ADTT	JPCP (INCHES)	CLASS 2 AB (INCHES)
5.0	5	6	6
6.0	25	6½	6
7.0	100	7½	6

9.3 SUBGRADE AND AGGREGATE BASE COMPACTION

Finish subgrade and aggregate base should be compacted in accordance with Section 5.6. Aggregate Base should meet the requirements for ¾-inch maximum Class 2 AB in accordance with Section 26-1.02B of the latest Caltrans Standard Specifications.

9.4 CUT-OFF CURBS

Saturated pavement subgrade or aggregate base can cause premature failure or increased maintenance of asphalt concrete pavements. This condition often occurs where landscape areas directly abut and drain toward pavements. If desired to install pavement cutoff barriers, they should be considered where pavement areas lie downslope of any landscape areas that are to be sprinklered or irrigated, and should extend to a depth of at least 4 inches below the base rock layer. Cutoff barriers may consist of deepened concrete curbs or deep-root moisture barriers.

If reduced pavement life and greater than normal pavement maintenance are acceptable to the owner, the cutoff barrier may be eliminated.

10.0 GROUND HEAT EXCHANGE

Based on our findings and review of the proposed development, we consider the site to be highly suitable for using a Ground Heat-Exchange (GHX) system to achieve energy savings and to potentially eliminate the need for outdoor air conditioner units, if desired.

For the thermal properties of the soil and groundwater conditions at the site, a closed-loop GHX system would likely be well suited and could be implemented on select buildings, or integrated into a project-wide system.

As project planning progresses into architectural design, we can meet with you, your architect, and your MEP designer to further assess and develop GHX energy saving opportunities and efficiencies.

11.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report presents geotechnical recommendations for design of the improvements discussed in Section 1 for the proposed restaurant project. If changes occur in the nature or design of the project, we should be allowed to review this report and provide additional recommendations, if any. It is the responsibility of the owner to transmit the information and recommendations of this report to the appropriate organizations or people involved in design of the project, including but not limited to developers, owners, buyers, architects, engineers, and designers. The conclusions and recommendations contained in this report are solely professional opinions and are valid for a period of no more than 2 years from the date of report issuance.

We strived to perform our professional services in accordance with generally accepted geotechnical engineering principles and practices currently employed in the area; no warranty is expressed or implied. There are risks of earth movement and property damages inherent in building on or with earth materials. We are unable to eliminate all risks; therefore, we are unable to guarantee or warrant the results of our services.

This report update is based on review of referenced previous reports by others, that are based upon field and other conditions discovered at the time of previous report preparation. We developed this report using subsurface exploration data performed by others. We assumed that the subsurface exploration data are representative of the actual subsurface conditions across the site. Considering possible underground variability of soil and groundwater, additional costs may be required to complete the project. We recommend that the owner establish a contingency fund to cover such costs. If unexpected conditions are encountered, ENGEO must be notified immediately to review these conditions and provide additional and/or modified recommendations, as necessary.

Our services did not include excavation sloping or shoring, soil volume change factors, flood potential, or a geohazard exploration. In addition, our scope did not include work to determine the existence of possible hazardous materials. If any hazardous materials are encountered during construction, the proper regulatory officials must be notified immediately.

This document must not be subject to unauthorized reuse, that is, reusing without written authorization of ENGEO. Such authorization is essential because it requires ENGEO to evaluate the document’s applicability given new circumstances, not the least of which is passage of time.

Actual field or other conditions will necessitate clarifications, adjustments, modifications or other changes to ENGEO’s documents. Therefore, ENGEO must be engaged to prepare the necessary clarifications, adjustments, modifications or other changes before construction activities commence or further activity proceeds. If ENGEO’s scope of services does not include onsite construction observation, or if other persons or entities are retained to provide such services, ENGEO cannot be held responsible for any or all claims arising from or resulting from the performance of such services by other persons or entities, and from any or all claims arising from or resulting from clarifications, adjustments, modifications, discrepancies or other changes necessary to reflect changed field or other conditions.

SELECTED REFERENCES

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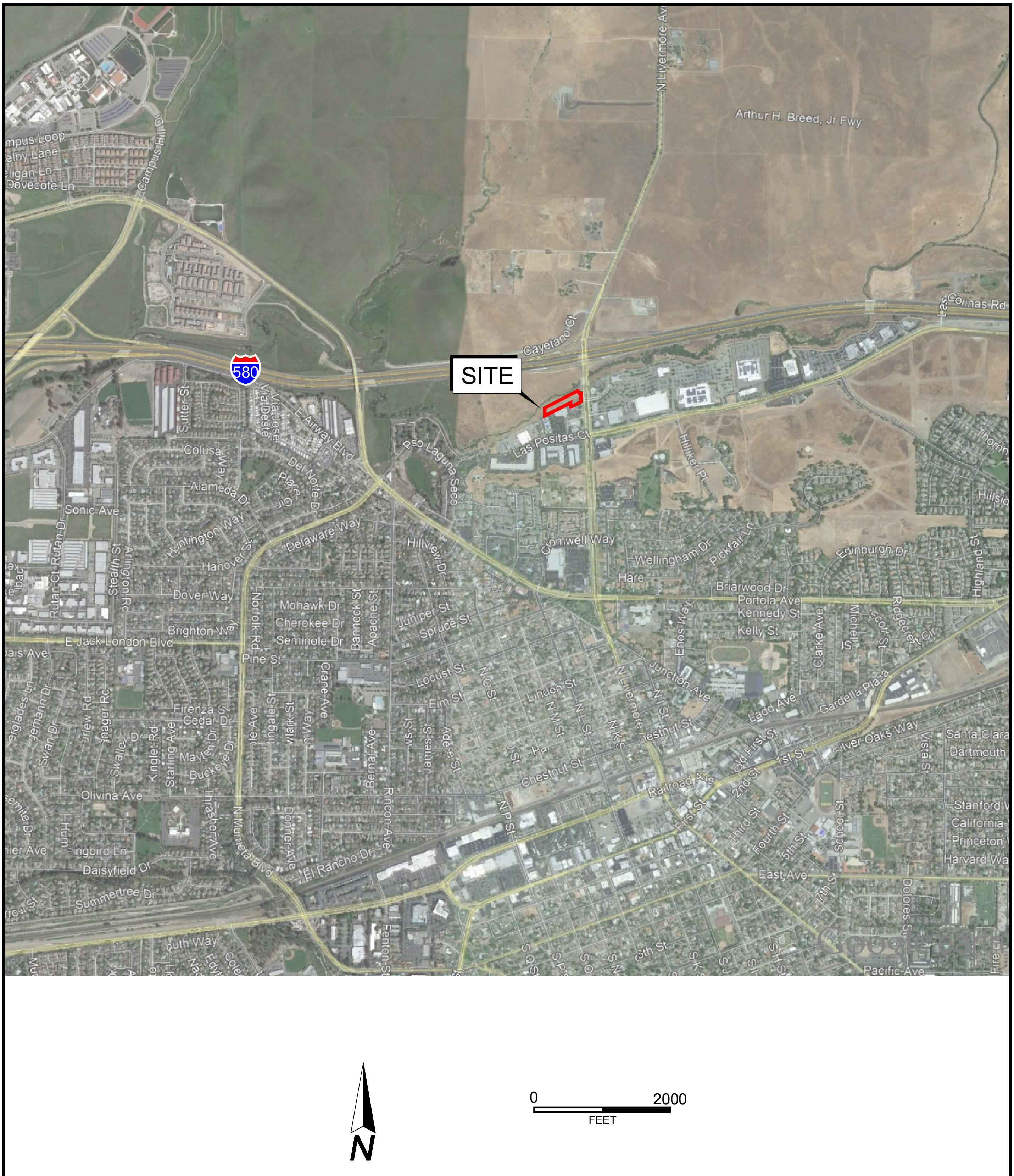
FIGURES



BASE MAP SOURCE: GOOGLE EARTH MAPPING SERVICE

ENGEO Expect Excellence	VICINITY MAP PROPOSED CHICK-FIL-A RESTAURANT LIVERMORE, CALIFORNIA	PROJECT NO.: 14986.000.000 SCALE: AS SHOWN DRAWN BY: LL CHECKED BY: UE	FIGURE NO. 1

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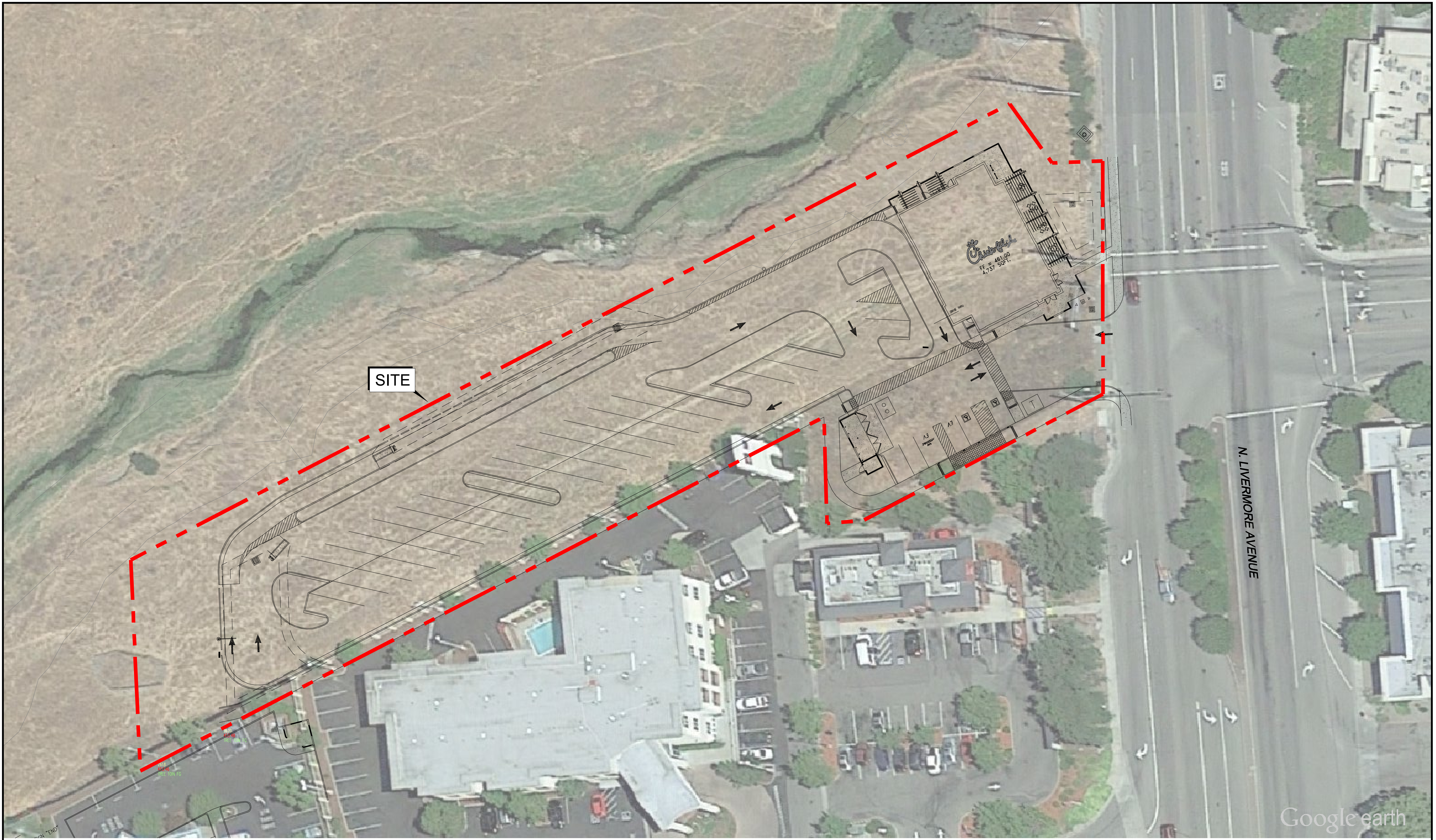

BASE MAP SOURCE: GOOGLE EARTH MAPPING SERVICE AND JOSEPH C. TRUXAW & ASSOCIATES, INC. (2018)

ENGEO
Expect Excellence

SITE PLAN

PROPOSED CHICK-FIL-A RESTAURANT
LIVERMORE, CALIFORNIA

PROJECT NO.: 14986.000.000
SCALE: AS SHOWN
DRAWN BY: LL
CHECKED BY: UE
FIGURE NO. 2

ORIGINAL FIGURE PRINTED IN COLOR

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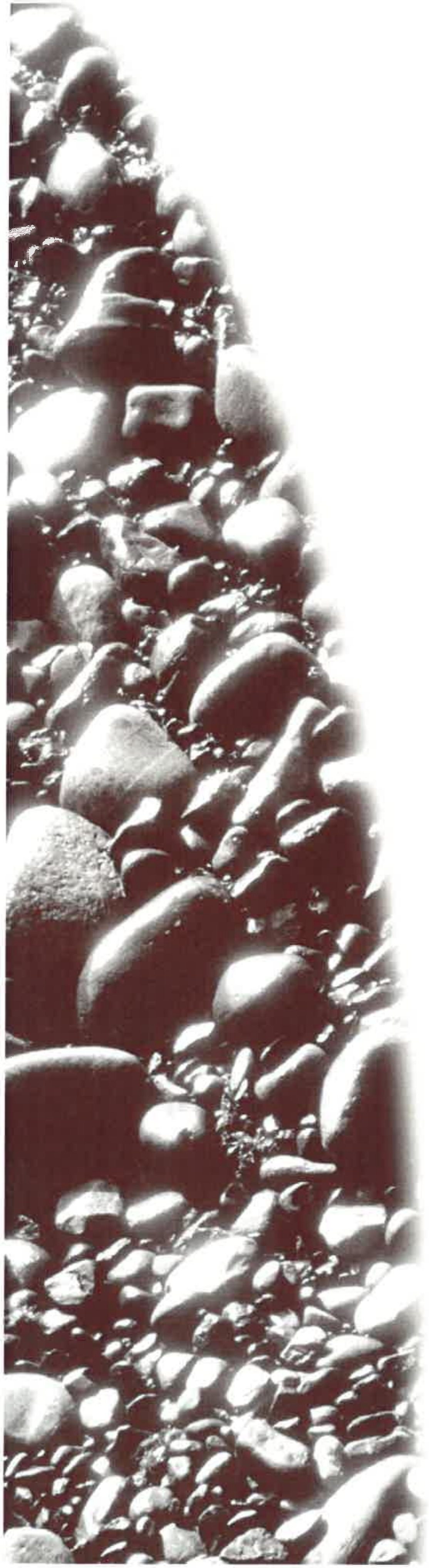
APPENDIX A

GEOTECHNICAL EXPLORATION (GILES, 2017)



● **RED**

Geotechnical Engineering Exploration and Analysis

Proposed Chick-fil-A Restaurant #3805
Livermore @ 580 FSU
SWC of N. Livermore Avenue and I-580 Freeway
Livermore, California

Prepared for:

Chick-fil-A, Inc.

Prepared by:

Giles Engineering Associates, Inc.

April 24, 2017
Project No. 2G-1606012

GILES
ENGINEERING ASSOCIATES, INC.

GILES
ENGINEERING ASSOCIATES, INC.
GEOTECHNICAL, ENVIRONMENTAL & CONSTRUCTION MATERIALS CONSULTANTS

- Atlanta, GA
- Baltimore, MD
- Dallas, TX
- Los Angeles, CA
- Manassas, VA
- Milwaukee, WI

April 24, 2017

Chick-fil-A, Inc.
15635 Alton Parkway, Suite 350
Irvine, California 92618

Attention: Ms. Beth Witt
Development Coordinator

Subject: Geotechnical Engineering Exploration and Analysis
Proposed Chick-fil-A Restaurant #3805
Livermore @ 580 FSU
SWC of N. Livermore Avenue and I-580 Freeway
Livermore, California
Project No. 2G-1606012

Dear Ms. Witt:

Giles Engineering Associates, Inc. (Giles) is pleased to present our *Geotechnical Engineering Exploration and Analysis* report prepared for the above-referenced project. Conclusions and recommendations developed from the exploration and analysis are discussed in the accompanying report.

We appreciate the opportunity to be of service on this project. If we may be of additional assistance, should geotechnical related problems occur or to provide construction observation and testing services, please do not hesitate to call at any time.

Respectfully submitted,

GILES ENGINEERING ASSOCIATES, INC.

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EXP 6-30-17

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40 GILES YEARS

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PROPOSED CHICK-FIL-A RESTAURANT #3805
LIVERMORE @ 580 FSU
SWC OF N. LIVERMORE AVENUE AND I-580 FREEWAY
LIVERMORE, CALIFORNIA
PROJECT NO. 2G-1606012

GEOTECHNICAL ENGINEERING EXPLORATION AND ANALYSIS

PROPOSED CHICK-FIL-A RESTAURANT #3805
LIVERMORE @ 580 FSU
SWC OF N. LIVERMORE AVENUE AND I-580 FREEWAY
LIVERMORE, CALIFORNIA
PROJECT NO. 2G-1606012

1.0 EXECUTIVE SUMMARY OUTLINE

The executive summary is provided solely for purposes of overview. Any party who relies on this report must read the full report. The executive summary omits a number of details, any one of which could be crucial to the proper application of this report.

Subsurface Conditions

- Site Class designation C is recommended for seismic design considerations.
- According to the California Department of Conservation- California Geological Survey, Geologic Map of the Livermore Quadrangle, California (2006), the site is located in an area underlain by Holocene stream terrace deposits consisting generally of sand, silt, clay and gravel.
- Soils encountered within our test borings generally consisted of stiff sandy clay and silty clay, and medium dense to very dense silty sand, clayey sand and sand with gravel and possible cobbles at deeper depths. The upper 10 feet of the soils were generally of finer soils (clay) and below 10 feet, the soils were generally granular (sand and gravel with possible cobbles).
- Groundwater was encountered at a depth of about 30 feet below existing ground surface within the deeper test boring (B-1).
- Moist to very moist soil conditions were encountered within some of the near surface soils during our subsurface investigation. Grading operations may require provisions for drying of soils prior to compaction.

Site Development

- Following site stripping, the exposed soils should be proof rolled with heavy construction equipment in the presence of the geotechnical engineer. Any soil that exhibits excessive deflection during proof rolling should be removed and replaced with engineered fill. Prior to placement of fill, the exposed surfaces should first be scarified to an approximate depth of at least 12 inches, moisture conditioned and then recompacted to at least 90% of the maximum dry density as determined by Modified Proctor (ASTM D 1557-00).
- The existing steep descending slope was evaluated to assess its stability. The results of this analysis indicate that the slope is stable with respect to static and pseudostatic global stability.
- A structural setback and mitigation has been incorporated into the project design due to the presence of the adjacent watercourse.

Building Foundation

- Shallow spread footing foundation system supported on suitable bearing soil may be designed for a maximum, net allowable soil bearing pressure of 3,000 psf.
- Minimum reinforcing in the strip footings is recommended to consist of four No. 5 bars (2 top and 2 bottom).

Building Floor Slab

- It is recommended that on grade slab be a minimum 4 inch thick slab-on-grade or turned-down slab, underlain by a minimum 4-inch thick granular material supported on a properly prepared subgrade.
- Minimum slab reinforcing recommended consisting of No. 3 rebars spaced at 18 inches on center, each way.
- The floor slab subgrade soils should be moisture conditioned and tested by the geotechnical engineer immediately prior to floor slab construction.

New Pavement

- Asphalt Pavements: 3 inches of asphaltic concrete underlain by 7 or 10 inches of base course in parking stall and drive lane areas, respectively.
- Portland Cement Concrete: 6 inches in thickness underlain by 4 inches of base course in high stress areas such as entrance/exit aprons, drive-thru lane and the trash enclosure-loading zone.
- Some increased pavement maintenance should be expected due to the presence of medium expansive soils.

RED - This site has been given a red designation due to the presence of the descending steep slope, due to presence of medium expansive soil and due to the required watercourse setback.

2.0 SCOPE OF SERVICES

This report provides the results of the *Geotechnical Engineering Exploration and Analysis* that Giles Engineering Associates, Inc. ("Giles") conducted regarding the proposed development. The *Geotechnical Engineering Exploration and Analysis* included several separate, but related, service areas referenced hereafter as the Geotechnical Subsurface Exploration Program, Geotechnical Laboratory Services, and Geotechnical Engineering Services. The scope of each service area was narrow and limited, as directed by our client and in consideration of the proposed project. The scope of each service area is briefly explained in this report.

Geotechnical-related recommendations for design and construction of the foundation and ground-bearing floor slab for the proposed building are provided in this report. Geotechnical-related recommendations are also provided for the proposed parking lot. Site preparation recommendations are also given; however, those recommendations are only preliminary since the means and methods of site preparation will depend on factors that were unknown when this report was prepared. Those factors include the weather before and during construction, the water table at the time of construction, subsurface conditions that are exposed during construction, and finalized details of the proposed development.

Giles conducted a Phase 1 Environmental Site Assessment for the subject site. A report documenting the results of that assessment have been provided under separate cover (2E-1606011, dated August 16, 2016).

3.0 SITES AND PROJECT DESCRIPTION

3.1 Site Description

The site is currently an irregular shaped vacant lot located at the southwest corner of North Livermore Avenue and the I-580 Freeway in the city of Livermore, California. Based on our Phase I report, the southeastern portion of the subject property was occupied by a residential property from at least 1949 through 2001, when the structures were demolished. The site is bordered on the north by an approximately 20-foot-high, 1:1 (h:v) slope that descends to Arroyo Las Positas (watercourse) then by the I-580 Freeway, on the east by N. Livermore Avenue, on the south by a Jack In The Box restaurant and Hawthorne Suites, and on the west by a vacant parcel.

Based on a review of the ALTA Survey, prepared by Joseph C. Truxaw & Associates, dated April 8, 2016, elevations within the site range from approximately El. 457.1 feet along the westerly end of the property to El. 460.4 feet near the northeast corner of the site near the top of the descending slope. The elevation of the toe of the descending slope is about El. 440. The adjacent northerly descending slope is covered by moderate vegetation that includes shrubs and occasional trees. The site is situated at approximately latitude 37.6991° North and longitude 121.7743° West.

3.2 Proposed Project Description

Based on our review of the site plan prepared by CRHO (project architect), it is our understanding that the proposed building is to be located in the northeasterly portion of the site and will be a single-story wood-frame modular structure with no basement or underground levels and will have a floor area of about 4,634 square feet. We were not provided with specific loading information for this project at the time of this report; however, based on our previous Chick-fil-A projects, we expect the maximum combined dead and live loads supported by the bearing walls and columns will be 2 to 3 kips per lineal foot (klf) and 40 to 50 kips, respectively. The live load supported by the floor slab is expected to be a maximum of 100 pounds per square foot (psf).

Other planned improvements include a paved drive thru and parking lot, menu board signs, a trash enclosure, a patio, concrete walkways and planter areas. Parking lot improvement, within the subject property, will include sidewalks, curbs and gutters, and underground utilities.

According to the *Conceptual Grading & Utility Plan*, prepared by Joseph C. Truxaw & Associates, Inc., dated April 13, 2017, the planned finished floor elevation for the proposed building is El. 461.00. Existing elevations within the building pad area are about El 460. Therefore, site grading is anticipated to include cuts and fills of less than 1 foot, exclusive of site preparation or over-excavation requirements.

The traffic loading on the proposed parking lot improvement is understood to predominantly consist of automobiles with occasional heavy trucks resulting from deliveries and trash removal. The parking lot pavement sections have been designed on the basis of a Traffic Index (TI) of 4.0 for the automobile traffic parking stalls (light duty) and a TI of 5.0 for drive lane areas (medium duty) and for a 20 year design life.

4.0 SUBSURFACE EXPLORATION

4.1 Subsurface Exploration

Prior to drilling, a Drill Permit (Permit # 2016078) was obtained from the Zone 7 Water Agency. Our subsurface exploration consisted of the drilling of eight (8) test borings (B-1 to B-8) to depths of approximately 5 to 51.5 feet below existing ground surfaces. The approximate test boring locations are shown in the Test Boring Location Plan (Figure 1). The Test Boring Location Plan and Test Boring Logs (Records of Subsurface Exploration) are enclosed in Appendix A. Field and laboratory test procedures and results are enclosed in Appendix B and C, respectively. The terms and symbols used on the Test Boring Logs are defined on the General Notes in Appendix D.

Our subsurface exploration included the collection of relatively undisturbed samples of subsurface soil materials for laboratory testing purposes. Bulk samples consisted of composite soil materials obtained at selected depth intervals from the borings. Relatively undisturbed samples were collected using a 3-inch outside-diameter, modified California split-spoon soil sampler (CS) lined with 1-inch high brass rings. The sampler was driven with successive 30-inch drops of a hydraulically operated, 140-pound automatic trip hammer. Blow counts for each 6-inch driving increment were recorded on the field exploration logs. The central portions of the driven core samples were placed in sealed containers and transported to our laboratory for testing.

Where deemed appropriate, standard split-spoon tests (SS), also called Standard Penetration Test (SPT), were also performed at selected depth intervals in accordance with the American Society for Testing Materials (ASTM) Standard Procedure D 1586. This method consists of mechanically driving an unlined standard split-barrel sampler 18 inches into the soil with successive 30-inch drops of the 140-pound automatic trip hammer. Blow counts for each 6-inch driving increment were recorded on the exploration logs. The number of blows required to drive the standard split-spoon sampler for the last 12 of the 18 inches was identified as the uncorrected standard penetration resistance (N). Disturbed soil samples from the unlined standard split-spoon samplers were placed in glass jars and transported to our laboratory for testing.

4.2 Subsurface Conditions

The subsurface conditions as subsequently described have been simplified somewhat for ease of report interpretation. A more detailed description of the subsurface conditions at the test boring locations is provided by the logs of the test borings enclosed in Appendix B of this report.

Soil

According to the California Department of Conservation- California Geological Survey, Geologic Map of the Livermore Quadrangle, California (2006), the site is located in an area underlain by Holocene stream terrace deposits consisting generally of sand, silt, clay and gravel.

Soils encountered within our test borings generally consisted of stiff sandy clay and silty clay, and medium dense to very dense silty sand, clayey sand and sand with gravel and possible cobbles at deeper depths. The upper 10 feet of the soils were generally of finer soils (clay) and below 10 feet the soils were generally granular (sand and gravel with possible cobbles).

Groundwater

Groundwater was encountered at a depth of about 30 feet below existing ground surface during our subsurface investigation within Test Boring B-1.

Fluctuations of the groundwater table, localized zones of perched water, and rise in soil moisture content should be anticipated during and after the rainy season. Irrigation of landscape areas on or adjacent to the site could also cause fluctuations of local or shallow perched groundwater levels.

4.3 Percolation Testing

A below grade storm water infiltration system is being considered at the site and the following information is provided.

Our percolation tests consisted of excavating an eight (8) inch diameter test holes. The bottom of each test hole was covered with about 2-inches of clean gravel then a 2-inch diameter perforated pvc casing was installed with clean coarse sand used to surround the outside casing. Testing involved presoaking the test holes and filling the test hole with water, and recording the drop in the water surface. Measurements were taken in approximately 30-minute period; refilling after every reading. The drop in water level over time is the percolation rate at the test locations. The percolation rates were reduced to account for the discharge of water from both the sides and bottom of the boring. The formula below was used to calculate for the tested infiltration rate.

Tested Infiltration Rate = $\Delta H (60r) / \Delta t (r + 2H_{avg})$

Where: r is the radius of the test hole (in)
ΔH is the change in height over the time interval (in)
Δt is the time interval (min)
Havg is the average head height over the time interval

The results obtained from our percolation testing are summarized below.

Test Number	Test Depth (feet)	Percolation Rate (in/hr)	Adjusted Infiltration Rate (in/hr)	Soil Type
B-7	5.0	3.3	0.06	Sandy Clay
B-8	5.0	3.7	0.07	Clayey Sand to Sandy Clay

Based on the results of this testing, it is our opinion that the site clayey soils have very low to negligible percolation rates and are not considered suitable for infiltration.

5.0 LABORATORY TESTING

Several laboratory tests were performed on selected samples considered representative of those encountered in order to evaluate the engineering properties of the on-site soils. The following are brief description of our laboratory test results.

In Situ Moisture and Density

Tests were performed on select samples from the test borings to determine the subsoil's dry density and natural moisture contents in accordance with Test Method ASTM 2216-10. The results of these tests are included in the Test Boring Logs enclosed in Appendix A.

Expansive Potential

To evaluate the expansive potential of the near surface soils encountered during our subsurface exploration, a composite sample collected from Test Boring B-1 (1 to 5 feet) was subjected to Expansive Index (EI) testing in accordance with Test Method ASTM D 4829-11. The result of our expansion index (EI) test indicates that the near surface sample has a medium expansion potential (EI=54).

Atterberg Limits

The Atterberg limits (liquid limit, plastic limit and plasticity index) were determined for a representative sample of the on-site soils in accordance with Test Method ASTM D 4318-00. The result of the Atterberg Limits is included on the Test Boring Logs enclosed in Appendix A.

Consolidation Test

Settlement prediction under anticipated load was made on the basis of a one-dimensional consolidation test. This test was performed in general conformance with Test Method ASTM D 2435. The test sample was inundated in order to evaluate the sudden increase in moisture condition (collapse/swell potential). Result of this test indicated that the tested near surface soils have very low swell potential (0.89%). The Consolidation test curve, Figure 3, is included in Appendix A.

Sieve Analysis

Sieve Analyses that include Passing No. 200 Sieve were performed on selected samples from various depths within Test Borings B-1, B-2, B-4, B-7 and B-8 to assist in soil classification and to aid in the liquefaction analysis. These tests were performed in accordance with Test Method ASTM D 1140-00. The results of these tests are presented in Test Boring Logs, Appendix A.

Direct Shear

The angle of internal friction and cohesion were determined for relatively undisturbed soil samples collected from Test Borings B-2 and B-4. These tests were performed in general accordance with Test Method No. ASTM D 3080-98. Three specimens were prepared for each test. The test specimens were artificially saturated, and then sheared under various normal loads. Results are graphically presented as Figures 4 and 5 in Appendix A.

Soluble Sulfate Analysis and Soil Corrosivity

A representative sample of the near surface soils which may contact shallow buried utilities and structural concrete was performed to determine the corrosion potential for buried ferrous metal conduits and the concentrations present of water soluble sulfate which could result in chemical attack of cement. These test results have been evaluated in accordance with criteria established by the Cast Iron Pipe Research Association, Ductile Iron Pipe Research Association, the American Concrete Institute and the National Association of Corrosion Engineers. The following table presents the results of our laboratory testing.

Parameter	**B-1** **1 to 5 feet**
pH	7.29
Chloride	85 ppm
Sulfate	0. 0009%
Resistivity	840 ohm-cm

The chloride content of near-surface soils was determined for a selected sample in accordance with California Test Method No. 422. The results of this test indicated that tested on-site soils have a Low exposure to chloride.

The soil pH and minimum resistivity values were determined in accordance with California Test Method No. 643. The test results for pH indicated the tested soil was nearly neutral. The results from the minimum resistivity test generally indicate that the tested soils have a severe corrosive potential when in contact with ferrous materials. Therefore, special protection for underground cast iron pipe or ductile pipe may be warranted depending on the actual materials in contact with the pipe. We recommend that a corrosion engineer review these results in order to provide specific recommendations for corrosion protection as well as appropriate recommendations for other types of buried metal structures.

A representative sample of the near surface soils which may contact shallow buried utilities and structural concrete was performed to determine the concentrations present of water soluble sulfate which could result in chemical attack of cement. Our laboratory test data indicated that near surface soils contain approximately 0.0009 percent of water soluble sulfates. Based on Section 1904.1 of the 2016 California Building Code (CBC), concrete that may be exposed to sulfate containing soils shall comply with the provisions of ACI 318-11, Section 4.3. Therefore, according to Table 4.3.1 of the ACI 318-11 a negligible exposure to sulfate can be expected for concrete placed in contact with the tested on-site soils. No special sulfate resistant cement is considered necessary for concrete which will be in contact with the tested on-site soils.

6.0 GEOLOGIC AND SEISMIC HAZARDS

6.1 Active Fault Zones

The project site is located in a highly seismic region of California within the influence of several fault systems. However, the site does not lie within the boundaries of an Earthquake Fault Zone as defined by the State of California in the Alquist-Priolo Earthquake Fault Zoning Act.

6.2 Seismic Hazard Zones

Our review of the published Seismic Hazard Evaluation report for the Livermore Quadrangle (where the subject site is located) indicates that the site is located within a designated Liquefaction Hazard Zone.

General types of ground failures that might occur as a consequence of severe ground shaking typically include landsliding, ground lurching and shallow ground rupture. The probability of occurrence of each type of ground failure depends on the severity of the earthquake, distance from faults, topography, subsoils and groundwater conditions, in addition to other factors. Based on our subsurface exploration, laboratory testing and the seismic designation for this site, all of the above effects of seismic activity are considered unlikely at the site.

7.0 CONCLUSIONS AND RECOMMENDATIONS

Conditions imposed by the proposed development have been evaluated on the basis of the assumed floor elevation and engineering characteristics of the subsurface materials encountered during our subsurface investigation and their anticipated behavior both during and after construction. Conclusions and recommendations presented for the design of building foundation and floor slab, and pavement along with site preparation recommendations and construction considerations are discussed in the following sections of this report.

Development of the proposed site entails soil and foundation oriented considerations with respect to the presence of variable strength fill and possible fill soils and grading associated with the existing drainage channel. Recommendations in this report are predicated upon site preparation, foundation and floor slab construction observed by the geotechnical engineer.

Slope Stability Limit Equilibrium Analysis

The stability of the existing slope configuration was evaluated along Section A-A' (as delineated on Figure 1) using the computer software program GSlope (Mitre Software Corporation). The GSlope program uses a search for the lowest factor of safety within a specified search grid. The GSlope analysis is based on limit equilibrium and incorporates the Bishop's Modified Method of analysis. For the pseudostatic (earthquake) analysis, a pseudostatic coefficient of 0.25g was utilized.

Laboratory testing (direct shear tests) were performed on undisturbed soil samples collected from the site to determine appropriate soil strength parameters for the stability analyses of the slope. The results of the direct shear testing are attached within Appendix A. For our analysis we utilized the direct shear soil strength parameters obtained with an angle of internal friction of 18 degrees with a 1000 psf cohesion for the upper soils and an angle of internal friction of 28 degrees and a cohesion value of 390 psf for the deeper soils. For our analysis, we reduced the cohesion value to 500 psf for the upper soils and 300 psf for the deeper soils as an added level of conservatism.

Cross Section A – A' Results

The existing slope configuration, along Section A-A', was analyzed for long-term and short-term (pseudostatic) stability, utilizing the soil strength parameters noted above. A building foundation line load was also applied at the location of the future building perimeter footings to assess impact of the nearby building. The results of these analyses indicated a static and pseudostatic factors of safety of 1.76 and 1.27, respectively. These values are greater than the typical required factor of safeties of 1.5 for the static and 1.1 for the pseudostatic conditions. Based on these results, it is our opinion that the existing slope is in a stable condition with respect to a deep-seated failure. A copy of the computer output for both the static and pseudostatic analyses is provided with Appendix A.

Watercourse Setback

According to the Alameda County Public Works Agency Engineering Design Guidelines (April 2008), no development shall be permitted within the setbacks provided in the Watercourse Ordinance. For existing bank slopes at 2 horizontal to 1 vertical, or steeper, the setback is established by drawing a line at a 2.5:1 (horizontal to vertical) inclination from the toe of existing bank to a point where it intercepts the ground surface. A 20-foot setback is then applied from the intercept point. However, we understand that development is allowed within this 20-foot .setback if a wall is constructed and extends below the imaginary setback projection. As noted on the Conceptual Grading & Utility Plans (Sections A &H on Sheet 4), a wall been designed to extend below the imaginary projection.

Impact of Site on Stability of Adjacent Properties

It is our opinion that the proposed grading and construction for the subject site will not affect adversely impact the stability of adjoining properties provided that grading and construction are performed in accordance with the recommendations provided herein and in accordance with local code guidelines.

7.1 Seismic Design Considerations

Faulting/Seismic Design Parameters

Research of available maps published by the California Geological Survey (CGS) indicates that the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. The potential for fault rupture through the site is, therefore, considered to be low. The site may however be subject to strong groundshaking during seismic activity. The proposed structure should be designed in accordance with the current version of the 2016 California Building Code (CBC) and applicable local codes. Based upon the encountered subsurface soils, a Site Class C is recommended for design.

According to the maps of known active fault per 2008 National Seismic Hazard Maps – Source Parameters to be used with the 2016 CBC, the Mt. Diablo Thrust, Greenville Connected and Calaveras (CN+CC+CS) faults are the closest known active faults and are located about 3.5, 4.2 and 8.1 miles, respectively, from the site and with an anticipated maximum moment magnitude (Mw) of 6.70, 7.00 and 7.03, respectively.

Within the International Code Council's 2015 International Building Code (IBC), the five-percent damped design spectral response accelerations at short periods, S_{DS}, and at 1-second period, S_{D1}, are used to determine the seismic design base shear. These parameters, which are a function of the site's seismicity and soil, are also used as parts of triggers for other code requirements. The following values are determined by using the program Java Ground Motion Parameter Calculator- Version 5.0.10 written by the ICC.

IBC 2015/ CBC 2016, Earthquake Loads	
Site Class Definition (Table 1613.5.2)	C
Mapped Spectral Response Acceleration Parameter, S_s (Figure 1613.5(3) for 0.2 second)	1.668
Mapped Spectral Response Acceleration Parameter, S_1 (Figure 1613.5(4) for 1.0 second)	0.600
Site Coefficient, F_a (Table 1613.5.3 (1) short period)	1.0
Site Coefficient, F_v (Table 1613.5.3 (2) 1-second period)	1.3
Adjusted Maximum Considered Earthquake Spectral Response Acceleration Parameter, S_{MS} (Eq. 16-37)	1.668
Adjusted Maximum Considered Earthquake Spectral Response Acceleration Parameter, S_{M1} (Eq. 16-38)	0.780
Design Spectral Response Acceleration Parameter, S_{DS} (Eq. 16-39)	1.112
Design Spectral Response Acceleration Parameter, S_{D1} (Eq. 16-40)	0.520

Liquefaction

According to the Seismic Hazard Zones map for the Livermore Quadrangle, published by the California Geological Survey (CGS), the site is located within an area that has been designated by the State Geologist as a "zone of required investigation" due to the potential for earthquake-induced liquefaction. Therefore, a site liquefaction evaluation consistent with the guidelines contained in DMG Special Publication 117A (2008) has been performed as part of the current investigation. Although groundwater was encountered at a depth of 30 feet below existing ground surface during our subsurface exploration, a historic high water level of 10 feet was adopted for the liquefaction analysis.

The peak ground acceleration was determined in accordance with Section 11.8.3 of 2010 ASCE 7 with the March 2013 errata. The horizontal acceleration was determined using the USGS U.S. Seismic Design Maps website and we incorporated a Site Class D. For this analysis, a PGA_M of 0.628g was obtained. A deaggregation analysis was performed to determine the predominant earthquake magnitude for a 2% probability of exceedance in 50 years (2,475 year return period). For this event, the predominant earthquake magnitude of 6.58 was obtained.

Our liquefaction study was based on the NCEER procedure (Youd & Idriss, 1998) using a peak ground acceleration of 0.628g and an earthquake magnitude of 6.58. The liquefaction analysis was performed using the computer program Liquefypro (version 5) developed by Civil Tech Software. The program is based on the most recent publications of the NCEER Workshop and SP117A Implementation. The result of this analysis indicates that the site soils within Boring B-1 are not potentially susceptible to soil liquefaction. The computer output file for this analysis is presented graphically as Plate A1 of Appendix A. The results of this analysis indicates that the site soils are not potentially susceptible to soil liquefaction. Some minimal (less than 1/10 inch) dry settlement is estimated.

7.2 Site Development Recommendations

The recommendations for site development as subsequently described are based upon the conditions encountered at the test boring locations. Due to elevated in-situ moistures of the site soils, grading operations may require provisions for drying of soils prior to compaction. In addition, due to the presence of moist to very moist and sensitive soils, the loads imposed by heavy rubber-tired equipment during grading may induce localized pumping of the subgrade that would require stabilization prior to fill placement. The grading contractor should include contingencies for air-drying of excessively moist soil, as well as the stabilization of excavation bottoms in their bids. Imported soils may be required if on-site soils cannot be air-dried on site due to space, time constraints, or weather.

Site Clearing

Clearing operations should include the removal of all landscape vegetation within the area of the proposed site improvements. Large shrubs to be removed should be grubbed out to include removal of their stumps and major root systems.

Should any unusual soil conditions or subsurface structures be encountered during demolition operations or during grading, they should be brought to the immediate attention of the project geotechnical consultant for corrective recommendations.

Existing Utilities

All existing utilities (if any) should be located. Utilities that will be preserved are recommended to be relocated outside the building area. Utilities that are not to be reused should be capped off and removed or properly abandoned in-place in accordance with local codes and ordinances. The excavations made for removed utilities are recommended to be backfilled with structural compacted fill. Underground utilities, which are to be reused or abandoned in-place, are recommended to be evaluated by the structural engineer and utility backfill is recommended to be evaluated by the geotechnical engineer, to determine their potential effect on the new development. If any existing utilities are to be preserved, grading operations must be carefully performed so as not to disturb or damage the existing utility.

Proofroll and Compact Subgrade

After site clearing and lowering of site grades where necessary, the subgrades within the proposed new building, pavement and drive thru areas should be proofrolled in the presence of the geotechnical engineer with appropriate rubber-tire mounted heavy construction equipment or a loaded truck to detect very loose/soft yielding soil which should be removed to a stable subgrade. Following proofrolling and completion of any necessary overexcavation, the subgrades should be scarified to a minimum depth of 12 inches, moisture conditioned or air dried, and recompacted to at least 90 percent of the Modified Proctor (ASTM D1557-00) maximum density. The upper 1 foot of the pavement subgrade should have minimum in-place density of at least 95% of the maximum dry density. Low areas and excavations may then be backfilled in lifts with suitable very low to medium expansion (EI less than 91) structural compacted fill. The selection, placement and compaction of structural fill should be performed in accordance with the project specifications.

The Guide Specifications included in Appendix D (Modified Proctor) of this report are recommended to be used, at a minimum, as an aid in developing the project specifications. The floor slab subgrade may need to be recompacted prior to slab construction due to weather and equipment traffic effects on the previously compacted soil.

Reuse of On-site Soil

On-site medium expansive soils may be reused as structural compacted fill provided they do not contain oversized materials and/or significant quantities of organic matter or other deleterious materials. Due to the moisture sensitivity of the site soils, care should be used in controlling the moisture content of the soils to achieve proper compaction for load bearing and pavement support. Some drying of the site soils is expected to be necessary prior to their use as engineered fill, based on the in-situ moisture contents of these soils. During inclement weather, drying is not expected to be feasible and use of a select fill may be necessary. All subgrade soil compaction as well as the selection, placement and compaction of new fill soils should be performed in accordance with the project specifications under engineering controlled conditions.

Subgrade Protection

The near surface soils that are expected to comprise the subgrade are sensitive to water and disturbance from construction activities. Unstable soil conditions will develop if the soils are exposed to moisture increases or are disturbed (rutted) by construction traffic. The site should be graded to prevent water from ponding within construction areas and/or flowing into excavations. Accumulated water must be removed immediately along with any unstable soil. Foundation concrete should be placed and excavations backfilled as soon as possible to protect the bearing grade. The degree of subgrade instability and associated remedial construction is dependent, in part, upon precautions taken by the contractor to protect the subgrade during site development.

Silt fences or other appropriate erosion control devices should be installed in accordance with local, state and federal requirements at the perimeter of the development areas to control sediment from erosion. Since silt fences or other erosion control measures are temporary structures, careful and continuous monitoring and periodic maintenance to remove accumulated soil and/or replacement should be anticipated.

Fill Placement

Material for engineered fill should be free of organic material, debris, and other deleterious substances, and should not contain fragments greater than 3 inches in maximum dimension. Fill soils should possess a very low to medium expansive potential (EI<91). On-site excavated soils that meet these requirements may be used to backfill the excavated new building and pavement areas.

All fill should be placed in 8-inch-thick maximum loose lifts, moisture conditioned and then compacted to at least 90 percent of the Modified Proctor maximum density. A representative of the project geotechnical consultant should be present on-site during grading operations to document proper placement and compaction of all fill, as well as to verify compliance with the other geotechnical recommendations presented herein.

Import Structural Fill

Any soils imported to the site for use as structural fill should consist of very low to low expansive (EI less than 51) soils. Materials designated for import should be submitted to the project geotechnical engineer no less than three working days for evaluation. In addition to expansion criteria, soils imported to the site should exhibit adequate shear strength characteristics for the recommended allowable soil bearing pressure, soluble sulfate content and corrosivity and pavement support characteristics.

7.3 Construction Considerations

Construction Dewatering

Groundwater was encountered at a depth of about 30 feet during our field exploration. However, shallower perched water conditions may occur due to seasonal precipitation and runoff characteristics of the site. Conventional filtered sump pumps placed in excavations are expected to be suitable for dewatering within shallow excavations should any excess water conditions be observed. Deeper excavations that extend into the water table may require a more elaborate dewatering system.

Soil Excavation

Some localized slope stability problems may be encountered in steep, unbraced excavations considering the nature of the subsoils. All excavations must be performed in accordance with CAL-OSHA requirements, which is the responsibility of the contractor. Shallow excavations may be adequately sloped for bank stability while deeper excavations or excavations where adequate back sloping cannot be performed may require some form of external support such as shoring or bracing.

7.4 Foundation Recommendations

Vertical Load Capacity

Upon completion of the recommended building pad proof rolling, scarification and recompaction, it is our opinion the proposed structure may be supported by a shallow foundation system. Foundations may be designed for a maximum, net, allowable soil-bearing pressure of 3,000 pounds per square foot (psf). Minimum foundation widths for walls and columns should be 16 and 24 inches, respectively, for bearing considerations, regardless of actual soil pressure. The maximum bearing value applies to combined dead and sustained live loads. This allowable soil bearing pressure may be increased by one-third for short term wind and/or seismic loads.

Reinforcing

The recommended minimum quantity of longitudinal reinforcing within continuous strip footings for geotechnical considerations is four No. 5 bars (2 top and 2 bottom) continuous through any intermittent column pad footings. The recommended quantity of reinforcing pertains to a minimum 12-inch thick and a maximum 24-inch wide footing; additional reinforcing may be necessary if a thinner or wider footing is used to develop equivalent rigidity. The reinforcing recommendation is intended to provide greater rigidity due to the presence of medium expansive onsite soils. A qualified structural engineer should determine the actual reinforcing details.

Lateral Load Resistance

Lateral load resistance will be developed by a combination of friction acting at the base of foundations and slabs and the passive earth pressure developed by footings below grade. Passive pressure and friction may be used in combination, without reduction, in determining the total resistance to lateral loads. A one-third increase in the passive pressure value may be used for short duration wind or seismic loads.

A coefficient of friction of 0.30 may be used with dead load forces for footings placed on compacted fill soil. An allowable passive earth pressure of 250 psf per foot of footing depth (pcf) below the lowest adjacent grade may be used for the sides of footings placed against structural fill. The maximum recommended allowable passive pressure is 2,000 psf.

Bearing Material Criteria

Evaluation of the foundation bearing soils is recommended to be performed by the geotechnical engineer at the time of foundation construction prior to placement of reinforcing steel. Soil suitable to serve as subgrade for support of foundations should exhibit at least a stiff comparative consistency ($q_u \geq 1.5$ tsf) for cohesive soils and/or a firm relative density (N-value of least 10) for non-cohesive soils for the recommended 3,000 psf allowable soil bearing pressure.. The depth of evaluation should be determined by the geotechnical engineer. If unsuitable bearing soils are encountered, they should be recompacted in-place if feasible, or excavated to a suitable bearing soil subgrade and to a lateral extent as defined by Item No. 3 of the enclosed Guide Specifications, with the excavation backfilled with structural compacted fill to develop a uniform bearing grade. Alternatively, footings may be locally stepped through any unsuitable bearing soil and be founded entirely upon competent materials. If stepping is desired, foundation details should be provided by the structural engineer.

Foundation Embedment

The California Building Code (CBC) requires a minimum 12-inch foundation embedment depth. However, it is recommended that exterior foundations extend at least 18 inches below the adjacent exterior grade for bearing capacity consideration. Interior footings may be supported at nominal depth below the floor. All footings must be protected against weather and water damage during and after construction, and must be supported within suitable bearing materials.

Estimated Foundation Movement

Post-construction total and differential settlement of a shallow foundation system designed and constructed in accordance with the recommendations provided in this report are estimated to be less than 1 and ½ inch, respectively, for static conditions. The estimated differential movement is anticipated to result in an angular distortion of about 0.002 inches per inch on the basis of a minimum clear span of 20 feet. The maximum estimated total and differential movement is considered within tolerable limits for the proposed structures provided it is considered in the structural design.

7.5 Floor Slab Recommendations

Subgrade

The floor slab subgrade should be prepared in accordance with the appropriate recommendations presented in the Site Development Recommendations section of this report. Foundation, utility trenches and other below-slab excavations should be backfilled with structural compacted fill in accordance with the project specifications. Due to the expansive nature of the subgrade soils, these soils must be maintained at a moisture content of about 2 to 4% above the soil's optimum moisture content (per ASTM D-1557) to a depth of 12 inches prior to concrete placement. Testing by the geotechnical engineer is recommended within 24 hours of concrete placement to document proper soil moisture conditioning.

Design

The floor of the proposed building may be designed and constructed as a slab-on-grade supported on a properly prepared subgrade. If desired, the floor slab may be constructed monolithically with foundations where the foundations consist of thickened sections thereby using a turned-down slab construction technique. Minimum slab reinforcing, for geotechnical considerations, is recommended to consist of No. 3 rebars at 18 inches on center, each way. Based on the recommended reinforcing, assumed live loading and medium expansion potential of the near surface soils, the slab is recommended to possess a minimum thickness of 4 inches. A qualified structural engineer should perform the actual design of the slab to ensure proper thickness and reinforcing.

The floor slab is recommended to be underlain by a 4-inch thick layer of granular material. A minimum 10-mil synthetic sheet should be placed below the floor slab to serve as a vapor retarder where required to protect moisture sensitive floor coverings (i.e. tile, or carpet, etc.). The sheets of the vapor retarder material should be evaluated for holes and/or punctures prior to placement and the edges overlapped and taped. If materials underlying the synthetic sheet contain sharp, angular particles, a layer of sand approximately 2 inches thick or a geotextile should be provided to protect it from puncture. An additional 2-inch thick layer of sand is recommended between the slab and the vapor retarder to promote proper curing. The sand layers above and below the synthetic sheeting may be used as a substitute for the granular material below the slab. Proper curing techniques are recommended to reduce the potential for shrinkage cracking and slab curling.

Estimated Settlement

Post-construction total and differential movement (settlement and/or heave) of the floor slab designed and constructed in accordance with the recommendations provided in this report are estimated to be less than ½ and ⅓ inch, respectively. The estimated differential movement is anticipated to occur across the short dimension of the structure. The maximum total and differential movement is considered within tolerable limits for the proposed structure, provided that the structural design adequately considers this distortion.

7.6 Retaining Wall Recommendations

Due to the existing site grades, it is possible that retaining walls may be needed for this site. The retaining wall(s) may be supported by conventional shallow spread footings designed for an allowable soil bearing pressure of 3,000 psf. A higher allowable soil bearing pressure may be possible but that determination should be based on a review of the locations and details of the planned wall.

Retaining walls may be designed for an allowable passive earth pressure of 250 pounds per square foot, per foot of depth, to a maximum value of 3,000 pounds per square foot. In addition, a coefficient of friction of 0.30 may be used with dead load forces for footings placed on competent soil, as determined by the geotechnical engineer. The recommended allowable soil bearing pressure and passive pressure may be increased by one-third for short term wind and/or seismic loads.

Design of walls should incorporate an adequate factor-of-safety against both over-turning and sliding (FS=1.5). The overturning resultant should also fall within the center third (kern) of the retaining wall footing for stability, or the design must be re-evaluated with a reduced bearing area.

Static Lateral Earth Pressures

Retaining walls should be designed to resist the applicable lateral earth pressures. On-site expansive soils are **not** recommended for use as backfill behind walls. Retaining wall backfill should consist of very low to low expansive soils and allow for a drainage layer as discussed in subsequent paragraphs. For very low to low expansive soils (EI less than 51) to be used as backfill materials, an active earth pressure of 40 pounds per cubic foot (equivalent fluid pressure) should be used assuming a level adjacent backfill and drained conditions. For walls to be restrained at the top, an at-rest pressure of 60 pcf should be used for design. All retaining walls should be supplied with a proper subdrain system. All walls should be designed to support any adjacent structural surcharge loads imposed by other nearby walls or footings and vehicles in addition to the above recommended active earth pressure.

Pea gravel, crushed rock or clean sand exhibiting a sand equivalent of 30 or greater may also be used for retaining wall backfill. If these materials are used as backfill, the retaining wall may be designed for active and at-rest earth pressures of 30 and 45 pounds per cubic foot (equivalent fluid pressure), respectively.

Drainage and Damp-proofing

Retaining walls are recommended to be designed for drained earth pressures and therefore, adequate drainage should be provided behind the walls. This can be accomplished by installing subdrains at the base of the walls. Wall footing-drains should consist of a system of filter material and perforated pipe. The perforated pipe system should consist of 4-inch diameter, schedule 40, PVC pipe or equivalent, embedded in 1 cubic foot of Class II Permeable Material (CALTRANS Standard Specifications, latest edition) or equivalent per lineal foot of pipe. Alternatively, ¾-inch open graded gravel or crushed rock enveloped in Mirafi 140 geofabric or equivalent may be used instead of the Class II Permeable Material. The pipe should be placed at the base of the wall, and then routed to a suitable area for discharge of accumulated water.

Wall backfill should be protected against infiltration of surface water. Backfill adjacent to walls should be sloped so that surface water drains freely away from the wall and will not pond. Damp-proofing of walls below-grade is recommended especially where moisture control is required by an approved waterproofing compound or covered with similar material to inhibit infiltration of moisture through the walls.

Wall Backfill

Retaining wall backfill behind the drainage layers should consist of very low to low-expansive soils with an E.I. less than 51, as determined by the ASTM D 4829-03 method. Wall backfill should not contain organic material, rubble, debris, and rocks or cemented fragments larger than 3 inches in greatest dimension. A 1 foot thick low-expansive cohesive layer, or pavement, should be placed at the surface to help prevent surface water intrusion. A geotextile or filter fabric should be placed between the granular drainage layers and adjacent soils (excavated face or compacted materials) to prevent fines from migrating into the drainage layers.

Backfill should be placed in lifts not exceeding 8 inches in thickness, moisture conditioned to slightly above optimum moisture content, and mechanically compacted throughout to at least 90 percent of the maximum dry density as determined by Modified Proctor (ASTM D 1557). Retaining walls should be properly braced prior to placement and compaction of backfill should be performed with extreme care not to damage the walls.

7.7 New Pavement

The following recommendations for the new pavement are intended for vehicular traffic associated with the restaurant development within the subject property.

New Pavement Subgrades

Following completion of the recommended subgrade preparation procedures, the subgrade in areas of new pavement construction are expected to consist of medium expansive soil. The anticipated subgrade soils are classified as a poor subgrade material with estimated R-value of 5-10 when properly prepared based on the Unified Soil Classification System designation of CL/CH. An R-value of 5 has been assumed in the preparation of the pavement design. It should however, be recognized that the City of Livermore may require a specific R-value test to verify the use of the following design. It is recommended that this testing, if required, be conducted following completion of rough grading in the proposed pavement areas so that the R-value test results are indicative of the actual pavement subgrade soils. Alternatively, a minimum code pavement section may be required if a specific R-value test is not performed. To use this R-value, all fill added to the pavement subgrade must have pavement support characteristics at least equivalent to the existing soils, and must be placed and compacted in accordance with the project specifications.

Asphalt Pavements

The following table presents recommended thicknesses for a new flexible pavement structure consisting of asphaltic concrete over a granular base, along with the appropriate CALTRANS specifications for proper materials and placement procedures. An alternate pavement section has been provided for use in parking stall areas due to the anticipated lower traffic intensity in these areas.

However, care must be used so that truck traffic is excluded from areas where the thinner pavement section is used, since premature pavement distress may occur. In the event that heavy vehicle traffic cannot be excluded from the specific areas, the pavement section recommended for drive lanes should be used throughout the parking lot.

ASPHALT PAVEMENTS

Materials	**Thickness (inches)**		**CALTRANS Specifications**
	Parking Stalls (TI=4.0)	**Drive Lanes (TI=5.0)**	
Asphaltic Concrete Surface Course (b)	1	1	Section 39, (a)
Asphaltic Concrete Binder Course (b)	2	2	Section 39, (a)
Crushed Aggregate Base Course	7	10	Section 26, Class 2 (R-value at least 78)

NOTES:
(a) Compaction to density between 95 and 100 percent of the 50-Blow Marshall Density
(b) The surface and binder course may be combined as a single layer placed in one lift if similar materials are utilized.

Pavement recommendations are based upon CALTRANS design parameters for a twenty-year design period and assume proper drainage and construction monitoring. It is, therefore, recommended that the geotechnical engineer monitors and tests subgrade preparation, and that the subgrade be evaluated immediately before pavement construction.

Portland Concrete Pavements

Portland Cement Concrete pavements are recommended in areas where traffic is concentrated such as the entrance/exit aprons as well as areas subjected to heavy loads such as the trash enclosure loading zone. The preparation of the subgrade soils within concrete pavement areas should be performed as previously described in this report. Portland Cement Concrete pavements in high stress areas are recommended to be at least 6 inches thick containing No. 3 bars at 18-inch on-center both ways placed at mid-height. The pavement should be constructed in accordance with Section 40 of the CALTRANS Standard Specifications. A minimum 4-inch thick layer of base course (CALTRANS Class 2) is recommended below the concrete pavement. This base course should be compacted to at least 95% of the material's maximum dry density.

The maximum joint spacing within all of the Portland Cement Concrete pavements is recommended to be 15 feet or less to control shrinkage cracking. Load transfer reinforcing is recommended at construction joints perpendicular to traffic flow if construction joints are not properly keyed. In this event, ¾-inch diameter smooth dowel bars, 18 inches in length placed at 12 inches on-center are recommended where joints are perpendicular to the anticipated traffic flow. Expansion joints are recommended only where the pavement abuts fixed objects such as light standard foundations. Tie bars are recommended at the first joint within the perimeter of the concrete pavement area. Tie bars are recommended to be No. 4 bars at 42-inch on-center spacings and at least 48 inches in length.

General Considerations

Pavement recommendations assume proper drainage and construction monitoring and are based on traffic loads as indicated previously. Pavement designs are based on either PCA or CALTRANS design parameters for twenty (20) year design period. However, these designs are also based on a routine pavement maintenance program and significant asphalt concrete pavement rehabilitation after about 8 to 10 years, in order to obtain a reasonable pavement service life. Due to the presence of expansive soils, some increased pavement maintenance should be expected.

7.8 Recommended Construction Materials Testing Services

The report was prepared assuming that Giles will perform Construction Materials Testing (CMT) services during construction of the proposed development. In general, CMT services are recommended (and expected) to at least include observation and testing of: foundation and pavement support soil and other construction materials. It might be necessary for Giles to provide supplemental geotechnical recommendations based on the results of CMT services and specific details of the project not known at this time.

7.9 Basis of Report

This report is based on Giles' proposal, which is dated June 21, 2016 and is referenced by Giles' proposal number 2GEP-1606025. The actual services for the project varied somewhat from those described in the proposal because of the conditions that were encountered while performing the services and in consideration of the proposed project.

This report is strictly based on the project description given earlier in this report. Giles must be notified if any parts of the project description or our assumptions are not accurate so that this report can be amended, if needed. This report is based on the assumption that the facility will be designed and constructed according to the codes that govern construction at the site.

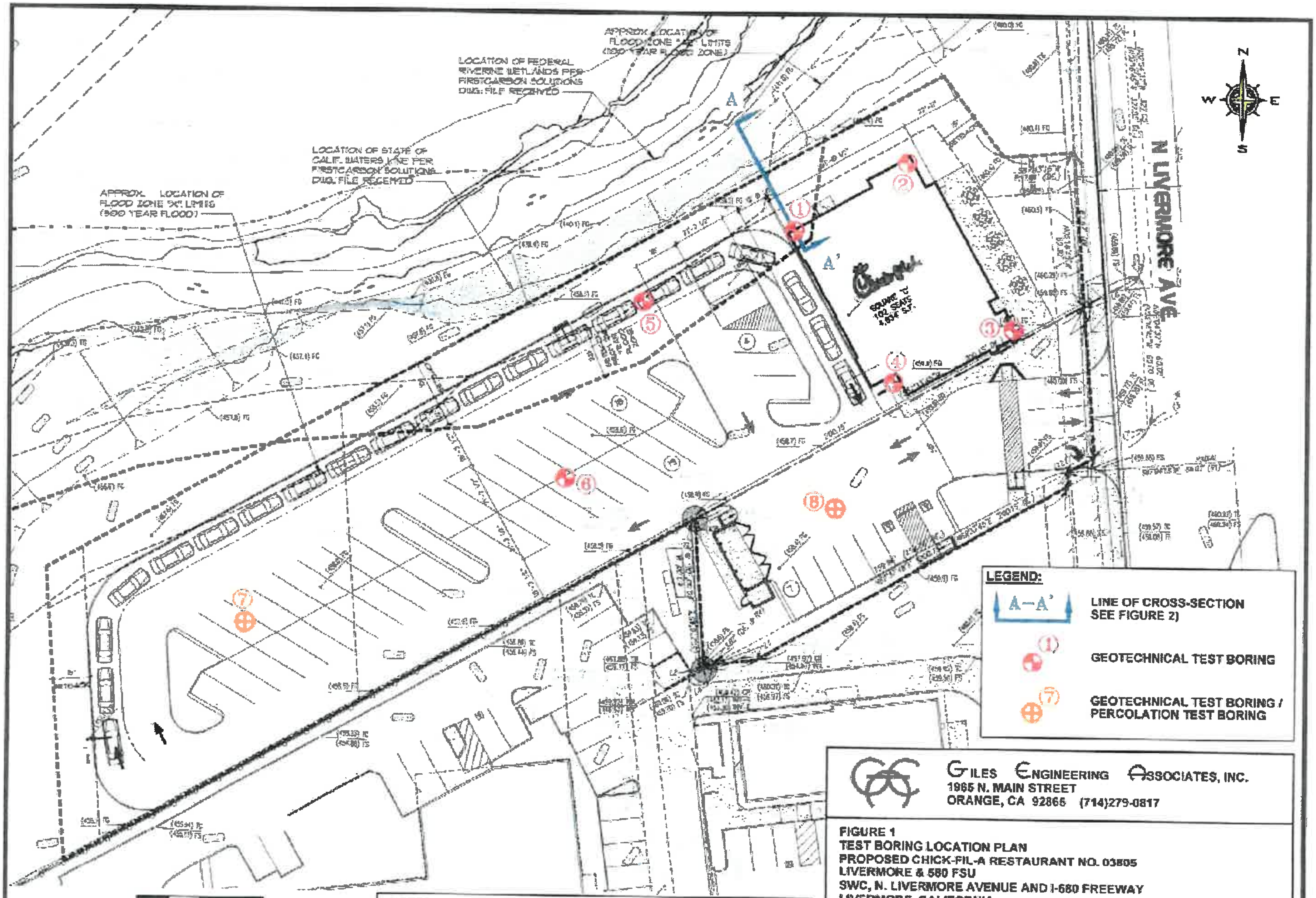
The conclusions and recommendations in this report are based on estimated subsurface conditions as shown on the *Records of Subsurface Exploration*. Giles must be notified if the subsurface conditions that are encountered during construction of the proposed development differ from those shown on the *Records of Subsurface Exploration* because this report will likely need to be revised. General comments and limitations of this report are given in the appendix.

APPENDIX A

FIGURES AND TEST BORING LOGS

The Test Boring Location Plan contained herein was prepared based upon information supplied by *Giles'* client, or others, along with *Giles'* field measurements and observations. The diagram is presented for conceptual purposes only and is intended to assist the reader in report interpretation.

The Test Boring Logs and related information enclosed herein depict the subsurface (soil and water) conditions encountered at the specific boring locations on the date that the exploration was performed. Subsurface conditions may differ between boring locations and within areas of the site that were not explored with test borings. The subsurface conditions may also change at the boring locations over the passage of time.



0 25' 50'

APPROXIMATE SCALE

LEGEND:

A–A'	LINE OF CROSS-SECTION SEE FIGURE 2)
● ①	GEOTECHNICAL TEST BORING
⊕ ⑦	GEOTECHNICAL TEST BORING / PERCOLATION TEST BORING

NOTES:

1.) TEST BORING LOCATIONS ARE APPROXIMATE.

2.) BASE MAP DEVELOPED FROM THE "PRELIMINARY SITE PLAN" (SHEET PSP-19), REV. 7-28-16, PREPARED BY C-R-H-O.

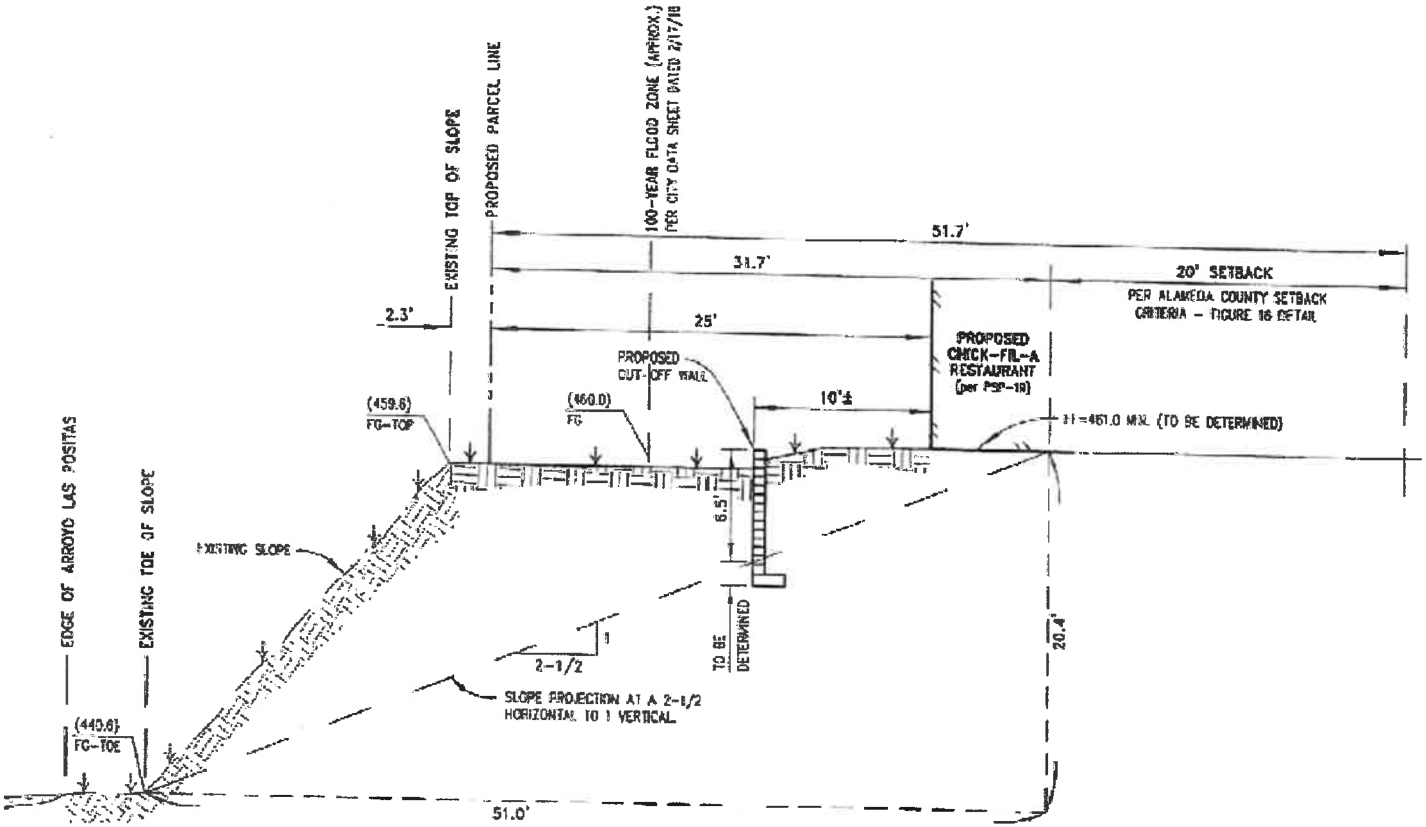
GILES ENGINEERING ASSOCIATES, INC.
1965 N. MAIN STREET
ORANGE, CA 92865 (714)279-0817

FIGURE 1
TEST BORING LOCATION PLAN
PROPOSED CHICK-FIL-A RESTAURANT NO. 03805
LIVERMORE & 580 FSU
SWC, N. LIVERMORE AVENUE AND I-580 FREEWAY
LIVERMORE, CALIFORNIA

DESIGNED	DRAWN	SCALE	DATE	REVISED
ELG	JSZ	approx. 1"=50'	09-12-16	--

PROJECT NO.: 2G-1606012

CAD No. 2g1606012-blp



GILES ENGINEERING ASSOCIATES, INC.
1965 N. MAIN STREET
ORANGE, CA 92865 (714)279-0817

FIGURE 2
CROSS-SECTION A-A'
PROPOSED CHICK-FIL-A RESTAURANT NO. 03805
LIVERMORE & 580 FSU
SWC, N. LIVERMORE AVENUE AND I-580 FREEWAY
LIVERMORE, CALIFORNIA

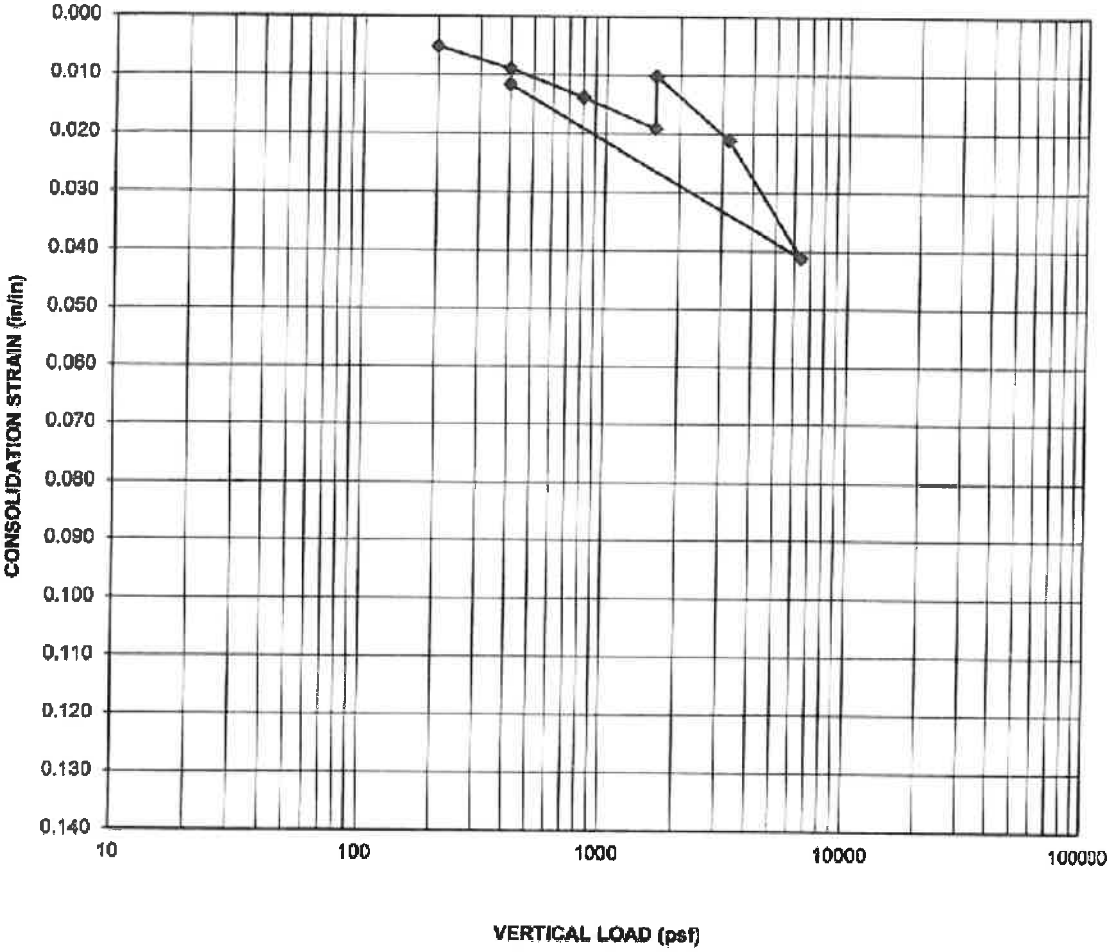
DESIGNED	DRAWN	SCALE	DATE	REVISED
ELG	JSZ	approx. 1"=10'	09-12-16	--

PROJECT NO.: 2G-1606012
CAD No. 2g1606012-cs1

NOTES:

1.) BASE MAP DEVELOPED FROM THE "SECTION A-A" PRINT DATE 8-26-16, PREPARED BY JOSEPH G. TRUXAW AND ASSOCIATES.

CONSOLIDATION / COLLAPSE TEST ASTM D2435/ASTM D5333



Classification	Sandy Clay
Boring No.	B-4
Sample No.	2-CS
Depth (ft.)	3.5 - 5.0
Elevation	
Liquid Limit	
Plastic Limit	
Specimen Diameter (in.)	2.42
Initial Specimen Thickness (in.)	1.00

Initial Moisture Content (%)	16.1
Final Moisture Content (%)	23.7
Natural Density (pcf)	121.0
Initial Dry Density (pcf)	104.2
Final Dry Density (pcf)	105.4
Swell at 1600 psf	0.89%

Sample inundated at 1600 psf pressure

Project:	CFA Livermore
Client:	Chick-fil-a Inc.
Project No.:	2G-1606012
Figure No.:	3

GILES ENGINEERING ASSOCIATES, INC.

-GEOTECHNICAL, ENVIRONMENTAL, AND CONSTRUCTION MATERIALS-
1965 NORTH MAIN STREET, ORANGE, CALIFORNIA
OFFICE: 714-279-0817 FAX : 714-279-9687

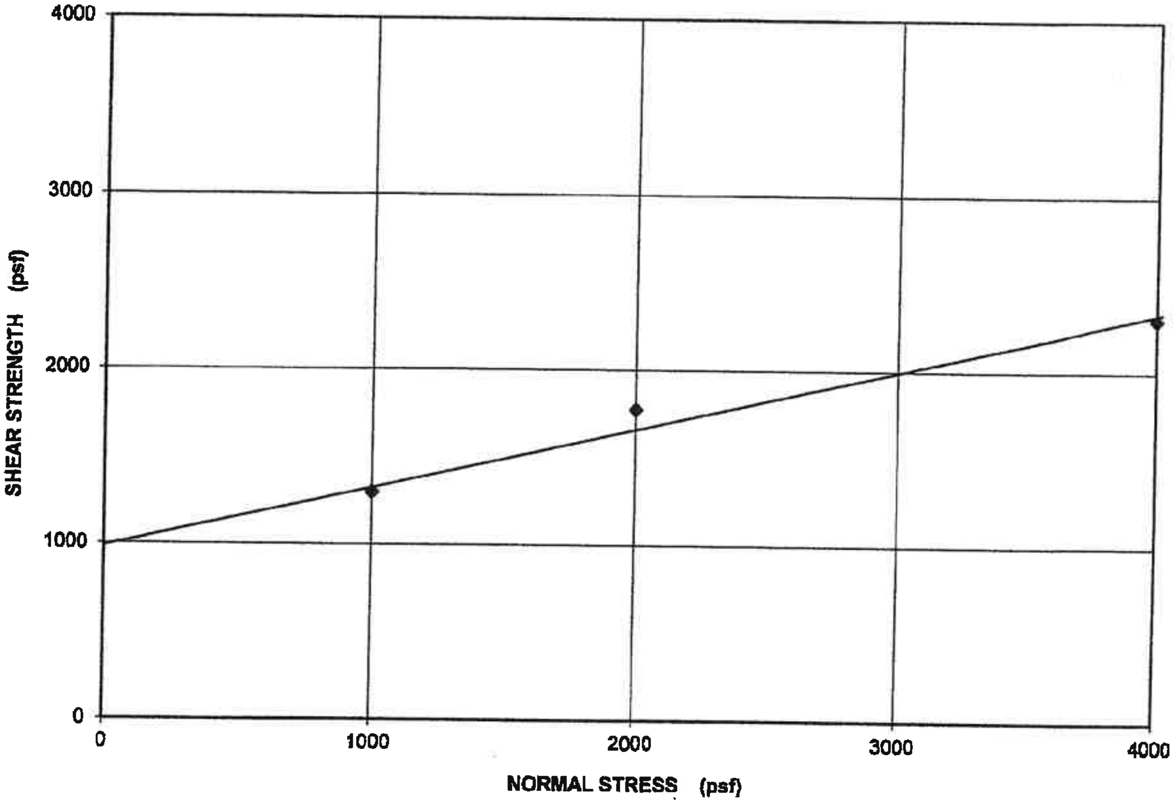
Giles Engineering Associates, Inc.

GEOTECHNICAL, ENVIRONMENTAL AND CONSTRUCTION MATERIALS CONSULTANTS

1965 N. MAIN STREET, ORANGE, CA 92865

OFFICE: 714-279-0817 FAX: 714-279-9887

UNDISTURBED DIRECT SHEAR TEST (ASTM D3080)



Classification Sandy Clay

Boring No. B-2 Depth 3.5 - 5.0 ft

Initial Specimen Properties:

Diameter (in.)	2.4
Height (in.)	1.0
Moisture Content (%)	12.3
Remolded Dry Density (pcf)	
Dry Density (pcf)	111.3
LL	
PL	
Ultimate - C (psi/psf)	1000
Ultimate - PHI (degrees)	18

PROJECT: CFA#3805, Livermore

CLIENT: chick-fil-A

PROJECT NO.: 2G-1606012

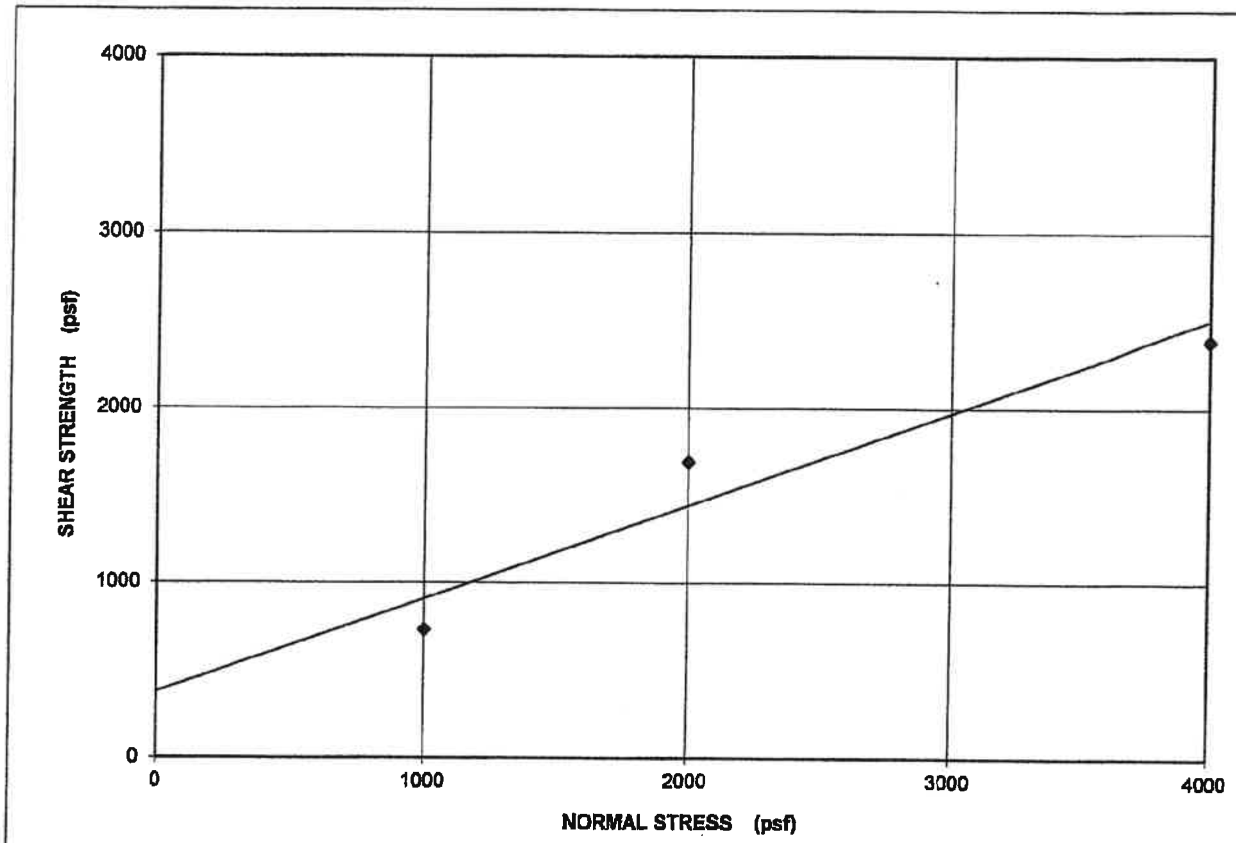
FIGURE NO. 4

Giles Engineering Associates, Inc.

GEOTECHNICAL, ENVIRONMENTAL AND CONSTRUCTION MATERIALS CONSULTANTS

1965 N. MAIN STREET, ORANGE, CA 92865
OFFICE: 714-279-0817 FAX: 714-279-9687

UNDISTURBED DIRECT SHEAR TEST (ASTM D3080)



Classification Silty Sand

Boring No. B-4 Depth 10.0 - 11.5 feet

		Initial Specimen Properties:	
		Diameter (in.)	2.4
		Height (in.)	1.0
PROJECT:	CFA#3805, Livermore	Moisture Content (%)	4.6
		Remolded Dry Density (pcf)	
CLIENT:	chick-fil-A	Dry Density (pcf)	120.6
		LL	
PROJECT NO.:	2G-1606012	PL	
		Ultimate - C (psi/psf)	390
FIGURE NO.	5	Ultimate - PHI (degrees)	28

BORING NO. & LOCATION: B-1	# TEST BORING LOG	**GILES ENGINEERING ASSOCIATES, INC.**
SURFACE ELEVATION: 459.5 feet	PROPOSED CHICK-FIL-A RESTAURANT #3805	
COMPLETION DATE: 08/17/16	I-580 & LIVERMORE AVENUE LIVERMORE, CA	
FIELD REP: JOSEPH HUYNH	PROJECT NO: 2G-1606012	

MATERIAL DESCRIPTION	**Depth (ft)**	**Elevation**	**Sample No. & Type**	**N**	**Q_u (tsf)**	**Q_p (tsf)**	**Q_s (tsf)**	**W (%)**	**PID**	**NOTES**
Light to Dark Sandy Clay to Clayey fine Sand - Moist			1 SS	15				18		LL=80 PL=30 PI=48
			2 SS	11				11		EI=54 (Medium)
Brown Silty fine to medium Sand to Light Gray Clayey Sand - Damp	10	450	3 SS	29				6		P_{200}=15%
			4 SS	30				6		
Bluish Gray Clayey fine to coarse Sand to Light Brown Clayey Sand to Sandy Clay, some Gravel, Possible Cobbles - Moist	20	440	5 SS	41				14		P_{200}=24%
			6 SS	74				12		
Bluish Gray Silty fine to coarse Sand to Clayey Sand, some Gravel, Possible Cobbles - Moist	∇ 30	430	7 SS	40/3"*				14		P_{200}=15%
			8 SS	50/2.5"*				15		
	40	420	9 SS	50/2"*				17		P_{200}=39%
			10 SS	50/6"*				9		
	50	410	11 SS	50/6"*				10		P_{200}=13%

Groundwater encountered at 30 feet
Boring Terminated at about 51.5 feet (EL. 408')

	Water Observation Data	**Remarks:**
∇	Water Encountered During Drilling: 30 ft.	SS = Standard Penetration Test
▼	Water Level At End of Drilling:	*N-Value Possibly influenced by Possible Cobbles
▨	Cave Depth At End of Drilling:	
▼	Water Level After Drilling:	
▨	Cave Depth After Drilling:	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION: B-2	
SURFACE ELEVATION: 460 feet	
COMPLETION DATE: 08/17/16	
FIELD REP: JOSEPH HUYNH	

TEST BORING LOG

PROPOSED CHICK-FIL-A RESTAURANT #3805

I-580 & LIVERMORE AVENUE
LIVERMORE, CA

PROJECT NO: 2G-1606012

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Light Brown to Gray Silty fine to medium Sand to Clayey Sand - Dry to Moist			1 SS	12				18		
Brown Silty fine to medium Sand, some Clay, some Gravel - Moist to Damp			2 CS	34				12		Dd=111.3 pcf
	5	455								
			3 CS	68				6		Dd=112.1 pcf
Light Brown fine to medium Sand, little Silt and Clay, some Gravel - Damp	10	450								
			4 CS	62				3		Dd=114.6 pcf P_{200}=9%
	15	445								
			5 SS	46				4		

No groundwater encountered
Boring Terminated at about 16.5 feet (EL. 443.5')

	Water Observation Data
▽	Water Encountered During Drilling: None
▼	Water Level At End of Drilling:
▨	Cave Depth At End of Drilling:
▼	Water Level After Drilling:
▨	Cave Depth After Drilling:

Remarks:

CS = California Split Spoon

SS = Standard Penetration Test

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION:
B-3

SURFACE ELEVATION:
459.5 feet

COMPLETION DATE:
08/17/16

FIELD REP:
JOSEPH HUYNH

TEST BORING LOG

PROPOSED CHICK-FIL-A RESTAURANT #3805

I-580 & LIVERMORE AVENUE
LIVERMORE, CA

PROJECT NO: 2G-1606012

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Light Brown Silty to Clayey fine to medium Sand - Moist			1 SS	12				25		
	5	455	2 SS	12				16		
Yellowish Brown Silty fine Sand, some Clay - Moist			3 SS	17				10		
	10	450								
Brown Gray Silty fine Sand to fine Sand with some Gravel, Possible Cobbles - Damp			4 SS	60*				3		
	15	445	5 SS	50/6"*				6		

No groundwater encountered
Boring Terminated at about 16.5 feet (EL. 443')

	Water Observation Data
▽	Water Encountered During Drilling: None
▽	Water Level At End of Drilling:
▨	Cave Depth At End of Drilling:
▼	Water Level After Drilling:
▨	Cave Depth After Drilling:

Remarks:

SS = Standard Penetration Test

*N-Value Possibly Influenced by Possible Cobbles

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION:
B-4

SURFACE ELEVATION:
460.2 feet

COMPLETION DATE:
08/17/16

FIELD REP:
JOSEPH HUYNH

TEST BORING LOG

PROPOSED CHICK-FIL-A RESTAURANT #3805

I-580 & LIVERMORE AVENUE
LIVERMORE, CA

PROJECT NO: 2G-1606012

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Dark Brown Silty fine Sand to Clayey fine Sand - Damp to Moist		460	1 SS	12				10		
Light Brown Sandy Clay to Clayey fine Sand - Moist	5	455	2 CS	35				16		Dd=104.2 pcf
Light Brown to Gray Silty fine Sand, some Clay - Moist			3 CS	49				8		Dd=113.0 pcf
Brown fine to medium Sand, little Silt, some Gravel, Possible Cobbles - Damp	10	450	4 CS	50/4"*				5		Dd=120.6 pcf P_{200}=9%
	15	445	5 SS	50/6"*				4		

No groundwater encountered
Boring Terminated at about 16.5 feet (EL. 443.7')

	Water Observation Data
∇	Water Encountered During Drilling: None
▼	Water Level At End of Drilling:
▨	Cave Depth At End of Drilling:
▼	Water Level After Drilling:
▨	Cave Depth After Drilling:

Remarks:

CS = California Split Spoon

SS = Standard Penetration Test

*N-Value Possibly influenced by Possible Cobbles

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION: B-5	# TEST BORING LOG	**GILES ENGINEERING ASSOCIATES, INC.**
SURFACE ELEVATION: 459 feet	PROPOSED CHICK-FIL-A RESTAURANT #3805 I-580 & LIVERMORE AVENUE LIVERMORE, CA	
COMPLETION DATE: 08/17/16		
FIELD REP: JOSEPH HUYNH	PROJECT NO: 2G-1606012	

MATERIAL DESCRIPTION		**Depth (ft)**	**Elevation**	**Sample No. & Type**	**N**	**Q_u (tsf)**	**Q_p (tsf)**	**Q_s (tsf)**	**W (%)**	**PID**	**NOTES**
Light Gray to Black Clayey fine Sand to Sandy Clay - Moist	▨		457.5	1 SS	8				17		
	▨	2.5									
Light Brown Clayey fine Sand - Moist	▨		455.0	2 SS	8				18		
		5.0									

No groundwater encountered
Boring Terminated at about 5 feet (EL. 454')

	Water Observation Data	**Remarks:**
▽	Water Encountered During Drilling:	SS = Standard Penetration Test
▽	Water Level At End of Drilling:	
░	Cave Depth At End of Drilling:	
▼	Water Level After Drilling:	
▒	Cave Depth After Drilling:	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION:
B-6

SURFACE ELEVATION:
459.5 feet

COMPLETION DATE:
08/17/16

FIELD REP:
JOSEPH HUYNH

TEST BORING LOG

PROPOSED CHICK-FIL-A RESTAURANT #3805

I-580 & LIVERMORE AVENUE
LIVERMORE, CA

PROJECT NO: 2G-1606012

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION		Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Black Silty Clay, some fine Sand - Dry to Moist	▨										
	▨		457.5	1 SS	10				16		
	▨	2.5									
Light Brown fine Sandy Clay - Very Moist	▨		455.0	2 SS	11				30		
		5.0									

No groundwater encountered
Boring Terminated at about 5 feet (EL. 454.5')

Water Observation Data

▽ Water Encountered During Drilling: None
▼ Water Level At End of Drilling:
▨ Cave Depth At End of Drilling:
▼ Water Level After Drilling:
▨ Cave Depth After Drilling:

Remarks:

SS = Standard Penetration Test

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

GILES LOG REPORT 2G-1606012.GPJ GILES.GDT 9/12/16

BORING NO. & LOCATION:
B-7

SURFACE ELEVATION:
459.2 feet

COMPLETION DATE:
08/17/16

FIELD REP:
JOSEPH HUYNH

TEST BORING LOG

PROPOSED CHICK-FIL-A RESTAURANT #3805

I-580 & LIVERMORE AVENUE
LIVERMORE, CA

PROJECT NO: 2G-1606012

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION		Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_a (tsf)	W (%)	PID	NOTES
Dark Brown fine Sandy Clay to Clayey fine Sand, some Gravel - Dry to Moist	▨	2.5	457.5	1 SS	11				10		
Dark Brown fine Sandy Clay to Silty Clay - Very Moist	▨	5.0	455.0	2 SS	9				37		

No groundwater encountered
Boring Terminated at about 5 feet (EL. 454.2')

Water Observation Data

∇ Water Encountered During Drilling: None
∇ Water Level At End of Drilling:
▨ Cave Depth At End of Drilling:
▼ Water Level After Drilling:
▨ Cave Depth After Drilling:

Remarks:

SS = Standard Penetration Test

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION:
B-8

SURFACE ELEVATION:
459.8 feet

COMPLETION DATE:
08/17/16

FIELD REP:
JOSEPH HUYNH

TEST BORING LOG

PROPOSED CHICK-FIL-A RESTAURANT #3805

I-580 & LIVERMORE AVENUE
LIVERMORE, CA

PROJECT NO: 2G-1606012

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Dark Gray Clayey fine Sand to fine Sandy Clay - Moist to Very Moist	2.5	457.5	1 SS	9				23		
Light Yellowish Brown Clayey fine Sand, some pockets of Sandy Clay - Moist	5.0	455.0	2 SS	14				13		

No groundwater encountered
Boring Terminated at about 5 feet (EL. 454.8')

	Water Observation Data	Remarks:
▽	Water Encountered During Drilling: None	SS = Standard Penetration Test
▼	Water Level At End of Drilling:	
▨	Cave Depth At End of Drilling:	
▼	Water Level After Drilling:	
▨	Cave Depth After Drilling:	

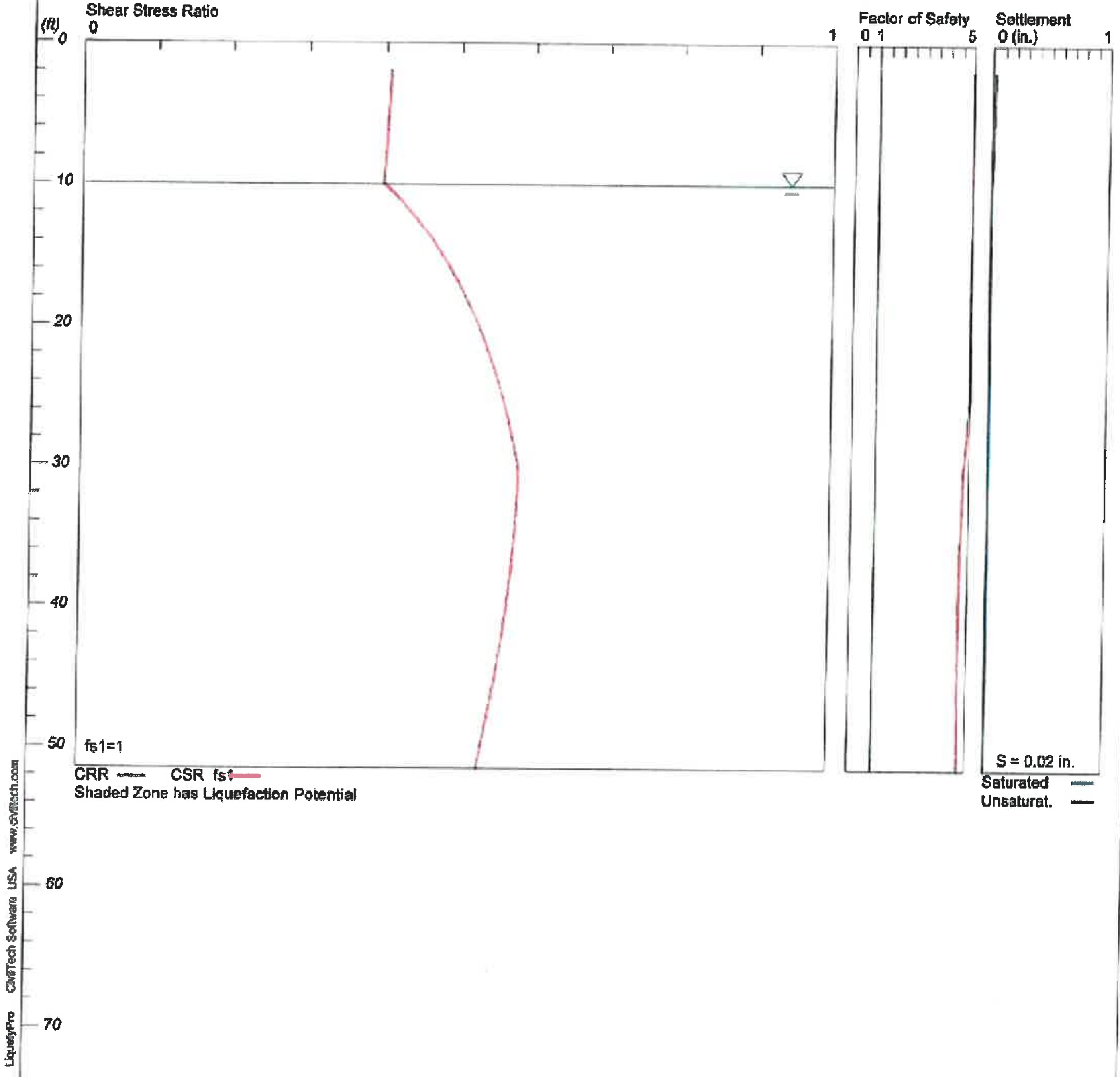
Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

LIQUEFACTION ANALYSIS

CFA #3805, Livermore, CA

Hole No.=B-1   Water Depth=10 ft

Magnitude=6.58
Acceleration=0.628g



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*************************************************************************************
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                                          LIQUEFACTION ANALYSIS SUMMARY

                                         Copyright by CivilTech Software
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*******************
       Font: Courier New, Regular, Size 8 is recommended for this report.
       Licensed to ,    9/1/2016         8:51:11 AM

       Input File Name: P:\Edgar Gatus\2G-1606012, CFA #3805, Livermore, CA\B-1.liq
       Title:  CFA #3805, Livermore, CA
       Subtitle:  2G-1606012

       Surface Elev.=
       Hole No.=B-1
       Depth of Hole= 51.50 ft
       Water Table during Earthquake= 10.00 ft
       Water Table during In-Situ Testing= 302.00 ft
       Max. Acceleration= 0.63 g
       Earthquake Magnitude= 6.58

Input Data:
       Surface Elev.=
       Hole No.=B-1
       Depth of Hole=51.50 ft
       Water Table during Earthquake= 10.00 ft
       Water Table during In-Situ Testing= 302.00 ft
       Max. Acceleration=0.63 g
       Earthquake Magnitude=6.58
       No-Liquefiable Soils:   CL, OL are Non-Liq. Soil

       1. SPT or BPT Calculation.
       2. Settlement Analysis Method: Tokimatsu/Seed
       3. Fines Correction for Liquefaction: Idriss/Seed
       4. Fine Correction for Settlement: During Liquefaction*
       5. Settlement Calculation in: All zones*
       6. Hammer Energy Ratio,                                 Ce = 1.25
       7. Borehole Diameter,                                       Cb= 1
       8. Sampling Method,                                         Cs= 1.2
       9. User request factor of safety (apply to CSR) ,    User= 1.1
          Plot one CSR curve (fs1=1)
       10. Use Curve Smoothing: Yes*
       * Recommended Options

       In-Situ Test Data:
       Depth   SPT     gamma    Fines
       ft              pcf      %
       ____________________________________
       2.00    15.00   120.00   80.00
       5.00    11.00   120.00   80.00
       10.00   29.00   120.00   15.00
       15.00   30.00   120.00   15.00
       20.00   41.00   120.00   24.00
       25.00   74.00   120.00   24.00
       30.00   40.00   120.00   15.00
       35.00   50.00   120.00   15.00
       40.00   50.00   120.00   39.00
       45.00   50.00   120.00   39.00
```

```
        50.00    50.00    120.00   13.00
        _____________________________________

Output Results:
        Settlement of Saturated Sands=0.00 in.
        Settlement of Unsaturated Sands=0.02 in.
        Total Settlement of Saturated and Unsaturated Sands=0.02 in.
        Differential Settlement=0.010 to 0.014 in.

        Depth    CRRm     CSRfs    F.S.     S_sat.   S_dry    S_all
        ft                                  in.      in.      in.
        _____________________________________________________________
        2.00     2.79     0.41     5.00     0.00     0.02     0.02
        2.50     2.79     0.41     5.00     0.00     0.02     0.02
        3.00     2.79     0.41     5.00     0.00     0.02     0.02
        3.50     2.79     0.40     5.00     0.00     0.02     0.02
        4.00     2.79     0.40     5.00     0.00     0.02     0.02
        4.50     2.79     0.40     5.00     0.00     0.01     0.01
        5.00     2.79     0.40     5.00     0.00     0.01     0.01
        5.50     2.79     0.40     5.00     0.00     0.01     0.01
        6.00     2.79     0.40     5.00     0.00     0.01     0.01
        6.50     2.79     0.40     5.00     0.00     0.01     0.01
        7.00     2.79     0.40     5.00     0.00     0.01     0.01
        7.50     2.79     0.40     5.00     0.00     0.01     0.01
        8.00     2.79     0.40     5.00     0.00     0.00     0.00
        8.50     2.79     0.40     5.00     0.00     0.00     0.00
        9.00     2.79     0.40     5.00     0.00     0.00     0.00
        9.50     2.79     0.40     5.00     0.00     0.00     0.00
        10.00    2.79     0.40     5.00     0.00     0.00     0.00
        10.50    2.79     0.41     5.00     0.00     0.00     0.00
        11.00    2.79     0.42     5.00     0.00     0.00     0.00
        11.50    2.79     0.43     5.00     0.00     0.00     0.00
        12.00    2.79     0.43     5.00     0.00     0.00     0.00
        12.50    2.79     0.44     5.00     0.00     0.00     0.00
        13.00    2.79     0.45     5.00     0.00     0.00     0.00
        13.50    2.79     0.46     5.00     0.00     0.00     0.00
        14.00    2.79     0.46     5.00     0.00     0.00     0.00
        14.50    2.79     0.47     5.00     0.00     0.00     0.00
        15.00    2.79     0.48     5.00     0.00     0.00     0.00
        15.50    2.79     0.48     5.00     0.00     0.00     0.00
        16.00    2.79     0.49     5.00     0.00     0.00     0.00
        16.50    2.79     0.49     5.00     0.00     0.00     0.00
        17.00    2.79     0.50     5.00     0.00     0.00     0.00
        17.50    2.79     0.50     5.00     0.00     0.00     0.00
        18.00    2.79     0.51     5.00     0.00     0.00     0.00
        18.50    2.79     0.51     5.00     0.00     0.00     0.00
        19.00    2.79     0.52     5.00     0.00     0.00     0.00
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        26.50    2.79     0.57     4.94     0.00     0.00     0.00
        27.00    2.79     0.57     4.91     0.00     0.00     0.00
```

```
  27.50    2.81    0.57    4.92    0.00    0.00    0.00
  28.00    2.80    0.57    4.88    0.00    0.00    0.00
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  31.50    2.74    0.58    4.72    0.00    0.00    0.00
  32.00    2.73    0.58    4.71    0.00    0.00    0.00
  32.50    2.73    0.58    4.70    0.00    0.00    0.00
  33.00    2.72    0.58    4.69    0.00    0.00    0.00
  33.50    2.71    0.58    4.68    0.00    0.00    0.00
  34.00    2.70    0.58    4.67    0.00    0.00    0.00
  34.50    2.70    0.58    4.66    0.00    0.00    0.00
  35.00    2.69    0.58    4.66    0.00    0.00    0.00
  35.50    2.68    0.58    4.65    0.00    0.00    0.00
  36.00    2.67    0.58    4.64    0.00    0.00    0.00
  36.50    2.67    0.58    4.64    0.00    0.00    0.00
  37.00    2.66    0.57    4.63    0.00    0.00    0.00
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  39.00    2.63    0.57    4.61    0.00    0.00    0.00
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  41.00    2.60    0.57    4.60    0.00    0.00    0.00
  41.50    2.60    0.56    4.60    0.00    0.00    0.00
  42.00    2.59    0.56    4.60    0.00    0.00    0.00
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  46.00    2.54    0.55    4.61    0.00    0.00    0.00
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  50.00    2.49    0.54    4.64    0.00    0.00    0.00
  50.50    2.48    0.53    4.64    0.00    0.00    0.00
  51.00    2.47    0.53    4.65    0.00    0.00    0.00
  51.50    2.47    0.53    4.65    0.00    0.00    0.00
_______________________________________________________________________
  * F.S.<1, Liquefaction Potential Zone
  (F.S. is limited to 5,  CRR is limited to 2,    CSR is limited to 2)

  Units:  Unit: qc, fs, Stress or Pressure = atm (1.0581tsf); Unit Weight =
pcf; Depth = ft; Settlement = in.

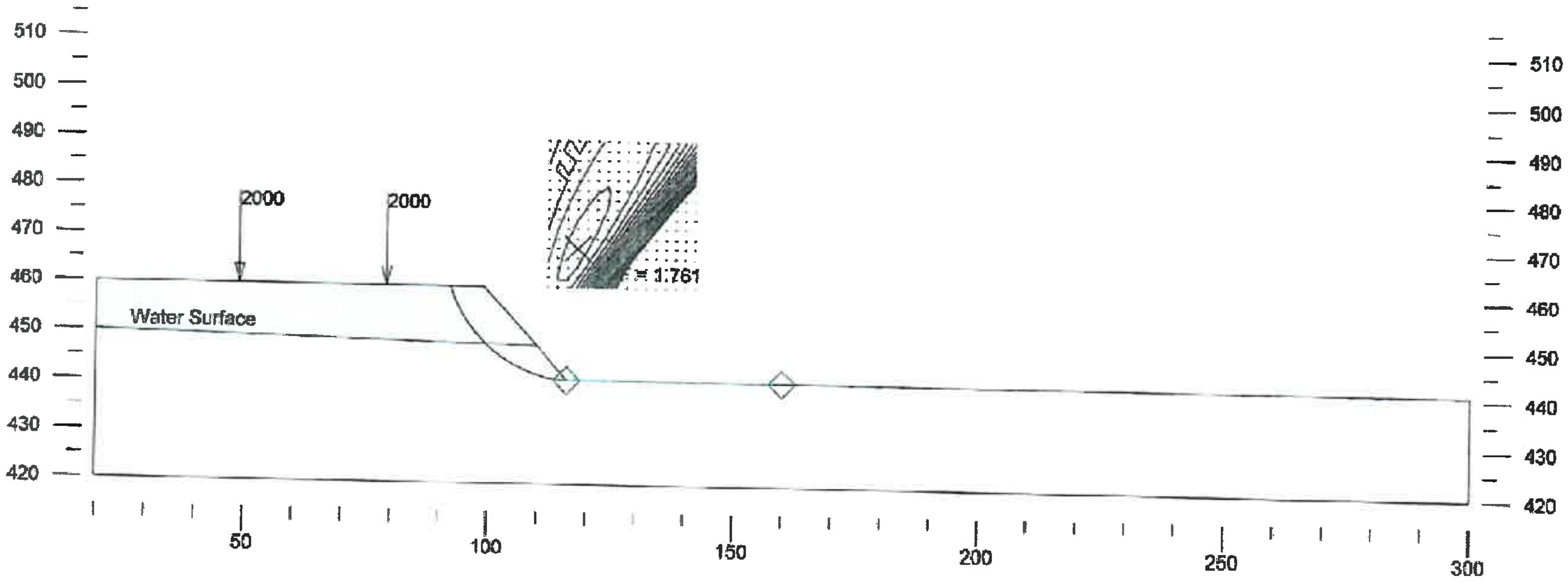
_____________________________________________________________________________
  1 atm (atmosphere) = 1 tsf (ton/ft2)
  CRRm             Cyclic resistance ratio from soils
  CSRsf            Cyclic stress ratio induced by a given earthquake (with user
request factor of safety)
  F.S.             Factor of Safety against liquefaction, F.S.=CRRm/CSRsf
  S_sat            Settlement from saturated sands
```

S_dry	Settlement from Unsaturated Sands
S_all	Total Settlement from Saturated and Unsaturated Sands
NoLiq	No-Liquefy Soils

	Gamma pcf	C psf	Phi deg	Piezo Surf.
Sandy Clay	125	500	18	0
Silty/Clayey Sand	130	300	28	0

Giles Engineering Associates Inc.

CFA Livermore-Preliminary Run
8/23/16



9/8/2016 2:42:37 PM P:\Bob Russell\Slope Info\CFA Livermore6.gsl Giles Engineering Associates Inc. F = 1.761

```
P:\Bob Russell\Slope Info\CFA Livermore6.gsl  09-08-2016 at 14:42:26


Minimum Bishop Factor of Safety this run:

    118.00    468.00    27.07    5    25      1.7606     0

Lowest results with no MAlpha value less than 0.3:        Min MAlpha

    118.00    468.00    27.07    5    25      1.7606   .5185

    120.00    472.00    31.26    5    26      1.7659   .5855

    118.00    470.00    29.07    5    28      1.7663   .5515

    120.00    470.00    29.27    5    25      1.7695   .5504

    116.00    464.00    23.00    5    25      1.7699   .4307
Lowest results with one MAlpha value less than 0.3:       Min MAlpha

    118.00    460.00    19.10    6    19      2.0083   .2141

    114.00    460.00    21.85    5    36      2.0426   .2966

    120.00    462.00    21.38    6    19      2.1165   .2925

    112.00    460.00    22.91    5    39      2.1380   .2821

    118.00    460.00    20.18    5    30      2.2306   .2656
Lowest results with two MAlpha values less than 0.3:      Min MAlpha

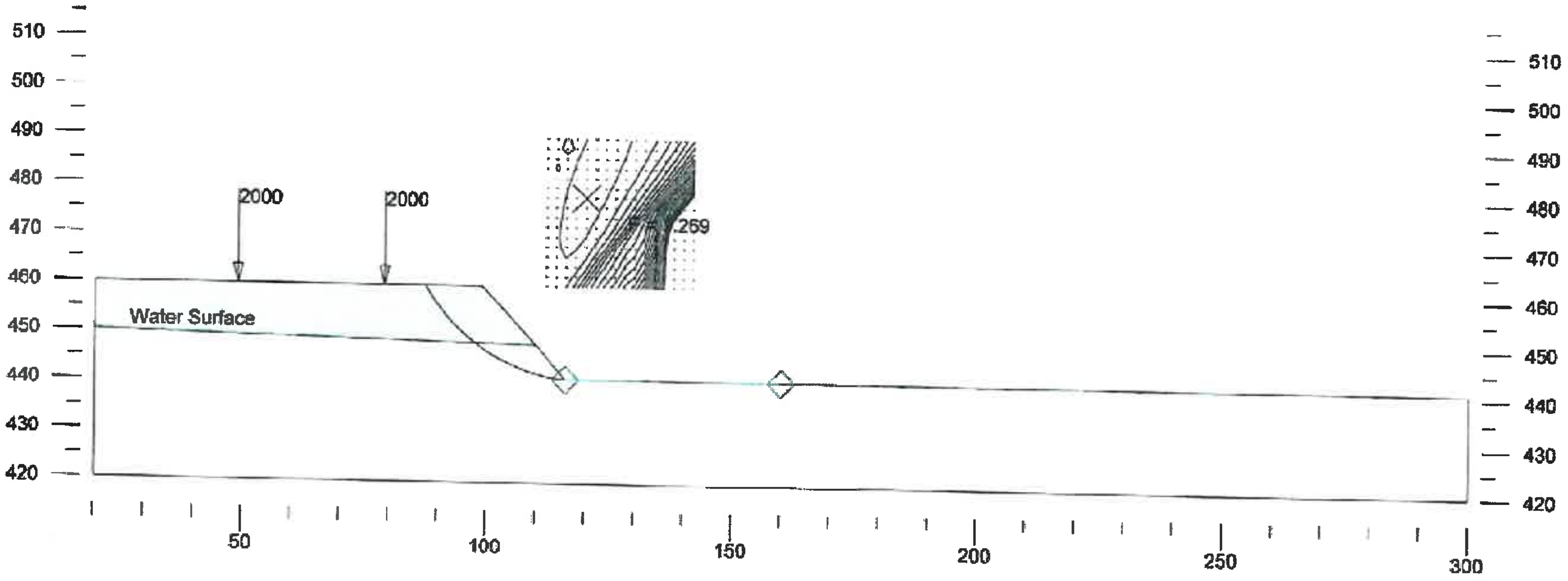
    130.00    464.00    31.28    4    30      5.1497   .1933
```

	Gamma pcf	C psf	Phi deg	Piezo Surf.
Sandy Clay	125	500	18	0
Silty/Clayey Sand	130	300	28	0

Seismic coefficient = 0.25

Giles Engineering Associates Inc.

CFA Livermore-Preliminary Run
8/23/16



9/8/2016 2:54:06 PM P:\Bob Russell\Slope Info\CFA Livermore6.gsl Giles Engineering Associates Inc. F = 1.269

```
P:\Bob Russell\Slope Info\CFA Livermore6.gsl  09-08-2016 at 14:43:47


Minimum Bishop Factor of Safety this run:

     120.00    478.00     37.22     3      31       1.2689      0

Lowest results with no MAlpha value less than 0.3:          Min MAlpha

     120.00    478.00     37.22     3      31       1.2689    .7259

     120.00    476.00     35.23     3      29       1.2700    .7039

     122.00    482.00     41.44     3      32       1.2703    .7618

     120.00    480.00     39.20     3      32       1.2704    .7468

     122.00    480.00     39.46     3      30       1.2709    .7439
Lowest results with one MAlpha value less than 0.3:          Min MAlpha

     118.00    460.00     19.10     5      19       1.5414    .2631

     116.00    460.00     32.53     4      33       1.9676    .2904

     124.00    460.00     26.45     4      48       2.1456    .2677

     116.00    462.00     40.99     3      41       2.1501    .2911

     124.00    460.00     33.18     4      34       2.1726    .2742
Lowest results with two MAlpha values less than 0.3:         Min MAlpha

     114.00    460.00     41.77     3      42       2.3098    .2746

     118.00    462.00     46.96     3      48       2.3794    .2890

     112.00    460.00     43.56     4      44       2.3867    .2638

     122.00    460.00     42.49     4      43       2.4188    .2680

     120.00    460.00     44.28     4      45       2.4267    .2611
```

APPENDIX B

FIELD PROCEDURES

The field operations were conducted in general accordance with the procedures recommended by the American Society for Testing and Materials (ASTM) designation D 420 entitled "Standard Guide for Sampling Rock and Rock" and/or other relevant specifications. Soil samples were preserved and transported to *Giles'* laboratory in general accordance with the procedures recommended by ASTM designation D 4220 entitled "Standard Practice for Preserving and Transporting Soil Samples." Brief descriptions of the sampling, testing and field procedures commonly performed by *Giles* are provided herein.

GENERAL FIELD PROCEDURES

Test Boring Elevations

The ground surface elevations reported on the Test Boring Logs are referenced to the assumed benchmark shown on the Boring Location Plan (Figure 1). Unless otherwise noted, the elevations were determined with a conventional hand-level and are accurate to within about 1 foot.

Test Boring Locations

The test borings were located on-site based on the existing site features and/or apparent property lines. Dimensions illustrating the approximate boring locations are reported on the Boring Location Plan (Figure 1).

Water Level Measurement

The water levels reported on the Test Boring Logs represent the depth of "free" water encountered during drilling and/or after the drilling tools were removed from the borehole. Water levels measured within a granular (sand and gravel) soil profile are typically indicative of the water table elevation. It is usually not possible to accurately identify the water table elevation with cohesive (clayey) soils, since the rate of seepage is slow. The water table elevation within cohesive soils must therefore be determined over a period of time with groundwater observation wells.

It must be recognized that the water table may fluctuate seasonally and during periods of heavy precipitation. Depending on the subsurface conditions, water may also become perched above the water table, especially during wet periods.

Borehole Backfilling Procedures

Each borehole was backfilled upon completion of the field operations. If potential contamination was encountered, and/or if required by state or local regulations, boreholes were backfilled with an "impervious" material (such as bentonite slurry). Borings that penetrated pavements, sidewalks, etc. were "capped" with Portland Cement concrete, asphaltic concrete, or a similar surface material. It must, however, be recognized that the backfill material may settle, and the surface cap may subside, over a period of time. Further backfilling and/or re-surfacing by *Giles'* client or the property owner may be required.

FIELD SAMPLING AND TESTING PROCEDURES

Auger Sampling (AU)

Soil samples are removed from the auger flights as an auger is withdrawn above the ground surface. Such samples are used to determine general soil types and identify approximate soil stratifications. Auger samples are highly disturbed and are therefore not typically used for geotechnical strength testing.

Split-Barrel Sampling (SS) – (ASTM D-1586)

A split-barrel sampler with a 2-inch outside diameter is driven into the subsoil with a 140-pound hammer free-falling a vertical distance of 30 inches. The summation of hammer-blows required to drive the sampler the final 12-inches of an 18-inch sample interval is defined as the "Standard Penetration Resistance" or N-value is an index of the relative density of granular soils and the comparative consistency of cohesive soils. A soil sample is collected from each SPT interval.

Shelby Tube Sampling (ST) – (ASTM D-1587)

A relatively undisturbed soil sample is collected by hydraulically advancing a thin-walled Shelby Tube sampler into a soil mass. Shelby Tubes have a sharp cutting edge and are commonly 2 to 5 inches in diameter.

Bulk Sample (BS)

A relatively large volume of soils is collected with a shovel or other manually-operated tool. The sample is typically transported to *Giles'* materials laboratory in a sealed bag or bucket.

Dynamic Cone Penetration Test (DC) – (ASTM STP 399)

This test is conducted by driving a 1.5-inch-diameter cone into the subsoil using a 15-pound steel ring (hammer), free-falling a vertical distance of 20 inches. The number of hammer-blows required to drive the cone 1¾ inches is an indication of the soil strength and density, and is defined as "N". The Dynamic Cone Penetration test is commonly conducted in hand auger borings, test pits and within excavated trenches.

- Continued -

Ring-Lined Barrel Sampling – (ASTM D 3550)

In this procedure, a ring-lined barrel sampler is used to collect soil samples for classification and laboratory testing. This method provides samples that fit directly into laboratory test instruments without additional handling/disturbance.

Sampling and Testing Procedures

The field testing and sampling operations were conducted in general accordance with the procedures recommended by the American Society for Testing and Materials (ASTM) and/or other relevant specifications. Results of the field testing (i.e. N-values) are reported on the Test Boring Logs. Explanations of the terms and symbols shown on the logs are provided on the appendix enclosure entitled "General Notes".

APPENDIX C

LABORATORY TESTING AND CLASSIFICATION

The laboratory testing was conducted under the supervision of a geotechnical engineer in accordance with the procedures recommended by the American Society for Testing and Materials (ASTM) and/or other relevant specifications. Brief descriptions of laboratory tests commonly performed by *Giles* are provided herein.

LABORATORY TESTING AND CLASSIFICATION

Photoionization Detector (PID)

In this procedure, soil samples are "scanned" in *Giles'* analytical laboratory using a Photoionization Detector (PID). The instrument is equipped with an 11.7 eV lamp calibrated to a Benzene Standard and is capable of detecting a minute concentration of **certain** Volatile Organic Compound (VOC) vapors, such as those commonly associated with petroleum products and some solvents. Results of the PID analysis are expressed in HNu (manufacturer's) units rather than actual concentration.

Moisture Content (w) (ASTM D 2216)

Moisture content is defined as the ratio of the weight of water contained within a soil sample to the weight of the dry solids within the sample. Moisture content is expressed as a percentage.

Unconfined Compressive Strength (qu) (ASTM D 2166)

An axial load is applied at a uniform rate to a cylindrical soil sample. The unconfined compressive strength is the maximum stress obtained or the stress when 15% axial strain is reached, whichever occurs first.

Calibrated Penetrometer Resistance (qp)

The small, cylindrical tip of a hand-held penetrometer is pressed into a soil sample to a prescribed depth to measure the soils capacity to resist penetration. This test is used to evaluate unconfined compressive strength.

Vane-Shear Strength (qs)

The blades of a vane are inserted into the flat surface of a soil sample and the vane is rotated until failure occurs. The maximum shear resistance measured immediately prior to failure is taken as the vane-shear strength.

Loss-on-Ignition (ASTM D 2974; Method C)

The Loss-on-Ignition (L.O.I.) test is used to determine the organic content of a soil sample. The procedure is conducted by heating a dry soil sample to 440°C in order to burn-off or "ash" organic matter present within the sample. The L.O.I. value is the ratio of the weight loss due to ignition compared to the initial weight of the dry sample. L.O.I. is expressed as a percentage.

Particle Size Distribution (ASTB D 421, D 422, and D 1140)

This test is performed to determine the distribution of specific particle sizes (diameters) within a soil sample. The distribution of coarse-grained soil particles (sand and gravel) is determined from a "sieve analysis," which is conducted by passing the sample through a series of nested sieves. The distribution of fine-grained soil particles (silt and clay) is determined from a "hydrometer analysis" which is based on the sedimentation of particles suspended in water.

Consolidation Test (ASTM D 2435)

In this procedure, a series of cumulative vertical loads are applied to a small, laterally confined soil sample. During each load increment, vertical compression (consolidation) of the sample is measured over a period of time. Results of this test are used to estimate settlement and time rate of settlement.

Classification of Samples

Each soil sample was visually-manually classified, based on texture and plasticity, in general accordance with the Unified Soil Classification System (ASTM D-2488-75). The classifications are reported on the Test Boring Logs.

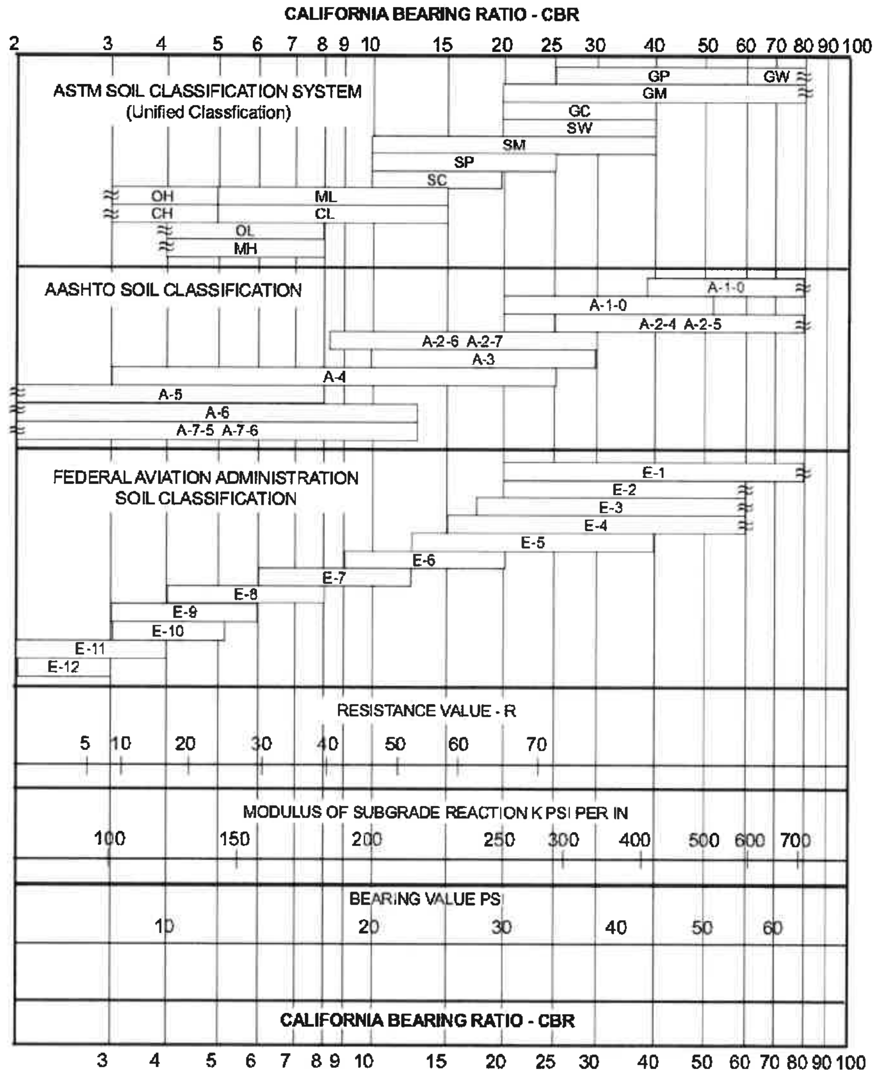
Laboratory Testing

The laboratory testing operations were conducted in general accordance with the procedures recommended by the American Society for Testing and Materials (ASTM) and/or other relevant specifications. Results of the laboratory tests are provided on the Test Boring Logs or other appendix enclosures. Explanation of the terms and symbols used on the logs is provided on the appendix enclosure entitled "General Notes."

California Bearing Ratio (CBR) Test ASTM D-1833

The CBR test is used for evaluation of a soil subgrade for pavement design. The test consists of measuring the force required for a 3-square-inch cylindrical piston to penetrate 0.1 or 0.2 inch into a compacted soil sample. The result is expressed as a percent of force required to penetrate a standard compacted crushed stone.

Unless a CBR test has been specifically requested by the client, the CBR is estimated from published charts, based on soil classification and strength characteristics. A typical correlation chart is below.



APPENDIX D

GENERAL INFORMATION

GUIDE SPECIFICATIONS FOR SUBGRADE AND PREPARATION FOR FILL, FOUNDATION, FLOOR SLAB AND PAVEMENT SUPPORT; AND SELECTION, PLACEMENT AND COMPACTION OF FILL SOILS USING MODIFIED PROCTOR PROCEDURES

1. Construction monitoring and testing of subgrades and grades for fill, foundation, floor slab and pavement; and fill selection, placement and compaction shall be performed by an experienced soils engineer and/or his representatives.

2. All compacted fill, subgrades, and grades shall be (a) underlain by suitable bearing material, (b) free of all organic frozen, or other deleterious material, and (c) observed, tested and approved by qualified engineering personnel representing an experienced soils engineer. Preparation of subgrades after stripping vegetation, organic or other unsuitable materials shall consist of (a) proofrolling to detect soft, wet, yielding soils or other unstable materials that must be undercut, (b) scarifying top 6 to 8 inches, (c) moisture conditioning the soils as required, and (d) recompaction to same minimum in-situ density required for similar material indicated under Item 5. Note: Compaction requirements for pavement subgrade are higher than other areas. Weather and construction equipment may damage compacted fill surface and reworking and retesting may be necessary for proper performance.

3. In overexcavation and fill areas, the compacted fill must extend (a) a minimum 1 foot lateral distance beyond the exterior edge of the foundation at bearing grade or pavement at subgrade and down to compacted fill subgrade on a maximum 0.5(H):1(v) slope, (b) 1 foot above footing grade outside the building, and (c) to floor subgrade inside the building. Fill shall be placed and compacted on a 5(H):1(V) slope or must be stepped or benched as required to flatten if not specifically approved by qualified personnel under the direction of an experienced soils engineer.

4. The compacted fill materials shall be free of deleterious, organic, or frozen matter, shall contain no chemicals that may result in the material being classified as "contaminated", and shall be low-expansive with a maximum Liquid Limit (ASTM D-423) and Plasticity Index (ASTM D-424) of 30 and 15, respectively, unless specifically tested and found to have low expansive properties and approved by an experienced soils engineer. The top 12 inches of compacted fill should have a maximum 3 inch particle diameter and all underlying compacted fill a maximum 6 inch diameter unless specifically approved by an experienced soils engineer. All fill material must be tested and approved under the direction of an experienced soils engineer prior to placement. If the fill is to provide non-frost susceptible characteristics, it must be classified as a clean GW, GP, SW or SP per Unified Soils Classification System (ASTM D-2487).

5. For structural fill depths less than 20 feet, the density of the structural compacted fill and scarified subgrade and grades shall not be less than 90 percent of the maximum dry density as determined by Modified Proctor (ASTM D-1557) with the exception of the top 12 inches of pavement subgrade which shall have a minimum in-situ density of 95 percent of maximum dry density, or 5 percent higher than underlying structural fill materials. Where the structural fill depth is greater than 20 feet, the portion below 20 feet should have a minimum in-place density of 95 percent of its maximum dry density or 5 percent higher than the top 20 feet. Cohesive soils shall not vary by more than -1 to +3 percent moisture content and granular soil ±3 percent from the optimum when placed and compacted or recompacted, unless specifically recommended/approved by the soils engineer observing the placement and compaction. Cohesive soils with moderate to high expansion potentials (PI>15) should, however, be placed, compacted and maintained prior to construction at a 3±1 percent moisture content above optimum moisture content to limit future heave. Fill shall be placed in layers with a maximum loose thickness of 8 inches for foundations and 10 inches for floor slabs and pavements, unless specifically approved by the soils engineer taking into consideration the type of materials and compaction equipment being used. The compaction equipment should consist of suitable mechanical equipment specifically designed for soil compaction. Bulldozers or similar tracked vehicles are typically not suitable for compaction.

6. Excavation, filing, subgrade grade preparation shall be performed in a manner and sequence that will provide drainage at all times and proper control of erosion. Precipitation, springs, and seepage water encountered shall be pumped or drained to provide a suitable working platform. Springs or water seepage encountered during grade/foundation construction must be called to the soils engineer's attention immediately for possible construction procedure revision or inclusion of an underdrain system.

7. Non-structural fill adjacent to structural fill should typically be placed in unison to provide lateral support. Backfill along walls must be placed and compacted with care to ensure excessive unbalanced lateral pressures do not develop. The type of fill material placed adjacent to below grade walls (i.e. basement walls and retaining walls) must be properly tested and approved by an experienced soils engineer with consideration for the lateral pressure used in the wall design.

8. Wherever, in the opinion of the soils engineer or the Owner's Representatives, an unstable condition is being created either by cutting or filling, the work should not proceed into that area until an appropriate geotechnical exploration and analysis has been performed and the grading plan revised, if found necessary.

GENERAL COMMENTS

The soil samples obtained during the subsurface exploration will be retained for a period of thirty days. If no instructions are received, they will be disposed of at that time.

This report has been prepared exclusively for the client in order to aid in the evaluation of this property and to assist the architects and engineers in the design and preparation of the project plans and specifications. Copies of this report may be provided to contractor(s), with contract documents, to disclose information relative to this project. The report, however, has not been prepared to serve as the plans and specifications for actual construction without the appropriate interpretation by the project architect, structural engineer, and/or civil engineer. Reproduction and distribution of this report must be authorized by the client and *Giles*.

This report has been based on assumed conditions/characteristics of the proposed development where specific information was not available. It is recommended that the architect, civil engineer and structural engineer along with any other design professionals involved in this project carefully review these assumptions to ensure they are consistent with the actual planned development. When discrepancies exist, they should be brought to our attention to ensure they do not affect the conclusions and recommendations provided herein. The project plans and specifications may also be submitted to *Giles* for review to ensure that the geotechnical related conclusions and recommendations provided herein have been correctly interpreted.

The analysis of this site was based on a subsoil profile interpolated from a limited subsurface exploration. If the actual conditions encountered during construction vary from those indicated by the borings, *Giles* must be contacted immediately to determine if the conditions alter the recommendations contained herein.

The conclusions and recommendations presented in this report have been promulgated in accordance with generally accepted professional engineering practices in the field of geotechnical engineering. No other warranty is either expressed or implied.

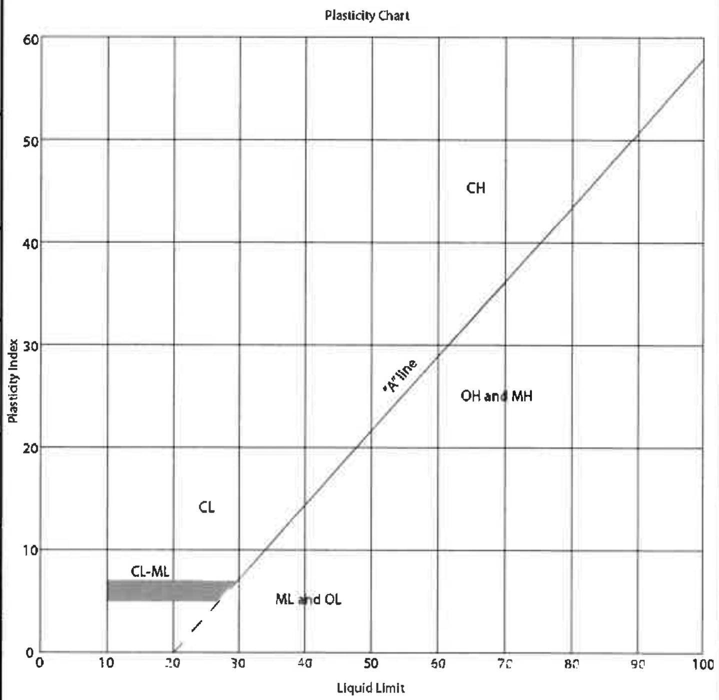
CHARACTERISTICS AND RATINGS OF UNIFIED SOIL SYSTEM CLASSES FOR SOIL CONSTRUCTION *

Class	Compaction Characteristics	Max. Dry Density Standard Proctor (pcf)	Compressibility and Expansion	Drainage and Permeability	Value as an Embankment Material	Value as Subgrade When Not Subject to Frost	Value as Base Course	Value as Temporary Pavement	
								With Dust Palliative	With Bituminous Treatment
GW	Good: tractor, rubber-tired, steel wheel or vibratory roller	125-135	Almost none	Good drainage, pervious	Very stable	Excellent	Good	Fair to poor	Excellent
GP	Good: tractor, rubber-tired, steel wheel or vibratory roller	115-125	Almost none	Good drainage, pervious	Reasonably stable	Excellent to good	Poor to fair	Poor	
GM	Good: rubber-tired or light sheepsfoot roller	120-135	Slight	Poor drainage, semipervious	Reasonably stable	Excellent to good	Fair to poor	Poor	Poor to fair
GC	Good to fair: rubber-tired or sheepsfoot roller	115-130	Slight	Poor drainage, impervious	Reasonably stable	Good	Good to fair **	Excellent	Excellent
SW	Good: tractor, rubber-tired or vibratory roller	110-130	Almost none	Good drainage, pervious	Very stable	Good	Fair to poor	Fair to poor	Good
SP	Good: tractor, rubber-tired or vibratory roller	100-120	Almost none	Good drainage, pervious	Reasonably stable when dense	Good to fair	Poor	Poor	Poor to fair
SM	Good: rubber-tired or sheepsfoot roller	110-125	Slight	Poor drainage, impervious	Reasonably stable when dense	Good to fair	Poor	Poor	Poor to fair
SC	Good to fair: rubber-tired or sheepsfoot roller	105-125	Slight to medium	Poor drainage, impervious	Reasonably stable	Good to fair	Fair to poor	Excellent	Excellent
ML	Good to poor: rubber-tired or sheepsfoot roller	95-120	Slight to medium	Poor drainage, impervious	Poor stability, high density required	Fair to poor	Not suitable	Poor	Poor
CL	Good to fair: sheepsfoot or rubber-tired roller	95-120	Medium	No drainage, impervious	Good stability	Fair to poor	Not suitable	Poor	Poor
OL	Fair to poor: sheepsfoot or rubber-tired roller	80-100	Medium to high	Poor drainage, impervious	Unstable, should not be used	Poor	Not suitable	Not suitable	Not suitable
MH	Fair to poor: sheepsfoot or rubber-tired roller	70-95	High	Poor drainage, impervious	Poor stability, should not be used	Poor	Not suitable	Very poor	Not suitable
CH	Fair to poor: sheepsfoot roller	80-105	Very high	No drainage, impervious	Fair stability, may soften on expansion	Poor to very poor	Not suitable	Very poor	Not suitable
OH	Fair to poor: sheepsfoot roller	65-100	High	No drainage, impervious	Unstable, should not be used	Very poor	Not suitable	Not suitable	Not suitable
Pt	Not suitable		Very high	Fair to poor drainage	Should not be used	Not suitable	Not suitable	Not suitable	Not suitable

* "The Unified Classification: Appendix A - Characteristics of Soil, Groups Pertaining to Roads and Airfields, and Appendix B - Characteristics of Soil Groups Pertaining to Embankments and Foundations," Technical Memorandum 357, U.S. Waterways Experiment Station, Vicksburg, 1953.

** Not suitable if subject to frost.

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

Major Divisions			*Group Symbols*		*Typical Names*	*Laboratory Classification Criteria*		
Coarse-grained soils (more than half of material is larger than No. 200 sieve size)	Gravels (More than half of coarse fraction is larger than No. 4 sieve size)	Clean gravels (little or no fines)	GW		Well-graded gravels, gravel-sand mixtures, little or no fines	Determine percentages of sand and gravel from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows: Less than 5 percent: GW, GP, SW, SP More than 12 percent: GM, GC, SM, SC 5 to 12 percent: *Borderline* cases requiring dual symbols[b]	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			GP		Poorly graded gravels, gravel-sand mixtrues, little or no fines		Not meeting all gradation requirements for GW	
		Gravels with fines (appreciable amount of fines)	GM[a]	d	Silty gravels, gravel-sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4	Limits plotting within shaded area, above "A" line with P.I. between 4 and 7 are *borderline* cases requiring use of dual symbols
				u				
			GC		Clayey gravels, gravel-sand-clay mixtures		Atterberg limits above "A" line or P.I. greater than 7	
	Sands (More than half of coarse fraction is smaller than No. 4 sieve size)	Clean sands (Little or no fines)	SW		Well-graded sands, gravelly sands, little or no fines		$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			SP		Poorly graded sands, gravelly sands, little or no fines		Not meeting all gradation requirements for SW	
		Sands with fines (Appreciable amount of fines)	SM[a]	d	Silty sands, sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4	Limits plotting within shaded area, above "A" line with P.I. between 4 and 7 are *borderline* cases requiring use of dual symbols
				u				
			SC		Clayey sands, sand-clay mixtures		Atterberg limits above "A" line or P.I. greater than 7	
Fine-grained soils (More than half 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	 Plasticity Chart		
			CL		Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty 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 sandy or silty soils, elastic silts			
			CH		Inorganic clays of high plasticity, fat clays			
			OH		Organic clays of medium to high plasticity, organic silts			
	Highly organic soils		Pt		Peat and other highly organic soils			

---

[a] Division of GM and SM groups into subdivisions of d and u are for roads and airfields only. Subdivision is based on Atterberg limits, suffix d used when L.L. is 28 or less and the P.I. is 6 or less; the suffix u is used when L.L. is greater than 28.

[b] Borderline classifications, used for soils possessing characteristics of two groups, are designated by combinations of group sympols. For example GW-GC, well-graded gravel-sand mixture with clay binder.

GENERAL NOTES

SAMPLE IDENTIFICATION

All samples are visually classified in general accordance with the Unified Soil Classification System (ASTM D-2487-75 or D-2488-75)

DESCRIPTIVE TERM (% BY DRY WEIGHT)

Trace:	1-10%
Little:	11-20%
Some:	21-35%
And/Adjective	36-50%

PARTICLE SIZE (DIAMETER)

Boulders:	8 inch and larger
Cobbles:	3 inch to 8 inch
Gravel:	coarse - ¾ to 3 inch
	fine – No. 4 (4.76 mm) to ¾ inch
Sand:	coarse – No. 4 (4.76 mm) to No. 10 (2.0 mm)
	medium – No. 10 (2.0 mm) to No. 40 (0.42 mm)
	fine – No. 40 (0.42 mm) to No. 200 (0.074 mm)
Silt:	No. 200 (0.074 mm) and smaller (non-plastic)
Clay:	No 200 (0.074 mm) and smaller (plastic)

SOIL PROPERTY SYMBOLS

Dd:	Dry Density (pcf)
LL:	Liquid Limit, percent
PL:	Plastic Limit, percent
PI:	Plasticity Index (LL-PL)
LOI:	Loss on Ignition, percent
Gs:	Specific Gravity
K:	Coefficient of Permeability
w:	Moisture content, percent
qp:	Calibrated Penetrometer Resistance, tsf
qs:	Vane-Shear Strength, tsf
qu:	Unconfined Compressive Strength, tsf
qc:	Static Cone Penetrometer Resistance (correlated to Unconfined Compressive Strength, tsf)
PID:	Results of vapor analysis conducted on representative samples utilizing a Photoionization Detector calibrated to a benzene standard. Results expressed in HNU-Units. (BDL=Below Detection Limit)
N:	Penetration Resistance per 12 inch interval, or fraction thereof, for a standard 2 inch O.D. (1⅜ inch I.D.) split spoon sampler driven with a 140 pound weight free-falling 30 inches. Performed in general accordance with Standard Penetration Test Specifications (ASTM D-1586). N in blows per foot equals sum of N-Values where plus sign (+) is shown.
Nc:	Penetration Resistance per 1¾ inches of Dynamic Cone Penetrometer. Approximately equivalent to Standard Penetration Test N-Value in blows per foot.
Nr:	Penetration Resistance per 12 inch interval, or fraction thereof, for California Ring Sampler driven with a 140 pound weight free-falling 30 inches per ASTM D-3550. Not equivalent to Standard Penetration Test N-Value.

DRILLING AND SAMPLING SYMBOLS

SS:	Split-Spoon
ST:	Shelby Tube – 3 inch O.D. (except where noted)
CS:	3 inch O.D. California Ring Sampler
DC:	Dynamic Cone Penetrometer per ASTM Special Technical Publication No. 399
AU:	Auger Sample
DB:	Diamond Bit
CB:	Carbide Bit
WS:	Wash Sample
RB:	Rock-Roller Bit
BS:	Bulk Sample
Note:	Depth intervals for sampling shown on Record of Subsurface Exploration are not indicative of sample recovery, but position where sampling initiated

SOIL STRENGTH CHARACTERISTICS

COHESIVE (CLAYEY) SOILS

COMPARATIVE CONSISTENCY	BLOWS PER FOOT (N)	UNCONFINED COMPRESSIVE STRENGTH (TSF)
Very Soft	0 - 2	0 - 0.25
Soft	3 - 4	0.25 - 0.50
Medium Stiff	5 – 8	0.50 - 1.00
Stiff	9 – 15	1.00 - 2.00
Very Stiff	16 – 30	2.00 - 4.00
Hard	31+	4.00+

NON-COHESIVE (GRANULAR) SOILS

RELATIVE DENSITY	BLOWS PER FOOT (N)
Very Loose	0 - 4
Loose	5 - 10
Firm	11 - 30
Dense	31 - 50
Very Dense	51+

DEGREE OF PLASTICITY	PI
None to Slight	0 - 4
Slight	5 - 10
Medium	11 - 30
High to Very High	31+

DEGREE OF EXPANSIVE POTENTIAL	PI
Low	0 - 15
Medium	15 - 25
High	25+

Important Information About Your
Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

The following information is provided to help you manage your risks.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual

subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely, on Your ASFE-Member Geotechncial Engineer for Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you ASFE-member geotechnical engineer for more information.

ASFE
The Best People on Earth

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e-mail: info@asfe.org www.asfe.org

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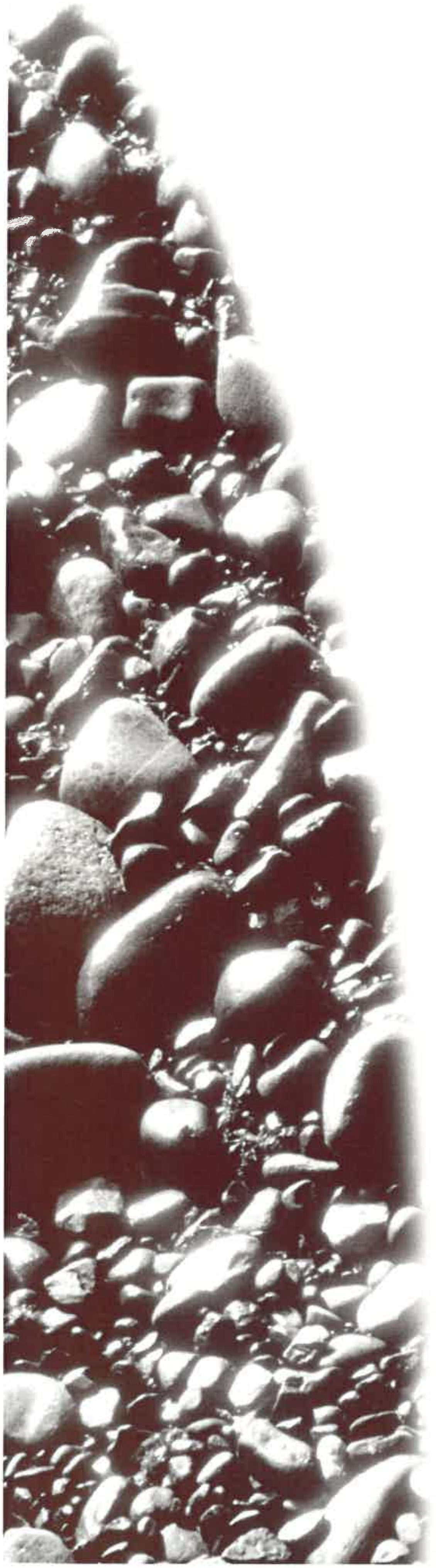
APPENDIX B

SUPPLEMENTAL GEOTECHNICAL EXPLORATION (GILES, 2018)



Supplemental Geotechnical Engineering Exploration and Analysis

Slope Stability Evaluation
Proposed Chick-fil-A Restaurant #3805
Livermore @ 580 FSU
SWC of N. Livermore Avenue and I-580 Freeway
Livermore, California

Prepared for:

Chick-fil-A, Inc.
Irvine, California

May 3, 2018
Giles Project No. 2G-1712002

GILES
ENGINEERING ASSOCIATES, INC.

GILES
ENGINEERING ASSOCIATES, INC.
GEOTECHNICAL, ENVIRONMENTAL & CONSTRUCTION MATERIALS CONSULTANTS

- Atlanta, GA
- Baltimore, MD
- Dallas, TX
- Los Angeles, CA
- Manassas, VA
- Milwaukee, WI

May 3, 2018

Chick-fil-A, Inc.
15365 Alton Parkway, Suite 350
Irvine, CA 92618

Attention Ms. Beth Witt
Development Coordinator

Subject: Slope Stability Evaluation
Proposed Chick-fil-A Restaurant #3805
Livermore @ 580 FSU
SWC of N. Livermore Avenue and I-580 Freeway
Livermore, California
Project No. 2G-1712002

Dear Ms. Witt:

In accordance with your request and authorization, Giles Engineering Associates, Inc. (Giles) performed an evaluation of the global stability of the existing slope at the above referenced project site. Giles previously conducted a *Geotechnical Engineering Exploration and Analysis* for a proposed new Chick-fil-A Restaurant at the site. That report is dated April 24, 2017 and is referenced by Giles' Project Number 2G-1606012.

The Slope Stability Evaluation addressed in this report was conducted due to the proximity of the proposed building to the existing slope and the existing conditions of the slope. This analysis is based on the subsurface conditions encountered at additional deeper borings (performed for this report), and the previous borings performed at the site, which are documented in the previously submitted *Geotechnical Engineering Exploration and Analysis.* The purpose of this report is to assist evaluation of the risk of slope movements affecting the proposed development.

1.0 SITE DESCRIPTION

The project site is located at the southwest corner of the intersection of North Livermore Avenue and the I-580 Freeway in the city of Livermore, California. The site is bordered on the north by an approximately 20-foot-high slope that descends to Arroyo Las Positas (watercourse). The existing slope generally has a 1:1 (horizontal:vertical) gradient, however, some portions of the slope are more steep and flatter areas also exist. At the time of the Geotechnical Subsurface Exploration Program, the slope was covered in tall grass and shrubs, and steeper areas had exposed soil. The crest of the slope was between approximately El. 456 and El. 460 and it is understood that the 100-Year Flood plain is between approximately El. 447 and El. 452.5. Those elevations are based on cross-sections and topographic contours shown on the *Arroyo Las Positas Creek Topographic Exhibit* prepared by Joseph C. Truxaw & Associates, Inc. and dated November 28, 2017. The site is shown on Figure 1, enclosed in Appendix A and further description of the project site is included in the previous geotechnical report.

2.0 GEOTECHNICAL SUBSURFACE EXPLORATION

To explore subsurface conditions, two test borings (Test Borings 9 and 10) were conducted at the site using a mechanical drill-rig and drilled to a depth of ±31 feet below-grade, as planned. The test borings were drilled near the crest of the existing slope. The test boring locations are shown on the *Test Boring Location Plan* (Figure 1), enclosed in Appendix A, along with the locations of the test borings performed for the previous report.

Our subsurface exploration included the collection of samples of subsurface soil materials for laboratory testing purposes and bulk samples that consisted of composite soil materials obtained at selected depth intervals from the borings. Relatively less disturbed samples were collected using a 3-inch outside-diameter, modified California split-spoon soil sampler (CS) lined with 1-inch high brass rings. The sampler was driven with successive 30-inch drops of a hydraulically operated, 140-pound automatic trip hammer. Blow counts for each 6-inch driving increment were recorded on the field exploration logs. The central portions of the driven core samples were placed in sealed containers and transported to our laboratory for testing.

Additionally, samples were collected at the test borings, at certain depths, using the Standard Penetration Test (SPT), conducted with the drill rig in accordance with ASTM Standard Procedure D1586. A brief description of the SPT is given in Appendix B along with descriptions of other field procedures. Immediately after sampling, select portions of the SPT samples were placed in containers that were labeled at the site for identification. Retained samples were transported to Giles' geotechnical laboratory. A Standard Penetration Resistance value (N-value) was determined from each SPT. N-values are reported on the *Test Boring Logs* (in Appendix A), which are records of the test borings.

The boreholes were backfilled upon completion; however, settlement and/or expansion of backfill material will likely occur, possibly creating a hazard that can lead to a threat of injury to people and animals. Borehole areas should, therefore, be carefully and routinely monitored by the property owner or others; settlement and/or expansion of backfill materials should be repaired immediately. Giles will not monitor or repair boreholes.

3.0 GEOTECHNICAL LABORATORY SERVICES

The retained samples were classified using the descriptive terms and particle-size criteria shown on the *General Notes* in Appendix D, and by using the Unified Soil Classification System (ASTM D 2488-75) as a general guide. The classifications are shown on the *Test Boring Logs*, along with horizontal lines that show estimated depths of material change. Field-related information pertaining to the test borings is also shown on the *Test Boring Logs*. For simplicity and abbreviation, terms and symbols are used on the *Test Boring Logs;* the terms and symbols are defined on the *General Notes*.

Calibrated penetrometer resistance, unit weight and moisture content tests were performed on select soil samples (SPT and California ring samples) to evaluate the soils general engineering properties. In addition, the following geotechnical laboratory tests were performed in general accordance with the corresponding ASTM standards:

- ASTM D4318 – Atterberg limits
- ASTM D1140 – Percentage of material finer than the No. 200 sieve
- ASTM D3080 – Consolidated-drained direct shear test
- ASTM D4767 – Consolidated-undrained triaxial shear strength (2 tests)

The Atterberg limits and percentage of material finer than the No. 200 sieve (P200) tests were performed on SPT and California ring samples. The direct shear and triaxial tests were performed, along with unit weight determinations, on California ring samples. The results of the direct shear and triaxial tests are on Figures 6 through 8 in Appendix A. The other test results are on the *Test Boring Logs* in Appendix A. Laboratory procedures are briefly described in Appendix C.

4.0 SUBSURFACE CONDITIONS

Because material sampling at the test borings was discontinuous, it was necessary for Giles to estimate conditions between sample intervals. Estimated conditions at the test borings (Test Borings 9 and 10) are briefly discussed in this section and are described in more detail on the *Test Boring Logs*. The conclusions and recommendations in this report are based only on the estimated conditions. Test Borings performed for the previous report (Test Borings 1 through 8) are reported in the *Test Boring Logs* for the previous report. Material conditions at those test borings are discussed in the previous report.

4.1. Native Soil

Native soil was at the surface of the test borings (beneath the existing grass and foliage) and present to the 31-foot test boring termination depths. In general, sandy silt and clay with variable amounts of sand and silt were encountered to approximately 6± to 8½± below-grade at the test borings. The cohesive soil had relatively high moisture contents and moderate to high strength characteristics, based on laboratory testing. Silty sand with gravel and silty gravel was encountered below the native, cohesive material. Sandy lean clay lenses were encountered within the native sand. The granular soil generally exhibited firm to very dense relative densities based on California sampler blow counts and SPT N-values.

4.2. Groundwater Conditions

Groundwater was encountered at depths of 25± and 19½± feet below the surface when the Test Borings 9 and 10 were performed, respectively. The groundwater conditions are only an estimate based on the conditions at the test borings. The actual conditions could differ and the water table might be higher than estimated. Additionally, it is understood that the 100-Year Flood plain is between approximately El. 447 and El. 452.5 and it is anticipated that ground. For the stability analysis conducted in this report, it is assumed that the groundwater table could temporarily rise to those elevations during a flood event.

5.0 CONCLUSIONS AND RECOMMENDATIONS

A slope stability analysis of the slope to the along the northern boundary of the site was performed using two representative cross sections (Section A'-A and Section B'-B), as shown on the enclosed Figure 1. It is understood that the slope is planned to be cut back to create a 1.5:1 (horizontal:vertical) gradient along the entire northern boundary of the site. Additionally, it understood that the face of the slope will be designed to limit erosion.

The topography used in the cross sections is based on topographic contours shown on the *Arroyo Las Positas Creek Topographic Exhibit* and assuming that the slope will be cut back to a 1.5:1 gradient from the toe of the slope, as defined in the above referenced plan. The analysis was performed using STABLPRO (Ensoft, Inc.) and the soil conditions encountered in the test borings. The analysis is based on limit equilibrium slope stability methods and incorporates the Bishop's Modified Method. Results of the slope stability analysis, along with conclusions and recommendations regarding the existing slope in conjunction with the proposed development are provided below.

Slope Stability Analysis

Several iterations of the slope stability analysis were performed for each profile to determine the lowest factor of safety for global stability of the site slope, assuming the slope is cut to a 1.5:1 gradient, within reasonable boundaries of the model. Static and pseudostatic analyses were performed on each profile. For the pseudostatic (earthquake) analysis, a pseudostatic horizontal coefficient of 0.25g was utilized. A 100-year flood and rapid drawdown condition were accounted for in the analyses based on the previously discussed 100-year flood elevations.

Laboratory testing (direct shear and triaxial shear strength tests) were performed on the ring samples collected from the site to determine the appropriate soil strength parameters for the stability analyses, as previously discussed. Results of those tests are reported on Figures 6 through 8 in appendix A. Additionally, unit weight testing was performed on samples and is reported on the *Test Boring Logs.* Soil parameters used for the analyses were based on the results of laboratory testing and are reported on the following table:

SLOPE STABILITY SOIL PARAMETERS			
Soil	**Unit Weight (psf)**(1)	**Cohesion (psf)** (1)	**Friction Angle (deg)**(1)
Upper Silt and Clay	120	100	30
Lower Silty Sand	130	300	40
1. Values estimated based on soil conditions encountered at the test borings and results of laboratory testing.			

Final results of stability analyses are shown on Figures 2 through 5. Based on the results of the slope stability analyses performed, and assuming that the slope will be cut to a 1.5:1 gradient, the slope is considered stable with regards to global stability. The factors of safety for static conditions were 2.0 and 2.7 for cross-sections A-A’ and B-B’, respectively. The factors of safety for pseudostatic conditions were 1.2 and 1.6 for cross-sections A-A’ and B-B’, respectively. The results of the analyses indicate that the slopes (when cut to a 1.5:1 gradient and protected from erosion), will be in a stable condition with respect to deep-seated rotational failure.

Slopes with a minimum safety factor greater than about 1.3 to 1.5, depending on conditions, are typically considered to be stable for static conditions. For the static condition evaluation, slopes with a minimum safety factor of less than 1.3 are considered to have a potential for instability, with the potential increasing as the minimum safety factor approaches 1.0. It is understood that a typical minimum factor of safety of 1.1 is required pseudostatic stability analysis.

Slope Protection Considerations

The stability analyses and results addressed above are based on surficial stabilization of the face of the slope, thereby protecting the slope from erosion and potential undercutting from the adjacent watercourse. If the slope is not adequately protected from erosion and undercutting, instability, possibly resulting in slope failure, may occur.

6.0 CLOSURE

This report is strictly based on the project description given earlier in this report. Giles must be notified if any parts of the project description or our assumptions are not accurate so that this report can be amended, if needed. This report is based on the assumption that the facility will be designed and constructed according to the codes that govern construction at the site.

The conclusions and recommendations in this report are based on estimated subsurface conditions as sown on the *Test Boring Logs*. Giles must be notified if the subsurface conditions that are encountered during construction of the proposed development differ from those shown on the *Test Boring Logs* because this report will likely need to be revised. General comments and limitations of this report are given in the enclosures.

The conclusions and recommendations presented in this report have been promulgated in accordance with generally accepted professional engineering practices in the field of geotechnical engineering. No other warranty is either expressed or implied.

We sincerely appreciate the opportunity to provide consulting services for the proposed project. Please contact the undersigned if there are questions concerning the report or if we may be of further service.

Very truly yours,

GILES ENGINEERING ASSOCIATES, INC.

Benjamin M. Stark, E.I.T.
Staff Professional I

David M. Cornale, P.E.
Senior Geotechnical Consultant

JOHN LUKE MAIER
CE 74894
CIVIL

John L. Maier, P.E.
Project Manager I

Distribution: Chick-fil-A, Inc.
Attn: Ms. Beth Witt (1 email: Beth.Witt@cfacorp.com)
Attn: Ms. Jennifer Daw (1 email: Jennifer.Daw@cfacorp.com)
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APPENDICES
Appendix A - Figures (8), Test Boring Logs (2)
Appendix B - Field Procedures
Appendix C - Laboratory Testing and Classification
Appendix D - General Information and Important Information About Your Geotechnical Report

2G-1712002-Letter/17geo04/bms/

APPENDIX A

Figures (8), Test Boring Logs (2)

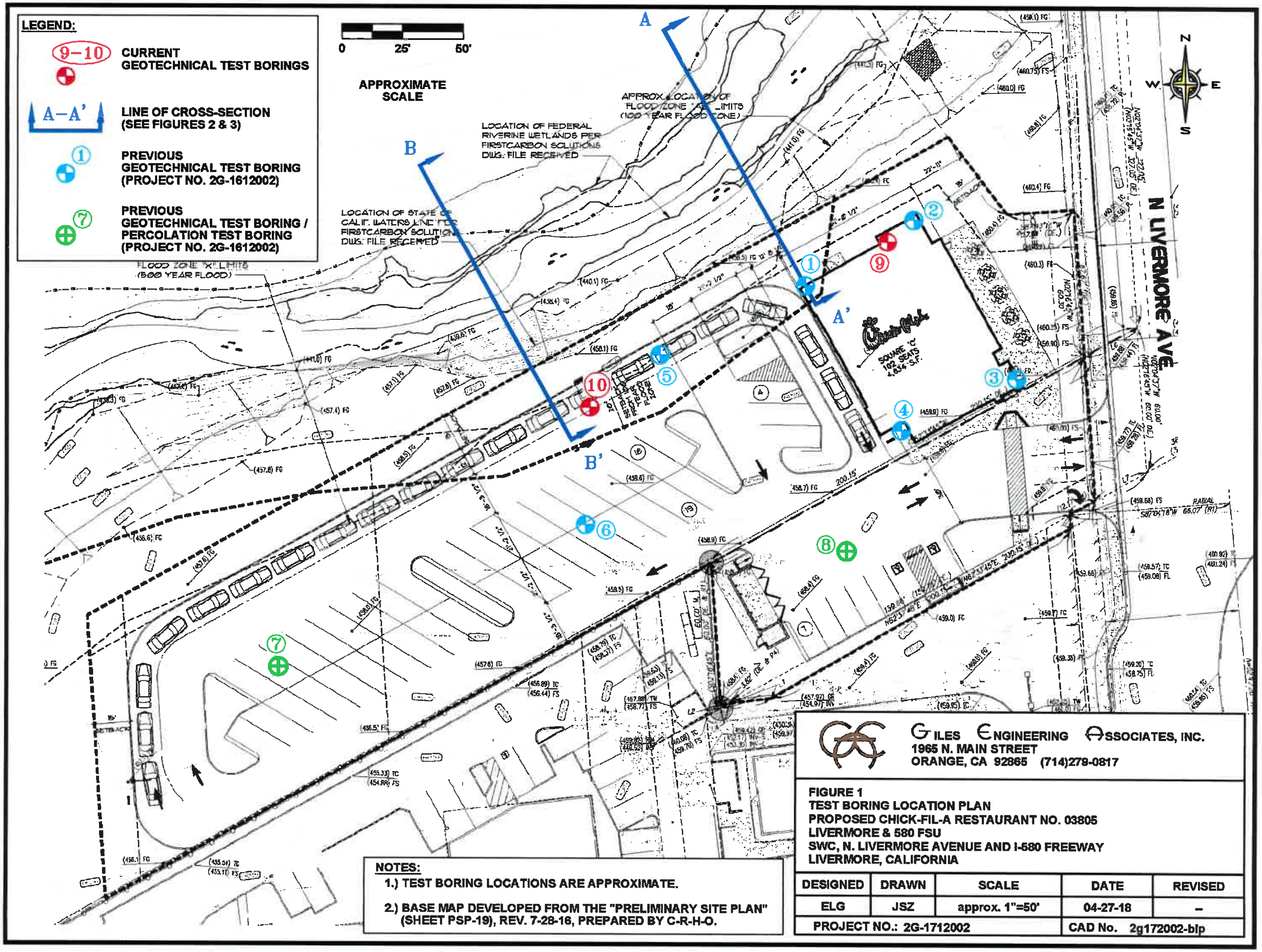
LEGEND:

9–10 ● CURRENT GEOTECHNICAL TEST BORINGS

A–A' LINE OF CROSS-SECTION (SEE FIGURES 2 & 3)

① ● PREVIOUS GEOTECHNICAL TEST BORING (PROJECT NO. 2G-1612002)

⑦ ⊕ PREVIOUS GEOTECHNICAL TEST BORING / PERCOLATION TEST BORING (PROJECT NO. 2G-1612002)



NOTES:

1.) TEST BORING LOCATIONS ARE APPROXIMATE.

2.) BASE MAP DEVELOPED FROM THE "PRELIMINARY SITE PLAN" (SHEET PSP-19), REV. 7-28-16, PREPARED BY C-R-H-O.

GILES ENGINEERING ASSOCIATES, INC.
1965 N. MAIN STREET
ORANGE, CA 92865 (714)279-0817

FIGURE 1
TEST BORING LOCATION PLAN
PROPOSED CHICK-FIL-A RESTAURANT NO. 03805
LIVERMORE & 580 FSU
SWC, N. LIVERMORE AVENUE AND I-580 FREEWAY
LIVERMORE, CALIFORNIA

DESIGNED	DRAWN	SCALE	DATE	REVISED
ELG	JSZ	approx. 1"=50'	04-27-18	–

PROJECT NO.: 2G-1712002
CAD No. 2g172002-blp

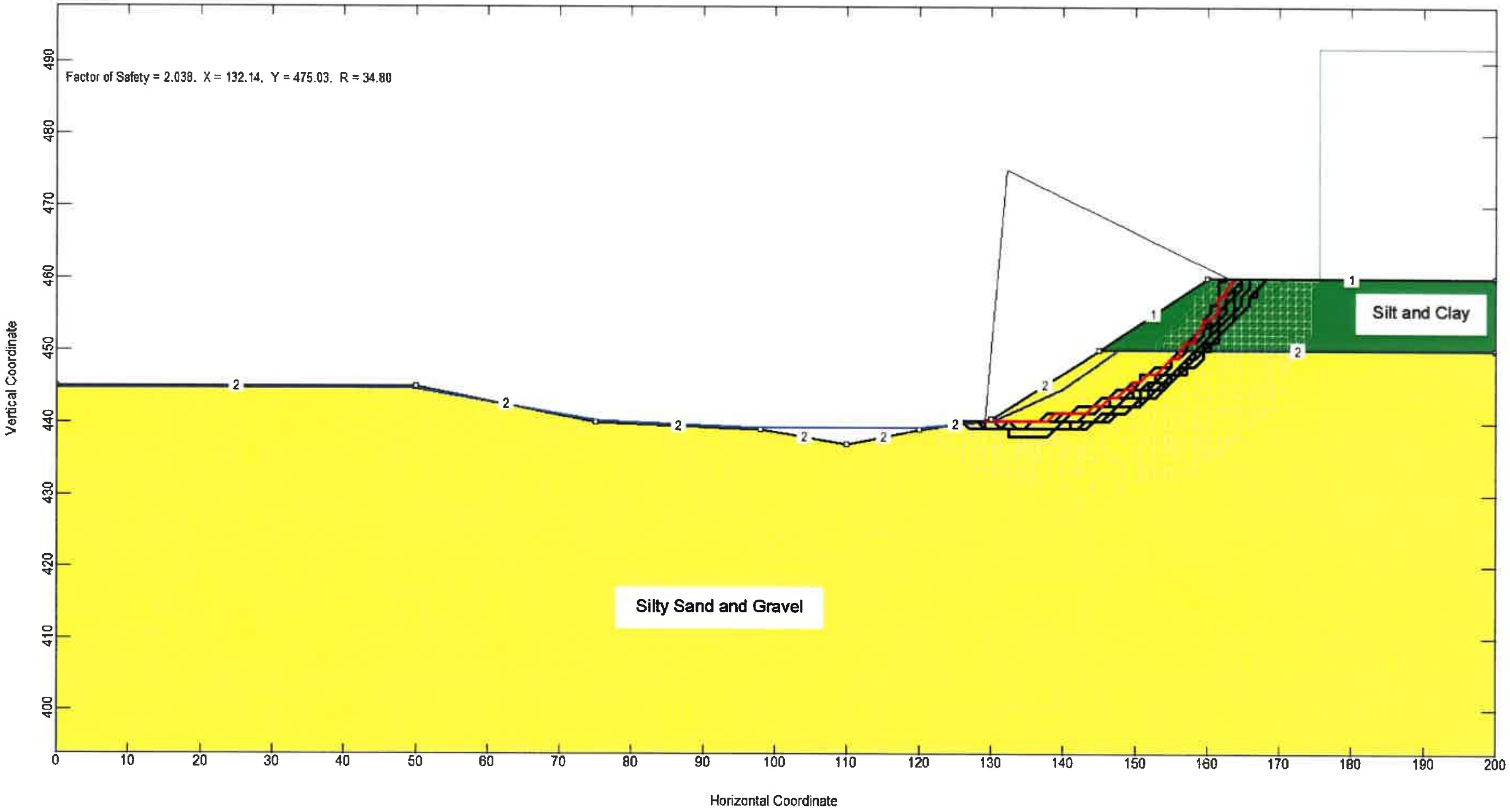


FIGURE 2 – SLOPE STABILITY ANALYSIS RESULTS (SECTION A-A') – Static Condition

Slope Stability Evaluation
Proposed Chick-fil-A Restaurant #3805
SWC of N. Livermore Avenue and I-580 Freeway
Livermore, California

GILES
ENGINEERING ASSOCIATES, INC.

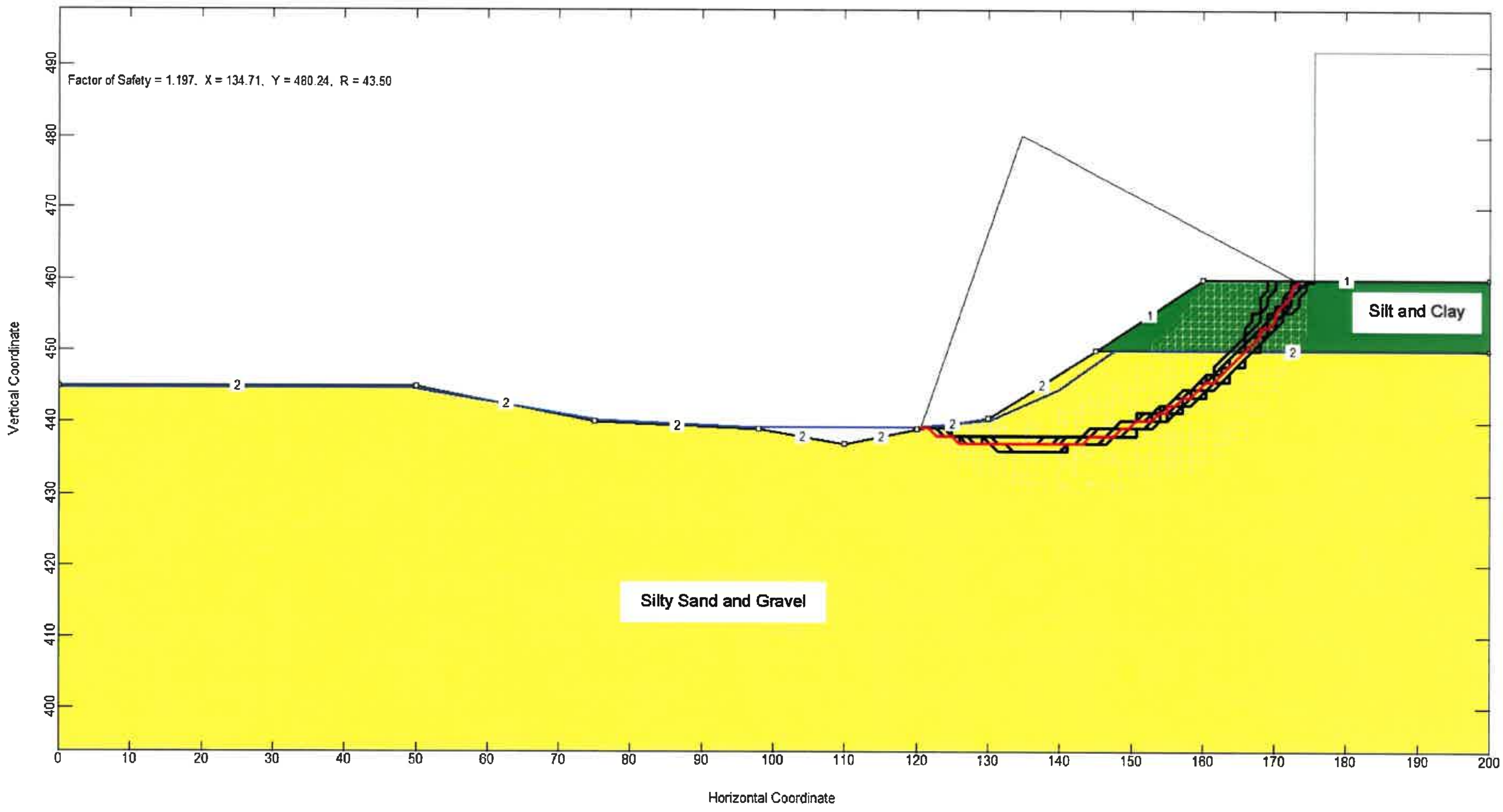


FIGURE 3 – SLOPE STABILITY ANALYSIS RESULTS (SECTION A-A') – Pseudostatic Condition

Slope Stability Evaluation
Proposed Chick-fil-A Restaurant #3805
SWC of N. Livermore Avenue and I-580 Freeway
Livermore, California

GILES
ENGINEERING ASSOCIATES, INC.

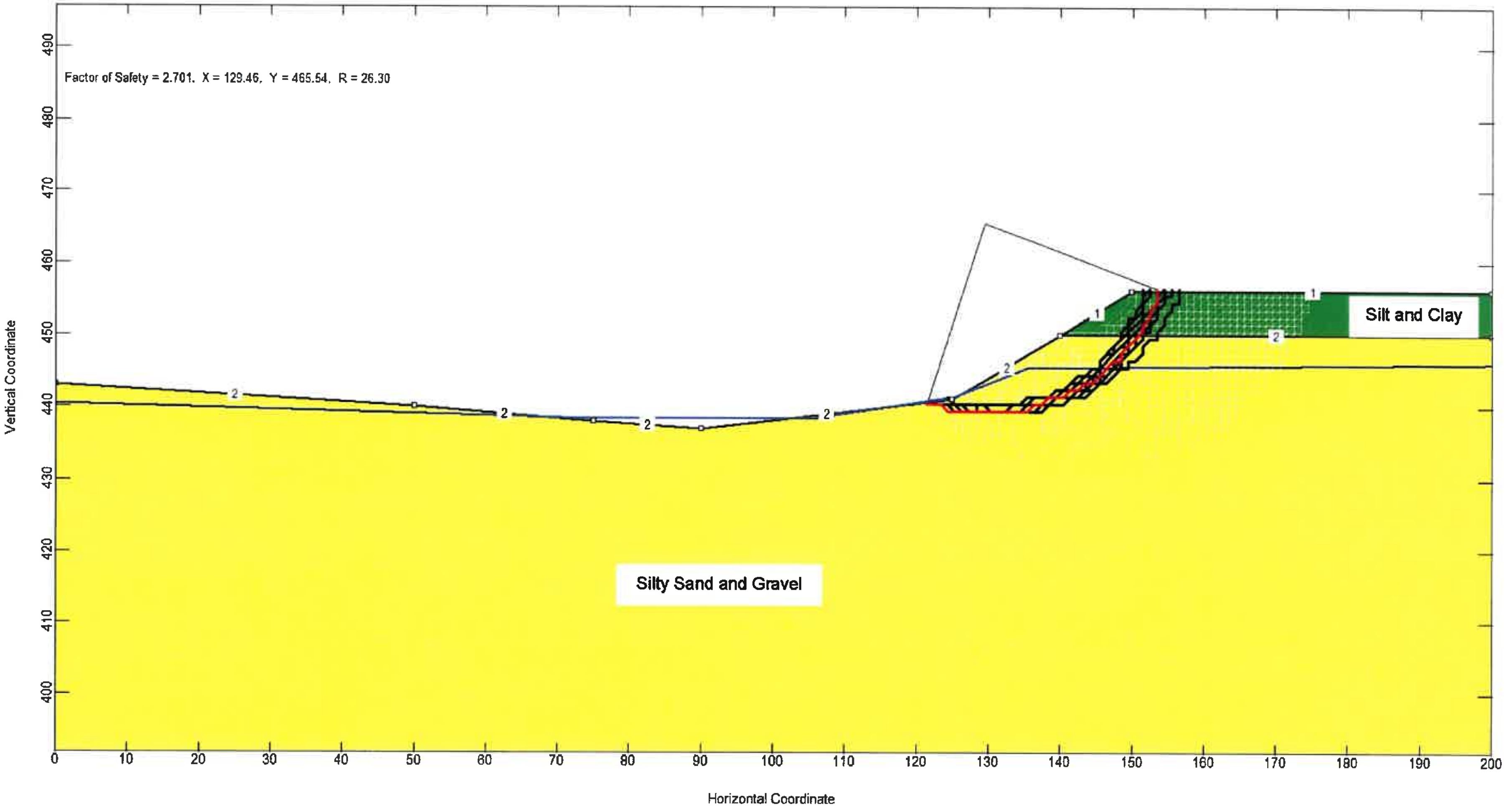


FIGURE 4 – SLOPE STABILITY ANALYSIS RESULTS (SECTION B-B') – Static Condition

Slope Stability Evaluation
Proposed Chick-fil-A Restaurant #3805
SWC of N. Livermore Avenue and I-580 Freeway
Livermore, California

GILES
ENGINEERING ASSOCIATES, INC.

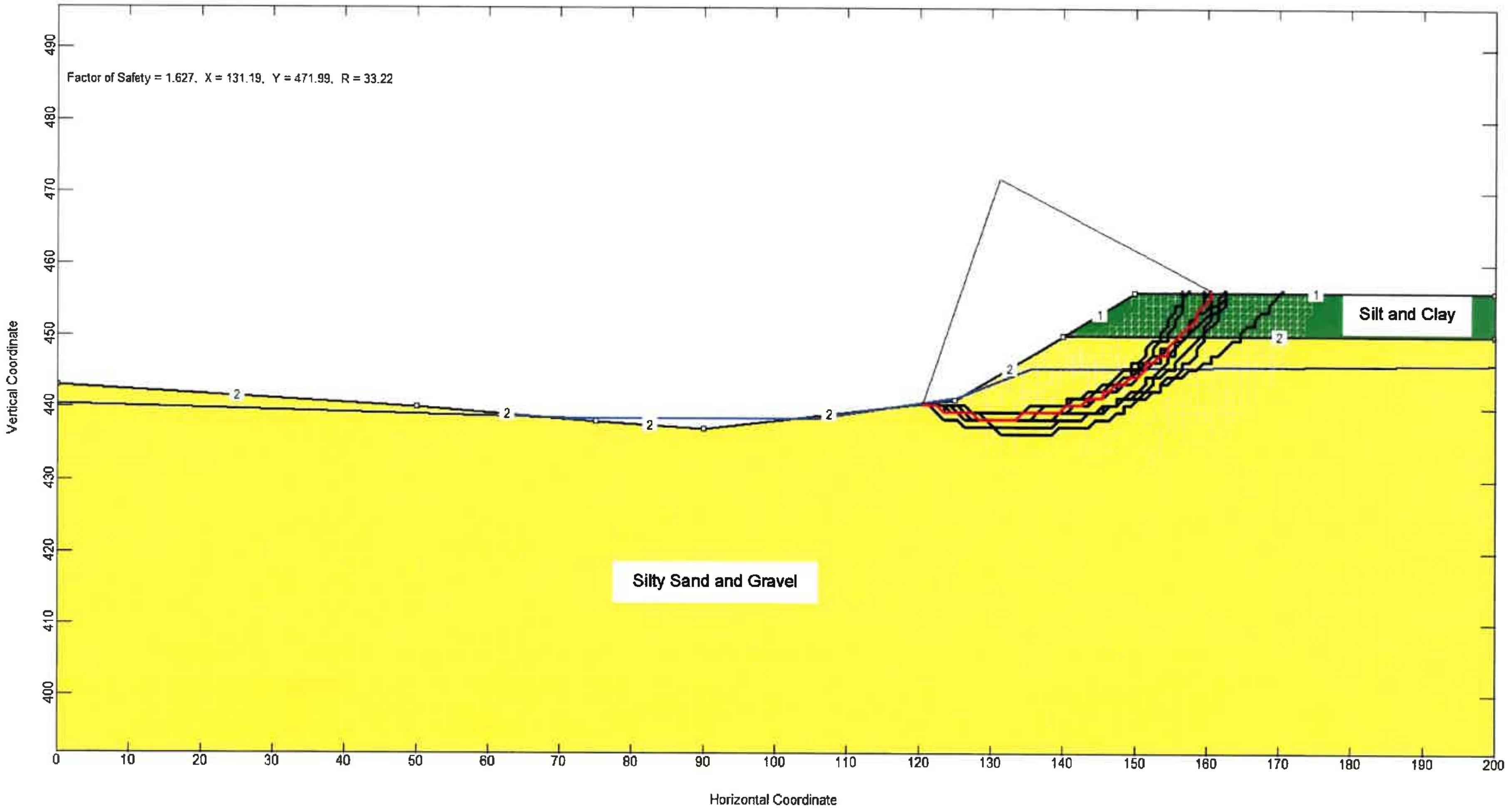
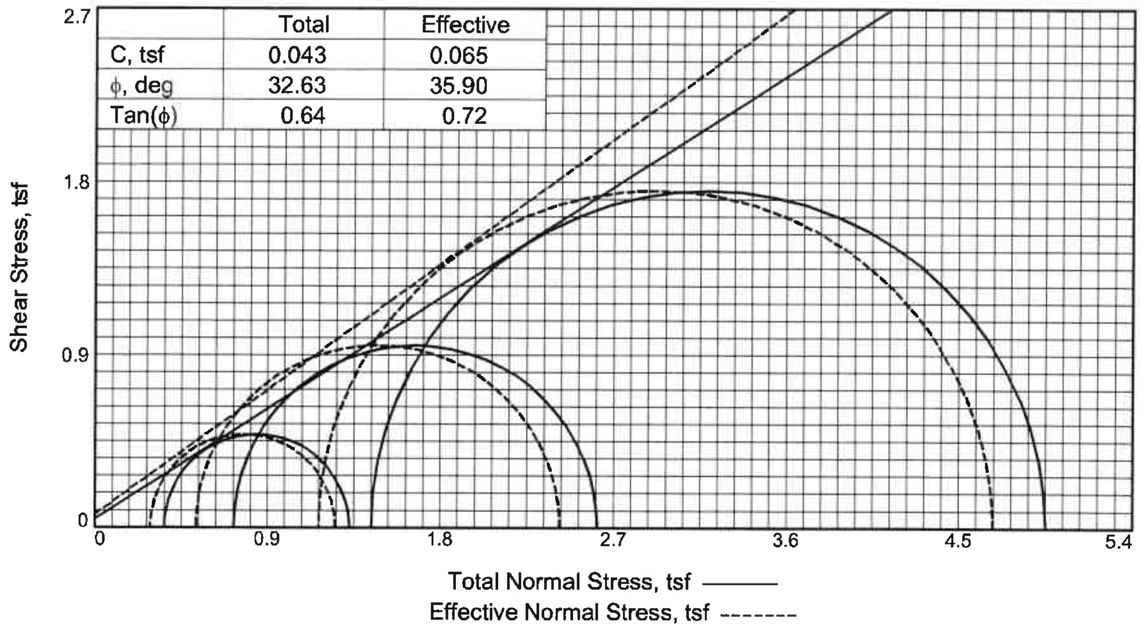


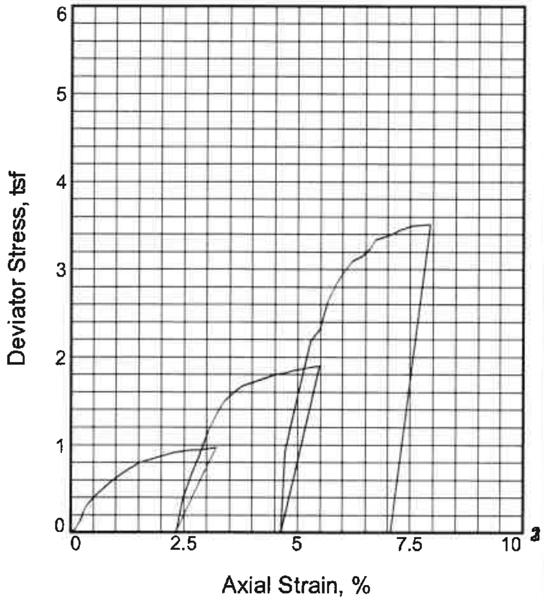
FIGURE 5 – SLOPE STABILITY ANALYSIS RESULTS (SECTION B-B') – Pseudostatic Condition

Slope Stability Evaluation
Proposed Chick-fil-A Restaurant #3805
SWC of N. Livermore Avenue and I-580 Freeway
Livermore, California

GILES
ENGINEERING ASSOCIATES, INC.



	Total	Effective
C, tsf	0.043	0.065
ϕ, deg	32.63	35.90
Tan(ϕ)	0.64	0.72



	Sample No.	1	2	3
Initial	Water Content, %	9.4	9.4	9.4
	Dry Density, pcf	113.8	113.8	113.8
	Saturation, %	52.5	52.5	52.5
	Void Ratio	0.4817	0.4817	0.4817
	Diameter, in.	2.400	2.400	2.400
	Height, in.	5.330	5.330	5.330
At Test	Water Content, %	23.3	22.3	21.0
	Dry Density, pcf	115.8	118.1	121.0
	Saturation, %	138.2	140.8	144.3
	Void Ratio	0.4554	0.4273	0.3935
	Diameter, in.	2.386	2.398	2.408
	Height, in.	5.298	5.142	4.981
Strain rate, in./min.		0.002	0.002	0.002
Eff. Cell Pressure, psi		5.00	10.00	20.00
Fail. Stress, tsf		0.97	1.90	3.51
	Total Pore Pr., tsf	1.87	1.99	2.07
	Strain, %	3.2	5.5	7.9
Ult. Stress, tsf				
	Total Pore Pr., tsf			
	Strain, %			
$\bar{\sigma}_1$ Failure, tsf		1.25	2.42	4.68
$\bar{\sigma}_3$ Failure, tsf		0.29	0.53	1.17

Type of Test:
CU with Pore Pressures

Sample Type:

Description: Light Gray and Yellow Brown Sandy Silt

Assumed Specific Gravity= 2.70

Remarks:

Client: CFA

Project:

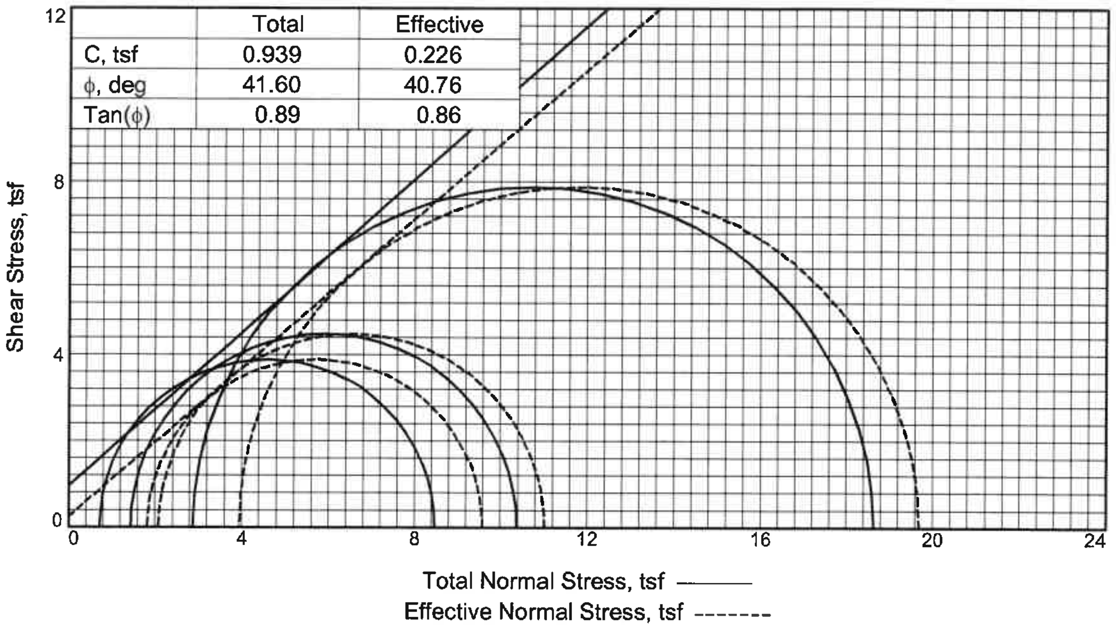
Source of Sample: Boring No. 9 **Depth:** 5 ft.

Sample Number: 2-CS

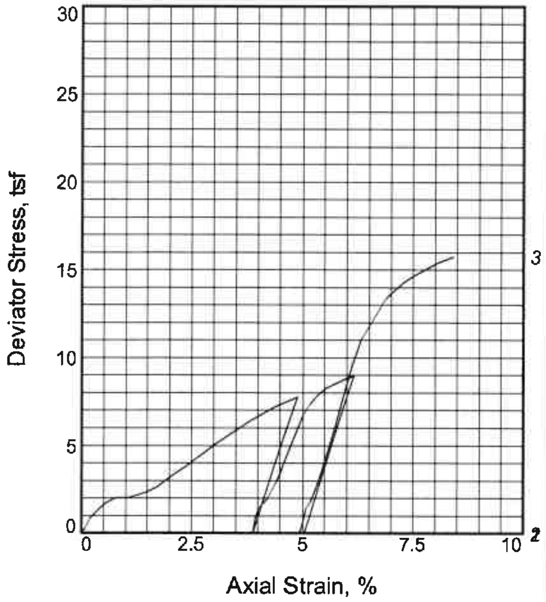
Proj. No.: 2G-1712002 **Date Sampled:** 3/21/2018

TRIAXIAL SHEAR TEST REPORT
GILES ENGINEERING ASSOC., INC.
Waukesha, Wisconsin

Figure 6



	Total	Effective
C, tsf	0.939	0.226
ϕ, deg	41.60	40.76
Tan(ϕ)	0.89	0.86



Sample No.		1	2	3
Initial	Water Content, %	11.0	11.0	11.0
	Dry Density, pcf	128.7	128.7	128.7
	Saturation, %	95.9	95.9	95.9
	Void Ratio	0.3093	0.3093	0.3093
	Diameter, in.	2.400	2.400	2.400
	Height, in.	5.250	5.250	5.250
At Test	Water Content, %	10.7	10.5	10.0
	Dry Density, pcf	130.3	131.1	132.4
	Saturation, %	98.9	98.9	98.8
	Void Ratio	0.2934	0.2854	0.2729
	Diameter, in.	2.390	2.433	2.440
	Height, in.	5.229	5.015	4.940
Strain rate, in./min.		0.022	0.022	0.022
Eff. Cell Pressure, psi		10.00	20.00	40.00
Fail. Stress, tsf		7.8	8.9	15.8
Total Pore Pr., tsf		0.3	0.8	0.4
Strain, %		4.9	6.1	8.4
Ult. Stress, tsf				
Total Pore Pr., tsf				
Strain, %				
$\bar{\sigma}_1$ Failure, tsf		9.6	11.0	19.7
$\bar{\sigma}_3$ Failure, tsf		1.8	2.1	3.9

Type of Test:
CU with Pore Pressures

Sample Type:

Description: Gray and Brown fine to coarse Sand, some Gravel, little Silt

Assumed Specific Gravity= 2.7

Remarks:

Figure 7

Client: CFA

Project:

Source of Sample: Boring No. 10 **Depth:** 26.5 ft.

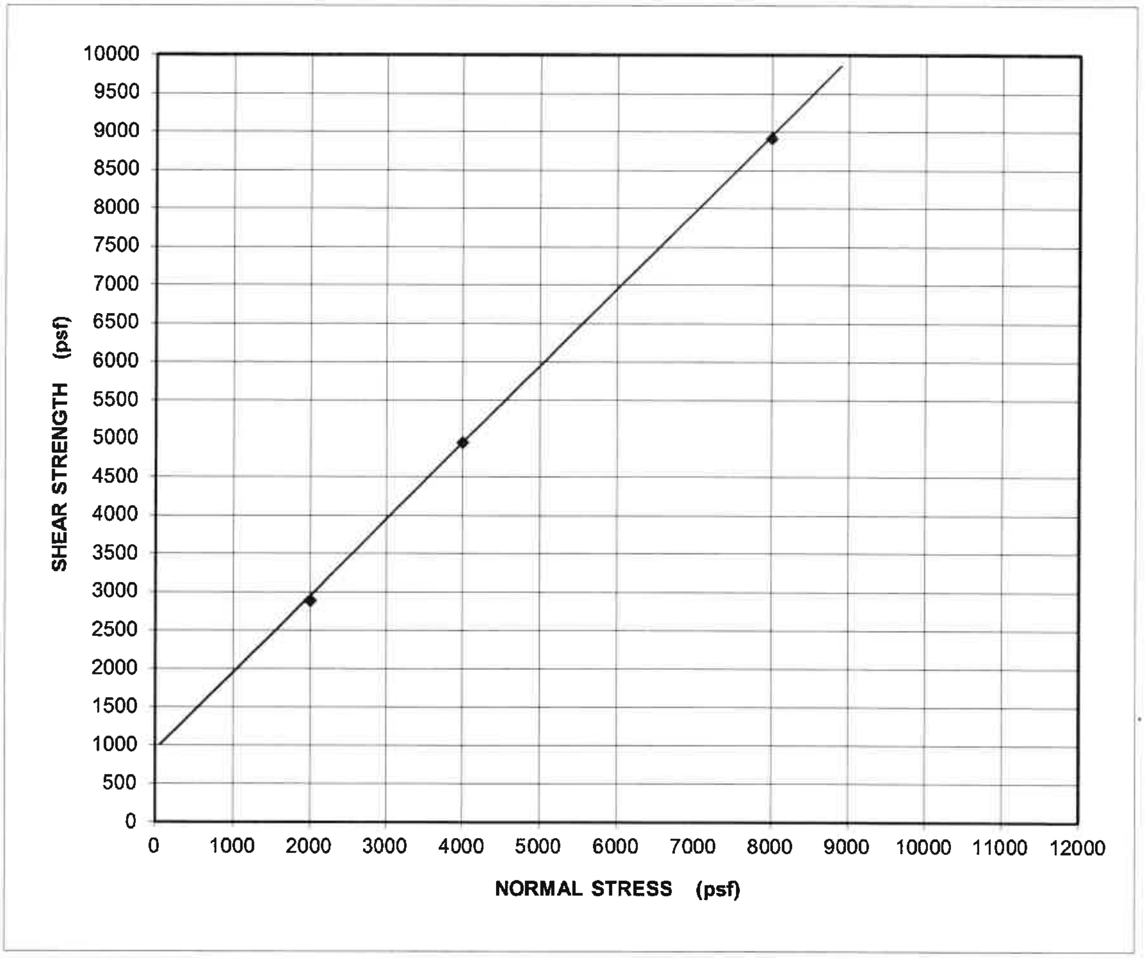
Sample Number: 7-CS

Proj. No.: 2G-1712002 **Date Sampled:** 3/21/18

TRIAXIAL SHEAR TEST REPORT
GILES ENGINEERING ASSOC., INC.
Waukesha, Wisconsin

Giles Engineering Associates, Inc.
GEOTECHNICAL, ENVIRONMENTAL AND CONSTRUCTION MATERIALS CONSULTANTS
N8 W22350 JOHNSON ROAD, SUITE A1/WAUKESHA, WI 53186/(262) 544-0118/FAX: (262) 549-5868

DIRECT SHEAR TEST (AASHTO T-236)



Classification Gray Silty Gravel, some fine to coarse Sand

Sample Source Boring 9 25 ft.

		Diameter (in.)	2.4
PROJECT:	CFA-Livermore, CA	Height (in.)	1.00
		Moisture Content (%)	11.0
CLIENT:	CFA	Dry Density (pcf)	126.6
Sample No.	7-SS		
PROJECT NO.:	2G-1712002	PHI (degrees)	45
Figure:	8		

REVIEWING ENGINEER: Steven P. Homar, P.E.

BORING NO. & LOCATION: 9	# TEST BORING LOG	**GILES ENGINEERING ASSOCIATES, INC.**
SURFACE ELEVATION: 460 feet	SLOPE STABILITY EVALUATION	
COMPLETION DATE: 03/21/18	CHICK-FIL-A RESTAURANT #3805 LIVERMORE AVENUE AND I-580 LIVERMORE, CALIFORNIA	
FIELD REP: JOE HUYNH	PROJECT NO: 2G-1712002	

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Dark Gray Silty Clay, little Sand-Damp (contains Calcareous deposits)			1-SS	19						
Light Gray and Yellow-Brown Mottled Sandy Silt-Damp	5	455	2-CS	41*						
			3-SS	18						
Brown and Gray-Brown Silt and fine to coarse Sand, some Gravel-Damp to Moist	10	450	4-CS	50/6"*		3.5		5		Dd=112.5 pcf P40=96% P200=51% Non-Plastic
	15	445	5-SS	54						
Gray Silty Gravel, and some fine to coarse Sand-Wet (includes Sandy lean Clay lenses)	20	440	6-CS	50/4"*				8		Dd=124.1 pcf P200=10% Non-Plastic
	▽ 25	435	7-SS	76						
	30	430	8-CS	50/5"*				7		Dd=117 pcf P200=8% Non-Plastic

Boring Terminated at about 31 feet (EL. 429')

	Water Observation Data	**Remarks:**
▽	Water Encountered During Drilling: 25 ft.	*Value is number of blows per foot for a California Ring Sampler driven with a 140 Lb. hammer falling 30 inches; NOT equivalent to SPT n-value
▼	Water Level At End of Drilling:	
░	Cave Depth At End of Drilling:	
▼	Water Level After Drilling:	
▨	Cave Depth After Drilling:	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION:
10

SURFACE ELEVATION:
458.5 feet

COMPLETION DATE:
03/21/18

FIELD REP:
JOE HUYNH

TEST BORING LOG

SLOPE STABILITY EVALUATION

CHICK-FIL-A RESTAURANT #3805
LIVERMORE AVENUE AND I-580
LIVERMORE, CALIFORNIA

PROJECT NO: 2G-1712002

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Dark Gray Sandy, Silty Clay-Damp			1-SS	8						
Gray-Brown and Yellow-Brown Clay, trace fine Sand-Damp to Moist	5	455	2-SS	8		2.0		46		P40=97% P200=90% LL=67;PI=19
Gray and Yellow-Brown Mottled Silty fine to medium Sand, trace Gravel-Moist		450	3-CS	31*		2.5		18		Dd=108.8 pep P40=95% P200=40% Non-Plastic
Gray-Brown Silty fine to medium Sand, little Gravel-Moist (includes Sandy lean Clay lenses)	10		4-SS	29						
Dark Gray-Brown Silty Gravel, some fine to coarse Sand-Damp	15	445	5-CS	75*						
Gray-Brown Clayey fine to medium Sand, some Gravel-Moist to Wet (includes Sandy lean Clay lenses)	▽ 20	440	6-SS	49				15		P200=26% LL=37; PI=15
Gray and Brown fine to coarse Sand, some Gravel, little Silt-Wet	25	435	7-CS	50/3*						
Gray Clayey fine to medium Sand, little Gravel-Wet (includes Sandy lean Clay lenses)	30	430	8-SS	50						

Boring Terminated at about 31 feet (EL. 427.5')

Water Observation Data

▽	Water Encountered During Drilling: 19.5 ft.
▼	Water Level At End of Drilling:
░	Cave Depth At End of Drilling:
▼	Water Level After Drilling:
▀	Cave Depth After Drilling:

Remarks:

*Value is number of blows per foot for a California Ring Sampler driven with a 140 Lb. hammer falling 30 inches; NOT equivalent to SPT n-value

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

APPENDIX B

FIELD PROCEDURES

The field operations were conducted in general accordance with the procedures recommended by the American Society for Testing and Materials (ASTM) designation D

420 entitled "Standard Guide for Sampling Rock and Rock" and/or other relevant specifications. Soil samples were preserved and transported to *Giles'* laboratory in general accordance with the procedures recommended by ASTM designation D 4220 entitled "Standard Practice for Preserving and Transporting Soil Samples." Brief descriptions of the sampling, testing and field procedures commonly performed by *Giles* are provided herein.

GENERAL FIELD PROCEDURES

Test Boring Elevations

The ground surface elevations reported on the Test Boring Logs are referenced to the assumed benchmark shown on the Boring Location Plan (Figure 1). Unless otherwise noted, the elevations were determined with a conventional hand-level and are accurate to within about 1 foot.

Test Boring Locations

The test borings were located on-site based on the existing site features and/or apparent property lines. Dimensions illustrating the approximate boring locations are reported on the Boring Location Plan (Figure 1).

Water Level Measurement

The water levels reported on the Test Boring Logs represent the depth of "free" water encountered during drilling and/or after the drilling tools were removed from the borehole. Water levels measured within a granular (sand and gravel) soil profile are typically indicative of the water table elevation. It is usually not possible to accurately identify the water table elevation with cohesive (clayey) soils, since the rate of seepage is slow. The water table elevation within cohesive soils must therefore be determined over a period of time with groundwater observation wells.

It must be recognized that the water table may fluctuate seasonally and during periods of heavy precipitation. Depending on the subsurface conditions, water may also become perched above the water table, especially during wet periods.

Borehole Backfilling Procedures

Each borehole was backfilled upon completion of the field operations. If potential contamination was encountered, and/or if required by state or local regulations, boreholes were backfilled with an "impervious" material (such as bentonite slurry). Borings that penetrated pavements, sidewalks, etc. were "capped" with Portland Cement concrete, asphaltic concrete, or a similar surface material. It must, however, be recognized that the backfill material may settle, and the surface cap may subside, over a period of time. Further backfilling and/or re-surfacing by *Giles'* client or the property owner may be required.

FIELD SAMPLING AND TESTING PROCEDURES

Auger Sampling (AU)

Soil samples are removed from the auger flights as an auger is withdrawn above the ground surface. Such samples are used to determine general soil types and identify approximate soil stratifications. Auger samples are highly disturbed and are therefore not typically used for geotechnical strength testing.

Split-Barrel Sampling (SS) – (ASTM D-1586)

A split-barrel sampler with a 2-inch outside diameter is driven into the subsoil with a 140-pound hammer free-falling a vertical distance of 30 inches. The summation of hammer-blows required to drive the sampler the final 12-inches of an 18-inch sample interval is defined as the "Standard Penetration Resistance" or N-value is an index of the relative density of granular soils and the comparative consistency of cohesive soils. A soil sample is collected from each SPT interval.

Shelby Tube Sampling (ST) – (ASTM D-1587)

A relatively undisturbed soil sample is collected by hydraulically advancing a thin-walled Shelby Tube sampler into a soil mass. Shelby Tubes have a sharp cutting edge and are commonly 2 to 5 inches in diameter.

Bulk Sample (BS)

A relatively large volume of soils is collected with a shovel or other manually-operated tool. The sample is typically transported to *Giles'* materials laboratory in a sealed bag or bucket.

Dynamic Cone Penetration Test (DC) – (ASTM STP 399)

This test is conducted by driving a 1.5-inch-diameter cone into the subsoil using a 15-pound steel ring (hammer), free-falling a vertical distance of 20 inches. The number of hammer-blows required to drive the cone 1¾ inches is an indication of the soil strength and density, and is defined as "N". The Dynamic Cone Penetration test is commonly conducted in hand auger borings, test pits and within excavated trenches.

- Continued -

Ring-Lined Barrel Sampling – (ASTM D 3550)

In this procedure, a ring-lined barrel sampler is used to collect soil samples for classification and laboratory testing. This method provides samples that fit directly into laboratory test instruments without additional handling/disturbance.

Sampling and Testing Procedures

The field testing and sampling operations were conducted in general accordance with the procedures recommended by the American Society for Testing and Materials (ASTM) and/or other relevant specifications. Results of the field testing (i.e. N-values) are reported on the Test Boring Logs. Explanations of the terms and symbols shown on the logs are provided on the appendix enclosure entitled "General Notes".

APPENDIX C

LABORATORY TESTING AND CLASSIFICATION

The laboratory testing was conducted under the supervision of a geotechnical engineer in accordance with the procedures recommended by the American Society for Testing and Materials (ASTM) and/or other relevant specifications. Brief descriptions of laboratory tests commonly performed by *Giles* are provided herein.

LABORATORY TESTING AND CLASSIFICATION

Photoionization Detector (PID)

In this procedure, soil samples are "scanned" in *Giles'* analytical laboratory using a Photoionization Detector (PID). The instrument is equipped with an 11.7 eV lamp calibrated to a Benzene Standard and is capable of detecting a minute concentration of **certain** Volatile Organic Compound (VOC) vapors, such as those commonly associated with petroleum products and some solvents. Results of the PID analysis are expressed in HNu (manufacturer's) units rather than actual concentration.

Moisture Content (w) (ASTM D 2216)

Moisture content is defined as the ratio of the weight of water contained within a soil sample to the weight of the dry solids within the sample. Moisture content is expressed as a percentage.

Unconfined Compressive Strength (qu) (ASTM D 2166)

An axial load is applied at a uniform rate to a cylindrical soil sample. The unconfined compressive strength is the maximum stress obtained or the stress when 15% axial strain is reached, whichever occurs first.

Calibrated Penetrometer Resistance (qp)

The small, cylindrical tip of a hand-held penetrometer is pressed into a soil sample to a prescribed depth to measure the soils capacity to resist penetration. This test is used to evaluate unconfined compressive strength.

Vane-Shear Strength (qs)

The blades of a vane are inserted into the flat surface of a soil sample and the vane is rotated until failure occurs. The maximum shear resistance measured immediately prior to failure is taken as the vane-shear strength.

Loss-on-Ignition (ASTM D 2974; Method C)

The Loss-on-Ignition (L.O.I.) test is used to determine the organic content of a soil sample. The procedure is conducted by heating a dry soil sample to 440°C in order to burn-off or "ash" organic matter present within the sample. The L.O.I. value is the ratio of the weight loss due to ignition compared to the initial weight of the dry sample. L.O.I. is expressed as a percentage.

Particle Size Distribution (ASTB D 421, D 422, and D 1140)

This test is performed to determine the distribution of specific particle sizes (diameters) within a soil sample. The distribution of coarse-grained soil particles (sand and gravel) is determined from a "sieve analysis," which is conducted by passing the sample through a series of nested sieves. The distribution of fine-grained soil particles (silt and clay) is determined from a "hydrometer analysis" which is based on the sedimentation of particles suspended in water.

Consolidation Test (ASTM D 2435)

In this procedure, a series of cumulative vertical loads are applied to a small, laterally confined soil sample. During each load increment, vertical compression (consolidation) of the sample is measured over a period of time. Results of this test are used to estimate settlement and time rate of settlement.

Classification of Samples

Each soil sample was visually-manually classified, based on texture and plasticity, in general accordance with the Unified Soil Classification System (ASTM D-2488-75). The classifications are reported on the Test Boring Logs.

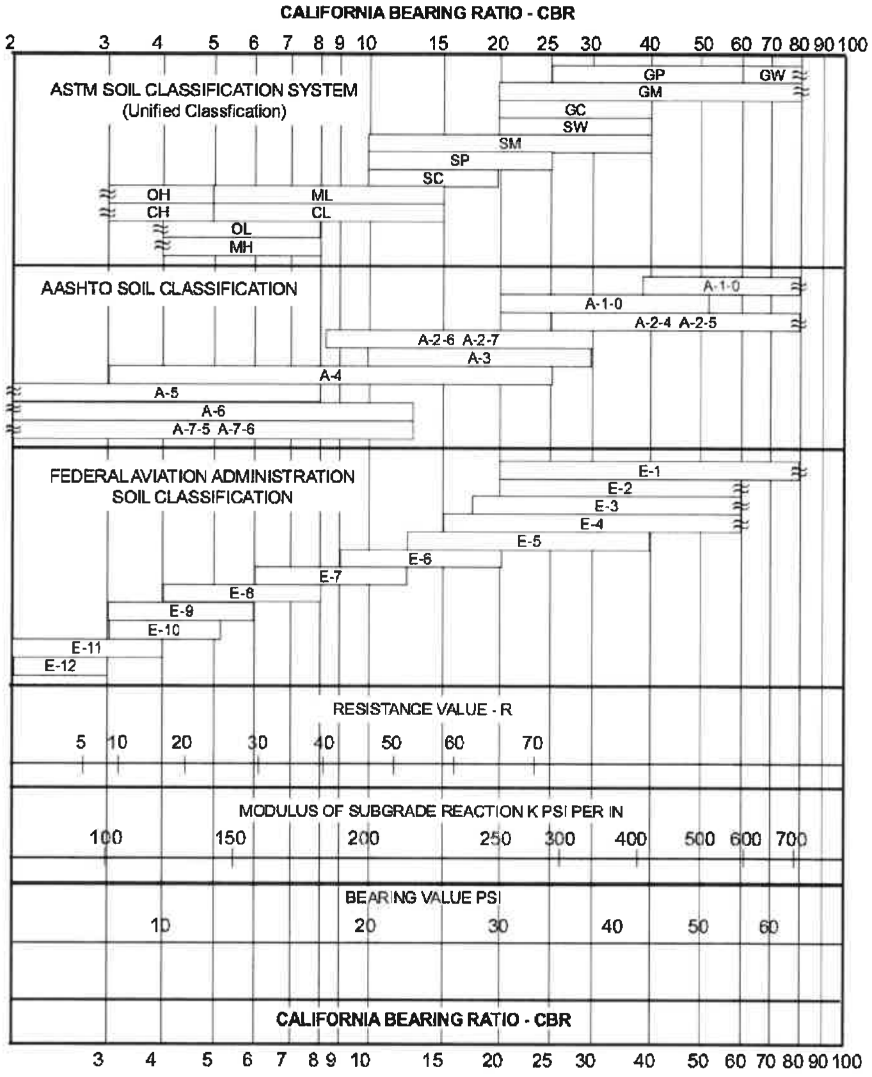
Laboratory Testing

The laboratory testing operations were conducted in general accordance with the procedures recommended by the American Society for Testing and Materials (ASTM) and/or other relevant specifications. Results of the laboratory tests are provided on the Test Boring Logs or other appendix enclosures. Explanation of the terms and symbols used on the logs is provided on the appendix enclosure entitled "General Notes."

California Bearing Ratio (CBR) Test ASTM D-1833

The CBR test is used for evaluation of a soil subgrade for pavement design. The test consists of measuring the force required for a 3-square-inch cylindrical piston to penetrate 0.1 or 0.2 inch into a compacted soil sample. The result is expressed as a percent of force required to penetrate a standard compacted crushed stone.

Unless a CBR test has been specifically requested by the client, the CBR is estimated from published charts, based on soil classification and strength characteristics. A typical correlation chart is below.



APPENDIX D

General Information and Important Information About Your Geotechnical Report

GUIDE SPECIFICATIONS FOR SUBGRADE AND PREPARATION FOR FILL, FOUNDATION, FLOOR SLAB AND PAVEMENT SUPPORT; AND SELECTION, PLACEMENT AND COMPACTION OF FILL SOILS USING MODIFIED PROCTOR PROCEDURES

1. Construction monitoring and testing of subgrades and grades for fill, foundation, floor slab and pavement; and fill selection, placement and compaction shall be performed by an experienced soils engineer and/or his representatives.

2. All compacted fill, subgrades, and grades shall be (a) underlain by suitable bearing material, (b) free of all organic frozen, or other deleterious material, and (c) observed, tested and approved by qualified engineering personnel representing an experienced soils engineer. Preparation of subgrades after stripping vegetation, organic or other unsuitable materials shall consist of (a) proofrolling to detect soft, wet, yielding soils or other unstable materials that must be undercut, (b) scarifying top 6 to 8 inches, (c) moisture conditioning the soils as required, and (d) recompaction to same minimum in-situ density required for similar material indicated under Item 5. Note: Compaction requirements for pavement subgrade are higher than other areas. Weather and construction equipment may damage compacted fill surface and reworking and retesting may be necessary for proper performance.

3. In overexcavation and fill areas, the compacted fill must extend (a) a minimum 1 foot lateral distance beyond the exterior edge of the foundation at bearing grade or pavement at subgrade and down to compacted fill subgrade on a maximum 0.5(H):1(v) slope, (b) 1 foot above footing grade outside the building, and (c) to floor subgrade inside the building. Fill shall be placed and compacted on a 5(H):1(V) slope or must be stepped or benched as required to flatten if not specifically approved by qualified personnel under the direction of an experienced soils engineer.

4. The compacted fill materials shall be free of deleterious, organic, or frozen matter, shall contain no chemicals that may result in the material being classified as "contaminated", and shall be low-expansive with a maximum Liquid Limit (ASTM D-423) and Plasticity Index (ASTM D-424) of 30 and 15, respectively, unless specifically tested and found to have low expansive properties and approved by an experienced soils engineer. The top 12 inches of compacted fill should have a maximum 3 inch particle diameter and all underlying compacted fill a maximum 6 inch diameter unless specifically approved by an experienced soils engineer. All fill material must be tested and approved under the direction of an experienced soils engineer prior to placement. If the fill is to provide non-frost susceptible characteristics, it must be classified as a clean GW, GP, SW or SP per Unified Soils Classification System (ASTM D-2487).

5 For structural fill depths less than 20 feet, the density of the structural compacted fill and scarified subgrade and grades shall not be less than 90 percent of the maximum dry density as determined by Modified Proctor (ASTM D-1557) with the exception of the top 12 inches of pavement subgrade which shall have a minimum in-situ density of 95 percent of maximum dry density, or 5 percent higher than underlying structural fill materials. Where the structural fill depth is greater than 20 feet, the portion below 20 feet should have a minimum in-place density of 95 percent of its maximum dry density or 5 percent higher than the top 20 feet. Cohesive soils shall not vary by more than -1 to +3 percent moisture content and granular soil ±3 percent from the optimum when placed and compacted or recompacted, unless specifically recommended/approved by the soils engineer observing the placement and compaction. Cohesive soils with moderate to high expansion potentials (PI>15) should, however, be placed, compacted and maintained prior to construction at a 3±1 percent moisture content above optimum moisture content to limit future heave. Fill shall be placed in layers with a maximum loose thickness of 8 inches for foundations and 10 inches for floor slabs and pavements, unless specifically approved by the soils engineer taking into consideration the type of materials and compaction equipment being used. The compaction equipment should consist of suitable mechanical equipment specifically designed for soil compaction. Bulldozers or similar tracked vehicles are typically not suitable for compaction.

6. Excavation, filing, subgrade grade preparation shall be performed in a manner and sequence that will provide drainage at all times and proper control of erosion. Precipitation, springs, and seepage water encountered shall be pumped or drained to provide a suitable working platform. Springs or water seepage encountered during grade/foundation construction must be called to the soils engineer's attention immediately for possible construction procedure revision or inclusion of an underdrain system.

7. Non-structural fill adjacent to structural fill should typically be placed in unison to provide lateral support. Backfill along walls must be placed and compacted with care to ensure excessive unbalanced lateral pressures do not develop. The type of fill material placed adjacent to below grade walls (i.e. basement walls and retaining walls) must be properly tested and approved by an experienced soils engineer with consideration for the lateral pressure used in the wall design.

8. Wherever, in the opinion of the soils engineer or the Owner's Representatives, an unstable condition is being created either by cutting or filling, the work should not proceed into that area until an appropriate geotechnical exploration and analysis has been performed and the grading plan revised, if found necessary.

GENERAL COMMENTS

The soil samples obtained during the subsurface exploration will be retained for a period of thirty days. If no instructions are received, they will be disposed of at that time.

This report has been prepared exclusively for the client in order to aid in the evaluation of this property and to assist the architects and engineers in the design and preparation of the project plans and specifications. Copies of this report may be provided to contractor(s), with contract documents, to disclose information relative to this project. The report, however, has not been prepared to serve as the plans and specifications for actual construction without the appropriate interpretation by the project architect, structural engineer, and/or civil engineer. Reproduction and distribution of this report must be authorized by the client and *Giles*.

This report has been based on assumed conditions/characteristics of the proposed development where specific information was not available. It is recommended that the architect, civil engineer and structural engineer along with any other design professionals involved in this project carefully review these assumptions to ensure they are consistent with the actual planned development. When discrepancies exist, they should be brought to our attention to ensure they do not affect the conclusions and recommendations provided herein. The project plans and specifications may also be submitted to *Giles* for review to ensure that the geotechnical related conclusions and recommendations provided herein have been correctly interpreted.

The analysis of this site was based on a subsoil profile interpolated from a limited subsurface exploration. If the actual conditions encountered during construction vary from those indicated by the borings, *Giles* must be contacted immediately to determine if the conditions alter the recommendations contained herein.

The conclusions and recommendations presented in this report have been promulgated in accordance with generally accepted professional engineering practices in the field of geotechnical engineering. No other warranty is either expressed or implied.

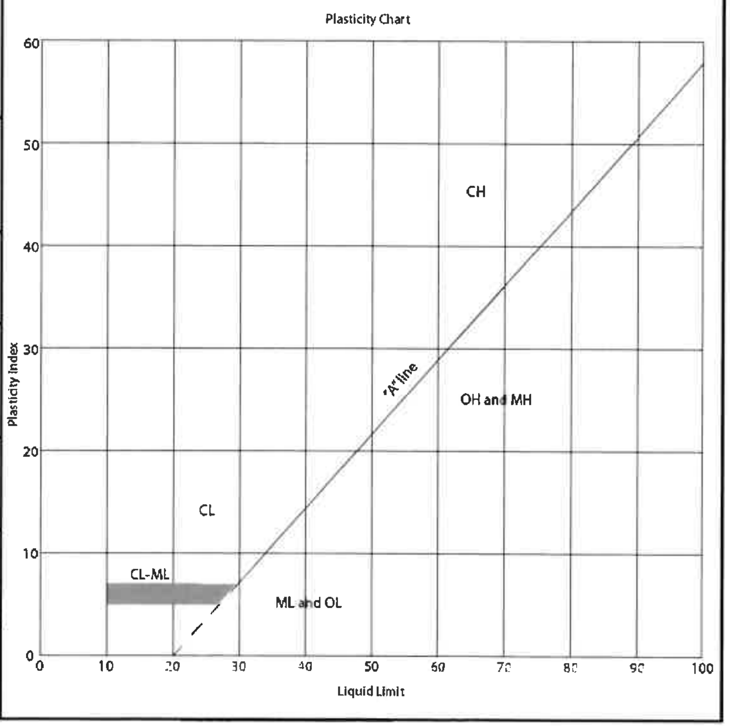
CHARACTERISTICS AND RATINGS OF UNIFIED SOIL SYSTEM CLASSES FOR SOIL CONSTRUCTION *

Class	Compaction Characteristics	Max. Dry Density Standard Proctor (pcf)	Compressibility and Expansion	Drainage and Permeability	Value as an Embankment Material	Value as Subgrade When Not Subject to Frost	Value as Base Course	Value as Temporary Pavement	
								With Dust Palliative	With Bituminous Treatment
GW	Good: tractor, rubber-tired, steel wheel or vibratory roller	125-135	Almost none	Good drainage, pervious	Very stable	Excellent	Good	Fair to poor	Excellent
GP	Good: tractor, rubber-tired, steel wheel or vibratory roller	115-125	Almost none	Good drainage, pervious	Reasonably stable	Excellent to good	Poor to fair	Poor	
GM	Good: rubber-tired or light sheepsfoot roller	120-135	Slight	Poor drainage, semipervious	Reasonably stable	Excellent to good	Fair to poor	Poor	Poor to fair
GC	Good to fair: rubber-tired or sheepsfoot roller	115-130	Slight	Poor drainage, impervious	Reasonably stable	Good	Good to fair **	Excellent	Excellent
SW	Good: tractor, rubber-tired or vibratory roller	110-130	Almost none	Good drainage, pervious	Very stable	Good	Fair to poor	Fair to poor	Good
SP	Good: tractor, rubber-tired or vibratory roller	100-120	Almost none	Good drainage, pervious	Reasonably stable when dense	Good to fair	Poor	Poor	Poor to fair
SM	Good: rubber-tired or sheepsfoot roller	110-125	Slight	Poor drainage, impervious	Reasonably stable when dense	Good to fair	Poor	Poor	Poor to fair
SC	Good to fair: rubber-tired or sheepsfoot roller	105-125	Slight to medium	Poor drainage, impervious	Reasonably stable	Good to fair	Fair to poor	Excellent	Excellent
ML	Good to poor: rubber-tired or sheepsfoot roller	95-120	Slight to medium	Poor drainage, impervious	Poor stability, high density required	Fair to poor	Not suitable	Poor	Poor
CL	Good to fair: sheepsfoot or rubber-tired roller	95-120	Medium	No drainage, impervious	Good stability	Fair to poor	Not suitable	Poor	Poor
OL	Fair to poor: sheepsfoot or rubber-tired roller	80-100	Medium to high	Poor drainage, impervious	Unstable, should not be used	Poor	Not suitable	Not suitable	Not suitable
MH	Fair to poor: sheepsfoot or rubber-tired roller	70-95	High	Poor drainage, impervious	Poor stability, should not be used	Poor	Not suitable	Very poor	Not suitable
CH	Fair to poor: sheepsfoot roller	80-105	Very high	No drainage, impervious	Fair stability, may soften on expansion	Poor to very poor	Not suitable	Very poor	Not suitable
OH	Fair to poor: sheepsfoot roller	65-100	High	No drainage, impervious	Unstable, should not be used	Very poor	Not suitable	Not suitable	Not suitable
Pt	Not suitable		Very high	Fair to poor drainage	Should not be used	Not suitable	Not suitable	Not suitable	Not suitable

* "The Unified Classification: Appendix A - Characteristics of Soil, Groups Pertaining to Roads and Airfields, and Appendix B - Characteristics of Soil Groups Pertaining to Embankments and Foundations," Technical Memorandum 357, U.S. Waterways Experiment Station, Vicksburg, 1953.

** Not suitable if subject to frost.

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

Major Divisions			*Group Symbols*		*Typical Names*	*Laboratory Classification Criteria*		
Coarse-grained soils (more than half of material is larger than No. 200 sieve size)	Gravels (More than half of coarse fraction is larger than No. 4 sieve size)	Clean gravels (little or no fines)	GW		Well-graded gravels, gravel-sand mixtures, little or no fines	Determine percentages of sand and gravel from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows: Less than 5 percent: GW, GP, SW, SP More than 12 percent: GM, GC, SM, SC 5 to 12 percent: *Borderline* cases requiring dual symbols[b]	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			GP		Poorly graded gravels, gravel-sand mixtrues, little or no fines		Not meeting all gradation requirements for GW	
		Gravels with fines (appreciable amount of fines)	GM[a]	d	Silty gravels, gravel-sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4	Limits plotting within shaded area, above "A" line with P.I. between 4 and 7 are *borderline* cases requiring use of dual symbols
				u				
			GC		Clayey gravels, gravel-sand-clay mixtures		Atterberg limits above "A" line or P.I. greater than 7	
	Sands (More than half of coarse fraction is smaller than No. 4 sieve size)	Clean sands (Little or no fines)	SW		Well-graded sands, gravelly sands, little or no fines		$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			SP		Poorly graded sands, gravelly sands, little or no fines		Not meeting all gradation requirements for SW	
		Sands with fines (Appreciable amount of fines)	SM[a]	d	Silty sands, sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4	Limits plotting within shaded area, above "A" line with P.I. between 4 and 7 are *borderline* cases requiring use of dual symbols
				u				
			SC		Clayey sands, sand-clay mixtures		Atterberg limits above "A" line or P.I. greater than 7	
Fine-grained soils (More than half 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			
			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 sandy or silty soils, elastic silts			
			CH		Inorganic clays of high plasticity, fat clays			
			OH		Organic clays of medium to high plasticity, organic silts			
	Highly organic soils		Pt		Peat and other highly organic soils			

[a] Division of GM and SM groups into subdivisions of d and u are for roads and airfields only. Subdivision is based on Atterberg limits, suffix d used when L.L. is 28 or less and the P.I. is 6 or less; the suffix u is used when L.L. is greater than 28.

[b] Borderline classifications, used for soils possessing characteristics of two groups, are designated by combinations of group sympols. For example GW-GC, well-graded gravel-sand mixture with clay binder.

GENERAL NOTES

SAMPLE IDENTIFICATION

All samples are visually classified in general accordance with the Unified Soil Classification System (ASTM D-2487-75 or D-2488-75)

DESCRIPTIVE TERM (% BY DRY WEIGHT)

Trace:	1-10%
Little:	11-20%
Some:	21-35%
And/Adjective	36-50%

PARTICLE SIZE (DIAMETER)

Boulders: 8 inch and larger	
Cobbles:	3 inch to 8 inch
Gravel:	coarse - ¾ to 3 inch
	fine – No. 4 (4.76 mm) to ¾ inch
Sand:	coarse – No. 4 (4.76 mm) to No. 10 (2.0 mm)
	medium – No. 10 (2.0 mm) to No. 40 (0.42 mm)
	fine – No. 40 (0.42 mm) to No. 200 (0.074 mm)
Silt:	No. 200 (0.074 mm) and smaller (non-plastic)
Clay:	No 200 (0.074 mm) and smaller (plastic)

SOIL PROPERTY SYMBOLS

Dd:	Dry Density (pcf)
LL:	Liquid Limit, percent
PL:	Plastic Limit, percent
PI:	Plasticity Index (LL-PL)
LOI:	Loss on Ignition, percent
Gs:	Specific Gravity
K:	Coefficient of Permeability
w:	Moisture content, percent
qp:	Calibrated Penetrometer Resistance, tsf
qs:	Vane-Shear Strength, tsf
qu:	Unconfined Compressive Strength, tsf
qc:	Static Cone Penetrometer Resistance (correlated to Unconfined Compressive Strength, tsf)
PID:	Results of vapor analysis conducted on representative samples utilizing a Photoionization Detector calibrated to a benzene standard. Results expressed in HNU-Units. (BDL=Below Detection Limit)
N:	Penetration Resistance per 12 inch interval, or fraction thereof, for a standard 2 inch O.D. (1⅜ inch I.D.) split spoon sampler driven with a 140 pound weight free-falling 30 inches. Performed in general accordance with Standard Penetration Test Specifications (ASTM D-1586). N in blows per foot equals sum of N-Values where plus sign (+) is shown.
Nc:	Penetration Resistance per 1¾ inches of Dynamic Cone Penetrometer. Approximately equivalent to Standard Penetration Test N-Value in blows per foot.
Nr:	Penetration Resistance per 12 inch interval, or fraction thereof, for California Ring Sampler driven with a 140 pound weight free-falling 30 inches per ASTM D-3550. Not equivalent to Standard Penetration Test N-Value.

DRILLING AND SAMPLING SYMBOLS

SS:	Split-Spoon
ST:	Shelby Tube – 3 inch O.D. (except where noted)
CS:	3 inch O.D. California Ring Sampler
DC:	Dynamic Cone Penetrometer per ASTM Special Technical Publication No. 399
AU:	Auger Sample
DB:	Diamond Bit
CB:	Carbide Bit
WS:	Wash Sample
RB:	Rock-Roller Bit
BS:	Bulk Sample
Note:	Depth intervals for sampling shown on Record of Subsurface Exploration are not indicative of sample recovery, but position where sampling initiated

SOIL STRENGTH CHARACTERISTICS

COHESIVE (CLAYEY) SOILS

COMPARATIVE CONSISTENCY	BLOWS PER FOOT (N)	UNCONFINED COMPRESSIVE STRENGTH (TSF)
Very Soft	0 - 2	0 - 0.25
Soft	3 - 4	0.25 - 0.50
Medium Stiff	5 – 8	0.50 - 1.00
Stiff	9 – 15	1.00 - 2.00
Very Stiff	16 – 30	2.00 - 4.00
Hard	31+	4.00+

NON-COHESIVE (GRANULAR) SOILS

RELATIVE DENSITY	BLOWS PER FOOT (N)
Very Loose	0 - 4
Loose	5 - 10
Firm	11 - 30
Dense	31 - 50
Very Dense	51+

DEGREE OF PLASTICITY	PI
None to Slight	0 - 4
Slight	5 - 10
Medium	11 - 30
High to Very High	31+

DEGREE OF EXPANSIVE POTENTIAL	PI
Low	0 - 15
Medium	15 - 25
High	25+

Important Information About Your
Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

The following information is provided to help you manage your risks.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual

subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely, on Your ASFE-Member Geotechncial Engineer for Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you ASFE-member geotechnical engineer for more information.

ASFE
The Best People on Earth

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Expect Excellence

ENGEO “BURIED PIER WALL DESIGN”
(MAY 8, 2019)

ENGEO
Expect Excellence

GEOTECHNICAL
ENVIRONMENTAL
WATER RESOURCES
CONSTRUCTION SERVICES

Project No.
14986.000.000

May 8, 2019

Mr. Mike Conn
Meridian Property Ventures LLC
2420 Camino Ramon, Suite 215
San Ramon, CA 94583

Subject: Proposed Chick-Fil-A Restaurant
North Livermore Road
Livermore, California

BURIED CAST-IN-DRILLED-HOLE PIER WALL DESIGN

Dear Mr. Conn:

As requested and with your authorization, this letter presents the final buried cast-in-drilled-hole (CIDH) pier wall design to address potential creek bank erosion of Arroyo Las Positas adjacent to the subject project in Livermore, California.

The buried pier wall will consist of a series of closely spaced, reinforced CIDH concrete piers constructed below grade and parallel to the creek bank. The design parameters and recommendations incorporated into the buried pier wall design are based on review of References 1 through 5. The final design consists of 24-inch-diameter piers reinforced with eight #8 longitudinal rebars, with piers spaced 6 feet on center, and extending 30 feet below existing grade.

The proposed buried pier wall location is based on recommendations contained in the creek scour evaluation (Reference 2), and considers the proposed restaurant and associated drive-thru locations. There is a minimum 8-foot offset from the back of the buried piers to the proposed restaurant and associated drive-thru. If at a future date the stream bank erodes to the pier wall, additional mitigation measures such as lagging and/or additional piers can be installed from within this offset area to address the concern at that time. We can work with you and the design team further to determine maintenance cost estimates.

The buried pier wall design calculations (Appendix A), slope stability analysis and summary (Appendix B), and construction drawings (Sheets 1 through 3) are attached to this letter.

Drilled pier construction recommendations are provided below.

DRILLED PIER CONSTRUCTION RECOMMENDATIONS

The following guidelines should be used during construction of drilled-pier foundations.

1. No two adjacent piers should be constructed concurrently.
2. Loose soils should be cleaned from the bottom of the pier boreholes using a cleanout bucket.

3. Pier boreholes should be inspected and approved by the geotechnical engineer prior to the installation of reinforcement. Extreme care in drilling, placement of steel, and the placing of concrete is essential to avoid excessive disturbance of pier boring walls. Drilling operations and concrete placement should be coordinated so that caisson holes are left open a minimum amount of time. Depressions at the tops of the piers, resulting from drilling operations or from any other cause, should be backfilled to prevent ponding. Concrete collars occurring at the tops of piers as a result of overpouring should be removed.
4. Concrete placement by pumping or tremie tube to the bottom of the pier borings is recommended. Specifications should require that sufficient space be provided in the reinforcing cage during fabrication to allow the insertion of a tremie tube for concrete placement. The reinforcing cage should be installed and the concrete pumped immediately after drilling is completed. At the time of concrete placement, the volume of concrete entering the drilled holes should be monitored to confirm that additional loss of ground has not occurred between drilling operations and the placement of concrete.
5. Sandy soils with gravels and possible cobbles were documented in References 3 and 4. These materials may present drilling challenges, and may be susceptible to caving. The drill rig should be capable of advancing through these material types to the proposed pier depths. If caving occurs, a temporary casing or wet construction method may be required during construction. Casings should have an outer diameter equal to or exceeding the pile diameter. Temporary casing should be placed tight-in-hole. The temporary casing should be retrieved as the concrete is being placed, while always maintaining at least a 5-foot head of concrete inside the casing.
6. Groundwater may be encountered during pier drilling, based on review of References 3 and 4. Groundwater levels can fluctuate due to seasonal rainfall amount, local irrigation, any groundwater-recharge program, and other conditions. If groundwater is encountered, it should be pumped prior to concrete placement.

If you have any questions regarding this project or would like to this letter, please do not hesitate to contact us.

Sincerely,

ENGEO Incorporated

REGISTERED PROFESSIONAL ENGINEER
ANDREW FIRMIN
No. 3025
GEOTECHNICAL
STATE OF CALIFORNIA

Jerry Chen

Andrew Firmin, GE

Uri Eliahu, GE
jc/af/ue/JF

Attachments: Selected References
Construction Drawings (Sheets 1 – 3)
Appendix A - Buried Pier Wall Design Calculations
Appendix B - Slope Stability Analysis and Summary

SELECT REFERENCES

1. ENGEO; CFA North Livermore Conceptual Buried Pier Wall Design; North Livermore Avenue, Livermore, California; May 14, 2018, Revised May 29, 2018; Project No. 14986.000.000

2. ENGEO; Geotechnical Exploration Update; Proposed Chick-Fil-A Restaurant; North Livermore Avenue; Livermore, California; March 1, 2019.

3. ENGEO; North Livermore Avenue Scour Analysis; North Livermore Avenue, Livermore, California; May 3, 2019; Project No. 14986.000.000

4. Giles Engineering Associates, Inc.; Geotechnical Engineering Exploration and Analysis; Proposed Chick-Fil-A Restaurant #3805, North Livermore Avenue and I-580; Livermore, California; April 24, 2017.

5. Giles Engineering Associates, Inc.; Slope Stability Evaluation; Proposed Chick-Fil-A Restaurant #3805, North Livermore Avenue and I-580; Livermore, California; May 3, 2018.

6. ENGEO; Response to City Comments; Proposed Chick-Fil-A Restaurant; North Livermore Avenue; Livermore, California; January 24, 2019.

CONSTRUCTION DRAWINGS (SHEETS 1 – 3)

14986.000.000
May 8, 2019

BURIED CAST-IN-DRILLED-HOLE PIER WALL

PROPOSED CHICK-FIL-A RESTAURANT
NORTH LIVERMORE AVENUE
LIVERMORE, CALIFORNIA

DESIGN BASIS:

* ENGEO, GEOTECHNICAL EXPLORATION UPDATE; PROPOSED CHICK-FIL-A RESTAURANT; NORTH LIVERMORE AVENUE; LIVERMORE, CALIFORNIA; MARCH 1, 2019.

*ENGEO, CREEK SCOUR EVALUATION AND RECOMMENDATIONS; PROPOSED CHICK-FIL-A RESTAURANT, NORTH LIVERMORE AVENUE, LIVERMORE, CALIFORNIA; MAY 3, 2019.

*GILES ENGINEERING ASSOCIATES, INC.; GEOTECHNICAL ENGINEERING EXPLORATION AND ANALYSIS; PROPOSED CHICK-FIL-A RESTAURANT #3805, NORTH LIVERMORE AVENUE AND I-580; LIVERMORE, CALIFORNIA; APRIL 24, 2017.

*GILES ENGINEERING ASSOCIATES, INC.; SLOPE STABILITY EVALUATION; PROPOSED CHICK-FIL-A RESTAURANT #3805, NORTH LIVERMORE AVENUE AND I-580; LIVERMORE CALIFORNIA; MAY 3, 2018.

EARTHWORK AND CONSTRUCTION NOTES:

1. CONSTRUCTION IS TO CONFORM TO THE CALIFORNIA BUILDING CODE, ACI-318, 2000 CAL-OSHA SAFETY ORDERS, AND ALL LOCAL BUILDING CODES.
2. THE PIER DRILLING SHALL HAVE PERIODIC OBSERVATION BY AN ENGEO REPRESENTATIVE PRIOR TO REINFORCING STEEL PLACEMENT. REFER TO STATEMENT OF SPECIAL INSPECTIONS.
3. SPECIAL INSPECTION IS REQUIRED PERIODICALLY FOR REINFORCING STEEL PLACEMENT AND CONTINUOUSLY FOR CONCRETE PLACEMENT. PIERS SHALL BE DRILLED TO SPECIFIED DEPTH AS SHOWN ON PLANS. REFER TO STATEMENT OF SPECIAL INSPECTIONS.
4. GRANULAR SOILS, INCLUDING GRAVELS AND POSSIBLE COBBLES WERE ENCOUNTERED IN GEOTECHNICAL BORINGS. THE DRILLING RIG SHOULD BE CAPABLE OF ADVANCING TO PLANNED PIER DEPTHS THROUGH SUCH SOIL TYPES.
5. TEMPORARY CASING MAY BE REQUIRED DURING CONSTRUCTION OF DRILLED PIERS.
6. GROUNDWATER WAS ENCOUNTERED DURING DRILLING OF GEOTECHNICAL BORINGS. IF ENCOUNTERED, GROUNDWATER SHOULD BE PUMPED FROM PIER HOLES PRIOR TO CONCRETE PLACEMENT. CONCRETE SHOULD BE PLACED USING THE TREMIE METHOD.
7. NO TWO ADJACENT PIER HOLES SHOULD BE CONSTRUCTED CONCURRENTLY.
8. THE CONTRACTOR SHALL VERIFY ALL DIMENSIONS.
9. EXISTING UTILITIES AND OTHER IMPROVEMENTS SHOWN ON THE TOPOGRAPHIC BASE MAP ARE BASED ON RECORD LOCATIONS. ADDITIONAL UTILITIES MAY BE PRESENT OR IN OTHER LOCATIONS. THE CONTRACTOR SHALL CONFIRM AND/OR DETERMINE THE LOCATION OF ALL UTILITIES AND DRILLING CLEARANCE PRIOR TO PROCEEDING WITH DRILLING OPERATIONS.
10. STOP DRILLING AND CONSULT THE ENGINEER IF UTILITY LINES, PIPING, FOUNDATIONS OR INDICATIONS OF OBSTRUCTIONS ARE ENCOUNTERED DURING DRILLING OF THE PIERS.
11. IF THE ACTUAL FIELD CONDITIONS VARY FROM THE ANTICIPATED CONDITIONS, NOTIFY THE GEOTECHNICAL ENGINEER.
12. EXCAVATION AND SHORING SHALL BE PERFORMED IN ACCORDANCE WITH OSHA, STATE, AND LOCAL JURISDICTIONAL REQUIREMENTS.
13. EARTHWORK SHALL BE IN CONFORMANCE WITH THE REFERENCED GEOTECHNICAL REPORT.
14. BACKFILLING SHALL BE PERFORMED USING LIGHT, HAND-OPERATED COMPACTION EQUIPMENT.

CONCRETE

1. CONCRETE SHALL CONFORM TO ACI 318.
2. CONCRETE SHALL HAVE A MINIMUM COMPRESSIVE STRENGTH OF 3,000 PSI.
3. CEMENT SHALL CONFORM TO ASTM C150, TYPE II OR V.
4. AGGREGATES SHALL BE NATURAL SANDS AND ROCK AGGREGATES THAT CONFORM TO ASTM C33.
5. MAXIMUM WATER-CEMENT RATIO OF 0.5.
6. PRIOR TO CONCRETE PLACEMENT, CIDH PIER EXCAVATIONS SHALL BE CLEAN OF DEBRIS AND STANDING WATER.

REINFORCING STEEL

1. REINFORCING STEEL SHALL BE ASTM A615, GRADE 60 DEFORMED BARS.
2. REINFORCING STEEL SHALL BE DETAILED, FABRICATED, PLACED AND LAPPED PER ACI DETAILING MANUAL 315.
3. PROVIDE 3" CLEARANCE FOR CONCRETE AGAINST EARTH, 2" FOR CONCRETE EXPOSED TO AIR, AND 1 1/2" FOR ALL INTERIOR EXPOSURE.

SHEET INDEX

STATEMENT OF SPECIAL INSPECTIONS

INSPECTION OF THE MATERIALS, INSTALLATION, FABRICATION, ERECTION, OR PLACEMENT OF COMPONENTS AND CONNECTIONS TO CONFIRM COMPLIANCE WITH THE CONSTRUCTION DOCUMENTS ACI 318 LATEST EDITION, AND 2016 CALIFORNIA BUILDING CODE IS REQUIRED. ITEMS REQUIRING SPECIAL INSPECTION SHALL BE AS FOLLOWS:

REQUIRED VERIFICATION AND INSPECTION OF SOILS

REQUIRED VERIFICATION AND INSPECTION TASK	CONTINUOUS	PERIODIC
1. VERIFY MATERIALS BELOW FOOTINGS ARE ADEQUATE TO ACHIEVE DESIGN BEARING CAPACITY	-	X
2. VERIFY EXCAVATIONS ARE EXTENDED TO PROPER DEPTH AND HAVE REACHED PROPER MATERIAL	-	X
3. PERFORM CLASSIFICATION AND TESTING OF CONTROLLED FILL MATERIALS	-	X
4. VERIFY USE OF PROPER MATERIALS, DENSITIES AND LIFT THICKNESSES DURING PLACEMENT, AND COMPACTION OF CONTROLLED FILL	X	-
5. PRIOR TO PLACEMENT OF FILL, OBSERVE SUBGRADE AND VERIFY THAT SITE HAS BEEN PREPARED PROPERLY	-	X
6. PIER DRILLING	-	X

REQUIRED VERIFICATION AND INSPECTION OF CONCRETE CONSTRUCTION

REQUIRED VERIFICATION AND INSPECTION TASK	CONTINUOUS	PERIODIC
1. INSPECTION OF REINFORCING STEEL AND PLACEMENT	-	X
2. VERIFY USE OF REQUIRED DESIGN MIX	-	X
3. AT THE TIME FRESH CONCRETE IS SAMPLED TO FABRICATE SPECIMENS FOR STRENGTH TESTS, PERFORM SLUMP TESTS, AND DETERMINE THE TEMPERATURE OF THE CONCRETE	X	-
4. INSPECTION OF CONCRETE PLACEMENT FOR PROPER APPLICATION TECHNIQUES	X	-
5. INSPECTION FOR MAINTENANCE OF SPECIFIED CURING TEMPERATURE AND TECHNIQUES	-	X
6. INSPECT FORMWORK FOR SHAPE, LOCATION, AND DIMENSIONS OF THE CONCRETE MEMBER BEING FORMED	-	X

2010 CROW CANYON PLACE
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FAX (888) 279-2698

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SANTA CLARITA - IRVINE - NEW ZEALAND

TITLE SHEET AND NOTES
BURIED CAST-IN-DRILLED-HOLE PIER WALL
PROPOSED CHICK-FIL-A RESTAURANT
LIVERMORE, CALIFORNIA

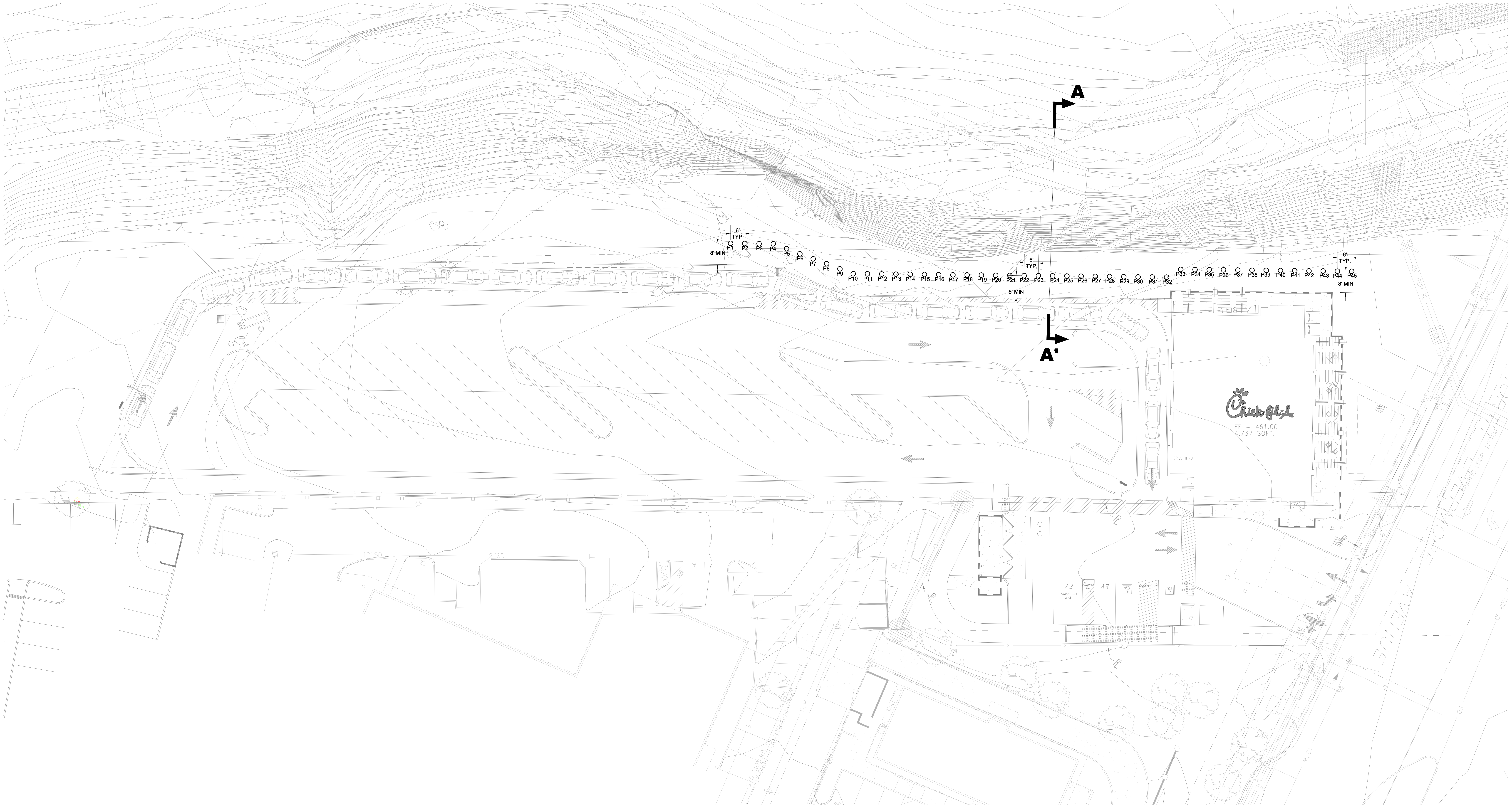
ANDREW FIRMIN
No. 71583

DESIGNED BY: AHF
DRAWN BY: EJ
CHECKED BY: UE
DATE: MAY 2019
SCALE: AS SHOWN

REV.	DATE	DESCRIPTION	BY
-			

SHEET NUMBER
1
OF 3 SHEETS

PROJECT NO.
14986.000.000



SITE PLAN

1"=20'

N

0 20
FEET

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

○ ○ ○ ○ ○ ○ ○ P45 PROPOSED BURIED PIER WALL WITH PIER NUMBER

A A' CROSS SECTION LOCATION SEE SHEET 3

BASE MAP SOURCE: JOSEPH C. TRUXAW & ASSOCIATES, INC. (2018)

G:\Drafting\DRAFTING2_Dwg_13000 Plus\14986\000\0519 - WALL\14986000000--2-SitePlan-0519.dwg Plot Date:5-08-19 EJimenez

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SITE PLAN

BURIED CAST-IN-DRILLED-HOLE PIER WALL
PROPOSED CHICK-FIL-A RESTAURANT
LIVERMORE, CALIFORNIA

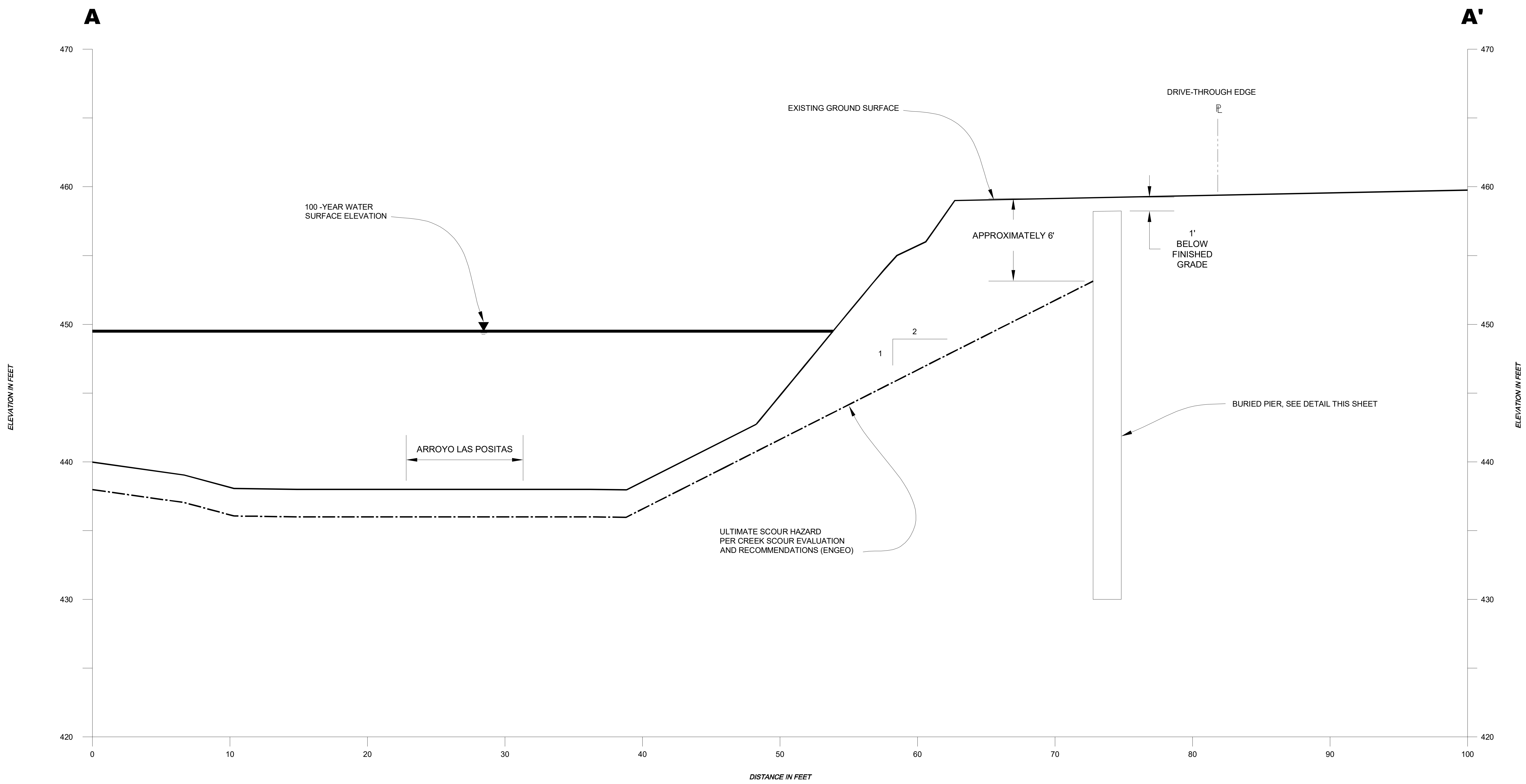
ANDREW FIRMIN
No. 71583
Exp. 12/31/2013

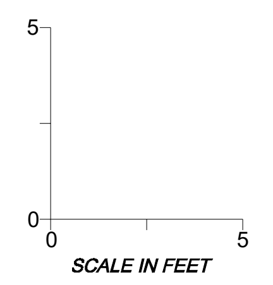
DESIGNED BY: AHF
DRAWN BY: EJ
CHECKED BY: UE
DATE: MAY 2019
SCALE: AS SHOWN

REV.	DATE	DESCRIPTION	BY

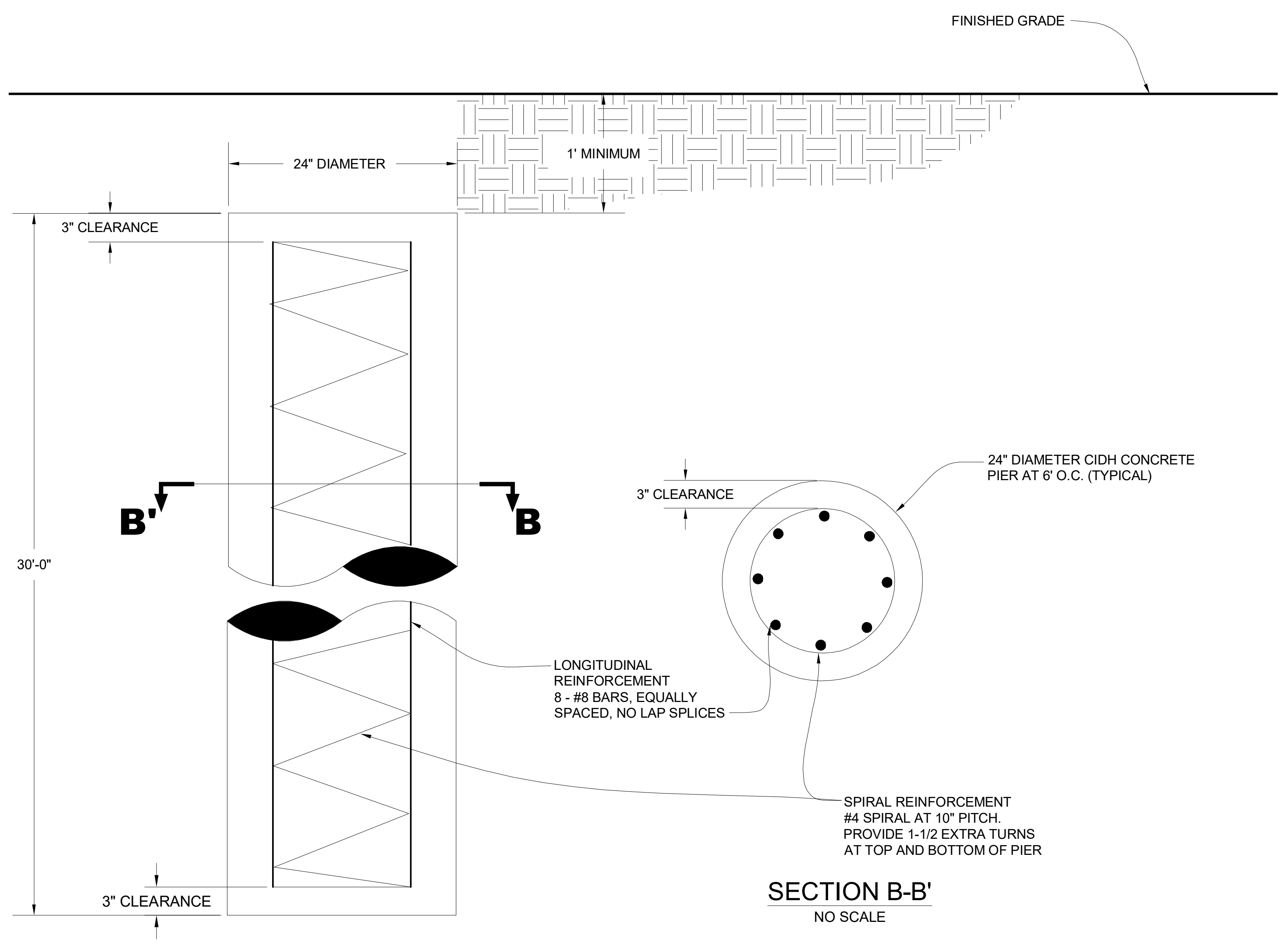
SHEET NUMBER
2
OF 3 SHEETS

PROJECT NO.
14986.000.000





SECTION A-A'
1"=5'



SECTION B-B'
NO SCALE

BURIED CAST-IN-DRILLED-HOLE (CIDH) PIER TYPICAL DETAIL
NO SCALE

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CROSS SECTION AND DETAIL
BURIED CAST-IN-DRILLED-HOLE PIER WALL
PROPOSED CHICK-FIL-A RESTAURANT
LIVERMORE, CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER
ANDREW FIRMIN
No. 71583
Exp. 12/31/2013
CIVIL
STATE OF CALIFORNIA

DESIGNED BY: AHF
DRAWN BY: EJ
CHECKED BY: UE
DATE: MAY 2019
SCALE: AS SHOWN

REV.	DATE	DESCRIPTION	BY
—			

SHEET NUMBER
3
OF 3 SHEETS

PROJECT NO.
14986.000.000

APPENDIX A

Buried Pier Wall Design Calculations

14986.000.000
May 8, 2019

APPENDIX A – DESIGN SUMMARY

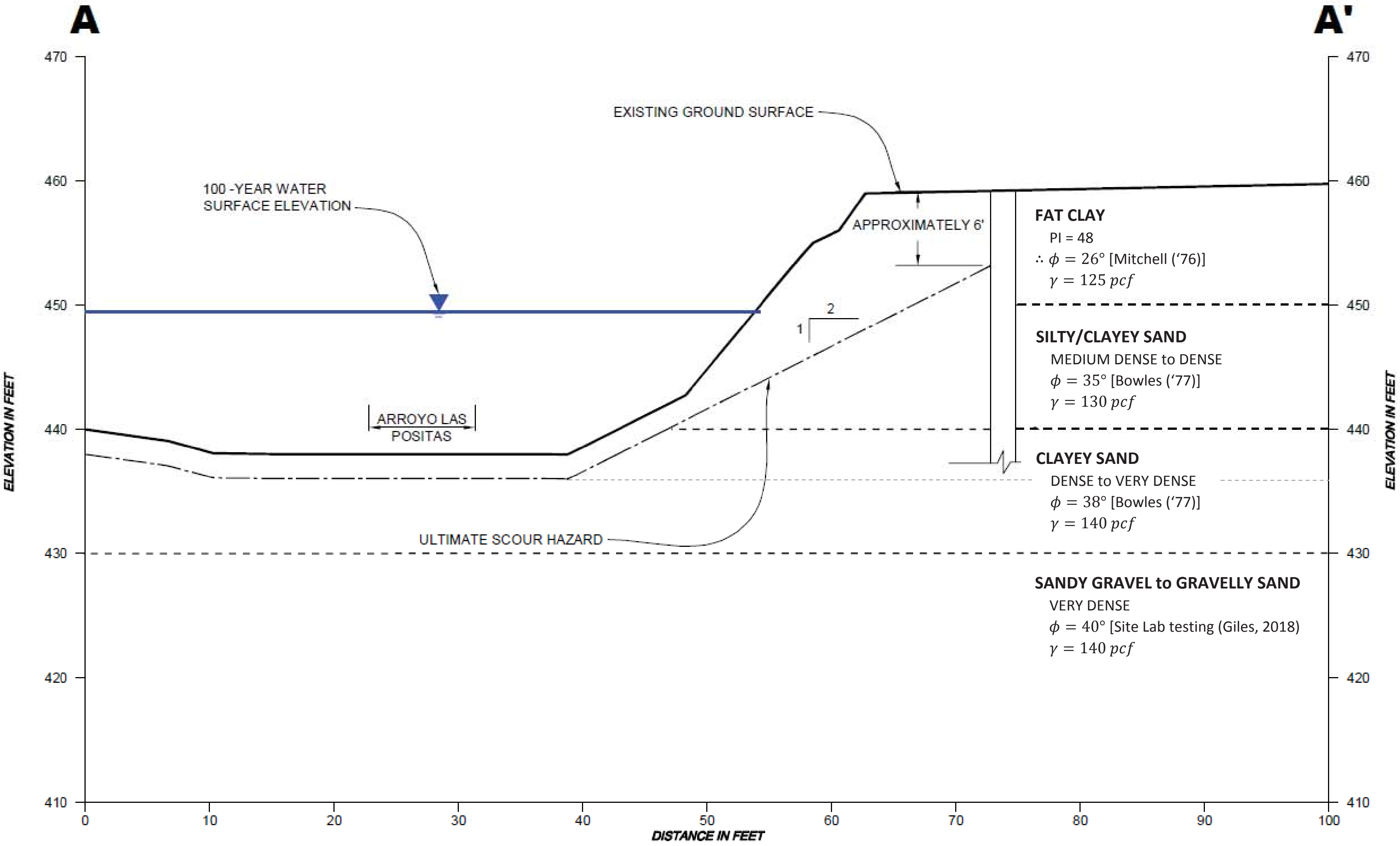
Buried Cast-in-Drilled-Hole Pier Wall Design
Proposed Chick-Fil-A Restaurant
Livermore, California

This design summary worksheet provides an outline of the design procedure utilized for the buried pier wall design.

1. We reviewed the referenced geotechnical reports to determine a generalized subsurface stratigraphy. We reviewed boring logs and associated laboratory test results for the site, as well as published references, to determine design parameters for the identified subsurface soil layers.
2. For buried pier wall design, we identified two loading cases. Case 1 assumed a static condition where the groundwater table is assumed to be at Elevation 440 feet, at the approximate base elevation of the arroyo. Case 2 assumed a rapid drawdown condition where the creek bank behind the pier wall retained the 100-year flood event water level at Elevation 450 feet, while the adjacent creek draws down to static water levels at Elevation 440 feet.
3. We used Rankine earth pressure theory to determine active and passive lateral earth pressure loading diagrams for Cases 1 and 2. We applied a factor of safety of 1.5 and 1.2 to calculated ultimate passive earth pressures for Cases 1 and 2, respectively. Passive earth pressure was applied over twice the effective width of the pier diameter, considering the effects of soil arching.
4. We utilized the computer program CivilTech (CT) Shoring (Version 8) to calculate the minimum required pier embedment based on the loading diagrams for Cases 1 and 2. The program also develops shear and moment diagrams and maximum shear forces and moments for the piles.
5. We utilized the computer program LPILE (Version 5) to calculate the ultimate bending moment capacity of the final pier design section, and compared the result to the computed maximum moment applied to the pile (Step 4 above).

We also performed slope stability analyses to evaluate the stability of the creek bank. A summary of our slope stability analyses is provided in Appendix B.

ASSUMED GEOMETRY AND SUBSURFACE SOIL CONDITIONS



SECTION A-A'

SCALE 1" = 10'

ACTIVE EARTH PRESSURE

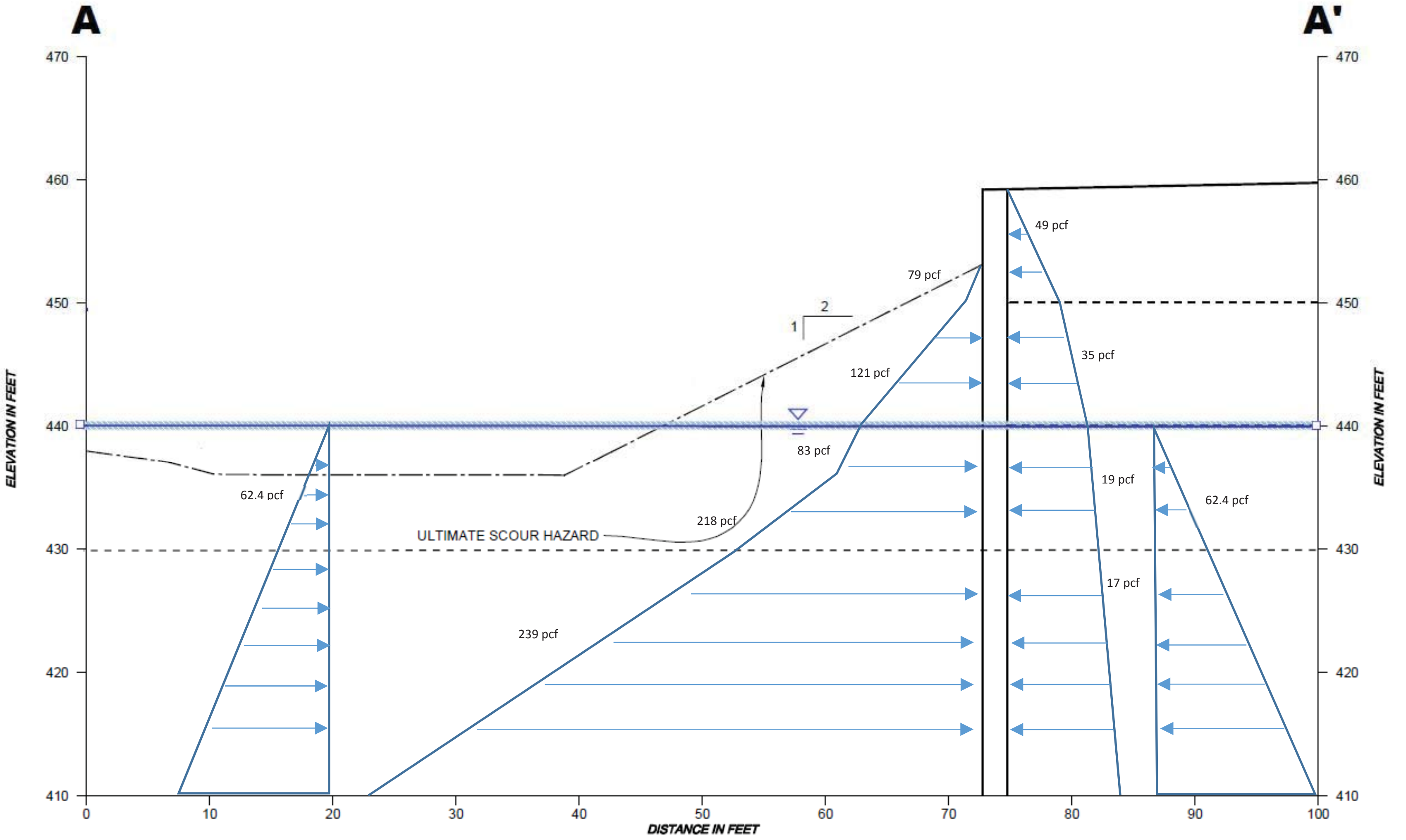
			Case 1: GWT @ 440'		*Case 2: GWT @ 450'*	
Layer	$\boldsymbol{\phi}$ (°)	$\boldsymbol{K_a}$	$\boldsymbol{\gamma'}$ **(pcf)**	$\boldsymbol{P_a}$ **(pcf)**	$\boldsymbol{\gamma'}$ **(pcf)**	$\boldsymbol{P_a}$ **(pcf)**
Fat Clay	26	$K_a = \tan^2(45 - \frac{\phi}{2}) = \mathbf{0.39}$	**125**	**49**	**125**	**49**
Silty/Clayey Sand	35	$K_a = \tan^2(45 - \frac{\phi}{2}) = \mathbf{0.27}$	**130**	**35**	130 – 62 = **68**	**18**
Clayey Sand	38	$K_a = \tan^2(45 - \frac{\phi}{2}) = \mathbf{0.24}$	140 – 62 = **78**	**19**	140 – 62 = **78**	**19**
Sandy gravel/gravelly sand	40	$K_a = \tan^2(45 - \frac{\phi}{2}) = \mathbf{0.22}$	140 – 62 = **78**	**17**	140 – 62 = **78**	**17**

PASSIVE EARTH PRESSURE (GWT at 440')

			Case 1: FoS = 1.5		*Case 2: FoS = 1.2*	
Layer	$\boldsymbol{\phi}$ (°)	$\boldsymbol{K_p}$	$\boldsymbol{\gamma'}$ **(pcf)**	$\boldsymbol{P_p}$ **(pcf)**	$\boldsymbol{\gamma'}$ **(pcf)**	$\boldsymbol{P_p}$ **(pcf)**
Fat Clay	26	$K_p = \mathbf{0.95*}$	**125**	**79**	**125**	**99**
Silty/Clayey Sand	35	$K_p = \mathbf{1.40*}$	**130**	**121**	**130**	**152**
Clayey Sand – 1	38	$K_p = \mathbf{1.60*}$	140 – 62 = **78**	**83**	140 – 62 = **78**	**104**
Clayey Sand – 2	38	$K_p = \tan^2(45 + \frac{\phi}{2}) = \mathbf{4.20}$	140 – 62 = **78**	**218**	140 – 62 = **78**	**273**
Sandy gravel/gravelly sand	40	$K_p = \tan^2(45 + \frac{\phi}{2}) = \mathbf{4.60}$	140 – 62 = **78**	**239**	140 – 62 = **78**	**299**

*From NAVFAC with sloping foreground of 2:1 (Horizontal:Vertical)

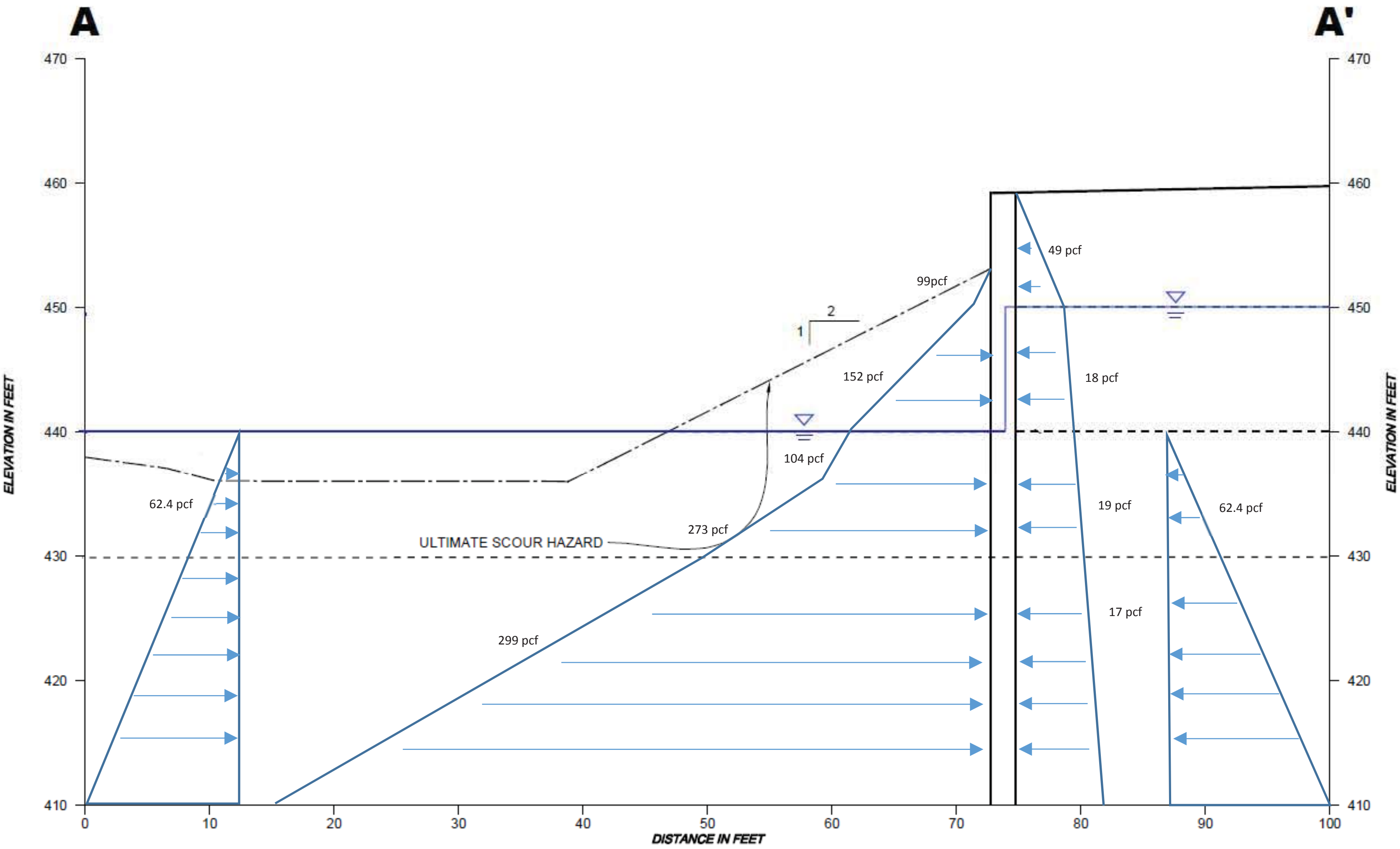
CASE 1 – STATIC CONDITIONS



SECTION A-A'

SCALE 1" = 10'

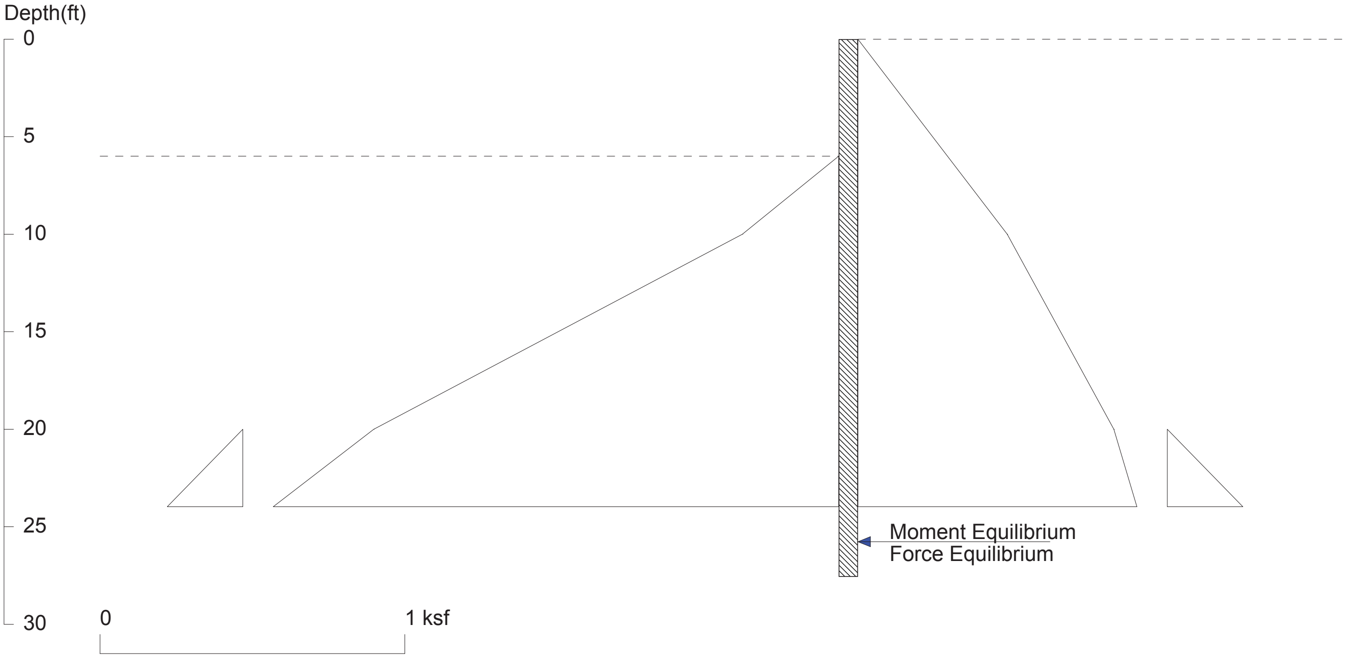
CASE 2 – RAPID DRAWDOWN



SECTION A-A'

SCALE 1" = 10'

Corner Pile Case



<ShoringSuite> CIVILTECH SOFTWARE USA www.civiltech.com

Licensed to Date: 5/6/2019

File: C:\Users\JChen\Desktop\CT Shoring\CFA.sh8

Wall Height=6.0 Pile Diameter=2.0 Pile Spacing=6.0 Wall Type: 2. Soldier Pile, Drilled

PILE LENGTH: Min. Embedment=21.57 Min. Pile Length=27.57

MOMENT IN PILE: Max. Moment=100.35 per Pile Spacing=6.0 at Depth=17.02

USE 30' MINIMUM
PIER LENGTH

PILE SELECTION:

Mu = 1.6 * 100 = 160 K-FT

Request Min. Section Modulus = 50.7 in3/pile=830.49 cm3/pile, Fy= 36 ksi = 248 MPa, Fb/Fy=0.66

-> Piles meet Min. Section Requirements: Top Deflection is shown in (in)

W8X58 (0.98) HP10X57 (0.76) W10X49 (0.82) HP12X53 (0.57) W12X40 (0.73)
HP13X60 (0.45) HP14X73 (0.31) W14X38 (0.58) W16X36 (0.50) HP16X88 (0.20)
W16X89 (0.17) HP16X101 (0.17) W16X100 (0.15) HP16X121 (0.14)

DRIVING PRESSURES (ACTIVE, WATER, & SURCHARGE):

Z1	P1	Z2	P2	Slope
0	0	10	0.490	0.049000
10	0.490	20	0.840	0.035000
20	0.840	30	1.030	0.019000
20	0	300	17.472	0.062400

PASSIVE PRESSURES:

Z1	P1	Z2	P2	Slope
6	0	10	0.316	.079
10	0.316	20	1.526	.121
20	1.526	24	1.858	.083
24	1.858	30	3.166	.218
20	0	300	17.472	.0624

ACTIVE SPACING:

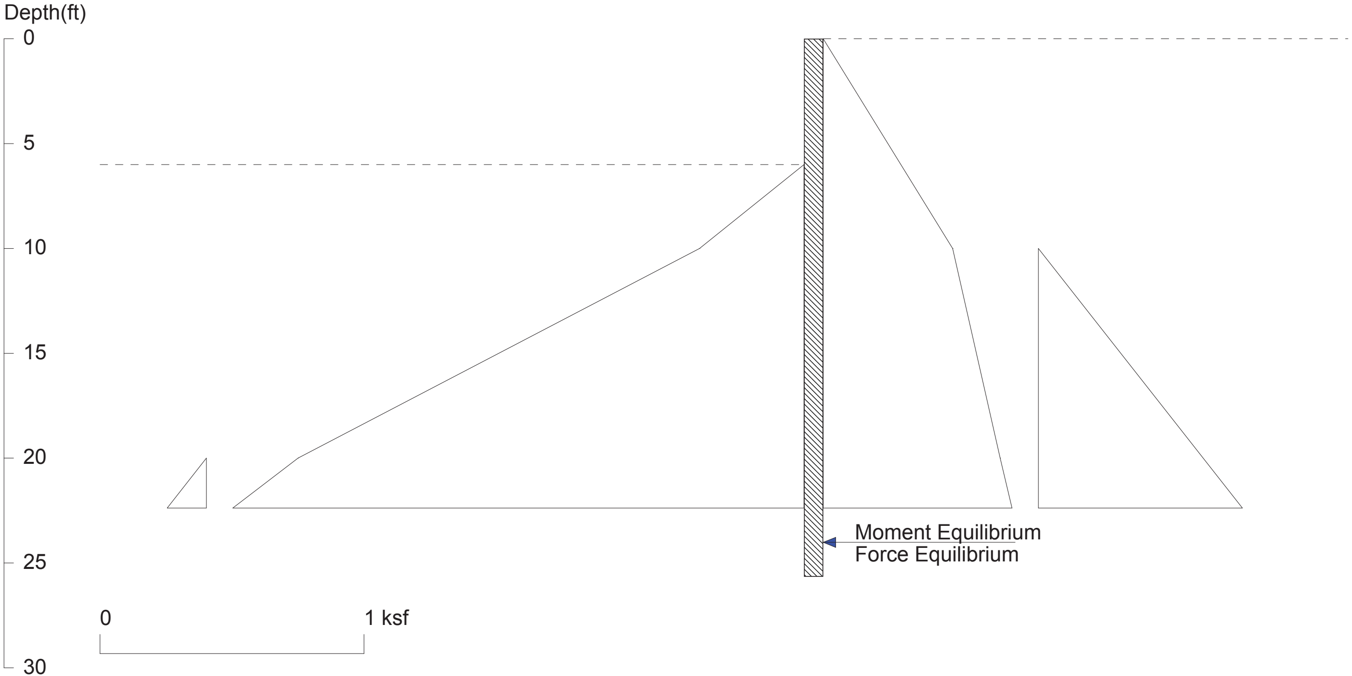
No.	Z depth	Spacing
1	0.00	6.00
2	10.00	6.00
3	10.01	2.00
4	300.00	2.00

PASSIVE SPACING:

No.	Z depth	Spacing
1	6.00	4.00
2	300.00	4.00

UNITS: Width,Spacing,Diameter,Length,and Depth - ft; Force - kip; Moment - kip-ft
Friction,Bearing,and Pressure - ksf; Pres. Slope - kip/ft3; Deflection - in

Corner Pile Case



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File: C:\Users\JChen\Desktop\CT Shoring\CFA_case 2.sh8

Wall Height=6.0 Pile Diameter=2.0 Pile Spacing=6.0 Wall Type: 2. Soldier Pile, Drilled

PILE LENGTH: Min. Embedment=19.65 Min. Pile Length=25.65
MOMENT IN PILE: Max. Moment=87.12 per Pile Spacing=6.0 at Depth=15.96

PILE SELECTION:
Request Min. Section Modulus = 44.0 in3/pile=721.00 cm3/pile, Fy= 36 ksi = 248 MPa, Fb/Fy=0.66
-> Piles meet Min. Section Requirements: Top Deflection is shown in (in)
W8X58 (0.78) HP10X57 (0.60) W10X45 (0.71) HP12X53 (0.45) W12X35 (0.62)
HP13X60 (0.35) HP14X73 (0.24) W14X34 (0.52) W16X31 (0.47) HP16X88 (0.16)
W16X89 (0.14) HP16X101 (0.14) W16X100 (0.12) HP16X121 (0.11)

DRIVING PRESSURES (ACTIVE, WATER, & SURCHARGE):

Z1	P1	Z2	P2	Slope
0	0	10	0.490	.049
10	0.490	20	0.670	.018
20	0.670	30	0.860	.019
10	0	300	18.096	.0624

PASSIVE PRESSURES:

Z1	P1	Z2	P2	Slope
6	0	10	0.396	.099
10	0.396	20	1.916	.152
20	1.916	24	2.332	.104
24	2.332	30	3.970	.273
20	0	300	17.472	.0624

ACTIVE SPACING:

No.	Z depth	Spacing
1	0.00	6.00
2	10.00	6.00
3	10.01	2.00
4	300.00	2.00

PASSIVE SPACING:

No.	Z depth	Spacing
1	6.00	4.00
2	300.00	4.00

UNITS: Width,Spacing,Diameter,Length,and Depth - ft; Force - kip; Moment - kip-ft
Friction,Bearing,and Pressure - ksf; Pres. Slope - kip/ft3; Deflection - in

```
================================================================================

                 LPILE Plus for Windows, Version 5.0 (5.0.47)

                Analysis of Individual Piles and Drilled Shafts
               Subjected to Lateral Loading Using the p-y Method

                         (c) 1985-2010 by Ensoft, Inc.
                              All Rights Reserved

================================================================================

This program is licensed to:

ENGEO Incorporated
ENGEO Incorporated

--------------------------------------------------------------------------------
                            Files Used for Analysis
--------------------------------------------------------------------------------

Path to file locations:      C:\Documents and Settings\JChen\Desktop\
Name of input data file:     CFA.lpd
Name of output file:         CFA.lpo
Name of plot output file:    CFA.lpp
Name of runtime file:        CFA.lpr


------------------------------------------------------------------------------
                          Time and Date of Analysis
------------------------------------------------------------------------------

               Date:  May  6, 2019     Time:  15:53:18


------------------------------------------------------------------------------
                                Problem Title
------------------------------------------------------------------------------

New LPILE Plus 5.0 Data File


------------------------------------------------------------------------------
                                Program Options
------------------------------------------------------------------------------

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 2:
- Computation of Ultimate Bending Moment of Cross Section (Section Design)



------------------------------------------------------------------------------
   Computations of Nominal Moment Capacity and Nonlinear Bending Stiffness
------------------------------------------------------------------------------

Number of sections =  1
```

```
Pile Section No.  1

The sectional shape is a circular drilled shaft (bored pile).

Outside Diameter                             =      24.0000 in

Material Properties:

Compressive Strength of Concrete             =        2.500 kip/in**2
Yield Stress of Reinforcement                =          60. kip/in**2
Modulus of Elasticity of Reinforcement =       29000. kip/in**2
Number of Reinforcing Bars                   =            8
Area of Single Bar                           =      0.60000 in**2
Number of Rows of Reinforcing Bars           =            5
Area of Steel                                =        4.800 in**2
Area of Shaft                                =      452.389 in**2
Percentage of Steel Reinforcement            =        1.061 percent
Cover Thickness (edge to bar center)         =        3.000 in

Unfactored Axial Squash Load Capacity        =      1239.13 kip

Distribution and Area of Steel Reinforcement

 Row          Area of         Distance to
Number     Reinforcement   Centroidal Axis
               in**2               in
------     -------------   ---------------
   1            0.600             9.000
   2            1.200             6.364
   3            1.200             0.000
   4            1.200            -6.364
   5            0.600            -9.000

Unfactored (Nominal) Moment Capacity at Concrete Strain of 0.003 =   2389.87934
in-kip

The analysis ended normally.
```

Mn = 2389 K-IN = 199 K-FT
PHI * Mn = 0.9 * 199 = 179 K-FT > 160 K-FT OK

APPENDIX B

Slope Stability Analysis and Summary

14986.000.000
May 8, 2019

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APPENDIX B – SLOPE STABILITY ANALYSIS AND SUMMARY

Buried Cast-in-Drilled-Hole Pier Wall Design
Proposed Chick-Fil-A Restaurant
Livermore, California

Limit equilibrium slope stability analyses were performed using the computer-aided program Slide (produced by Rocscience) and following the Bishop simplified method. We analyzed potential circular slip surfaces through Cross Section A-A’.

To evaluate the stability of slopes under seismic conditions, we used a “pseudostatic” method of analysis. The pseudostatic method models the effects of transient earthquake loading on a potential slide mass by using an equivalent sustained horizontal force that is the product of a seismic coefficient and the weight of the potential slide mass. We conservatively selected a seismic coefficient of 0.25g. The selected seismic coefficient considers a displacement threshold of up to 6 inches. We targeted static and pseudo-static factors of safety of 1.5 and 1.0, respectively.

The soil parameters used in the analyses are summarized below and are based on review of the referenced geotechnical reports for the project, our experience with similar soil conditions in the project area, and published correlations.

Table 1. Slope Stability Analysis Material Properties

MATERIAL	γ'	DRAINED STRENGTHS		UNDRAINED STRENGTHS	
		C’	ϕ’	C	ϕ
Clay/Silt	125	200	30	1000	0
Silty/Clayey Sand	130	0	35	0	35
Clayey Sand	140	0	38	0	38
Sandy Gravel to Gravelly Sand	140	0	40	0	40

Note: γ’ = Moist Unit Weight (pcf)
C’ = Effective Cohesion (psf)
ϕ’ = Effective Angle of Internal Friction (Degrees)
C = Total Stress Cohesion (psf)
ϕ = Total Stress Angle of Internal Friction (Degrees)

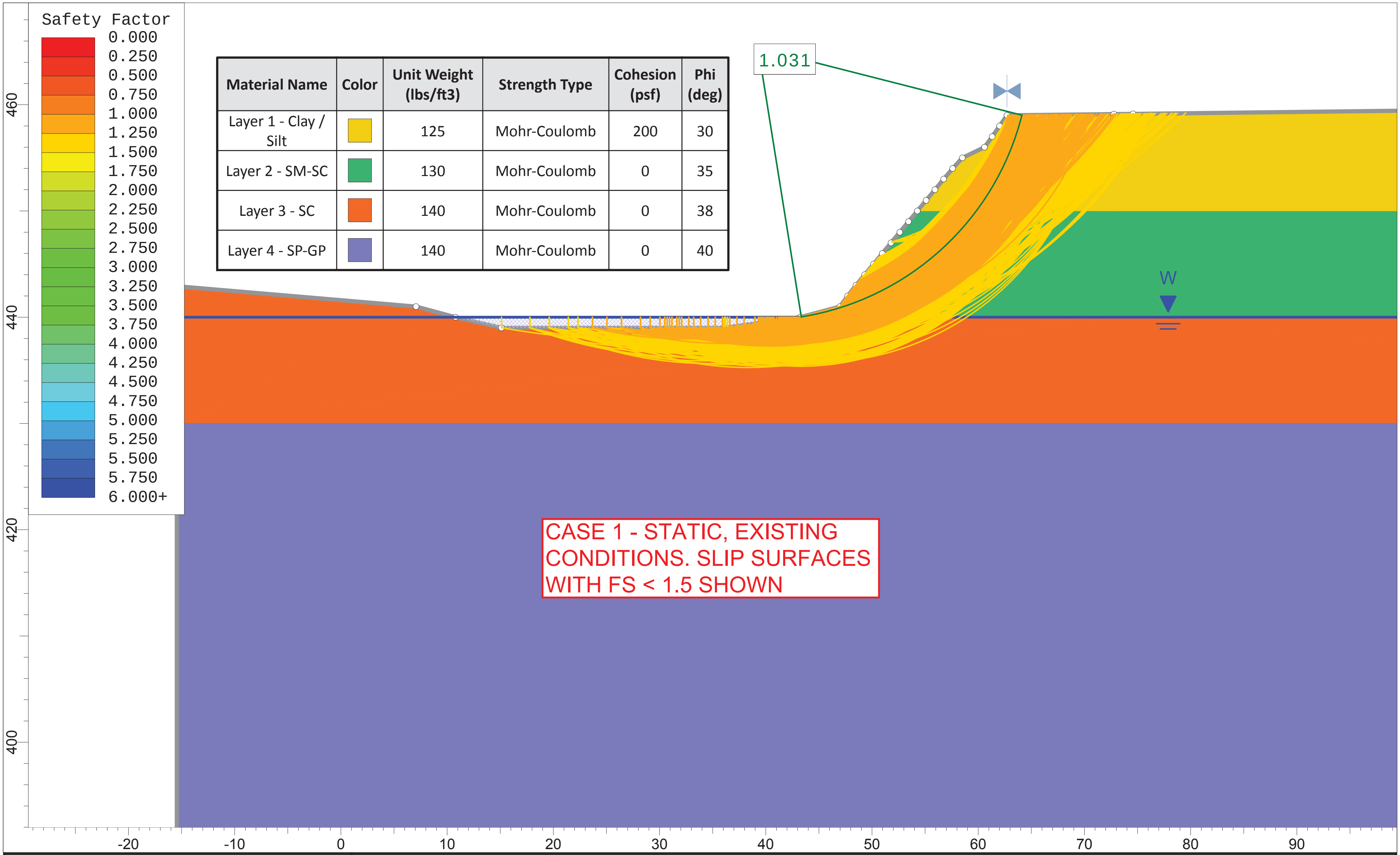
The buried piers were modeled as passive pile supports with shear capacity equal to the computed pier shear strength divided by the out-of-plane spacing of 6 feet.

We performed limit equilibrium slope stability analyses of four different conditions:

1. Existing conditions under static loading
2. Post-construction conditions (buried pier wall installed) under static loading
3. Post-construction conditions under pseudostatic loading
4. Post-construction conditions under rapid-drawdown loading

Results indicate the existing ½: to ¾:1 (horizontal:vertical) bank is marginally stable (i.e., factor of safety is approximately equal to 1.0) under static conditions. Based on results of the slope stability analysis and the referenced creek scour evaluation, there is potential for portions of the bank with this steep inclination to regress or flatten over the lifetime of the project; although based on our review of aerial photographs, observed bank retreat in this area has been minor dating back to at least 1940.

Static slope stability analyses with the buried pier wall installed (Case 2) indicate that slip surfaces that intersected the pier wall in the static existing conditions analysis (Case 1) now have a factor of safety greater than 1.5 for static conditions. Acceptable static and pseudo-static factors of safety of at least 1.5 and 1.0, respectively, are achieved for potential slip surfaces that extend beyond the buried piers and into the site development area.



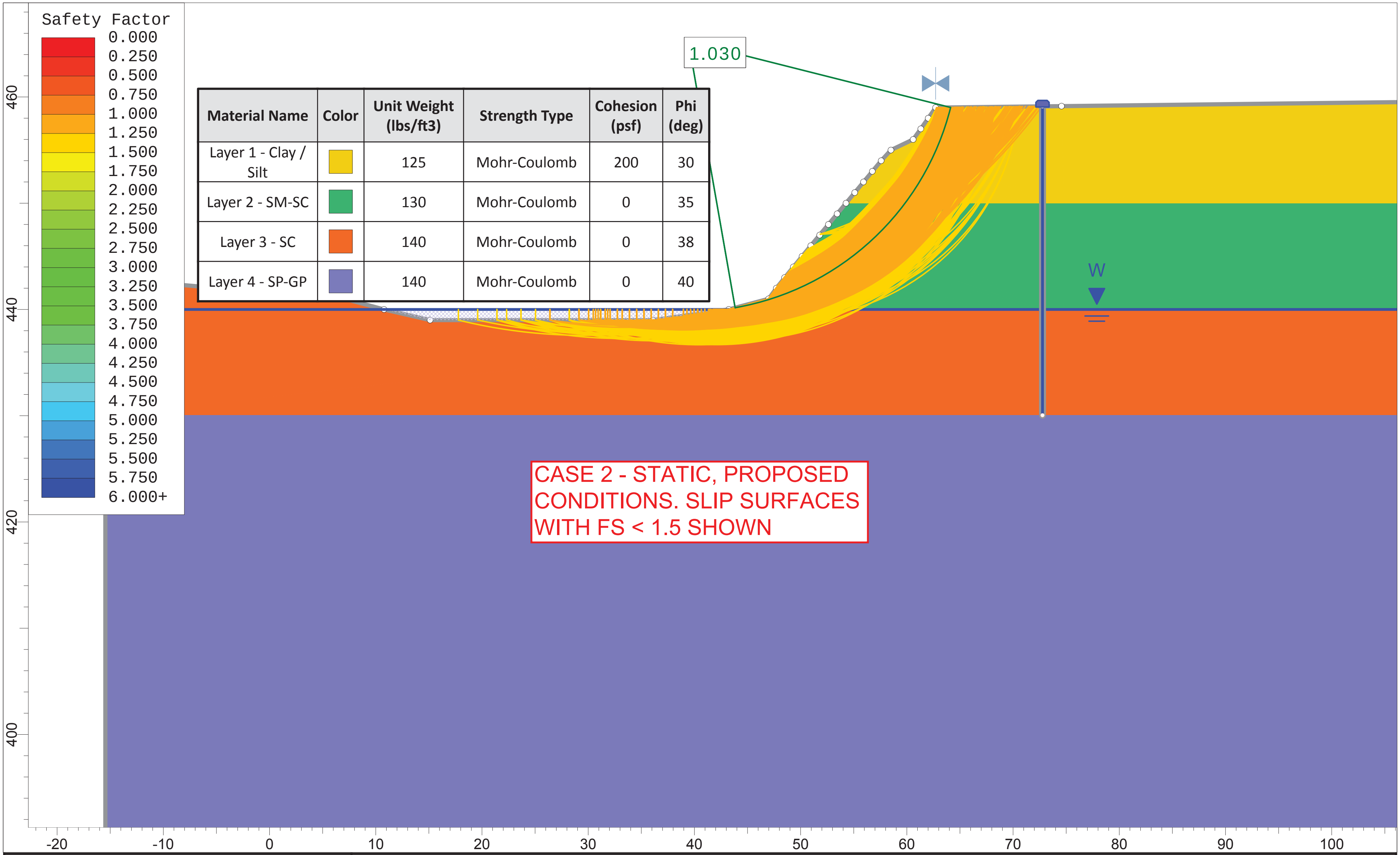
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Layer 1 - Clay / Silt	🟨	125	Mohr-Coulomb	200	30
Layer 2 - SM-SC	🟩	130	Mohr-Coulomb	0	35
Layer 3 - SC	🟧	140	Mohr-Coulomb	0	38
Layer 4 - SP-GP	🟪	140	Mohr-Coulomb	0	40

CASE 1 - STATIC, EXISTING CONDITIONS. SLIP SURFACES WITH FS < 1.5 SHOWN

rocscience

SLIDEINTERPRET 8.021

Project	SLIDE - An Interactive Slope Stability Program
Analysis Description	
Drawn By	
Scale	1:150
Company	
Date	4/25/2019, 1:42:47 PM
File Name	Slide.slmd



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Layer 1 - Clay / Silt	🟨	125	Mohr-Coulomb	200	30
Layer 2 - SM-SC	🟩	130	Mohr-Coulomb	0	35
Layer 3 - SC	🟧	140	Mohr-Coulomb	0	38
Layer 4 - SP-GP	🟪	140	Mohr-Coulomb	0	40

CASE 2 - STATIC, PROPOSED CONDITIONS. SLIP SURFACES WITH FS < 1.5 SHOWN

rocscience

SLIDEINTERPRET 8.021

Project

SLIDE - An Interactive Slope Stability Program

Analysis Description

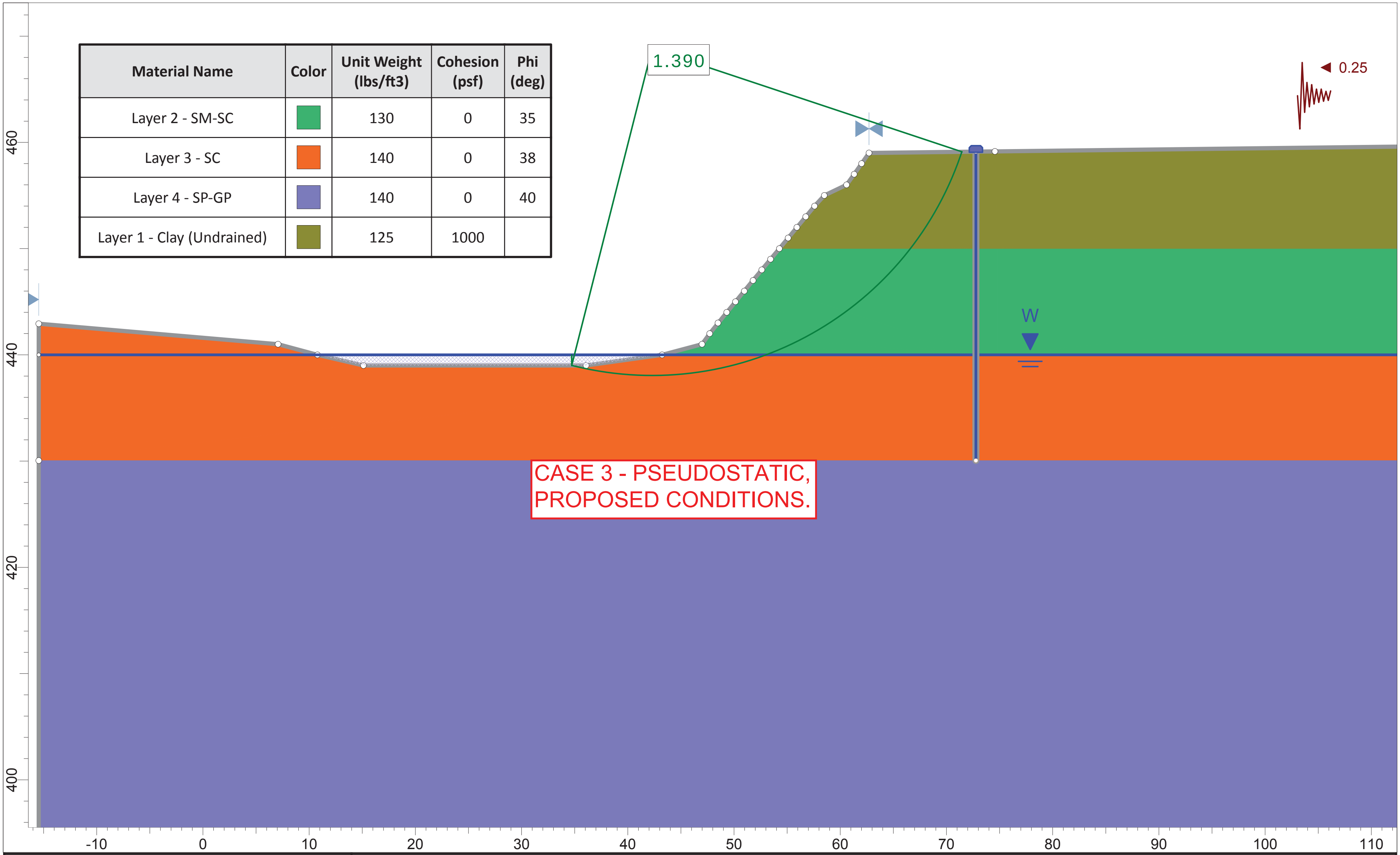
Drawn By

Scale 1:150

Company

Date 4/25/2019, 1:42:47 PM

File Name Slide.slmd

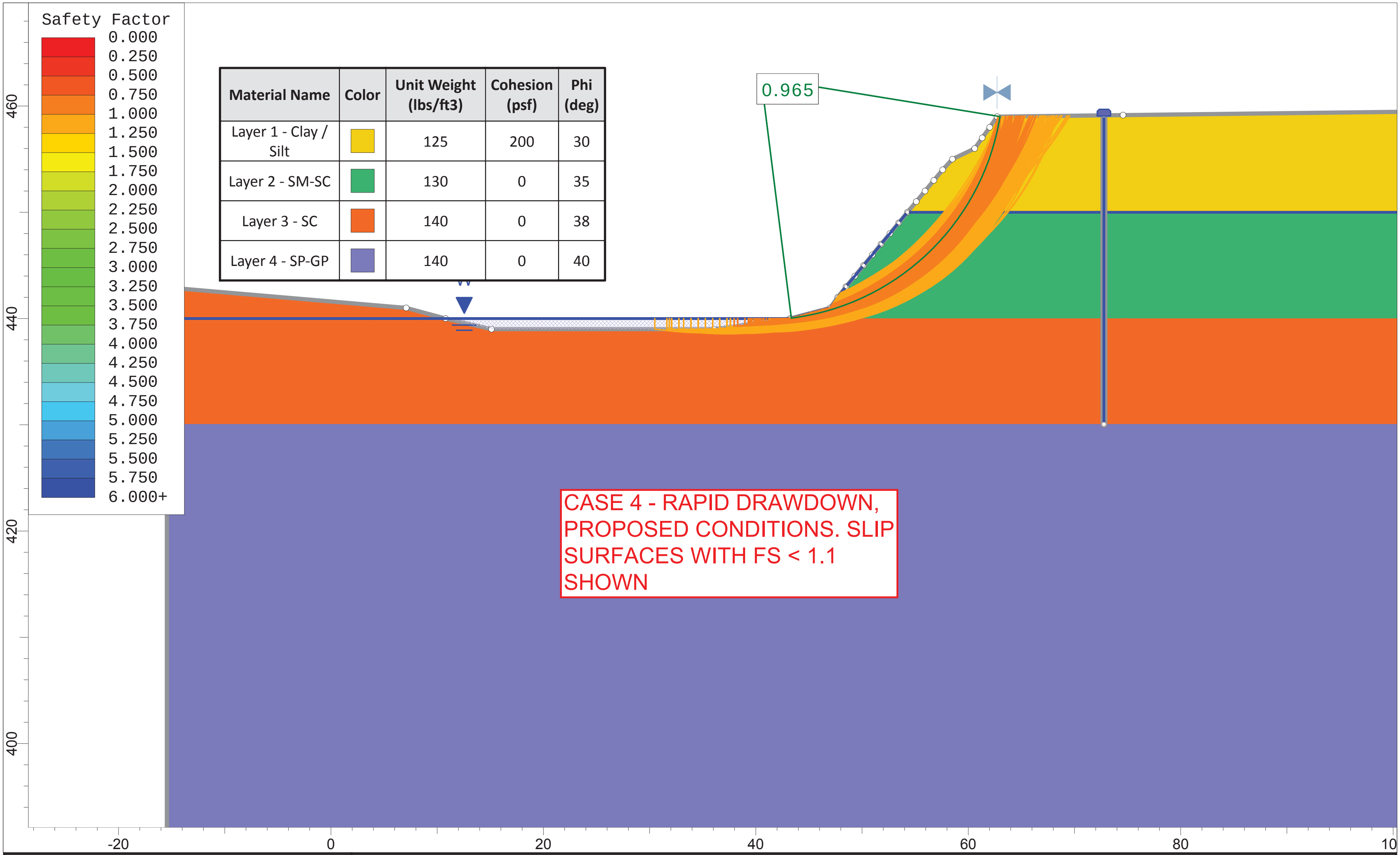


Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Layer 2 - SM-SC	🟩	130	0	35
Layer 3 - SC	🟧	140	0	38
Layer 4 - SP-GP	🟪	140	0	40
Layer 1 - Clay (Undrained)	🟫	125	1000	

CASE 3 - PSEUDOSTATIC,
PROPOSED CONDITIONS.

Project	SLIDE - An Interactive Slope Stability Program
Analysis Description	
Drawn By	
Scale	1:150
Company	
Date	4/25/2019, 1:42:47 PM
File Name	Slide.slmd

SLIDEINTERPRET 8.021



Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Layer 1 - Clay / Silt	🟨	125	200	30
Layer 2 - SM-SC	🟩	130	0	35
Layer 3 - SC	🟧	140	0	38
Layer 4 - SP-GP	🟪	140	0	40

CASE 4 - RAPID DRAWDOWN, PROPOSED CONDITIONS. SLIP SURFACES WITH FS < 1.1 SHOWN

rocscience

SLIDEINTERPRET 8.021

Project
SLIDE - An Interactive Slope Stability Program

Analysis Description

Drawn By

Scale 1:150

Company

Date 4/25/2019, 1:42:47 PM

File Name Slide.slmd

ENGEO "CREEK SCOUR AND GEOMORPHOLOGY EVALUATION AND RECOMMENDATIONS" (MAY 3, 2019, REVISED MARCH 5, 2020)

ENGEO
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GEOTECHNICAL
ENVIRONMENTAL
WATER RESOURCES
CONSTRUCTION SERVICES

Project No.
14986.000.000

May 3, 2019
Latest Revision March 5, 2020

Mr. Indrajit Obeysekere
Meridian Property Ventures LLC
2420 Camino Ramon, Suite 215
San Ramon, CA 94583

Subject: Proposed Chick-Fil-A Restaurant
North Livermore Avenue
Livermore, California

CREEK SCOUR AND GEOMORPHOLOGY EVALUATION AND RECOMMENDATIONS

Dear Mr. Obeysekere:

At your request, we are providing supplemental recommendations for the proposed Chick-Fil-A Restaurant project (Site) in regards to Arroyo Las Positas Creek that passes through the northern portion of the Site. A portion of the creek immediately downstream of the bridge at North Livermore Avenue was evaluated for lateral migration potential. We have proposed a buried pier wall design to address continued creek bank erosion of Arroyo Las Positas.

As part of this evaluation, we met with Mr. Jeff Tang of the Livermore-Amador Valley Zone 7 Water Agency (Zone 7 Agency) on March 8, 2019, to discuss the buried pier wall design concept, as well as the scope of this study. Zone 7 Agency prepared a review of our previous conceptual buried pier wall design (Reference 3), indicating that the concept appeared feasible in their opinion (Reference 13, also attached). The approach utilized in this study is based on that meeting as well as our experience on similar projects.

To evaluate potential future creek erosion at the location of the proposed buried wall, we are providing additional recommendations based on the following.

1. Review of previous studies performed on the subject property regarding erosion potential.

2. Hydraulic modeling of the creek reach, supplemented with the hydrologic and hydraulic model prepared by Livermore-Amador Valley Zone 7 Water Agency (Zone 7 Agency, 2018) to assess stream velocity and shear stresses in the subject reach of Arroyo Las Positas.

3. A Bank Erosion Index study to estimate scour potential based on streambank characteristics.

4. A geomorphic study to assess the potential for long-term systematic scour of the greater Arroyo Las Positas creek system which would potentially lower the creek bed elevations in the vicinity of the proposed project.

The intent of these recommendations is to establish an ultimate creek scour hazard for buried wall improvements being proposed in this area. These recommendations should be incorporated into the final design of the buried pier wall so that any additional potential soil loss due to future erosion of the creek bank is considered in the ultimate design of the wall system.

PREVIOUS STUDY

We previously prepared a conceptual buried pier wall design dated May 29, 2018 (Reference 3). As part of this study, we evaluated a buried pier wall solution to address continued bank erosion of Arroyo Las Positas adjacent to the subject Chick-Fil-A restaurant project. A conceptual buried pier wall plan is attached as Figure 1.

We performed a detailed review of historical aerial photographs and topographic maps as part of our Geotechnical Exploration Update for the project (Reference 4). The creek banks in the immediate project vicinity appear to exhibit minimal regression or bank retreat dating back to at least 1940 (earliest aerial photograph available and reviewed). A final buried pier wall design, including structural calculations and construction plans that will incorporate recommendations contained in this report is in progress and will be submitted separately.

SOIL OBSERVATIONS AND TESTING

As referenced in the Geotechnical Engineering Exploration and Analysis (Giles, 2017), according to the Livermore Quadrangle, California (2006), the Site is underlain by Holocene stream terrace deposits consisting generally of sand, silt, clay, and gravel. Soils encountered within test borings generally consisted of stiff sandy clay and silty clay, and medium dense to very dense silty sand, clayey sand and sand with gravel and possible cobbles at deeper depths. The upper 10 feet of the soils were generally fine-grained soils (clay) and below 10 feet, the soils were generally granular (sand and gravel with possible cobbles).

CREEK OBSERVATIONS

We performed a site reconnaissance on March 4, 2019. We observed historic erosion of the left creek bank between Survey Stations 1077 and 1104, as shown on Figure 2. The Arroyo Las Positas Creek has a bottom width of approximately 10 feet with banks up to approximately 15 to 20 feet in height within the area of observed bank erosion. The upper portions of the eroded creek bank are nearly vertical and have sparse vegetation suggesting that the creek bank is experiencing some erosional scour. Within this reach, the creek bank slope has a gradient of approximately 1¼:1 (horizontal:vertical) and is lightly vegetated with brush and grasses, then transitions into a nearly vertical exposed face. Soils exposed on the lower half of the creek bank appear to consist of silty sand with minor cobbles and light vegetation. Soils exposed on the upper half of the creek bank appear to consist of predominantly fine-grained silt and clay soils.

HYDRAULIC ANALYSIS

A hydraulic analysis was performed using the Hydraulic Engineering Center River Analysis System (HEC-RAS) Version 5.0.5 computer program published by the United States Army Corps of Engineers (USACE). HEC-RAS performs one-dimensional hydraulic analyses for natural channels to calculate water surface profiles and velocities in steady, gradually varied flow conditions. The basic HEC-RAS computational procedure is based on the solution of the one-dimensional energy equation. Energy losses consist of friction losses (based on Manning's equation). Surveyed information, dated April 8, 2016, and provided by Joseph C. Truxaw and Associates, Inc., was used for the cross-sectional geometries of the creek reach. The cross-section data were georeferenced from AutoCAD Civil 3D into the HEC-RAS geometry editor. Surveyed cross-section stations for the reach between North Livermore Avenue and Portola Avenue are shown on Figure 2. Zone 7 Water Agency's HEC-RAS model extends between approximately Las Colinas Road and Portola Avenue.

A 100-year event was used to model a steady-state peak hydrologic flow rate of 6,570 cubic feet per second (cfs) through the reach. This 100-year flow rate was furnished by Zone 7 Water Agency for this reach based on its hydrologic model at North Livermore Avenue.

The value of the Manning's roughness coefficient (*n*) establishes frictional resistance in the channel and is thus related to the modeling of channel velocity and water surface profile by the HEC-RAS program. In accordance with Table 3.1 of the USACE HEC-RAS Hydraulic Manual (USACE, 2016), an 'n' value was selected that corresponds to the hydraulic roughness created by vegetation and other factors encountered throughout the study reach. This value is based on recommended minimum and maximum values developed for a variety of vegetative and morphological conditions similar to those found in the channel. The following table summarizes the use of the coefficient in the modeling based on visual observations of the current channel and overbank conditions.

TABLE 1: Manning's 'N' Value

MANNING'S 'N' VALUE	DESCRIPTION
0.035 (active channel)	Clean, straight channel, some stones and weeds, no rifts or deep pools
0.035 (overbank)	Scattered brush; heavy weeds

Photographs of the creek are shown on Figure 3, which depict the types of established vegetation in the channels and banks.

The hydraulic model is based on 'normal depth' boundary conditions, whereby HEC-RAS calculates an initial water surface profile based on the bed slope of the creek. Estimated bed slopes for the creek of 0.008 ft/ft at the upstream and downstream end were used as boundary conditions for computational purposes. The model was run under a "mixed flow regime" in order to capture both subcritical and supercritical flow conditions. Dimensionless channel expansion and contraction energy losses were computed using an expansion coefficient of 0.3 and a contraction coefficient of 0.1.

The following table summarizes the range of velocities and shear stresses (of left bank) calculated from the HEC-RAS program for the reach of creek adjacent to and just upstream of the proposed Chick-Fil-A. Stations 1104 and 1077 (bolded) approximately correspond to the identified area of bank erosion. HEC-RAS output data are attached.

TABLE 2: ENGEO Range of Calculated Velocities and Shear Stresses (100-Year)

HEC-RAS STATION	100-Year WSE (ft)	CHANNEL VELOCITY (ft/sec)	ESTIMATED SHEAR STRESS (psf) LEFT OF BANK
1153	449.65	8.43	0.48
1126	449.57	7.65	0.34
1104	**449.60**	**6.88**	**0.28**
1077	**449.52**	**7.25**	**0.40**
1044	449.49	7.40	0.34
1019	449.48	6.83	0.16
979	449.38	8.01	0.31

To further estimate velocities and shear stresses to which the eroding bank may be exposed, Zone 7’s model was compared for stations within the vicinity of eroding bank. Station 28695 (bolded) corresponds most closely to the area of bank erosion. Stations shown below have been approximately plotted on Figure 2 for comparison.

TABLE 3: Zone 7 Range of Calculated Velocities (100-Year)

HEC-RAS STATION	100-YEAR WSE (ft)	CHANNEL VELOCITY (ft/sec)	ESTIMATED SHEAR STRESS (psf) LEFT OF BANK
28695	**451.04**	**6.6**	**0.06**
28545	450.20	8.93	0.08
28446	449.59	11.41	0.27
28393	449.27	11.63	0.17

In general, similar velocities and 100-year water surface elevations for stations in the general area of bank erosion were calculated for both models.

It is our opinion that the actual velocities at the creek bottom and banks are substantially lower than what is furnished in the HEC-RAS studies, since HEC-RAS calculates average velocities across a channel, and the velocities are actually not uniformly distributed in the creek section. Studies performed by Barfield (Barfield, 1981) indicate that, due to friction along the walls and bottom of an open channel section, the actual velocity at the boundary of a creek channel is approximately one-half the calculated “average” velocity. Accordingly, actual velocities likely approach about 4 feet per second for the existing 100-year recurrence interval event at the channel bottom where the water is in contact with the bed material.

Based on research published by the United States Army Corps of Engineers (USACE, 1994), which provides erosion threshold guidance for flood control channels, the allowable mean velocity for an unlined/unvegetated channel comprising coarse sand to fine gravel is 5.0 feet per second (fps) and 2.0 fps for sandy silt earth materials (within the bank slope). Soil borings located near the creek characterized the subsurface conditions at the flowline of creek as silty sand and gravel.

Based on soil stratigraphy and erosion threshold guidance, it is reasonable to expect moderate bank erosion potential within the sandy silt materials even with consideration to the assumption that velocities are non-uniform, if channel banks remain relatively unvegetated.

Permissible shear stress values have been developed by the International Erosion Control Association (IECA, 1996) to provide guidance for erosion potential in natural channels. According to the IECA, the maximum permissible shear stress in a vegetated channel with this type of vegetation is 1.00 lb/sq ft. Calculated left bank shear stresses within the area of significant erosion are below this threshold. However, the steepness of the bank does not appear to allow deep-rooted vegetation to establish in this area, which would help stabilize the bank to the shear stresses estimated in the hydraulic model. Thus, some potential for bank erosion appears to exist in the area based on shear stress calculations.

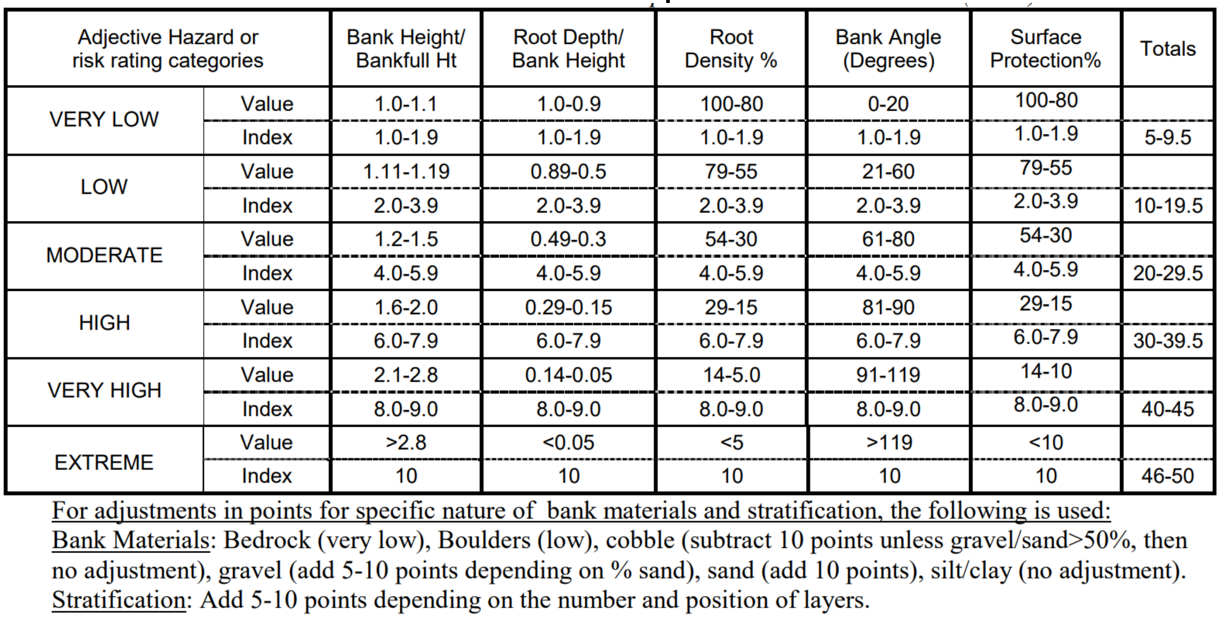
Moreover, based upon the stratigraphy of soils, the steep inclination of portions of the bank, and the calculated depth of flow during a 100-year recurrence interval storm event, it is reasonable to expect the potential for bank sloughing under a rapid drawdown condition. This is due to a loss of soil shear strength that may occur during saturated bank conditions following drawdown of stream water-surface elevations. During rapid drawdown, the stabilizing effect of the water on the upstream face is lost, but the remaining pore-water pressures within the embankment may remain high following a large rain event and/or high river stage event. Based on our experience with local creeks in the Livermore area and the hydraulic data presented above, rapid-drawdown-related sloughing issues could flatten the bank slope to an inclination of approximately 2:1, over the long term.

We therefore estimate some potential for erosion on the subject stream bank based on velocity and shear stress calculations, and the potential for rapid drawdown sloughing to occur based on calculated water surface elevations and bank geometry.

BANK EROSION HAZARD INDEX ANALYSIS

In order to estimate the limit of the scour hazard potential of the subject stream bank we performed a Bank Erosion Hazard Index (BEHI) using a method developed by Rosgen. An excerpt of this is provided in Table 4.

TABLE 4: Streambank Characteristics used to Develop BEHI



Adjective Hazard or risk rating categories		Bank Height/ Bankfull Ht	Root Depth/ Bank Height	Root Density %	Bank Angle (Degrees)	Surface Protection%	Totals
VERY LOW	Value	1.0-1.1	1.0-0.9	100-80	0-20	100-80	
	Index	1.0-1.9	1.0-1.9	1.0-1.9	1.0-1.9	1.0-1.9	5-9.5
LOW	Value	1.11-1.19	0.89-0.5	79-55	21-60	79-55	
	Index	2.0-3.9	2.0-3.9	2.0-3.9	2.0-3.9	2.0-3.9	10-19.5
MODERATE	Value	1.2-1.5	0.49-0.3	54-30	61-80	54-30	
	Index	4.0-5.9	4.0-5.9	4.0-5.9	4.0-5.9	4.0-5.9	20-29.5
HIGH	Value	1.6-2.0	0.29-0.15	29-15	81-90	29-15	
	Index	6.0-7.9	6.0-7.9	6.0-7.9	6.0-7.9	6.0-7.9	30-39.5
VERY HIGH	Value	2.1-2.8	0.14-0.05	14-5.0	91-119	14-10	
	Index	8.0-9.0	8.0-9.0	8.0-9.0	8.0-9.0	8.0-9.0	40-45
EXTREME	Value	>2.8	<0.05	<5	>119	<10	
	Index	10	10	10	10	10	46-50

For adjustments in points for specific nature of bank materials and stratification, the following is used:
Bank Materials: Bedrock (very low), Boulders (low), cobble (subtract 10 points unless gravel/sand>50%, then no adjustment), gravel (add 5-10 points depending on % sand), sand (add 10 points), silt/clay (no adjustment).
Stratification: Add 5-10 points depending on the number and position of layers.

Source: Rosgen, 1996

Where:

1. Ratio of bank height to bankfull height: This value is estimated as approximately one for this area given the large floodplain on the opposite bank of the subject reach of creek. The floodplain allows larger flows above bankfull depth to move away from the creek bank and spread to the northern side of the creek.

2. Ratio of root depth to bank height: Root depth to height (**RDH**) is the ratio of the average plant root depth to the bank height, expressed as a percent (e.g., roots extending 2 feet into a 4-foot-tall bank = 0.50).

3. Root density: Root density (**RD**), expressed as a percent, is the proportion of the streambank surface covered (and protected) by plant roots (e.g. a bank whose slope is half covered with roots = 50 percent).

4. Surface protection: Surface protection (**SP**) is the percentage of the stream bank covered (and therefore protected) by plant roots, downed logs, branches, rocks, etc. In many streams, surface protection and root density are synonymous.

5. Bank angle: Bank angle (**BA**) is the angle of the “lower bank” – the bank from the waterline at base flow to the top of the bank, as opposed to benches that are higher on the floodplain.

The following parameters were identified for the current conditions of the adjacent bank to the proposed development based on Table 4.

TABLE 5: Arroyo Las Positas Streambank Current Characteristics

ADJECTIVE HAZARD OR RISK RATING CATEGORIES	BANK HEIGHT/BANKFULL HEIGHT (AVERAGE)	ROOT DEPTH/BANK HEIGHT	ROOT DENSITY (%)	BANK ANGLE (DEGREE)	SURFACE PROTECTION (%)	BANK MATERIAL ADJUSTMENT
Value	1.03	0.03	<5	60*	25	10 (sand)
Index	1.2	10	10	3.9	6.5	10
						Total: 41.6

*Approximated for section of the most severe bank erosion based on recent field observations (March 2019) and survey data (Truxaw and Associates, 2016)

Based on a BEHI of 41.6, there is currently potential for erosion of the bank adjacent to the proposed development; although, based on our review of aerial photographs, observed bank retreat in this area has been minor dating back to 1940.

Following an erosion or rapid drawdown issue that would potentially relax the inclination of the slope to a 2:1 (horizonal;vertical) inclination, and establishment of woody vegetation on the bank, the streambank characteristics from Table 5 would be modified to the following:

TABLE 6: Arroyo Las Positas Streambank Characteristics following Rapid Drawdown

ADJECTIVE HAZARD OR RISK RATING CATEGORIES	BANK HEIGHT/BANKFULL HEIGHT (AVERAGE)	ROOT DEPTH/BANK HEIGHT	ROOT DENSITY (%)	BANK ANGLE (DEGREE)	SURFACE PROTECTION (%)	BANK MATERIAL ADJUSTMENT
Value	1.01	0.33	100	26	45	0*
Index	1.2	5	1.5	2.5	4.3	0
						Total: 14.50

*Silt/Clay exposed on bank following rapid drawdown sloughing

Based on a BEHI of 14.5, there would be a low-erosion potential for the bank adjacent to the proposed development assuming the bank ultimately scours to a 2:1 slope and moderate vegetation cover, with some woody vegetation, establishes on the bank. We estimate that scour may occur until this condition is achieved and the 2:1 bank would remain relatively stable based on the modified inclination, which will contribute to erosion resistance, including the ability for deep-rooted vegetation to establish.

GEOMORPHIC STUDY

Based on our historic review, it appears that the majority of the site has remained largely unchanged since 1940. Arroyo Las Positas, located adjacent to the northern boundary of the site, appears to be relatively stable during years with available photos. A meander northwest of the site was realigned/straightened sometime between 1950 and 1958. This realignment appears to be within the incised channel and not due to erosion of the southern bank. Whether this adjustment was natural or due to grading activities is unknown; however, it does not appear to be related to the construction of Interstate 580 (then US 50). In the same general area, a small bridge or culvert was constructed across Arroyo Las Positas between 1960 and 1965. In conjunction with this structure, a small access roadway appears to have been cut into the south bank of Arroyo Las Positas, extending onto the Site. This structure is no longer evident in the 1987 aerial photographs, and the roadway appears to have been filled in. Current topography shows a relatively gentle drainage swale that discharges into the creek in this general area.

Based on the historic review described above in addition to morphological features of the stream provided below in Table 6, the arroyo would be characterized as an F5 Stream Type (Rosgen, 1996).

TABLE 7: Arroyo Las Positas Morphological Features

CHANNEL BED	WIDTH/DEPTH RATIO	ENTRENCHMENT RATIO	SINUOSITY	SLOPE
Sand	>12 (Moderate to High)	1.4- 2.2 (Moderate)	>1.2 (Moderate)	<0.02

The higher width/depth ratio of F5 Stream types is associated with the depositional characteristics of the stream bed and active lateral migration tendencies. The F5 stream type is susceptible to shifts in both lateral and vertical stability as a result of changes in flow.

A major indicator for the potential of long-term erosion in a fluvial system is the measurement of the system's bed slope, which would be similar to that of other systems, which are in a state of erosion/deposition equilibrium. The concept of an 'equilibrium' bed slope is based on principles of fluvial geomorphology and suggests that a creek will evolve to adjust its longitudinal slope over the long term, such that the system transports its sediment load without either net deposition or erosion. If the bed slope of the creek is too steep, for example, the creek will tend to erode its channel until an ‘equilibrium’ channel slope is obtained. Often this occurs due to changes in hydrologic flow rates in a system due to watershed urbanization, which is more commonly known as “hydromodification.”

Based on the survey information collected by Joseph C. Truxaw and Associates, Inc. for this reach of creek, it is apparent that the creek bed has formed a minor pool and riffle system in the channel bed since the survey reveals several undulations in the flowline as the creek progresses downstream. Since this is the case, the bed slope of the creek was estimated by the slope of the Energy Grade Line (EGL) computed by HEC-RAS, which provides a reasonable indicator of the overall creek slope. The creek’s EGL is between 0.0025 and 0.008 ft/ft for the HEC-RAS model. Based on our experience with the geomorphology of other creeks in the San Francisco East Bay Region, these values are approximately in the range of an ‘equilibrium’ bed slope for a fluvial system with a similar discharge during a 100-year rain event.

Thus, because the bed slope of the creek is in the range of what one would expect for an ‘equilibrium’ slope condition for a fluvial system of this size, long-term erosion associated with watershed hydromodification in this reach of Arroyo Las Positas is considered to be unlikely.

The potential for downcutting of the creek bed was also evaluated for this reach of Arroyo Las Positas Creek as part of our geomorphic study. Increases in instantaneous flow velocities, such as during a 100-year flow event, can remove the roughness of the streambed and steepen the slope resulting in downcutting or knick points. We observed a knick point corresponding to the stations shown below within our and Zone 7’s HEC-RAS model, approximately 300 feet downstream of the unstable bank.

TABLE 8: Observed Knick Point Location

STATION (MODEL)	POTENTIAL CHANNEL BED ELEVATION CHANGE (feet)
577 (ENGEO)	1.61
28246 (Zone 7)	2.32

The potential channel bed elevation change was based on a combination of bed slope and energy grade line data for the two models. Over time, this knick point may migrate upstream and continue to deepen as the stream incision and floodplain disconnect. For the purpose of this scour analysis, we have assumed that this knick point would migrate upstream.

Significant signs of meandering, braiding, or berming were not observed downstream of the proposed pier wall design that would reasonably be expected to affect the scour within this area. Minor channel widening has been noted at various areas downstream. We are not aware of significant urbanization land-use changes within the surrounding areas upstream, which may increase flow to the creek. We were unable to assess downstream conditions of the creek at Portola Avenue. However, portions of the channel below Portola Avenue appear to be stable based on our review of historic photographs.

CONCLUSIONS

The study indicates potential for a scour hazard to ultimately develop along the creek bank at the location where the buried pier wall will be constructed. The potential scour hazard can be summarized as follows:

TABLE 9: Ultimate Scour Hazard

SCOUR HAZARD COMPONENT	EXISTING CONDITION	SCOUR HAZARD POTENTIAL
Slope Inclination (horizontal:vertical)	½:1	2:1
Creek Bed (relative elevation ft.)	0	-2 ft.

With consideration to the stratigraphy of slope soils, determined bank erosion potential, and hydraulic and geomorphic analyses, it is reasonable to expect rapid drawdown erosion issues with a future ultimate creek scour hazard of 2:1 (horizontal:vertical). This ultimate condition at a 2:1 slope would likely have a stabilizing effect on the bank based on the estimated modified BEHI, and more robust vegetation could establish on a slope with this inclination.

Based on our geomorphic study, we have additionally assumed 2 feet of potential downcutting as a result of the stream type and knick point observed 500 feet downstream of the eroding bank. As shown in Figure 1, following slope regression and potential downcutting, an estimated 6 feet of pier wall may be potentially exposed under the projected slope conditions. This approach is considered conservative with the assumption that the ultimate creek scour hazard would occur concurrently with the point of ultimate scour and downcutting migration followed by a large flow event.

Based on the results of this study, and our review of historic erosion within the creek, the proposed buried pier wall design would allow for more than a 50-year design life in our opinion. In addition, since the buried piers can be constructed entirely from behind top of bank, there will be no disturbance to the Arroyo and no permitting requirements with resource agencies having jurisdiction within the creek.

Finally, the construction of the proposed buried pier wall will have no direct downstream or upstream impacts to the Arroyo since, at the ultimate scour hazard condition, the 100-year water surface does not intersect any portion of the buried piers, as shown on Figure 1.

If you have any questions, please do not hesitate to contact us.

Sincerely,

ENGEO Incorporated

Brooke Spruit

Brooke Spruit
bs/jb/ue/cjn

Jonathan Buck, PE

REGISTERED PROFESSIONAL ENGINEER
JONATHAN BUCK
No. 67302
CIVIL
STATE OF CALIFORNIA

Attachments: List of Selected References
Figures
Zone 7 Agency – Review Letter
HEC-RAS Output

SELECTED REFERENCES

1. Barfield, N.B., R.C. Warner, and C.T. Haan. 1981. Applied Hydrology and Sedimentology for Disturbed Areas. Oklahoma Technical Press (Stillwater, OK).

2. Dave Rosgen; Applied River Morphology; Published 1996; Second Edition.

3. ENGEO; Conceptual Buried Pier Wall Design; Proposed Restaurant; North Livermore Road, Livermore, California; May 14, 2018, Revised May 29, 2018.

4. ENGEO; Geotechnical Exploration Update; Proposed Chick-Fil-A Restaurant; North Livermore Avenue; Livermore, California; March 1, 2019.

5. Giles Engineering Associates, Inc.; DRAFT Geotechnical Engineering Exploration and Analysis; Proposed Chick-Fil-A Restaurant #3805, North Livermore Avenue and I-580; Livermore, California; September 13, 2016.

6. Giles Engineering Associates, Inc.; Slope Stability Evaluation; Proposed Chick-Fil-A Restaurant #3805, North Livermore Avenue and I-580; Livermore, California; May 3, 2018.

7. Joseph C. Truxaw & Associates, Inc.; ALTA/NSPS Land Title Survey; Livermore Avenue at I-580; April 8, 2016.

8. Joseph C. Truxaw & Associates, Inc.; Slope Topography Exhibit, Livermore Avenue at I-580; Livermore, California; April 12, 2018.

9. United States Army Corps of Engineers (USACE); HEC-RAS River Analysis System; Hydraulic Reference Manual Version 5.0; February 2016.

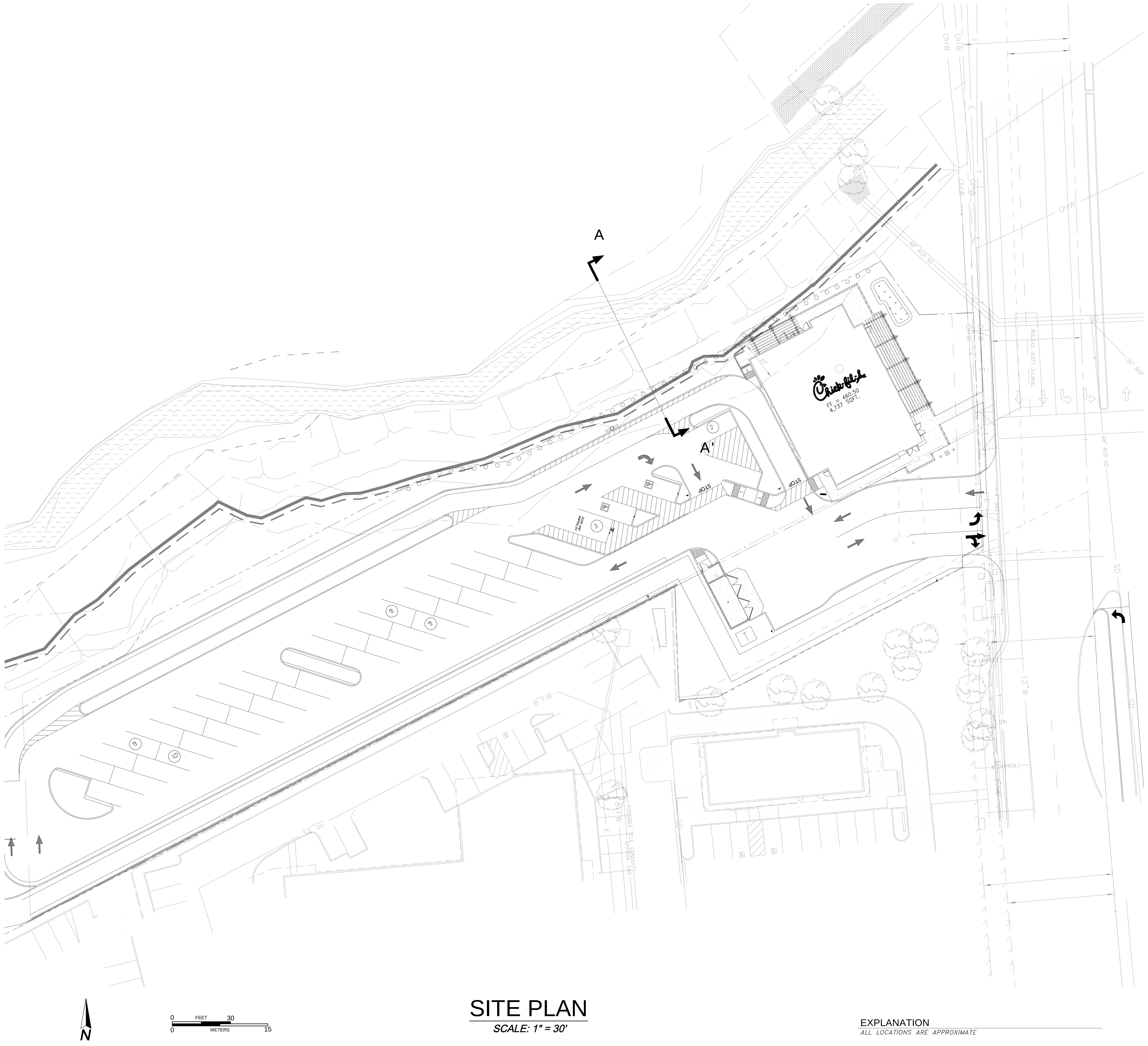
10. USACE; Hydraulic Stability of Natural Channels EM 1110-2-1418, 1994.

11. IECA; Design Procedures for Channel Protection and Stream Bank Stabilization; 1996.

12. Zone 7 Agency; HEC-RAS Line H, Positas River, Seco-Cayeano Reach; Arroyo Las Positas Creek; 2018.

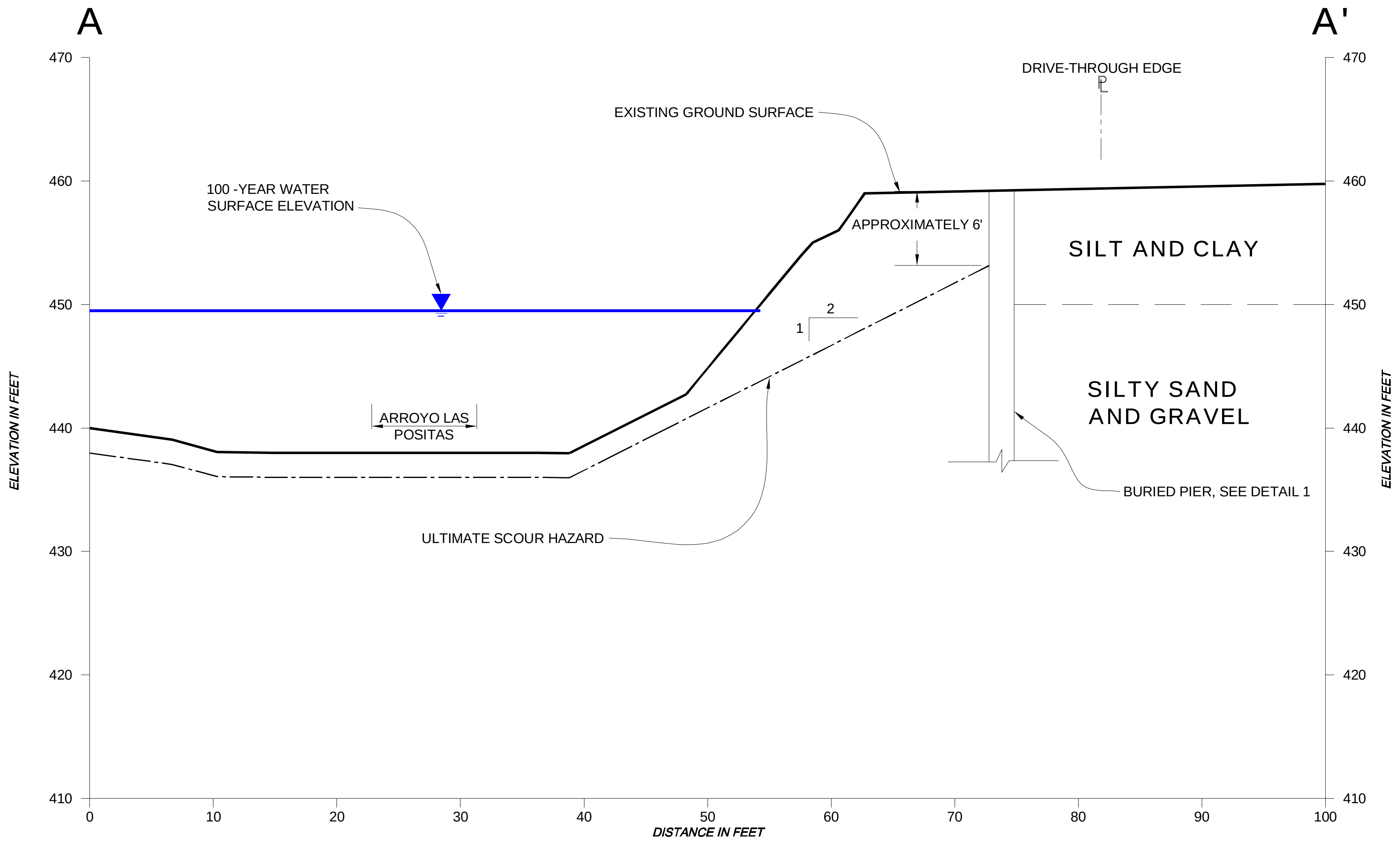
13. Zone 7 Agency; Review of ENGEO; Conceptual Buried Pier Wall Design Proposed for Chick-Fil-A Development at North Livermore Avenue, Livermore, California; March 18, 2019.

FIGURES



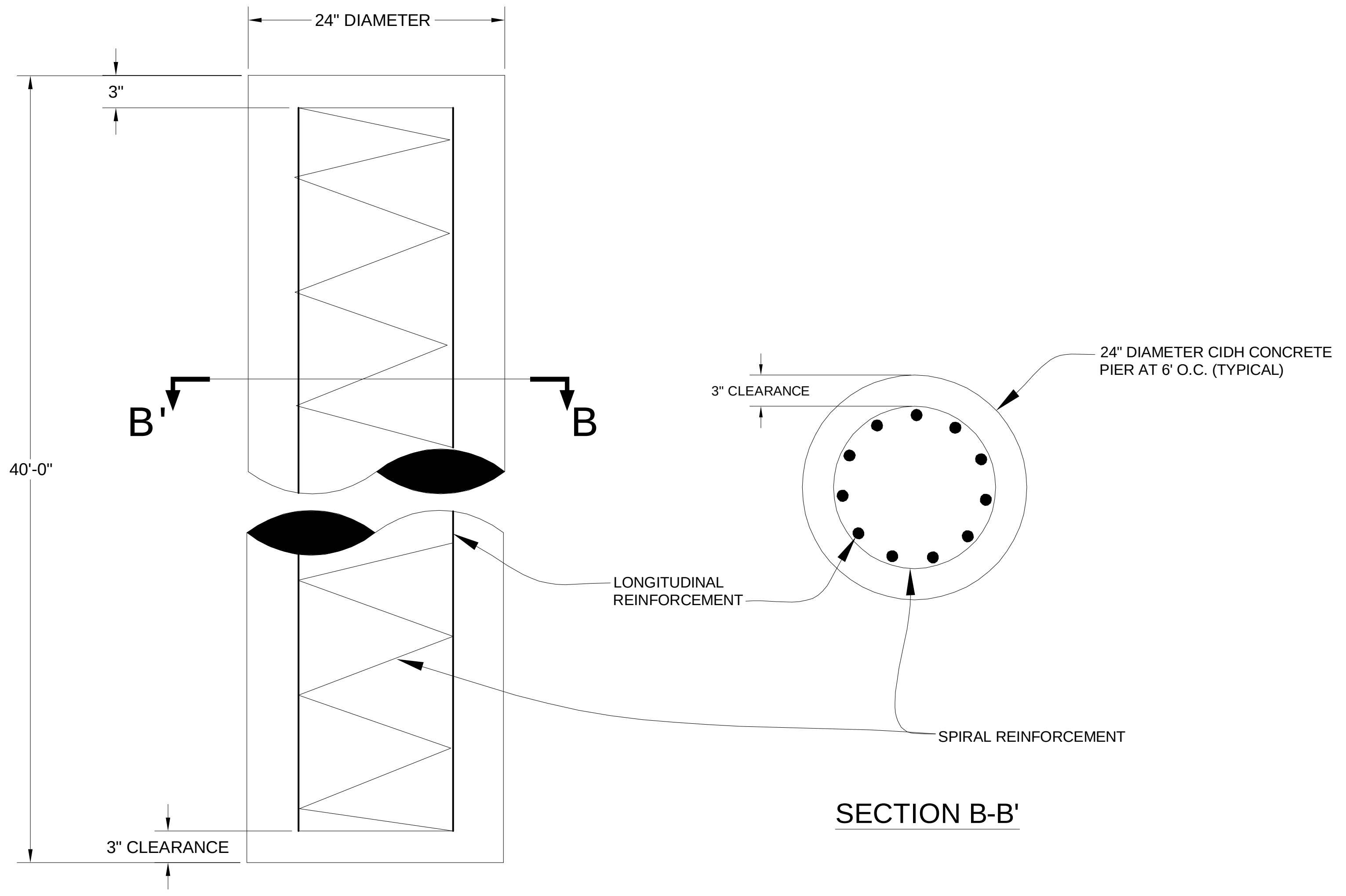
SITE PLAN

SCALE: 1" = 30'



SECTION A-A'

SCALE 1" = 10'



BURIED CAST-IN-DRILLED-HOLE (CIDH) PIER TYPICAL DETAIL

① *NO SCALE*

EXPLANATION

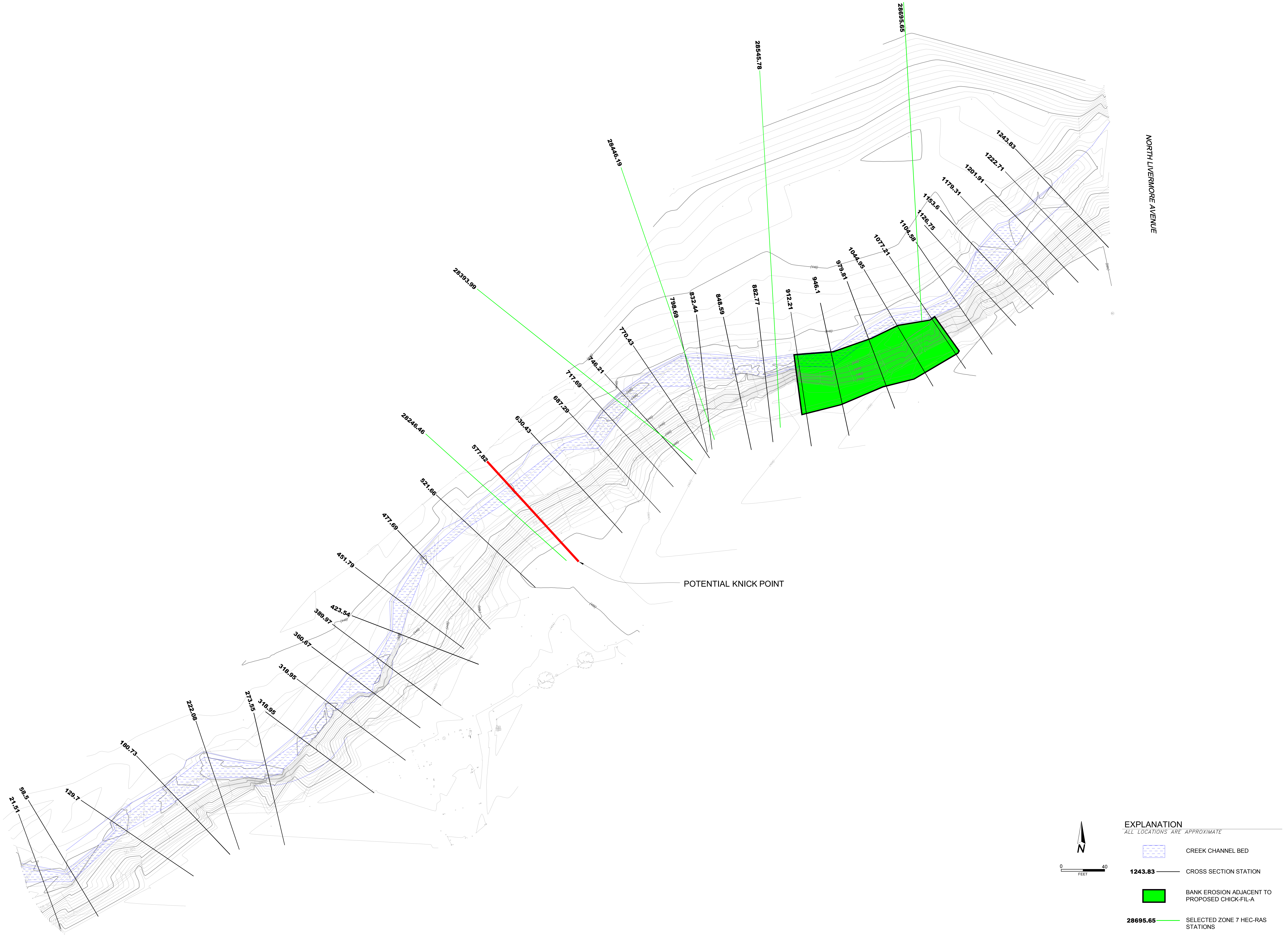
ALL LOCATIONS ARE APPROXIMATE

○ ○ ○ ○ ○ ○ ○	PROPOSED BURIED PIER WALL
A ↑___↑ A'	CROSS SECTION LOCATION

BASE MAP SOURCE: UNKNOWN

ENGEO Expect Excellence	CONCEPTUAL BUIRED PIER WALL DESIGN CHICK-FIL-A NORTH LIVERMORE LIVERMORE, CALIFORNIA	PROJECT NO.: 14986.000.000 SCALE: AS SHOWN DRAWN BY: JV CHECKED BY: JB	FIGURE NO. 1

ORIGINAL FIGURE PRINTED IN COLOR



EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

▨	CREEK CHANNEL BED
1243.83 ———	CROSS SECTION STATION
🟩	BANK EROSION ADJACENT TO PROPOSED CHICK-FIL-A
28695.65 ———	SELECTED ZONE 7 HEC-RAS STATIONS

BASE MAP SOURCE: UNKNOWN

ENGEO
Expect Excellence

HEC-RAS STATIONING ALONG ARROYO
LAS POSITAS REACH
CHICK-FIL-A NORTH LIVERMORE
LIVERMORE, CALIFORNIA

PROJECT NO.: 14986.000.000
SCALE: AS SHOWN
DRAWN BY: LL
CHECKED BY: JB
FIGURE NO. 2

ORIGINAL FIGURE PRINTED IN COLOR

PHOTO 1

VIEW OF ARROYO LAS POSITAS FACING DOWNSTREAM

PHOTO 2

VIEW OF ARROYO LAS POSITAS FACING UPSTREAM

ENGEO
Expect Excellence

SITE PHOTOGRAPHS
CHICK-FIL-A NORTH LIVERMORE
LIVERMORE, CALIFORNIA

PROJECT NO.: 14986.000.000
SCALE: AS SHOWN
DRAWN BY: JV
CHECKED BY: JB

FIGURE NO.
3

G:\Drafting\DRAFTING2_Dwg_13000 Plus\14986\000\041819-CreekStudy\14986000000-CreekStudy-3-SITEPHOTOS-0419.dwg

ORIGINAL FIGURE PRINTED IN COLOR

ZONE 7 AGENCY
Review Letter

14986.000.000
May 3, 2019
Latest Revision March 5, 2020

ALAMEDA COUNTY FLOOD CONTROL AND WATER CONSERVATION DISTRICT, ZONE 7
100 NORTH CANYONS PARKWAY • LIVERMORE, CA 94551 • PHONE (925) 454-5000 • FAX (925) 454-5727

March 18, 2019

Mr. Indrajit Obeysekere – Vice President
Meridian
2420 Camino Ramon, Suite 215
San Ramon, CA 94583

Subject: Review of ENGEO; Conceptual Buried Pier Wall Design Proposed for Chick-fil-A Development at North Livermore Ave., Livermore

Dear Indrajit,

Thank you for meeting with Zone 7 on Mar 8, 2018, to discuss your proposed plans for the Chick-fil-A development and the conceptual pier wall design proposed for bank stabilization. As we had discussed, the City of Livermore had requested that Zone 7 review the proposed bank stabilization enhancement utilizing a buried cast-in-drilled-hole pier design to protect the development's embankment from potential future erosion. The City had also indicated that as part of the development, a 22 acre parcel adjacent to the development was to be annexed into the City and that it was their desire, at some point in the future, to give the portion of Arroyo las Positas to Zone 7.

While Zone 7 has never employed this type of pier design for bank stabilization, it appears to be a feasible solution that may address potential creek scour and erosion with minimal environmental impacts, provided the following are addressed prior to final design:

1. ENGEO has prepared the necessary hydraulic and scour analyses to determine the potential impacts and the extent of scouring that may occur. This should determine how deep the piers should be installed in the ground, as well as the length required for stabilization; Zone 7 is happy to provide a copy of our hydraulic model to utilize as a basis to prepare an existing condition model;
2. ENGEO has prepared a potential mitigation plan on how, if the area around the buried piers are scoured and eroded, will they be repaired and have they identified sufficient areas required to conduct such repairs in limited working space, taking into account potential environmental impacts at that time.

With regards to the City's desire to annex the 22-acre parcel and give up the portion of Arroyo las Positas to Zone 7 at some time in the future, at this time, Zone 7 will not be accepting any properties at this time.

Please acknowledge this understanding and let me know if you have further questions.

Sincerely,

Jeff Tang

Jeff Tang, P.E.

Cc: Mike Conn, Meridian, 2420 Camino Ramon, Suite 215, San Ramon, CA 94583
Jonathan Buck, ENGEO, 2010 Crow Canyon Pl., Suite 250, San Ramon, CA 94583
Andy Firmin, ENGEO
Uri Eliahu, ENGEO
Carol Mahoney, Zone 7
Joe Seto, Zone 7

HEC-RAS Output

14986.000.000
May 3, 2019
Latest Revision March 5, 2020

```
                        HEC-RAS HEC-RAS 5.0.6 November 2018
                          U.S. Army Corps of Engineers
                         Hydrologic Engineering Center
                               609 Second Street
                               Davis, California


            X     X  XXXXXX    XXXX        XXXX       XX      XXXX
            X     X  X        X    X       X   X     X  X    X
            X     X  X        X            X   X    X    X   X
            XXXXXXX  XXXX     X       XXX  XXXX     XXXXXX    XXXX
            X     X  X        X            X  X     X    X        X
            X     X  X        X    X       X   X    X    X        X
            X     X  XXXXXX    XXXX        X    X   X    X   XXXXX



PROJECT DATA
Project Title: 14986_ArroyoLosPostas_EG
Project File : 14986_ArroyoLosPost.prj
Run Date and Time: 4/19/2019 3:15:33 PM

Project in English units



PLAN DATA

Plan Title: Plan 06
Plan File : g:\Active Projects\_14000 to 15999\14986\14986000000\HECRAS\Current
Conditions\14986_ArroyoLosPost.p06

           Geometry Title: ALP2
           Geometry File : g:\Active Projects\_14000 to
15999\14986\14986000000\HECRAS\Current Conditions\14986_ArroyoLosPost.g02

           Flow Title    : 14986_ArroyoLosPositas_EG
           Flow File     : g:\Active Projects\_14000 to
15999\14986\14986000000\HECRAS\Current Conditions\14986_ArroyoLosPost.f01

Plan Summary Information:
Number of:  Cross Sections =   36    Multiple Openings  =    0
            Culverts       =    0    Inline Structures  =    0
            Bridges        =    0    Lateral Structures =    0

Computational Information
    Water surface calculation tolerance  =  0.01
    Critical depth calculation tolerance =  0.01
```

```
    Maximum number of iterations         =  20
    Maximum difference tolerance         =  0.3
    Flow tolerance factor                =  0.001

Computation Options
    Critical depth computed only where necessary
    Conveyance Calculation Method: At breaks in n values only
    Friction Slope Method:         Average Conveyance
    Computational Flow Regime:     Mixed Flow



FLOW DATA

Flow Title: 14986_ArroyoLosPositas_EG
Flow File : g:\Active Projects\_14000 to 15999\14986\14986000000\HECRAS\Current
Conditions\14986_ArroyoLosPost.f01

Flow Data (cfs)

  River           Reach           RS                    PF 1
  ALP             ALP             1243.83               6570


Boundary Conditions


  River           Reach           Profile                       Upstream
     Downstream


  ALP             ALP             PF 1                       Normal S = 0.008
  Normal S = 0.008




GEOMETRY DATA

Geometry Title: ALP2
Geometry File : g:\Active Projects\_14000 to 15999\14986\14986000000\HECRAS\Current
Conditions\14986_ArroyoLosPost.g02

CROSS SECTION


RIVER: ALP
REACH: ALP                RS: 1243.83
```

```
INPUT
Description:
Station Elevation Data    num=      29
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  461.92   10.64  461.5512.42999  459.51   21.47  459.9126.67999  459.34
   29.98  459.1446.03999  450.8458.53999  444.04   62.78   442.7   73.17  441.51
      80  438.4680.39999  438.23   83.87  438.06   85.91  437.52   88.39  437.32
89.00999   437.6   93.85  439.8299.14999  440.19  105.23  439.81  117.63  445.42
  119.85  446.23  120.91  446.47  141.14   448.2  153.49   448.9  167.65  449.72
   189.4  454.97  192.29  455.86  196.84  457.27  204.06  459.52

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .03558.53999    .035  117.63    .035

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
      58.53999  117.63            21.12   21.12   21.12             .1       .3

CROSS SECTION


RIVER: ALP
REACH: ALP                RS: 1222.71

INPUT
Description:
Station Elevation Data    num=      25
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  460.34   30.87  460.33   33.64  460.2840.31999  459.78   62.42  448.39
71.59999  442.57      85  440.15      90  439.31    96.5   437.7    96.6   437.7
    98.7   437.8      99   437.9   99.95   438.5  103.42  439.72  103.64  439.88
  124.21  442.75  133.12  446.44  133.42  446.44   152.8  446.95  156.93  447.34
  197.24  449.67  215.11  453.99  217.51  454.73  221.28   455.9  233.06  459.56

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .03571.59999    .035  124.21    .035

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
      71.59999  124.21             20.8    20.8    20.8             .1       .3

CROSS SECTION


RIVER: ALP
REACH: ALP                RS: 1201.91

INPUT
Description:
```

```
Station Elevation Data    num=      24
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  460.4326.09001  460.4431.48001  460.3535.42001  460.0547.85001  453.67
65.67001  442.3582.48001  440.4784.23001  440.1985.87001  440.1587.35001   438.5
   91.39     43898.37001   438.5  104.16  440.97  118.61  443.96  126.33  444.95
  128.88     446  133.87  446.07  172.96   447.1  182.56  448.01  210.07   449.6
  222.51  452.62  224.19  453.14  226.84  453.96   245.6  459.78

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .03565.67001    .035  118.61    .035

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
      65.67001  118.61             22.6    22.6    22.6             .1       .3

CROSS SECTION


RIVER: ALP
REACH: ALP                RS: 1179.31

INPUT
Description:
Station Elevation Data    num=      33
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  460.4616.45999  460.61   18.59  460.6123.07001  460.5329.49001   459.5
   30.58  458.7837.10001  454.42   56.56  442.07    58.3  441.88   62.67  441.19
   66.75     439   70.39   437.8   73.12     43881.25999     439   97.19   439.5
   109.2     440  111.56     440  113.32  441.24  121.07   444.1  122.49  445.67
  139.28  445.65  147.74  445.62  153.47  445.67  223.78  447.51  226.81  447.58
   229.6  447.84  245.27  451.54   250.3   453.1  274.35  460.55  275.14  461.21
  278.23  461.96  280.17  461.96  280.53  461.97

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .035   56.56    .035  121.07    .035

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
         56.56  121.07            25.71   25.71   25.71             .1       .3

CROSS SECTION


RIVER: ALP
REACH: ALP                RS: 1153.6

INPUT
Description:
Station Elevation Data    num=      52
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
```

```
       0  460.662.279999  460.569.700012  460.21   29.31  460.28   40.84  460.32
   46.16  460.3750.48001  459.23   59.97  459.2768.57999  452.48   69.34  451.98
72.76001  448.6474.29999  447.85   77.59  445.4980.64999  443.3980.67001  443.37
85.04999  442.4890.51999  440.5991.17001   440.291.42001   440.291.64001  440.13
   96.48  437.62   99.61  436.94  103.91  436.64  104.76  436.81  112.96  438.55
  115.63  438.99  117.03  439.19   118.1  439.68  127.37  440.27   129.7  440.82
  130.39  440.82  134.68  441.19  137.03  441.38  141.36  441.75  149.59  445.42
  150.56  445.42  187.44  445.32  190.41  445.43  196.21  445.45  196.68  445.45
   200.7  445.53  241.01     445  286.43  452.74  320.62  458.98  325.29  459.71
  330.47  460.51  335.97  462.78  338.04  463.85  338.25     464  342.59  464.58
  346.61   465.1  349.72   465.5

Manning's n Values         num=       3
     Sta    n Val     Sta    n Val     Sta    n Val
       0    .03591.17001    .035   118.1    .035

Bank Sta: Left   Right     Lengths: Left Channel   Right      Coeff Contr.   Expan.
      91.17001   118.1             26.85   26.85   26.85             .1       .3

CROSS SECTION


RIVER: ALP
REACH: ALP                 RS: 1126.75

INPUT
Description:
Station Elevation Data    num=      57
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  460.74   13.37  460.1137.92999   460.241.07999  460.2250.10999  460.02
56.70999  459.9158.32999  459.9159.64999  458.8960.15997   458.460.71997  457.97
84.17999  442.5287.76999  440.2289.66998  438.7689.85999   438.7   90.09  438.64
93.81998  437.08   93.87  437.0993.94998  437.0996.71997  437.1599.37997  438.49
  101.45  438.58  101.47  438.58   105.8  440.01  106.42  440.02  116.64  440.03
  117.32  440.18  121.49  440.99  123.24  441.93  127.99  442.62  132.11   443.1
  136.33  443.56  142.05  443.99  191.29  445.43  205.53  445.41  207.23   445.4
  239.85  444.54  289.32  453.84  304.33  456.17  323.03  459.09  326.04  460.37
   326.5  460.48  328.62   461.1  332.64  462.09  332.78  462.13  333.55  462.28
  341.07  463.93  341.42     464   349.5  465.47     352     466  353.55  466.74
  356.17     468  356.98  468.18  360.33  468.93   365.1     470  369.06  470.84
  374.98     472  379.11  472.41

Manning's n Values         num=       3
     Sta    n Val     Sta    n Val     Sta    n Val
       0    .03584.17999    .035  142.05    .035

Bank Sta: Left   Right     Lengths: Left Channel   Right      Coeff Contr.   Expan.
      84.17999  142.05             22.17   22.17   22.17             .1       .3

CROSS SECTION
```

```
RIVER: ALP
REACH: ALP                RS: 1104.58

INPUT
Description:
Station Elevation Data    num=      65
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  460.8516.70999  460.07   34.88  460.13      40  460.1555.76999  459.78
60.47998  459.768.45999  448.11   75.41  442.62   79.12  440.5383.13998  439.18
84.45999  439.3985.57999  439.4190.97998   438.1   91.25  437.98   93.13  438.31
   95.72  438.2595.88998  438.2596.04999  438.3496.17999  438.3498.92999  439.12
   99.41  439.28  100.76   439.3  114.72     440  116.78  440.51  120.74  441.68
  124.47  442.18  131.04  442.88  134.22  443.13  143.28  443.83     153  444.54
  183.84  445.42  220.97  445.36  241.27  444.82  243.62  444.76  278.43  451.29
   298.9  454.48  323.25  458.28  324.23  458.44  324.24  458.53  324.51  461.98
  324.68     462  326.58  462.15  328.52   462.3  328.86  462.32  336.69  462.92
   350.8     464   354.2  465.41  355.29     466  356.94  466.82  358.47   467.6
  358.81  467.77  358.88  467.81  358.93  467.84     359  467.88  359.22     468
  359.63  468.09  359.65  468.09  360.63  468.27  365.36  469.21  369.39     470
  372.68  470.62  378.03  471.62  379.99     472  384.94  472.53  385.49  472.59

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .035   75.41    .035     153    .035

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
         75.41     153            27.37   27.37   27.37             .1       .3

CROSS SECTION


RIVER: ALP
REACH: ALP                RS: 1077.21

INPUT
Description:
Station Elevation Data    num=      66
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  460.65 7.73999  460.1211.29001  459.95   13.13  459.9623.42999     460
43.98001  459.54   51.75  459.37   60.03  447.3861.89999  444.7165.29999  443.57
   69.62  442.7977.07001  438.1177.20001     43877.35999  437.9778.04001  438.09
   80.44  438.03   82.06  437.8884.04999  437.71   84.22  437.69   85.22  437.65
87.26001  438.16   87.41  438.2487.42999  438.24   94.22  438.83  101.71  439.49
  103.63  440.05  110.66  441.67  114.98  442.13  123.14  442.86  134.67  443.71
   147.2  444.69   151.6  445.01  165.47   445.4  223.94  445.32  236.12     445
  236.46  445.06  254.78   448.5  281.15  452.61  300.22  455.58   313.8   457.7
  313.87  458.39  313.91  459.02  314.09  461.33  317.32  461.69  318.81  461.85
  319.78     462  325.14  462.43  325.96  462.49  329.28  462.73  346.04     464
```

```
  346.06     464  346.91   464.4  347.02  464.45  347.31   464.6  348.14  464.97
  348.51  465.14  349.06  465.41  350.19     466  351.14  466.43   352.2  466.91
  354.51     468  357.31  468.82  361.49     470  370.48  471.59  372.75     472
   378.5  472.62

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .035   69.62    .035  134.67    .035

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
         69.62  134.67            32.26   32.26   32.26             .1       .3

CROSS SECTION


RIVER: ALP
REACH: ALP                RS: 1044.95

INPUT
Description:
Station Elevation Data    num=      74
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0    462.4899902     4629.769989  460.7410.35999  460.69   14.88  459.89
   20.66  459.9129.06998  459.71   38.62  459.4853.67999  458.72   59.94  448.24
60.04999  448.21   63.59  444.4663.70999  444.3966.51999  442.8369.84998  440.26
69.95999  440.14   69.97  440.1372.03998  440.01   72.06  440.0174.82999  439.77
   78.63  439.4578.63998  439.4583.06998  439.1987.23999   438.787.35999  438.66
87.84998  438.67   91.12  436.97   93.56  436.71   93.78  436.6996.07999  437.08
99.53998  437.81   99.78  437.83  102.32  438.35  102.72  438.44  107.92  440.16
  113.83  441.33  117.35  441.66   129.1  442.72   140.4  443.59  151.55  444.47
  162.82  445.36  173.98  445.47   240.9  445.27  241.17  445.27  275.69  450.65
   290.2  452.92  316.69  457.05  316.82  458.31  316.89  459.45  316.99  460.75
  318.86  460.96  319.73  461.05  323.44  461.61   325.2  461.83  327.24  461.98
   327.5     462  332.05   462.4  344.54  463.46  349.74     464  350.42  464.45
  350.72  464.55  351.25  464.78  353.07     466  356.49  467.83  356.77     468
  356.85     468  364.44  469.53  365.56  469.76  366.04  469.85  366.32  469.91
  366.79     470  366.88     470  375.69  471.17  376.24  471.25

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .03569.84998    .035  117.35    .035

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
      69.84998  117.35            25.57   25.57   25.57             .1       .3

CROSS SECTION


RIVER: ALP
REACH: ALP                RS: 1019.38
```

```
INPUT
Description:
Station Elevation Data    num=      84
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0     46210.48001     462   18.34  460.4418.51999  460.4118.86002  460.36
    19.5  460.2219.67999  460.19   19.78  459.5623.14001  459.6435.35001  459.37
   40.53  459.2458.42999  457.7659.82001  457.7171.20999  448.0775.42001  443.76
82.79999   437.683.11002  437.5885.39001  438.0390.29999  438.3390.79999  438.29
91.14999  438.2791.67999  438.2496.14999  437.96   96.37  437.94  100.63  436.98
  100.75  436.96  100.88  436.89  100.89   436.9  102.84  437.33  105.17  437.69
  105.24  437.71  108.42  438.28  108.44  438.29  108.52  438.31   108.6  438.33
   127.5  441.12  131.04  441.45  132.59  441.65  140.42   442.7  158.23  444.03
  174.19  445.29  202.53  445.59  239.94  445.46  251.97  445.48  253.48  445.49
  274.85  448.82  283.83  450.22  323.12  456.35  323.31  458.22  323.41  459.91
  323.43  460.12  323.74  460.16  323.89  460.18  324.51  460.27   324.8  460.31
  325.15  460.33  326.51  460.45  327.17  460.51  329.36  460.71  331.21  460.89
  342.76     462  352.17  463.23  353.75  463.43  357.77     464  360.67  465.53
  361.56     466  361.64  466.01  361.67  466.01  361.74  466.02  363.86  467.01
  365.15  467.59  366.01     468  366.06     468  366.26  468.01  366.31  468.01
  369.46  468.58  372.15  469.01  372.72  469.11  373.51  469.23  377.78     470
  379.23  470.15  380.96  470.35  381.07  470.36  384.19   470.7

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .03575.42001    .035  140.42    .035

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
      75.42001  140.42            39.57   39.57   39.57             .1       .3

CROSS SECTION


RIVER: ALP
REACH: ALP                RS: 979.81

INPUT
Description:
Station Elevation Data    num=      62
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0     462 6.48999     4627.660004  461.778.259979  461.689.349976  461.52
11.42999  461.07      12  460.9712.31998  459.8812.32999  458.9723.03998  459.22
60.50998  458.4463.19998  458.3769.69998     45871.85999   457.888.20999  439.35
   89.19  438.41   89.69  438.09   94.84  437.5897.44998  437.3299.78998  437.57
  103.49  437.89  103.93  437.85   104.3  437.93     106  438.34  110.18  438.65
  115.62   439.5  117.64  439.64  132.97  440.44  137.85  441.77  140.46  442.12
   144.4  442.25  156.52  442.68  172.78  444.05  187.05  445.15  218.86  445.61
  228.29  445.71  231.15  445.69  234.37  445.69  263.82  445.77  267.98  446.42
  269.72   446.7  325.72  455.43  325.98   458.1  326.04  458.99  326.18     459
  330.41  459.59  333.12     460  346.35  461.09  351.54  461.51  356.98  461.98
```

```
  357.28     462  359.31  462.91  361.35   463.5  362.36  463.85  362.79     464
  363.46  464.37  366.61     466  370.42  466.98  374.81     468  377.87  468.63
  384.44     470  389.02  470.41

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .03588.20999    .035  117.64    .035

Bank Sta: Left   Right     Lengths: Left Channel   Right     Coeff Contr.   Expan.
      88.20999  117.64            33.71   33.71   33.71             .1       .3

CROSS SECTION


RIVER: ALP
REACH: ALP                RS: 946.1

INPUT
Description:
Station Elevation Data    num=      58
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  458.4221.70999  458.0439.04999  458.0847.17999  458.1148.42999  458.04
57.01001  457.9558.57999  455.62   67.13  438.19   67.22  438.15   69.91  437.44
71.32001  437.81   74.25  435.7975.89001  435.8377.35001  435.9978.39001  435.85
81.45999  435.4581.57999  435.46   84.87  438.5285.04001  438.5489.60999  439.14
   95.92  439.11   98.41  439.26  105.84  440.95  107.43  441.36  114.19  441.64
  119.16  441.84   134.5  442.72  142.14  443.01  167.69  444.98  195.11  445.37
  201.26  445.66  214.27  446.31  237.41   446.4  242.83  446.86  291.09  454.39
  291.39  457.44  291.42  457.63  294.04     458  307.94   459.9  308.68     460
  309.94  460.15  310.26  460.19  315.65   460.9  317.29  461.12  317.94  461.21
  323.62     462  324.81  462.64  327.48     464  329.55  465.17  330.98     466
  331.13  466.01  331.26  466.01  335.83  466.64  341.27  467.37  345.69  467.97
  345.94     468  346.29  468.07  351.75  469.04

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .035   67.13    .035   84.87    .035

Bank Sta: Left   Right     Lengths: Left Channel   Right     Coeff Contr.   Expan.
         67.13   84.87            33.89   33.89   33.89             .1       .3

CROSS SECTION


RIVER: ALP
REACH: ALP                RS: 912.21

INPUT
Description:
Station Elevation Data    num=      56
```

```
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  457.8728.94998  457.9331.26999  457.9433.47998  457.9736.65997  457.91
42.42999  457.39   48.84  451.4954.91998  445.06   60.37  438.78   61.06  435.91
63.31998  434.7463.40997  434.7765.32999  435.4468.72998  435.7772.59999  435.34
75.21999  436.7477.18999  438.2878.15999   438.685.87999  438.3786.47998  438.37
87.27998  438.5796.28999   440.6  100.53   440.9  116.69  442.35     125  442.76
  155.12  444.22  162.89  444.69  164.13  444.74  172.66  445.14  182.22   445.6
  219.12  447.44   223.9  447.45  239.65  448.79  266.74  453.02  266.91  454.74
  267.06  455.72  267.15     456     268     456   268.8  456.11  282.48     458
  282.49     458  282.66  458.02  285.89  458.43  291.16   459.2  296.57     460
  300.15  461.78  300.44  461.89   300.7     462  300.89  462.13  304.29     464
  307.76  465.51  309.91     466  309.93     466  310.33  466.04  311.52  466.18
  320.58  467.19

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .035   60.37    .03578.15999    .035

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
         60.3778.15999            29.44   29.44   29.44             .1       .3

CROSS SECTION


RIVER: ALP
REACH: ALP                RS: 882.77

INPUT
Description:
Station Elevation Data    num=      75
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0   457.816.64999  457.8225.73001  457.5927.13998  457.35   27.44  457.35
   32.12  456.52   41.28  453.4541.85001  453.4342.23001  453.3143.26999  452.04
   47.28  446.59   48.05  445.55    48.3  445.39   51.36  443.41   51.94  443.01
   54.12  441.61   54.38  441.5357.03999  441.03   58.14  440.5261.73999  440.04
    62.8  439.99   65.08  438.28   67.08  437.39    67.3  435.98   68.63  434.88
   68.97   434.9   71.38  435.02   73.81  434.9878.28999  436.22   80.17   436.4
   83.84   437.3   86.09  438.6286.45999  438.83   91.34   439.8   92.17  439.82
   95.08   440.1  102.38  440.72  122.59  443.14  133.89  443.82  139.25  444.16
  154.03  444.77  189.25  446.47  202.78   447.4  248.28  451.04  249.57  451.15
  250.68  451.32  250.69  451.42  251.98  456.04  252.37  456.04  255.35  456.03
  256.22  456.03   257.2  456.02  260.01  456.02  261.74  456.01  265.12  456.01
  266.73     456  277.48  457.41  277.93  457.47   278.4  457.52  281.31  457.97
  281.49  457.97  281.68  457.98  282.67  457.98  283.36     458   283.4     458
  284.86  458.65  287.92     460  288.81  460.46  289.31  460.76  291.37     462
  293.78  463.08   295.7     464   295.8  464.01  295.81  464.01  300.06   464.9

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .035    62.8    .035  102.38    .035
```

```
Bank Sta: Left   Right     Lengths: Left Channel   Right     Coeff Contr.   Expan.
          62.8  102.38           34.18   34.18   34.18            .1       .3

CROSS SECTION


RIVER: ALP
REACH: ALP                RS: 848.59

INPUT
Description:
Station Elevation Data    num=      70
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  457.6822.26001  457.0526.23001  456.3727.10001  456.35   33.77  455.17
   41.39  451.7341.96001  448.7546.46001  445.54   49.53  443.54   50.05  443.14
52.12001  442.6154.49001  441.97   55.05  441.94   60.06  441.51   61.75  441.14
   62.52  439.7664.32001   439.666.07001  438.39   67.28  438.15   69.83   438.2
70.62001  437.2873.95001  437.4778.71001   437.681.01001  437.4685.34001   436.8
85.40001  436.82   87.55  436.61   87.77  436.59   87.81  436.61   87.89  436.62
   90.33  436.61   92.58  437.78   93.63   438.5   95.03  438.9696.60001  439.21
   99.39  439.95  103.11  440.75  107.44  441.25  133.22  443.91  143.21   444.5
  144.29  444.56  145.43  444.64  200.23  448.18  202.29  448.28   212.5  448.89
  220.55  449.94  220.58  451.48  220.63  452.28  224.87  454.18  226.78  454.93
  229.61     456  229.85     456  233.54   456.5  234.97  456.68  238.06  457.08
  241.93  457.56  242.29  457.61  245.15     458  245.17     458  247.38   458.3
  253.35  459.11  254.71   459.3  255.59  459.42  259.73     460  262.14  460.48
  271.71     462  273.17  462.37  273.66  462.48  274.77  462.78  276.13  463.09

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .035   60.06    .035  107.44    .035

Bank Sta: Left   Right     Lengths: Left Channel   Right     Coeff Contr.   Expan.
         60.06  107.44           16.15   16.15   16.15            .1       .3

CROSS SECTION


RIVER: ALP
REACH: ALP                RS: 832.44

INPUT
Description:
Station Elevation Data    num=      65
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  457.62   28.37   456.8   33.59  455.91   34.73  455.8836.57001  455.55
   39.33   454.3   43.97  452.34   44.92  451.89   49.03  448.45   49.37  447.48
    54.3  446.554.79001  446.26   55.97  445.64   56.81  445.16    63.2  441.23
64.35001  441.03   66.12  441.09    67.3  440.8871.49001  439.9971.99001  439.88
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   74.08  438.8180.32001  438.59   82.87  438.26   84.05  438.07   89.94  437.55
89.99001  437.54   90.03  437.53   93.28  436.92   93.36  436.91   93.42  436.96
   99.25  436.26    99.3  436.25  104.34  439.79  108.08  440.29  122.54  442.26
  133.36  443.18  136.25  443.38  143.17  443.87  145.87  444.05     156  444.68
  182.01  446.51  186.04  446.75  201.34  448.75   201.4  451.69  201.49  453.21
  206.48  455.45  206.55  455.47  206.61  455.48  206.69  455.49  206.79   455.5
  208.79  455.69  211.42  455.96  211.85     456  211.99     456  212.21  456.01
  212.39  456.01  217.78  456.55  224.33   457.2  225.04  457.26  227.35  457.47
  232.07     458  243.25  459.21  250.24     460  254.09  460.64  257.44  461.16

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .035    63.2    .035  108.08    .035

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
          63.2  108.08            33.75   33.75   33.75             .1       .3

CROSS SECTION


RIVER: ALP
REACH: ALP                RS: 798.69

INPUT
Description:
Station Elevation Data    num=      59
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  457.3228.23001  456.47   30.33  456.4130.64001  456.42   31.22  456.35
   32.83  456.0534.95001  455.4636.23001  455.02    42.8  452.6643.35001  452.46
   43.38  452.4543.40001  452.4349.71001  448.6357.98001  444.8658.34001  444.68
66.37001  441.2366.65001  441.0872.21001   439.8   72.44  439.7978.07001  439.13
78.60001  439.0482.37001  438.54   86.11  437.45      90  436.97   92.38  435.99
94.96001  435.24      95  435.22   102.2  436.07  102.31  436.18  102.55   436.2
  106.01  439.77  133.02  442.49  139.25  443.12  139.58  443.16  139.64  443.17
  168.02  446.88  168.13   451.6  169.07     452  175.34  453.39  175.39   453.4
  178.33     454   181.1  454.48  181.72  454.59  182.71  454.77  184.74  455.18
  188.61     456  188.65     456  203.81  457.94  204.24     458  214.77  459.38
  219.51     460  231.54   461.8   232.9     462  236.68  462.62  245.33     464
  245.36     464     249  464.22   255.8  464.62  271.32  465.51

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .03566.37001    .035  106.01    .035

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
      66.37001  106.01            28.26   28.26   28.26             .1       .3

CROSS SECTION
```

```
RIVER: ALP
REACH: ALP                RS: 770.43

INPUT
Description:
Station Elevation Data    num=      73
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  456.927.160004   456.7   25.59  456.19   28.34  456.2533.64999  455.65
33.69998  455.6439.28999  453.7539.40999   453.745.82999  450.4552.46999  447.38
52.48999  447.3752.51999  447.3558.59999  444.3158.67999  444.2866.32999  440.15
66.40999  440.1371.60999  439.8471.92999   439.876.42999  439.0578.26999   438.8
81.37999  437.7483.53999  437.53   86.12  436.7987.43999  436.3488.43999  435.76
90.62999  435.1996.01999  433.4797.09999  433.0497.19998  433.05  103.82  439.33
  118.27  440.75  118.84  441.32   123.1  441.75   124.1  441.85  148.94  445.28
  153.34  445.86   153.8  445.92  153.81     446  153.85  446.01  154.52     450
  154.54     450  154.55  450.01  154.56  450.01  154.67  450.03  164.39     452
  167.01  452.42  176.47     454     178  454.35   179.3  454.64  181.09  455.04
   185.3     456  190.95  456.92  191.26  456.97  193.83  457.39  197.57     458
  197.72     458  202.39  458.75     210     460  211.39  460.23  217.79  461.25
  221.31  461.81  222.56     462  223.27  462.09  223.79  462.16  226.22   462.5
  233.07  463.45  236.76     464  236.85     464  238.31  464.07  239.34  464.11
  239.62  464.13   241.1  464.19  264.52  465.22

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .03566.32999    .035  118.27    .035

Bank Sta: Left   Right     Lengths: Left Channel   Right      Coeff Contr.   Expan.
      66.32999  118.27              24.22   24.22   24.22             .1       .3

CROSS SECTION


RIVER: ALP
REACH: ALP                RS: 746.21

INPUT
Description:
Station Elevation Data    num=      71
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  456.56   19.39  456.0219.92999  456.03    22.2  455.52   28.36  454.33
37.98999  452.4339.75999  451.6244.48999  449.49   45.37  449.08   52.95  445.56
53.28999  445.39   53.72  445.1462.82999  440.6668.00999  439.4768.18999  439.46
68.25999  439.45   68.39   439.4   68.98  438.6473.54999  438.1673.98999  438.11
   78.72  436.85   80.78  436.0882.92999  435.24   82.97  435.2186.79999  434.84
92.28999  434.5792.32999  434.57    94.7  434.7995.53999  434.8795.56999  434.88
98.03999  439.05   98.09  439.17  109.42  439.85  111.02  440.06  122.63  442.36
  128.02   443.1  138.71  443.97  140.44  444.26  141.23  444.38   142.1  444.52
  144.98  444.97  151.54  449.59  152.48     450  152.66     450  157.15  450.97
  160.53     452  160.57  452.01  160.72  452.01  166.23  453.44  167.57  453.78
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   168.5     454  168.57     454  168.61  454.01  169.17  454.11  172.41  454.72
  175.03  455.21  179.13     456  179.19     456   179.7  456.12  180.26  456.25
   180.6  456.33  181.24  456.47  187.71     458  189.96  458.53  196.15     460
  202.79  461.18  207.19     462  209.27  462.26  222.36     464  240.76  464.84
   242.4   464.9

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .03568.00999    .03598.03999    .035

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
      68.0099998.03999            28.52   28.52   28.52              .1       .3

CROSS SECTION


RIVER: ALP
REACH: ALP                RS: 717.69

INPUT
Description:
Station Elevation Data    num=      69
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  456.337.529999  455.97   17.89  455.3522.85001  453.91   26.98  453.03
33.92999  451.19   35.41  450.9342.71001  448.47   44.08  448.1544.24001   448.1
52.53999  446.0260.96001  443.55   61.77  443.08   66.84  441.1967.03999  441.07
69.89999  439.4380.46001  436.42   80.47  436.41   80.47  436.42   88.53  436.04
   90.38  435.9693.17999  436.47   93.48  436.44   96.53  437.0398.71001  437.66
   98.87  437.65   98.94  437.64   99.03  437.78  107.53  437.81   108.4  437.87
  113.19  439.24  115.24  439.63  123.11  440.77  133.83  442.34  137.13  442.49
  147.17  443.55  150.51   444.1  151.18  446.76  152.72     448  158.24  449.88
  158.77     450   158.9     450  159.95  450.23  161.08  450.51  163.25  451.04
  163.39  451.08  166.83  451.92  167.01  451.96  167.14     452  168.63  452.47
  173.07     454  178.38  455.69  179.55     456  180.15  456.03  187.21  457.57
  188.63  457.88   188.8  457.91  189.19     458  197.06  459.74  198.26     460
  201.53  460.62  208.96     462  217.13   463.1  220.24  463.52  222.56  463.83
  223.08  463.89  223.85     464  223.91     464   243.5   464.7

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .03569.89999    .035  137.13    .035

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
      69.89999  137.13             30.4    30.4    30.4              .1       .3

CROSS SECTION


RIVER: ALP
REACH: ALP                RS: 687.29
```

```
INPUT
Description:
Station Elevation Data    num=      60
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  455.23   28.59  454.47   32.03  454.07   41.09  452.0242.98999  451.32
   49.84  447.81   58.92   441.7   61.31  440.3666.28999   439.1   73.27  439.09
   79.62  438.72    80.2   438.786.67999  437.72   86.94  437.6689.89999  436.93
   90.95  436.85   95.53  436.44   96.38  436.47   98.36  436.52  101.66  436.62
  106.37  438.75  117.24   440.5  117.36  440.52  136.64  442.66  138.93  442.91
  144.96  443.46  148.13   447.3  148.99     448  153.01  449.72  153.62     450
  154.14  450.29  157.19     452  165.53  453.99  165.55     454  167.04  454.48
  171.79     456  171.89  456.06  172.08  456.18  174.85  457.98  174.95     458
  175.04  458.01  175.79  458.53   178.1     460  178.52     460  178.67  460.01
  179.26  460.02  180.32  460.04  180.67  460.05  181.11  460.05  191.38  461.18
  196.33  461.79  198.04     462  198.93   462.1  199.23  462.13  206.55  462.88
  217.25     464  218.87  464.05  222.11  464.16  222.26  464.17  229.06   464.4

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .035   61.31    .035  144.96    .035

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
         61.31  144.96            56.86   56.86   56.86             .1       .3

CROSS SECTION


RIVER: ALP
REACH: ALP                RS: 630.43

INPUT
Description:
Station Elevation Data    num=      70
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  455.33   20.56  455.32   29.87   454.531.06999  454.4139.50999  451.86
    41.7  451.0950.03999  448.2656.42999   445.860.28999  444.0265.70999  440.14
70.52999  438.7970.60999  438.7474.66999  438.07   77.56  438.2680.68999  436.41
81.84999  435.4485.60999  435.49   88.87  435.23   90.91  435.11   92.23  435.04
   95.62  435.1298.46999  436.97  104.06  438.71  106.68  439.25  111.38  440.58
  125.29  441.27  126.34  441.35  131.88  441.73  133.05  441.88  133.32  445.04
  133.33  445.06  133.56     446  135.84     446  136.04  446.16  136.36  446.36
  137.07  446.86  138.73     448  140.83  449.84   140.9  449.92  141.01     450
  141.07  450.04  141.09  450.05  141.12  450.06  141.14  450.08  141.27  450.15
  143.38  451.19  144.71     452   147.5  453.47   148.1  453.77  148.61     454
  148.65     454  149.66  454.56  150.36  454.93   152.6     456  155.43  457.15
   157.5     458  158.89  458.56  162.41     460  167.21  461.25  170.02     462
  170.18     462  170.42  462.01  176.92  462.46  183.12  462.88   184.4  462.96
  185.49  463.04  188.34  463.23  190.25  463.35  195.04  463.58   195.4   463.6
```

```
Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .03565.70999    .035  111.38    .035

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
      65.70999  111.38            52.61   52.61   52.61             .1       .3

CROSS SECTION


RIVER: ALP
REACH: ALP                RS: 577.82

INPUT
Description:
Station Elevation Data    num=      57
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0   454.91.590012  454.93   18.58  454.91   24.25  454.56   26.53  454.22
31.76001  452.7534.73001  451.9934.85001  451.9535.48001  451.63   45.38   446.3
49.85001  443.64    55.7  440.17   64.59  439.69   67.91  439.1870.82001  438.47
   72.84  436.8977.06001  436.8281.29001  436.1283.83001  435.9585.24001  435.85
   87.88  435.92   92.27   437.5   93.19  437.5996.93001  437.65   97.87     438
  101.66  438.06  125.04     441  126.95   444.3  127.68     446     129     446
  129.14  446.07  129.64  446.31  133.03     448  135.28  449.13  137.05     450
  139.42  451.16  141.12     452  143.54  453.24  143.74  453.34  144.95     454
  145.98  454.63  147.38  455.41  148.48     456  152.51  457.39  153.27  457.64
   154.3     458  154.35  458.01  156.79  458.86  157.98  459.26  158.61  459.45
  159.11   459.6  160.03  459.96  160.12     460   160.3  460.01  160.69  460.01
  170.89  461.03  173.35  461.27

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .035    55.7    .035  125.04    .035

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
          55.7  125.04            56.16   56.16   56.16             .1       .3

CROSS SECTION


RIVER: ALP
REACH: ALP                RS: 521.66

INPUT
Description:
Station Elevation Data    num=      73
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  454.63   15.62  454.96   16.02  454.95   16.38  454.92   27.11  454.82
27.96001  454.81   33.25  452.97   37.78  451.25    43.5  449.21   46.39   448.9
    54.3  447.95    59.2  445.88   62.69   444.5   70.52  441.68   74.56  440.72
```

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77.85001  440.0479.85001  439.39      80  439.34   83.66   438.783.99001  438.65
   84.27  438.65   91.14  438.7691.21001  438.75   91.83  438.67    97.3  438.15
   99.98  437.51  102.48  436.34  106.08   436.2  108.59  435.58  110.44  435.19
  112.21  435.04  114.18  434.87  115.83   435.2  116.57  435.35  119.07   438.2
  121.64  438.57   122.1  438.65  128.89  439.85  129.44  439.91  154.47  441.76
  154.88  441.76  154.91  441.77  154.93  442.03  154.98  442.37  154.99   442.4
     155  442.43  155.92     446  157.15     446  157.51  446.26  158.28   446.8
  159.78  447.87  159.94     448  160.11  448.11  160.17  448.16  160.31  448.25
  160.83  448.61  162.78     450  163.86  450.74  165.72     452  167.26  453.09
  168.55     454  168.62     454  168.81  454.02  168.87  454.02  168.88  454.03
  169.04  454.03   170.7  454.49  173.06  455.12   175.9     456  214.54     456
  223.55  457.27  224.48  457.39  225.72  457.56

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .035   74.56    .035  154.47    .035

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
         74.56  154.47             43.97   43.97   43.97             .1       .3

CROSS SECTION


RIVER: ALP
REACH: ALP                RS: 477.69

INPUT
Description:
Station Elevation Data    num=      63
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  453.783.389999  453.474.410004  453.39   11.63  449.78   18.16  447.19
   19.41   446.723.03999  445.94   28.63  444.7629.56999   444.6   40.11  443.21
   50.03  440.1651.50999  439.68   51.66  439.66    60.5  437.69   60.87  437.69
   69.53  435.27    70.2  435.18    74.7  435.32   75.02  435.3776.46001  435.57
   79.61  435.78   83.86  435.78   86.42  435.86   87.91  436.68   89.03  436.92
   92.33  437.4996.35001  438.37  106.55  440.21  109.81  440.57  111.25  440.56
  124.96  440.55  134.73  440.49  135.45  440.75  135.56  441.99  136.54  443.03
  136.58  443.07  139.39     446  140.33     446  142.98  447.49  143.85     448
  145.42  449.27  146.51     450  147.56  450.52  150.78     452  151.67  452.25
  155.82  453.32  157.48  453.82  158.13     454  158.17     454  158.37  454.01
  158.56  454.01   159.1  454.04  159.24  454.04  166.15  454.82  174.28  455.72
  175.44  455.83  177.06     456  180.12     456  184.63  456.01  200.36  456.01
  201.87     456  216.93     456  232.64  457.72

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .035   50.03    .035  109.81    .035

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
         50.03  109.81              25.9    25.9    25.9             .1       .3
```

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CROSS SECTION


RIVER: ALP
REACH: ALP                RS: 451.79

INPUT
Description:
Station Elevation Data    num=      66
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  453.8417.45999  452.87   32.36  446.8542.50999  443.5352.64999     442
   57.97  441.36   60.89  440.4364.04999  439.666.53999  438.3468.37999  435.67
69.04999  435.16   69.25  434.8771.92999   435.173.81999   435.8   74.81  436.17
79.20999  435.9379.62999  435.79   79.86   435.8   80.17  435.7980.67999  435.88
      96  437.19    96.7  437.33  105.58  437.97   107.4   438.3   112.8  439.22
  119.22  440.33  125.99  440.32   150.5  440.77  152.68  444.85  153.01  446.02
  153.22  446.02  154.66     446  154.74  446.05  155.21  446.18  155.84  446.54
  155.98  446.62  156.59   446.9  157.28  447.23  159.02     448  161.82  449.44
  162.97     450  163.78  450.38  167.28     452   168.7  452.25  170.01  452.45
     173  452.78  175.08  452.99  176.68  453.12  177.92  453.21  178.48  453.35
  179.91  453.72  180.41  453.78  181.38     454  181.73     454  182.05  454.01
  186.41  454.01  186.91  454.02  206.37  455.48  208.58  455.62  209.85   455.7
  213.41     456  213.82     456  220.64  456.01  224.39  456.01  226.62  456.02
   231.9  456.02

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .035   57.97    .035  119.22    .035

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
         57.97  119.22            28.25   28.25   28.25             .1       .3

CROSS SECTION


RIVER: ALP
REACH: ALP                RS: 423.54

INPUT
Description:
Station Elevation Data    num=      79
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  453.55   10.03  452.9718.35001  452.29   22.34  451.6824.56999  451.11
42.10001  445.4242.24001  445.38   44.31  444.68   46.94  443.74   50.27  443.05
   51.69  442.74   51.89  442.49   52.52   441.3   56.34  437.6956.35001  437.68
   57.52  436.43   58.69     435   59.06  434.6260.64999   434.462.31999  434.19
63.81999  434.2964.92999  434.36   66.14  434.47   66.88  435.04   68.64  435.19
70.00999   436.470.96001  436.52   72.64  436.6286.06999  437.4287.21001  437.51
   93.33  437.2394.92999  437.12  105.37  438.79  111.23  439.74  111.41  439.77
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  112.24  439.79  119.97  440.13  131.11  440.32  148.52  440.85  155.26  440.97
  155.49  444.32  156.09  445.98   157.5  445.97  157.55  445.97  158.04  445.96
  158.71  445.98  159.27     446  163.84     448  163.86     448  163.96  448.01
  164.44  448.05   164.6  448.05  165.42  448.44  166.91   449.1   167.3  449.26
  167.82   449.5  168.03  449.58  168.27  449.69  168.98     450  169.53  450.23
  176.17     452  179.14  452.33  180.09  452.41  180.95  452.48  184.78  452.62
  191.91  453.13  203.96     454  214.93  454.78  232.17     456  232.83     456
  236.57  456.01  241.15  456.01  243.48  456.02  247.33  456.02  248.48  456.03
  250.11  456.03  251.21  456.04  251.49  456.04  253.64  456.11

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .035   52.52    .035  111.23    .035

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
         52.52  111.23            33.57   33.57   33.57              .1       .3

CROSS SECTION


RIVER: ALP
REACH: ALP                RS: 389.97

INPUT
Description:
Station Elevation Data    num=      64
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  453.7826.63998  452.93    44.5  448.36   48.58  447.88   56.98  443.95
   57.16  443.39    57.5  441.7257.67999  442.58   58.83  438.27    61.3  437.32
   63.53   435.4   64.42  435.33   66.81  435.34   75.69   435.881.81999  436.11
   86.16  436.3987.17999  436.3887.60001  436.4292.81999  436.8593.99001  436.93
98.99001  437.61  108.37  438.33   117.8  439.14  126.18  439.57  128.76  439.72
  156.01  441.41  156.99  441.47  157.43  441.41  162.64  440.71  162.67  442.26
  167.57  445.39  173.09  445.92  174.68  445.93  175.12  445.93  175.48  445.94
  177.08  445.95  177.69  445.96  178.49  445.98  179.89     446  189.53  447.33
  189.77  447.35  193.86  447.91  194.37     448   195.5  448.53  198.42     450
     207  451.02  215.25     452  215.34     452  224.33  452.64  229.27  452.98
  231.21   453.1  235.24  453.38  246.65     454  248.31   454.1  252.33  454.35
  253.32  454.41  256.52  454.61  257.17  454.65  260.53  454.86  263.81  455.08
  276.73     456  278.25     456  280.95  456.01  293.03  456.36

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .035    57.5    .035  156.01    .035

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
          57.5  156.01             29.3    29.3    29.3              .1       .3

CROSS SECTION
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RIVER: ALP
REACH: ALP                RS: 360.67

INPUT
Description:
Station Elevation Data    num=      72
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  453.8717.56998  453.45   18.81   453.4   29.56     45232.88998  451.56
44.51999  446.71   53.87  440.9156.92999  439.34   59.12  438.17   62.22  435.15
62.34998  435.0366.22998  434.3367.31998   433.868.09998  433.91   70.12  434.17
   72.37  436.27   75.66  436.24   84.53  437.0484.97998  437.0889.13998  437.39
   98.81  438.01   108.1  438.59  118.33  439.06  127.98   439.5  137.76  440.05
  145.98  440.56  151.43  440.87  153.82  440.55  154.98   440.4  155.05  443.62
  156.12   444.3  159.89  444.66  160.96  444.84  161.09  444.86  162.24  445.01
  163.91  445.25  165.05  445.42  165.61  445.47  166.55  445.56  168.15  445.69
  171.33  445.98  171.77  445.98  171.99  445.99  172.03     446  177.11  446.64
  181.92  447.18  184.02   447.4  185.19  447.52  185.34  447.54  186.74  447.66
  187.18  447.69  193.14  447.95  194.05     448  198.48  448.25  199.37  448.31
  199.74  448.32  200.35  448.34  204.17   448.6  224.56     450  224.84     450
  225.76  450.03  225.83  450.03  225.99  450.04  251.18  451.41   261.8     452
  299.41   453.1  299.81   453.1  300.89  453.12  302.16  453.13  302.46  453.14
  322.77  453.45  345.49  453.82

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .035   53.87    .035  151.43    .035

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
         53.87  151.43            41.72   41.72   41.72             .1       .3

CROSS SECTION


RIVER: ALP
REACH: ALP                RS: 318.95

INPUT
Description:
Station Elevation Data    num=      80
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  453.7722.71002  453.96   24.78  454.1834.64001  454.2837.43002   454.1
43.79001   453.782.45001  438.3183.39001  438.1584.02002  438.02   84.88  437.44
85.10001   437.2   86.34  435.9187.36002  435.25   87.69  435.0588.89999  434.64
   89.69  434.13   91.09  434.2892.64001  434.1193.46002  434.1294.86002  434.14
96.64999  434.83   96.91  434.8497.18002  434.8799.74002  435.34  105.58  436.81
  106.29  436.83  107.95  437.04  109.99   437.1  132.12  437.57  138.98   438.1
  144.27  438.41  164.72  439.14  176.26  439.62  183.39  439.86   183.9  439.88
  183.93  439.88  184.07  439.89  204.17  444.01  208.04  444.01  209.45     444
  217.73     444  219.77  445.04  221.78     446  221.89     446  221.95  446.01
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  225.26  446.84  229.62     448   232.8  448.58  237.56  449.44  240.66     450
   240.7     450  241.24  450.03  241.26  450.03  244.16   450.2  257.49  450.72
     265  450.98  266.61  451.01  267.14  451.02  267.69  451.02  274.16  451.09
  276.79  451.13  278.21  451.15  282.66  451.21  289.28  451.36  297.91  451.58
  300.52  451.66  309.86     452  309.92     452  313.69  452.17  317.54  452.33
  321.81  452.41  329.13  452.65  354.79  453.29  355.87  453.32  357.02  453.35
  364.91  453.56  379.79     454  384.31     454  385.51  454.01  390.79  454.02

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .03582.45001    .035  183.39    .035

Bank Sta: Left   Right     Lengths: Left Channel   Right     Coeff Contr.   Expan.
      82.45001  183.39             45.4    45.4    45.4             .1       .3

CROSS SECTION


RIVER: ALP
REACH: ALP                RS: 273.55

INPUT
Description:
Station Elevation Data    num=      66
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  452.6819.45999  452.6834.73999  453.44   47.41  453.6952.31998  453.32
55.57999   453.164.14999  452.5673.01999  447.1173.60999  447.14   78.56  446.76
   85.09   444.3   91.37  435.4592.71999  433.7593.63998  433.9595.40999  434.34
95.56999  434.34  100.82  434.73  102.73   435.4  103.41  435.55  108.95  436.55
  109.13  436.56  109.23  436.57  109.63  436.59  110.05   436.6  113.62  436.74
  120.46  437.03  129.54  437.31  133.37  437.58  142.69  438.18  147.18   438.6
  158.96  439.03  185.27  439.87  190.24  439.77  199.49  441.07  214.59  443.18
  218.67     444  222.04     444  223.03  444.41  226.76     446  227.22  446.18
  231.37     448  233.09  448.63  235.05  449.26  236.31   449.7  237.37  449.92
  237.69     450  239.95  450.04  240.56  450.04  241.79  450.08  242.94  450.08
  244.16  450.17  254.66  450.97  262.41  451.49  266.92     452  266.95     452
  267.96  452.03  268.15  452.03  268.68  452.04  269.25  452.05  294.92  453.79
  297.48  453.96  297.55  453.97  297.66  453.97  297.69  453.98  298.05     454
  312.71     454

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .035   85.09    .035  190.24    .035

Bank Sta: Left   Right     Lengths: Left Channel   Right     Coeff Contr.   Expan.
         85.09  190.24            51.47   51.47   51.47             .1       .3

CROSS SECTION
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```
RIVER: ALP
REACH: ALP                RS: 222.08

INPUT
Description:
Station Elevation Data    num=      69
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  452.613.470001   452.6   10.12  452.5614.89999  452.5518.41998  452.54
   21.56  452.3723.66998  452.3930.70999  452.4834.42999  452.5235.69998  452.53
36.73999  452.5469.19998  452.5576.57999  452.9180.08998  452.6488.55998  452.67
   94.62  452.58  102.62   452.1  107.51  451.61  126.98  439.49  130.22   437.5
  130.59  437.04  133.89  436.87  138.18  433.84  140.63  433.96  142.84  434.06
  142.99  434.08  148.72  434.33  151.73  434.08  153.38  434.06  158.28  433.97
  160.87  434.29  168.13   438.4  168.73  438.41  170.71  438.44  176.62   438.5
  192.58  438.69  197.77  438.85  209.57  439.66  213.84  439.58  234.21  442.42
  234.46  442.45  235.89   442.6  240.78   443.2  245.01  443.73  245.11  443.89
  247.03     444  248.88     444  250.08  444.38  250.16  444.41  250.79  444.68
  252.78  445.42  254.25  445.99  254.32     446  256.67  446.46  259.31  446.93
  265.16  447.99  265.21     448  265.23     448  277.14     450  279.06  450.29
   290.5     452   290.7  452.01  296.48  452.43  297.06  452.46  299.53   452.6
  303.22  452.84   305.8  453.02  324.39     454  334.55     454

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .035  126.98    .035  209.57    .035

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
         126.98  209.57             41.35   41.35   41.35              .1       .3

CROSS SECTION


RIVER: ALP
REACH: ALP                RS: 180.73

INPUT
Description:
Station Elevation Data    num=      84
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  453.158.190002  453.1226.67999  453.0742.96002   452.253.79001  452.32
58.10001  452.3760.39001   452.461.17001  452.4161.81001  452.4288.82001  452.42
90.65001  452.24   101.4  452.27  107.77  450.17  135.31  438.88  135.61  438.73
  140.58  437.42  140.81  437.36   141.6   436.4  143.04  435.24  145.68  434.76
  149.33  433.45  149.42  433.41  149.44  433.41  152.13  433.02  156.25  432.93
  158.39  432.39  158.85  432.27  161.37  432.85  162.88   435.4  167.89  436.93
  172.76  437.19  190.97  438.37  196.38  438.53  207.17  438.84  217.12  439.04
  221.94  439.35  223.41  439.32  229.91  440.18  229.98  440.19  230.41  440.23
  231.84   440.4  233.05  440.54  234.26  442.63  235.36  442.69  236.69  442.76
  237.04  442.78   237.4  442.81  238.95  442.92  241.05  443.09  241.35  443.12
  242.14  443.17  244.24  443.31  246.94   443.5  247.91  443.58  248.98  443.67
```

```
   249.3  443.69  249.63  443.72  252.49  443.94  253.11  443.99  253.27     444
  253.62     444  257.27  445.64  258.72     446  258.81  446.01  258.86  446.01
  261.15  446.44  261.73  446.58  262.15  446.69  262.98  446.91  264.63  447.36
  266.08  447.74  266.73  447.89  267.12     448   267.2     448  267.26  448.01
  267.28  448.01  270.82  448.63  272.08  448.85  272.79  448.97  279.34     450
  279.66     450  289.26  450.63  295.48  451.03  306.57  451.67

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .035  135.31    .035  223.41    .035

Bank Sta: Left   Right     Lengths: Left Channel   Right     Coeff Contr.   Expan.
        135.31  223.41            51.03   51.03   51.03             .1       .3

CROSS SECTION


RIVER: ALP
REACH: ALP                RS: 129.7

INPUT
Description:
Station Elevation Data    num=      78
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0   453.815.38998  453.7648.82001  451.9771.14999  452.2371.97998  452.24
72.42999  452.25   77.94  452.25   84.31  451.6386.29999  451.63   108.9  444.16
  110.09  443.67  116.66  440.43  116.84  440.38  124.16  436.45  126.39  436.81
  126.49  436.73  128.36  436.79  132.26  436.19  135.35  435.04  137.74  433.71
  144.14  433.56  144.46  433.55  146.46  432.92  149.18  433.68  150.93  433.84
  152.94  436.72  155.53  437.12  157.53  437.37  166.93  437.24   179.7  438.02
  190.68  437.66   204.6   438.6  206.06  438.66  211.86  438.92  213.97  440.32
  214.84  440.63   214.9  441.98  215.96     442  220.03  442.81  221.67  442.93
   223.9  443.11  224.07  443.12  224.25  443.14  225.53  443.24  227.06  443.37
   227.6  443.42  228.17  443.47  230.28  443.66  233.03  443.92  233.91     444
  233.99     444  235.88  444.57   240.6     446  240.64     446  240.68  446.01
  240.74  446.01  240.81  446.02  240.83  446.02  243.58  446.87  244.29  447.08
  247.27     448  264.84  449.44  265.98  449.53  271.33     450  276.21  450.39
  296.67     452  296.69     452  296.98  452.01  297.81  452.01  301.76  452.15
  301.82  452.15  304.57  452.24  314.72  452.52  314.96  452.52  324.14  452.74
  331.09  452.88   333.3  452.92  337.43  452.94

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .035  116.66    .035  214.84    .035

Bank Sta: Left   Right     Lengths: Left Channel   Right     Coeff Contr.   Expan.
        116.66  214.84             71.2    71.2    71.2             .1       .3

CROSS SECTION
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RIVER: ALP
REACH: ALP                RS: 58.5

INPUT
Description:
Station Elevation Data    num=      74
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  453.941.940002  453.945.269989  453.95   10.53  453.95   23.66  453.98
25.91998  453.98   41.22     45441.35001     454   43.31  453.7545.19998   453.5
50.51999  451.9451.42999  451.8452.45999  451.75   52.97  451.7253.23999  451.64
55.10999  451.6660.16998  451.73   60.81  451.7495.14999  437.5195.29999  437.49
95.53999  437.49   95.64  437.46   96.67  437.12  100.57  432.49  104.19  432.23
  104.68  432.17  107.23  431.86  107.47   431.9  107.53  431.91  109.35  433.45
  110.46  434.29  110.94  434.56  119.33  436.12   129.7  436.53  134.88  436.35
  139.47  437.44  142.74  437.74  162.07  436.67  164.36   436.6  173.93   438.5
  181.29  437.59  191.03  438.04  193.25  439.52  201.18  441.56  202.43     442
  209.75   443.4  213.16     444  215.82  444.51  219.37  445.19  223.33     446
   224.8  446.24  225.13  446.29  227.06  446.58  228.97  446.89  235.13  447.67
     236  447.79   236.4  447.85  237.64     448  240.56  448.31  242.24   448.5
  244.41  448.74  245.11  448.81  254.26     450  259.55  450.28  263.38  450.46
  267.51  450.65  269.98  450.77  272.06  450.86   277.9  451.14  296.19  451.89
  296.72  451.91  298.74     452  302.13  452.07  332.43  452.23

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .03595.14999    .035  193.25    .035

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
      95.14999  193.25            36.99   36.99   36.99             .1       .3

CROSS SECTION


RIVER: ALP
REACH: ALP                RS: 21.51

INPUT
Description:
Station Elevation Data    num=     104
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  453.3229.71002  453.7440.51001  453.99   40.75     45458.49002     454
   61.28  453.5361.83002  453.4462.64001   453.3   63.19  453.21   64.69  452.95
   67.72  452.4168.35001   452.369.98001     452   72.25   450.5      73     450
73.20001  449.83   73.25  449.7973.35001  449.7175.30002     44876.64001  446.76
77.45001     44677.74002  445.95   78.06  445.94   79.28  445.2679.73001   445.1
   79.91  443.0580.08002  443.0880.26001  443.09   80.28  442.6880.58002   442.4
   85.69  439.61   87.02  438.7794.18001  437.1294.32001  436.95   94.47  436.93
  100.48  436.13  100.88  435.88  101.82  436.13   107.7  435.88  115.69   434.1
  117.84  434.28  118.31  434.32  122.53  431.81  124.76   432.4  125.19  432.55
```

```
  128.09  433.06  130.68  433.06  134.18  436.91  134.38  436.93  137.55  436.43
  138.11  435.91  142.04   434.7  159.84     437  164.18  438.69  165.71  438.63
  166.69  438.59  167.18  438.57  178.24  437.99  186.25  437.58  186.43  437.56
  186.62  437.57  186.67   437.6  186.84  437.64  186.94  437.67  186.98   437.7
  187.09  437.73   188.3  438.18  198.88     442  207.56  443.27  212.78     444
   221.8  445.08  225.28  445.46  230.09  445.98  230.37  445.98  230.63  445.99
  231.08  445.99  231.28     446  231.34     446  232.59  446.22  233.09  446.33
  233.37   446.4  234.53  446.67  234.59  446.68  234.75  446.71  238.93  447.67
  240.56  447.92  241.05     448  245.51  448.38  251.16  448.89  255.44  449.28
  255.96  449.33  257.09  449.44  259.51  449.65  260.65  449.74  263.55     450
  277.22  450.62  281.47  450.82  285.34  451.01  308.64     452  309.13     452
  311.69  452.01  312.97  452.01  337.93  452.04  338.54  452.04

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .035   85.69    .035  198.88    .035

Bank Sta: Left   Right    Coeff Contr.   Expan.
         85.69  198.88             .1       .3



SUMMARY OF MANNING'S N VALUES

River:ALP

      Reach          River Sta.       n1         n2         n3

 ALP                  1243.83           .035      .035      .035
 ALP                  1222.71           .035      .035      .035
 ALP                  1201.91           .035      .035      .035
 ALP                  1179.31           .035      .035      .035
 ALP                  1153.6            .035      .035      .035
 ALP                  1126.75           .035      .035      .035
 ALP                  1104.58           .035      .035      .035
 ALP                  1077.21           .035      .035      .035
 ALP                  1044.95           .035      .035      .035
 ALP                  1019.38           .035      .035      .035
 ALP                  979.81            .035      .035      .035
 ALP                  946.1             .035      .035      .035
 ALP                  912.21            .035      .035      .035
 ALP                  882.77            .035      .035      .035
 ALP                  848.59            .035      .035      .035
 ALP                  832.44            .035      .035      .035
 ALP                  798.69            .035      .035      .035
 ALP                  770.43            .035      .035      .035
 ALP                  746.21            .035      .035      .035
 ALP                  717.69            .035      .035      .035
 ALP                  687.29            .035      .035      .035
 ALP                  630.43            .035      .035      .035
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 ALP                    577.82            .035      .035      .035
 ALP                    521.66            .035      .035      .035
 ALP                    477.69            .035      .035      .035
 ALP                    451.79            .035      .035      .035
 ALP                    423.54            .035      .035      .035
 ALP                    389.97            .035      .035      .035
 ALP                    360.67            .035      .035      .035
 ALP                    318.95            .035      .035      .035
 ALP                    273.55            .035      .035      .035
 ALP                    222.08            .035      .035      .035
 ALP                    180.73            .035      .035      .035
 ALP                    129.7             .035      .035      .035
 ALP                    58.5              .035      .035      .035
 ALP                    21.51             .035      .035      .035




SUMMARY OF REACH LENGTHS

River: ALP

      Reach            River Sta.      Left     Channel    Right

 ALP                    1243.83          21.12     21.12     21.12
 ALP                    1222.71           20.8      20.8      20.8
 ALP                    1201.91           22.6      22.6      22.6
 ALP                    1179.31          25.71     25.71     25.71
 ALP                    1153.6           26.85     26.85     26.85
 ALP                    1126.75          22.17     22.17     22.17
 ALP                    1104.58          27.37     27.37     27.37
 ALP                    1077.21          32.26     32.26     32.26
 ALP                    1044.95          25.57     25.57     25.57
 ALP                    1019.38          39.57     39.57     39.57
 ALP                    979.81           33.71     33.71     33.71
 ALP                    946.1            33.89     33.89     33.89
 ALP                    912.21           29.44     29.44     29.44
 ALP                    882.77           34.18     34.18     34.18
 ALP                    848.59           16.15     16.15     16.15
 ALP                    832.44           33.75     33.75     33.75
 ALP                    798.69           28.26     28.26     28.26
 ALP                    770.43           24.22     24.22     24.22
 ALP                    746.21           28.52     28.52     28.52
 ALP                    717.69            30.4      30.4      30.4
 ALP                    687.29           56.86     56.86     56.86
 ALP                    630.43           52.61     52.61     52.61
 ALP                    577.82           56.16     56.16     56.16
 ALP                    521.66           43.97     43.97     43.97
 ALP                    477.69            25.9      25.9      25.9
 ALP                    451.79           28.25     28.25     28.25
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 ALP                  423.54           33.57     33.57     33.57
 ALP                  389.97            29.3      29.3      29.3
 ALP                  360.67           41.72     41.72     41.72
 ALP                  318.95            45.4      45.4      45.4
 ALP                  273.55           51.47     51.47     51.47
 ALP                  222.08           41.35     41.35     41.35
 ALP                  180.73           51.03     51.03     51.03
 ALP                  129.7             71.2      71.2      71.2
 ALP                  58.5             36.99     36.99     36.99
 ALP                  21.51
```

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SUMMARY OF CONTRACTION AND EXPANSION COEFFICIENTS
River: ALP


      Reach          River Sta.     Contr.    Expan.

 ALP                  1243.83         .1        .3
 ALP                  1222.71         .1        .3
 ALP                  1201.91         .1        .3
 ALP                  1179.31         .1        .3
 ALP                  1153.6          .1        .3
 ALP                  1126.75         .1        .3
 ALP                  1104.58         .1        .3
 ALP                  1077.21         .1        .3
 ALP                  1044.95         .1        .3
 ALP                  1019.38         .1        .3
 ALP                  979.81          .1        .3
 ALP                  946.1           .1        .3
 ALP                  912.21          .1        .3
 ALP                  882.77          .1        .3
 ALP                  848.59          .1        .3
 ALP                  832.44          .1        .3
 ALP                  798.69          .1        .3
 ALP                  770.43          .1        .3
 ALP                  746.21          .1        .3
 ALP                  717.69          .1        .3
 ALP                  687.29          .1        .3
 ALP                  630.43          .1        .3
 ALP                  577.82          .1        .3
 ALP                  521.66          .1        .3
 ALP                  477.69          .1        .3
 ALP                  451.79          .1        .3
 ALP                  423.54          .1        .3
 ALP                  389.97          .1        .3
 ALP                  360.67          .1        .3
 ALP                  318.95          .1        .3
```

ALP	273.55	.1	.3
ALP	222.08	.1	.3
ALP	180.73	.1	.3
ALP	129.7	.1	.3
ALP	58.5	.1	.3
ALP	21.51	.1	.3

ENGEO "RESPONSE TO CITY COMMENTS"
(OCTOBER 17, 2019)

ENGEO
Expect Excellence

GEOTECHNICAL
ENVIRONMENTAL
WATER RESOURCES
CONSTRUCTION SERVICES

Project No.
14986.000.000

October 17, 2019

Mr. Mike Conn
Meridian Property Ventures LLC
2420 Camino Ramon, Suite 215
San Ramon, CA 94583

Subject: Proposed Restaurant
North Livermore Avenue
Livermore, California

RESPONSE TO CITY COMMENTS

References:
1. City of Livermore; Chick-fil-A Application - Incomplete, North Livermore Avenue, Livermore, California; Email dated September 30, 2019.
2. ENGEO; Buried Cast-in-Drilled-Hole Pier Wall Design, Proposed Chick-Fil-A Restaurant, North Livermore Avenue; Livermore, California; May 8, 2019.
3. ENGEO; Creek Scour and Geomorphology Evaluation and Recommendations; Proposed Chick-Fil-A Restaurant, North Livermore Avenue, Livermore, California; May 3, 2019 revised August 23, 2019.
4. ENGEO; Geotechnical Exploration Update; Proposed Chick-Fil-A Restaurant; North Livermore Avenue; Livermore, California; August 23, 2019.

Dear Mr. Conn:

As requested, this letter documents our response to select comments provided by the City of Livermore (Reference 1) regarding the proposed restaurant submittal for the subject project in Livermore, California. The select comments are provided in *italics*, and our response is provided below.

City Comments
City staff is in general agreement with the design approach for the creek bank subject to final design, peer review, and addressing the following comments:

- *The design is premised on the notion that slope stability in the 'long term' is achieved with a 2:1 bank slope and assuming 2 feet of potential downcutting as a result of migration of an observed knick point downstream. The preliminary analysis indicates that improvements will likely be protected over the design life of the project but the analysis does not address any potential creek issues related to (accelerated or sudden) sloughing of the creek bank.*
- *The ENGEO "Response to City Comments" letter states that piers will be "20 feet below the bottom of the adjacent creek" whereas the Pier Wall Design Memorandum uses 30' min pier length with top of bank at over 20' from creek bed. It appears that the response letter refers to depth below the 100-year flood level. Please clarify.*

ENGEO Response

The referenced documents we previously submitted (References 2 through 4) are not preliminary in nature, and should be considered a final design submittal.

The final buried pier wall design (Reference 2) is based on the “ultimate creek scour hazard” as described in the creek scour and geomorphology evaluation (Reference 3). Based on our analysis, the potential for further downcutting and regression of the creek bank is not anticipated to impact the proposed restaurant improvements. The proposed scour countermeasures are intended to function even in the unlikely event that additional (accelerated or sudden) regression of the slope between the buried wall and flowline of the creek does occur. This approach allows the waterway to form its own bed and bank through natural processes and is consistent with San Francisco Bay Regional Water Quality Control Board guidance for stream maintenance and restoration. If at a future date the stream bank erodes and exposes a portion of the pier wall, additional mitigation measures such as lagging and/or additional piers can be installed from within the offset area between the restaurant improvements and the piers to address the concern.

In regards to the final buried pier length of 30 feet as shown in Reference 2, the preliminary concept estimated 24-inch-diameter heavily reinforced piers, spaced 6 feet on center, and extending approximately 40 feet below existing grade. After performing the final buried pier wall design (Reference 2) based on the “ultimate creek scour hazard” as described in the creek scour and geomorphology evaluation (Reference 3), we were able to refine the initial conceptual pier depth estimate. Final pier design consists of 24-inch-diameter piers reinforced with eight #8 longitudinal rebars, with piers spaced 6 feet on center, and extending 30 feet below existing grade.

If you have any questions, please contact us.

Sincerely,

ENGEO Incorporated

REGISTERED PROFESSIONAL ENGINEER
URI ELIAHU
No. 2166
GEOTECHNICAL
STATE OF CALIFORNIA

Andrew Firmin, GE

Uri Eliahu, GE

af/jb/ue/cjn

ENGEO "RESPONSE TO CITY COMMENTS"
(JANUARY 24, 2019)

ENGEO
Expect Excellence

GEOTECHNICAL
ENVIRONMENTAL
WATER RESOURCES
CONSTRUCTION SERVICES

Project No.
14986.000.000

January 24, 2019

Mr. Mike Conn
Meridian Property Ventures LLC
2420 Camino Ramon, Suite 215
San Ramon, CA 94583

Subject: Proposed Restaurant
North Livermore Avenue
Livermore, California

RESPONSE TO CITY COMMENTS

References:

1. City of Livermore; Chick-fil-A Development Project, 1754 North Livermore Avenue, Livermore, California; July 2, 2018.
2. ENGEO; Conceptual Buried Pier Wall Design, Proposed Restaurant, North Livermore Road; Livermore, California; May 14, 2018, latest revision dated May 29, 2018.
3. Cornerstone Earth Group; Geotechnical Peer Review, Chick-fil-A Restaurant, North Livermore Avenue, Livermore, California; August 27, 2018.

Dear Mr. Conn:

As requested, this letter documents our response to comments provided by the City of Livermore (Reference 1) regarding the conceptual buried pier wall design (Reference 2) to address potential creek bank erosion of Arroyo Las Positas adjacent to the subject project in Livermore, California.

We provide background information, design assumptions, and conceptual plans for the buried pier wall design in Reference 2. In our experience, this design concept is a common method to address potential creek bank erosion. Cornerstone Earth Group concurred in its peer review letter (Reference 3) that our pier wall solution is expected to stabilize the site for over 50 years, far longer than the expected useful life of the improvements constructed by Meridian. This design alternative provides superior bank stabilization compared to other options (discussed below), and additionally provides the following benefits to the project:

- As outlined in Reference 2, installation of closely spaced drilled piers extending an estimated 20 feet below the bottom of the adjacent creek will improve slope stability concerns identified in the project geotechnical reports by providing additional lateral resistance through the reinforced concrete piers. No geomorphological study is required for this alternative as it is not dependent on understanding the fluvial path of the creek.
- Since the buried piers can be constructed entirely from behind top of bank, there will be no disturbance to the Arroyo and no permitting requirements with resource agencies having jurisdiction within the creek. The buried pier approach is the least environmentally impactful of all of the alternatives we evaluated, as it does not disturb native flora and fauna both on the face of the creek bank and the bed of the Arroyo.

- Our solution is also the least invasive aesthetically as the Arroyo and the bank face is currently highly visible from North Livermore Avenue. A solution that is entirely below grade will be much more acceptable to the community than the more invasive options evaluated in this letter.
- In the unlikely event that bank erosion extends toward the buried piers, it will be arrested at the wall interface. Soil arching, a fundamental geotechnical engineering concept, would redistribute soil pressures to the rigid concrete piers and not allow the erosion to propagate further toward the project improvements.

Based on our review, potential alternatives to address bank stabilization for the subject project include a full buried retaining wall, revegetation and rock slope protection of the existing creek bank, or 'hard' structural solutions on the existing creek bank (such as shotcrete application). A brief summary of each potential alternative is provided below. We will further quantify and evaluate these conclusions if we receive preliminary support from staff for the solutions we are proposing.

1. Buried retaining wall. This option would provide a similar result to the selected approach, but presents several additional challenges related to 1) constructability of a full below-grade retaining wall, 2) Occupational Safety and Health Administration (OSHA) compliance issues, and 3) subsurface retaining wall drainage considerations. The last challenge would likely require discharge to Arroyo Las Positas, and therefore this alternative would also create permitting challenges with resource agencies and potentially significant environmental impacts to the creek and the riparian habitat within the creek bank. No geomorphological study is required for this alternative as it is not dependent on understanding the fluvial path of the creek.
2. Revegetation and rock slope protection. This alternative would only reduce the potential for localized scour and erosion of protected portions of the creek bank. Compared to the buried pier wall, this alternative does not improve overall slope stability of the creek bank and requires extensive permitting with resource agencies with creek jurisdiction regarding the potentially significant impacts to the creek and the riparian habitat within the creek bank. In addition, because adding vegetation and rock slope protection to one area of a creek impacts the overall hydrology of the creek, a geomorphological study would likely be required. It is likely that such a solution could cause significant community opposition as well due to the impact to sight lines from North Livermore Avenue.
3. 'Hard' structural solutions on the existing creek bank. This alternative would provide similar benefit and challenges compared to the revegetation and rock slope protection option. However, we anticipate this option is a non-starter since permitting agencies do not generally approve these types of 'hard' solutions in creek environments. It is likely that this solution will also result in significant community opposition due to the impact to sight lines from North Livermore Avenue.

As discussed, we propose to perform a final design for the buried pier wall that will include supplemental field exploration, final design recommendations, hydrologic evaluation of Arroyo Las Positas to confirm our preliminary design assumptions, structural calculations, and construction plans. However, we are seeking concurrence with City of Livermore staff on our methodology and potential solution. During this process, we can work with you and the design team to determine construction and maintenance cost estimates.

If you have any questions, please contact us.

Sincerely,

ENGEO Incorporated

Andrew Firmin, GE

Uri Eliahu, GE

REGISTERED PROFESSIONAL ENGINEER
URI ELIAHU
No. 2166
GEOTECHNICAL
STATE OF CALIFORNIA

GILES ENGINEERING "GEOTECHNICAL ENGINEERING EXPLORATION AND ANALYSIS"
(APRIL 24, 2017)

🔴 **RED**

Geotechnical Engineering Exploration and Analysis

Proposed Chick-fil-A Restaurant #3805
Livermore @ 580 FSU
SWC of N. Livermore Avenue and I-580 Freeway
Livermore, California

Prepared for:

Chick-fil-A, Inc.

Prepared by:

Giles Engineering Associates, Inc.

April 24, 2017
Project No. 2G-1606012

GILES
ENGINEERING ASSOCIATES, INC.

GILES
ENGINEERING ASSOCIATES, INC.
GEOTECHNICAL, ENVIRONMENTAL & CONSTRUCTION MATERIALS CONSULTANTS

· Atlanta, GA
· Baltimore, MD
· Dallas, TX
· Los Angeles, CA
· Manassas, VA
· Milwaukee, WI

April 24, 2017

Chick-fil-A, Inc.
15635 Alton Parkway, Suite 350
Irvine, California 92618

Attention: Ms. Beth Witt
Development Coordinator

Subject: Geotechnical Engineering Exploration and Analysis
Proposed Chick-fil-A Restaurant #3805
Livermore @ 580 FSU
SWC of N. Livermore Avenue and I-580 Freeway
Livermore, California
Project No. 2G-1606012

Dear Ms. Witt:

Giles Engineering Associates, Inc. (Giles) is pleased to present our *Geotechnical Engineering Exploration and Analysis* report prepared for the above-referenced project. Conclusions and recommendations developed from the exploration and analysis are discussed in the accompanying report.

We appreciate the opportunity to be of service on this project. If we may be of additional assistance, should geotechnical related problems occur or to provide construction observation and testing services, please do not hesitate to call at any time.

Respectfully submitted,

GILES ENGINEERING ASSOCIATES, INC.

C 070687
EXP 6-30-17
CIVIL

Edgar L. Gatus, P.E.
Assistant Branch Manager

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GEOTECHNICAL ENGINEERING EXPLORATION AND ANALYSIS

PROPOSED CHICK-FIL-A RESTAURANT #3805
LIVERMORE @ 580 FSU
SWC OF N. LIVERMORE AVENUE AND I-580 FREEWAY
LIVERMORE, CALIFORNIA
PROJECT NO. 2G-1606012

1.0 EXECUTIVE SUMMARY OUTLINE

The executive summary is provided solely for purposes of overview. Any party who relies on this report must read the full report. The executive summary omits a number of details, any one of which could be crucial to the proper application of this report.

Subsurface Conditions

- Site Class designation C is recommended for seismic design considerations.
- According to the California Department of Conservation- California Geological Survey, Geologic Map of the Livermore Quadrangle, California (2006), the site is located in an area underlain by Holocene stream terrace deposits consisting generally of sand, silt, clay and gravel.
- Soils encountered within our test borings generally consisted of stiff sandy clay and silty clay, and medium dense to very dense silty sand, clayey sand and sand with gravel and possible cobbles at deeper depths. The upper 10 feet of the soils were generally of finer soils (clay) and below 10 feet, the soils were generally granular (sand and gravel with possible cobbles).
- Groundwater was encountered at a depth of about 30 feet below existing ground surface within the deeper test boring (B-1).
- Moist to very moist soil conditions were encountered within some of the near surface soils during our subsurface investigation. Grading operations may require provisions for drying of soils prior to compaction.

Site Development

- Following site stripping, the exposed soils should be proof rolled with heavy construction equipment in the presence of the geotechnical engineer. Any soil that exhibits excessive deflection during proof rolling should be removed and replaced with engineered fill. Prior to placement of fill, the exposed surfaces should first be scarified to an approximate depth of at least 12 inches, moisture conditioned and then recompacted to at least 90% of the maximum dry density as determined by Modified Proctor (ASTM D 1557-00).
- The existing steep descending slope was evaluated to assess its stability. The results of this analysis indicate that the slope is stable with respect to static and pseudostatic global stability.
- A structural setback and mitigation has been incorporated into the project design due to the presence of the adjacent watercourse.

Building Foundation

- Shallow spread footing foundation system supported on suitable bearing soil may be designed for a maximum, net allowable soil bearing pressure of 3,000 psf.
- Minimum reinforcing in the strip footings is recommended to consist of four No. 5 bars (2 top and 2 bottom).

Building Floor Slab

- It is recommended that on grade slab be a minimum 4 inch thick slab-on-grade or turned-down slab, underlain by a minimum 4-inch thick granular material supported on a properly prepared subgrade.
- Minimum slab reinforcing recommended consisting of No. 3 rebars spaced at 18 inches on center, each way.
- The floor slab subgrade soils should be moisture conditioned and tested by the geotechnical engineer immediately prior to floor slab construction.

New Pavement

- Asphalt Pavements: 3 inches of asphaltic concrete underlain by 7 or 10 inches of base course in parking stall and drive lane areas, respectively.
- Portland Cement Concrete: 6 inches in thickness underlain by 4 inches of base course in high stress areas such as entrance/exit aprons, drive-thru lane and the trash enclosure-loading zone.
- Some increased pavement maintenance should be expected due to the presence of medium expansive soils.

RED - This site has been given a red designation due to the presence of the descending steep slope, due to presence of medium expansive soil and due to the required watercourse setback.

2.0 SCOPE OF SERVICES

This report provides the results of the *Geotechnical Engineering Exploration and Analysis* that Giles Engineering Associates, Inc. ("Giles") conducted regarding the proposed development. The *Geotechnical Engineering Exploration and Analysis* included several separate, but related, service areas referenced hereafter as the Geotechnical Subsurface Exploration Program, Geotechnical Laboratory Services, and Geotechnical Engineering Services. The scope of each service area was narrow and limited, as directed by our client and in consideration of the proposed project. The scope of each service area is briefly explained in this report.

Geotechnical-related recommendations for design and construction of the foundation and ground-bearing floor slab for the proposed building are provided in this report. Geotechnical-related recommendations are also provided for the proposed parking lot. Site preparation recommendations are also given; however, those recommendations are only preliminary since the means and methods of site preparation will depend on factors that were unknown when this report was prepared. Those factors include the weather before and during construction, the water table at the time of construction, subsurface conditions that are exposed during construction, and finalized details of the proposed development.

Giles conducted a Phase 1 Environmental Site Assessment for the subject site. A report documenting the results of that assessment have been provided under separate cover (2E-1606011, dated August 16, 2016).

3.0 SITES AND PROJECT DESCRIPTION

3.1 Site Description

The site is currently an irregular shaped vacant lot located at the southwest corner of North Livermore Avenue and the I-580 Freeway in the city of Livermore, California. Based on our Phase I report, the southeastern portion of the subject property was occupied by a residential property from at least 1949 through 2001, when the structures were demolished. The site is bordered on the north by an approximately 20-foot-high, 1:1 (h:v) slope that descends to Arroyo Las Positas (watercourse) then by the I-580 Freeway, on the east by N. Livermore Avenue, on the south by a Jack In The Box restaurant and Hawthorne Suites, and on the west by a vacant parcel.

Based on a review of the ALTA Survey, prepared by Joseph C. Truxaw & Associates, dated April 8, 2016, elevations within the site range from approximately El. 457.1 feet along the westerly end of the property to El. 460.4 feet near the northeast corner of the site near the top of the descending slope. The elevation of the toe of the descending slope is about El. 440. The adjacent northerly descending slope is covered by moderate vegetation that includes shrubs and occasional trees. The site is situated at approximately latitude 37.6991° North and longitude 121.7743° West.

3.2 Proposed Project Description

Based on our review of the site plan prepared by CRHO (project architect), it is our understanding that the proposed building is to be located in the northeasterly portion of the site and will be a single-story wood-frame modular structure with no basement or underground levels and will have a floor area of about 4,634 square feet. We were not provided with specific loading information for this project at the time of this report; however, based on our previous Chick-fil-A projects, we expect the maximum combined dead and live loads supported by the bearing walls and columns will be 2 to 3 kips per lineal foot (klf) and 40 to 50 kips, respectively. The live load supported by the floor slab is expected to be a maximum of 100 pounds per square foot (psf).

Other planned improvements include a paved drive thru and parking lot, menu board signs, a trash enclosure, a patio, concrete walkways and planter areas. Parking lot improvement, within the subject property, will include sidewalks, curbs and gutters, and underground utilities.

According to the *Conceptual Grading & Utility Plan*, prepared by Joseph C. Truxaw & Associates, Inc., dated April 13, 2017, the planned finished floor elevation for the proposed building is El. 461.00. Existing elevations within the building pad area are about El 460. Therefore, site grading is anticipated to include cuts and fills of less than 1 foot, exclusive of site preparation or over-excavation requirements.

The traffic loading on the proposed parking lot improvement is understood to predominantly consist of automobiles with occasional heavy trucks resulting from deliveries and trash removal. The parking lot pavement sections have been designed on the basis of a Traffic Index (TI) of 4.0 for the automobile traffic parking stalls (light duty) and a TI of 5.0 for drive lane areas (medium duty) and for a 20 year design life.

4.0 SUBSURFACE EXPLORATION

4.1 Subsurface Exploration

Prior to drilling, a Drill Permit (Permit # 2016078) was obtained from the Zone 7 Water Agency. Our subsurface exploration consisted of the drilling of eight (8) test borings (B-1 to B-8) to depths of approximately 5 to 51.5 feet below existing ground surfaces. The approximate test boring locations are shown in the Test Boring Location Plan (Figure 1). The Test Boring Location Plan and Test Boring Logs (Records of Subsurface Exploration) are enclosed in Appendix A. Field and laboratory test procedures and results are enclosed in Appendix B and C, respectively. The terms and symbols used on the Test Boring Logs are defined on the General Notes in Appendix D.

Our subsurface exploration included the collection of relatively undisturbed samples of subsurface soil materials for laboratory testing purposes. Bulk samples consisted of composite soil materials obtained at selected depth intervals from the borings. Relatively undisturbed samples were collected using a 3-inch outside-diameter, modified California split-spoon soil sampler (CS) lined with 1-inch high brass rings. The sampler was driven with successive 30-inch drops of a hydraulically operated, 140-pound automatic trip hammer. Blow counts for each 6-inch driving increment were recorded on the field exploration logs. The central portions of the driven core samples were placed in sealed containers and transported to our laboratory for testing.

Where deemed appropriate, standard split-spoon tests (SS), also called Standard Penetration Test (SPT), were also performed at selected depth intervals in accordance with the American Society for Testing Materials (ASTM) Standard Procedure D 1586. This method consists of mechanically driving an unlined standard split-barrel sampler 18 inches into the soil with successive 30-inch drops of the 140-pound automatic trip hammer. Blow counts for each 6-inch driving increment were recorded on the exploration logs. The number of blows required to drive the standard split-spoon sampler for the last 12 of the 18 inches was identified as the uncorrected standard penetration resistance (N). Disturbed soil samples from the unlined standard split-spoon samplers were placed in glass jars and transported to our laboratory for testing.

4.2 Subsurface Conditions

The subsurface conditions as subsequently described have been simplified somewhat for ease of report interpretation. A more detailed description of the subsurface conditions at the test boring locations is provided by the logs of the test borings enclosed in Appendix B of this report.

Soil

According to the California Department of Conservation- California Geological Survey, Geologic Map of the Livermore Quadrangle, California (2006), the site is located in an area underlain by Holocene stream terrace deposits consisting generally of sand, silt, clay and gravel.

Soils encountered within our test borings generally consisted of stiff sandy clay and silty clay, and medium dense to very dense silty sand, clayey sand and sand with gravel and possible cobbles at deeper depths. The upper 10 feet of the soils were generally of finer soils (clay) and below 10 feet the soils were generally granular (sand and gravel with possible cobbles).

Groundwater

Groundwater was encountered at a depth of about 30 feet below existing ground surface during our subsurface investigation within Test Boring B-1.

Fluctuations of the groundwater table, localized zones of perched water, and rise in soil moisture content should be anticipated during and after the rainy season. Irrigation of landscape areas on or adjacent to the site could also cause fluctuations of local or shallow perched groundwater levels.

4.3 Percolation Testing

A below grade storm water infiltration system is being considered at the site and the following information is provided.

Our percolation tests consisted of excavating an eight (8) inch diameter test holes. The bottom of each test hole was covered with about 2-inches of clean gravel then a 2-inch diameter perforated pvc casing was installed with clean coarse sand used to surround the outside casing. Testing involved presoaking the test holes and filling the test hole with water, and recording the drop in the water surface. Measurements were taken in approximately 30-minute period; refilling after every reading. The drop in water level over time is the percolation rate at the test locations. The percolation rates were reduced to account for the discharge of water from both the sides and bottom of the boring. The formula below was used to calculate for the tested infiltration rate.

Tested Infiltration Rate = ΔH (60r) / Δt (r + 2Havg)

Where: r is the radius of the test hole (in)
ΔH is the change in height over the time interval (in)
Δt is the time interval (min)
Havg is the average head height over the time interval

The results obtained from our percolation testing are summarized below.

Test Number	Test Depth (feet)	Percolation Rate (in/hr)	Adjusted Infiltration Rate (in/hr)	Soil Type
B-7	5.0	3.3	0.06	Sandy Clay
B-8	5.0	3.7	0.07	Clayey Sand to Sandy Clay

Based on the results of this testing, it is our opinion that the site clayey soils have very low to negligible percolation rates and are not considered suitable for infiltration.

5.0 LABORATORY TESTING

Several laboratory tests were performed on selected samples considered representative of those encountered in order to evaluate the engineering properties of the on-site soils. The following are brief description of our laboratory test results.

In Situ Moisture and Density

Tests were performed on select samples from the test borings to determine the subsoil's dry density and natural moisture contents in accordance with Test Method ASTM 2216-10. The results of these tests are included in the Test Boring Logs enclosed in Appendix A.

Expansive Potential

To evaluate the expansive potential of the near surface soils encountered during our subsurface exploration, a composite sample collected from Test Boring B-1 (1 to 5 feet) was subjected to Expansive Index (EI) testing in accordance with Test Method ASTM D 4829-11. The result of our expansion index (EI) test indicates that the near surface sample has a medium expansion potential (EI=54).

Atterberg Limits

The Atterberg limits (liquid limit, plastic limit and plasticity index) were determined for a representative sample of the on-site soils in accordance with Test Method ASTM D 4318-00. The result of the Atterberg Limits is included on the Test Boring Logs enclosed in Appendix A.

Consolidation Test

Settlement prediction under anticipated load was made on the basis of a one-dimensional consolidation test. This test was performed in general conformance with Test Method ASTM D 2435. The test sample was inundated in order to evaluate the sudden increase in moisture condition (collapse/swell potential). Result of this test indicated that the tested near surface soils have very low swell potential (0.89%). The Consolidation test curve, Figure 3, is included in Appendix A.

Sieve Analysis

Sieve Analyses that include Passing No. 200 Sieve were performed on selected samples from various depths within Test Borings B-1, B-2, B-4, B-7 and B-8 to assist in soil classification and to aid in the liquefaction analysis. These tests were performed in accordance with Test Method ASTM D 1140-00. The results of these tests are presented in Test Boring Logs, Appendix A.

Direct Shear

The angle of internal friction and cohesion were determined for relatively undisturbed soil samples collected from Test Borings B-2 and B-4. These tests were performed in general accordance with Test Method No. ASTM D 3080-98. Three specimens were prepared for each test. The test specimens were artificially saturated, and then sheared under various normal loads. Results are graphically presented as Figures 4 and 5 in Appendix A.

Soluble Sulfate Analysis and Soil Corrosivity

A representative sample of the near surface soils which may contact shallow buried utilities and structural concrete was performed to determine the corrosion potential for buried ferrous metal conduits and the concentrations present of water soluble sulfate which could result in chemical attack of cement. These test results have been evaluated in accordance with criteria established by the Cast Iron Pipe Research Association, Ductile Iron Pipe Research Association, the American Concrete Institute and the National Association of Corrosion Engineers. The following table presents the results of our laboratory testing.

Parameter	**B-1** **1 to 5 feet**
pH	7.29
Chloride	85 ppm
Sulfate	0. 0009%
Resistivity	840 ohm-cm

The chloride content of near-surface soils was determined for a selected sample in accordance with California Test Method No. 422. The results of this test indicated that tested on-site soils have a Low exposure to chloride.

The soil pH and minimum resistivity values were determined in accordance with California Test Method No. 643. The test results for pH indicated the tested soil was nearly neutral. The results from the minimum resistivity test generally indicate that the tested soils have a severe corrosive potential when in contact with ferrous materials. Therefore, special protection for underground cast iron pipe or ductile pipe may be warranted depending on the actual materials in contact with the pipe. We recommend that a corrosion engineer review these results in order to provide specific recommendations for corrosion protection as well as appropriate recommendations for other types of buried metal structures.

A representative sample of the near surface soils which may contact shallow buried utilities and structural concrete was performed to determine the concentrations present of water soluble sulfate which could result in chemical attack of cement. Our laboratory test data indicated that near surface soils contain approximately 0.0009 percent of water soluble sulfates. Based on Section 1904.1 of the 2016 California Building Code (CBC), concrete that may be exposed to sulfate containing soils shall comply with the provisions of ACI 318-11, Section 4.3. Therefore, according to Table 4.3.1 of the ACI 318-11 a negligible exposure to sulfate can be expected for concrete placed in contact with the tested on-site soils. No special sulfate resistant cement is considered necessary for concrete which will be in contact with the tested on-site soils.

6.0 GEOLOGIC AND SEISMIC HAZARDS

6.1 Active Fault Zones

The project site is located in a highly seismic region of California within the influence of several fault systems. However, the site does not lie within the boundaries of an Earthquake Fault Zone as defined by the State of California in the Alquist-Priolo Earthquake Fault Zoning Act.

6.2 Seismic Hazard Zones

Our review of the published Seismic Hazard Evaluation report for the Livermore Quadrangle (where the subject site is located) indicates that the site is located within a designated Liquefaction Hazard Zone.

General types of ground failures that might occur as a consequence of severe ground shaking typically include landsliding, ground lurching and shallow ground rupture. The probability of occurrence of each type of ground failure depends on the severity of the earthquake, distance from faults, topography, subsoils and groundwater conditions, in addition to other factors. Based on our subsurface exploration, laboratory testing and the seismic designation for this site, all of the above effects of seismic activity are considered unlikely at the site.

7.0 CONCLUSIONS AND RECOMMENDATIONS

Conditions imposed by the proposed development have been evaluated on the basis of the assumed floor elevation and engineering characteristics of the subsurface materials encountered during our subsurface investigation and their anticipated behavior both during and after construction. Conclusions and recommendations presented for the design of building foundation and floor slab, and pavement along with site preparation recommendations and construction considerations are discussed in the following sections of this report.

Development of the proposed site entails soil and foundation oriented considerations with respect to the presence of variable strength fill and possible fill soils and grading associated with the existing drainage channel. Recommendations in this report are predicated upon site preparation, foundation and floor slab construction observed by the geotechnical engineer.

Slope Stability Limit Equilibrium Analysis

The stability of the existing slope configuration was evaluated along Section A-A' (as delineated on Figure 1) using the computer software program GSlope (Mitre Software Corporation). The GSlope program uses a search for the lowest factor of safety within a specified search grid. The GSlope analysis is based on limit equilibrium and incorporates the Bishop's Modified Method of analysis. For the pseudostatic (earthquake) analysis, a pseudostatic coefficient of 0.25g was utilized.

Laboratory testing (direct shear tests) were performed on undisturbed soil samples collected from the site to determine appropriate soil strength parameters for the stability analyses of the slope. The results of the direct shear testing are attached within Appendix A. For our analysis we utilized the direct shear soil strength parameters obtained with an angle of internal friction of 18 degrees with a 1000 psf cohesion for the upper soils and an angle of internal friction of 28 degrees and a cohesion value of 390 psf for the deeper soils. For our analysis, we reduced the cohesion value to 500 psf for the upper soils and 300 psf for the deeper soils as an added level of conservatism.

Cross Section A – A' Results

The existing slope configuration, along Section A-A', was analyzed for long-term and short-term (pseudostatic) stability, utilizing the soil strength parameters noted above. A building foundation line load was also applied at the location of the future building perimeter footings to assess impact of the nearby building. The results of these analyses indicated a static and pseudostatic factors of safety of 1.76 and 1.27, respectively. These values are greater than the typical required factor of safeties of 1.5 for the static and 1.1 for the pseudostatic conditions. Based on these results, it is our opinion that the existing slope is in a stable condition with respect to a deep-seated failure. A copy of the computer output for both the static and pseudostatic analyses is provided with Appendix A.

Watercourse Setback

According to the Alameda County Public Works Agency Engineering Design Guidelines (April 2008), no development shall be permitted within the setbacks provided in the Watercourse Ordinance. For existing bank slopes at 2 horizontal to 1 vertical, or steeper, the setback is established by drawing a line at a 2.5:1 (horizontal to vertical) inclination from the toe of existing bank to a point where it intercepts the ground surface. A 20-foot setback is then applied from the intercept point. However, we understand that development is allowed within this 20-foot .setback if a wall is constructed and extends below the imaginary setback projection. As noted on the Conceptual Grading & Utility Plans (Sections A &H on Sheet 4), a wall been designed to extend below the imaginary projection.

Impact of Site on Stability of Adjacent Properties

It is our opinion that the proposed grading and construction for the subject site will not affect adversely impact the stability of adjoining properties provided that grading and construction are performed in accordance with the recommendations provided herein and in accordance with local code guidelines.

7.1 Seismic Design Considerations

Faulting/Seismic Design Parameters

Research of available maps published by the California Geological Survey (CGS) indicates that the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. The potential for fault rupture through the site is, therefore, considered to be low. The site may however be subject to strong groundshaking during seismic activity. The proposed structure should be designed in accordance with the current version of the 2016 California Building Code (CBC) and applicable local codes. Based upon the encountered subsurface soils, a Site Class C is recommended for design.

According to the maps of known active fault per 2008 National Seismic Hazard Maps – Source Parameters to be used with the 2016 CBC, the Mt. Diablo Thrust, Greenville Connected and Calaveras (CN+CC+CS) faults are the closest known active faults and are located about 3.5, 4.2 and 8.1 miles, respectively, from the site and with an anticipated maximum moment magnitude (Mw) of 6.70, 7.00 and 7.03, respectively.

Within the International Code Council's 2015 International Building Code (IBC), the five-percent damped design spectral response accelerations at short periods, S_{DS}, and at 1-second period, S_{D1}, are used to determine the seismic design base shear. These parameters, which are a function of the site's seismicity and soil, are also used as parts of triggers for other code requirements. The following values are determined by using the program Java Ground Motion Parameter Calculator- Version 5.0.10 written by the ICC.

IBC 2015/ CBC 2016, Earthquake Loads	
Site Class Definition (Table 1613.5.2)	C
Mapped Spectral Response Acceleration Parameter, S_s (Figure 1613.5(3) for 0.2 second)	1.668
Mapped Spectral Response Acceleration Parameter, S_1 (Figure 1613.5(4) for 1.0 second)	0.600
Site Coefficient, F_a (Table 1613.5.3 (1) short period)	1.0
Site Coefficient, F_v (Table 1613.5.3 (2) 1-second period)	1.3
Adjusted Maximum Considered Earthquake Spectral Response Acceleration Parameter, S_{MS} (Eq. 16-37)	1.668
Adjusted Maximum Considered Earthquake Spectral Response Acceleration Parameter, S_{M1} (Eq. 16-38)	0.780
Design Spectral Response Acceleration Parameter, S_{DS} (Eq. 16-39)	1.112
Design Spectral Response Acceleration Parameter, S_{D1} (Eq. 16-40)	0.520

Liquefaction

According to the Seismic Hazard Zones map for the Livermore Quadrangle, published by the California Geological Survey (CGS), the site is located within an area that has been designated by the State Geologist as a "zone of required investigation" due to the potential for earthquake-induced liquefaction. Therefore, a site liquefaction evaluation consistent with the guidelines contained in DMG Special Publication 117A (2008) has been performed as part of the current investigation. Although groundwater was encountered at a depth of 30 feet below existing ground surface during our subsurface exploration, a historic high water level of 10 feet was adopted for the liquefaction analysis.

The peak ground acceleration was determined in accordance with Section 11.8.3 of 2010 ASCE 7 with the March 2013 errata. The horizontal acceleration was determined using the USGS U.S. Seismic Design Maps website and we incorporated a Site Class D. For this analysis, a PGA_M of 0.628g was obtained. A deaggregation analysis was performed to determine the predominant earthquake magnitude for a 2% probability of exceedance in 50 years (2,475 year return period). For this event, the predominant earthquake magnitude of 6.58 was obtained.

Our liquefaction study was based on the NCEER procedure (Youd & Idriss, 1998) using a peak ground acceleration of 0.628g and an earthquake magnitude of 6.58. The liquefaction analysis was performed using the computer program Liquefypro (version 5) developed by Civil Tech Software. The program is based on the most recent publications of the NCEER Workshop and SP117A Implementation. The result of this analysis indicates that the site soils within Boring B-1 are not potentially susceptible to soil liquefaction. The computer output file for this analysis is presented graphically as Plate A1 of Appendix A. The results of this analysis indicates that the site soils are not potentially susceptible to soil liquefaction. Some minimal (less than 1/10 inch) dry settlement is estimated.

7.2 Site Development Recommendations

The recommendations for site development as subsequently described are based upon the conditions encountered at the test boring locations. Due to elevated in-situ moistures of the site soils, grading operations may require provisions for drying of soils prior to compaction. In addition, due to the presence of moist to very moist and sensitive soils, the loads imposed by heavy rubber-tired equipment during grading may induce localized pumping of the subgrade that would require stabilization prior to fill placement. The grading contractor should include contingencies for air-drying of excessively moist soil, as well as the stabilization of excavation bottoms in their bids. Imported soils may be required if on-site soils cannot be air-dried on site due to space, time constraints, or weather.

Site Clearing

Clearing operations should include the removal of all landscape vegetation within the area of the proposed site improvements. Large shrubs to be removed should be grubbed out to include removal of their stumps and major root systems.

Should any unusual soil conditions or subsurface structures be encountered during demolition operations or during grading, they should be brought to the immediate attention of the project geotechnical consultant for corrective recommendations.

Existing Utilities

All existing utilities (if any) should be located. Utilities that will be preserved are recommended to be relocated outside the building area. Utilities that are not to be reused should be capped off and removed or properly abandoned in-place in accordance with local codes and ordinances. The excavations made for removed utilities are recommended to be backfilled with structural compacted fill. Underground utilities, which are to be reused or abandoned in-place, are recommended to be evaluated by the structural engineer and utility backfill is recommended to be evaluated by the geotechnical engineer, to determine their potential effect on the new development. If any existing utilities are to be preserved, grading operations must be carefully performed so as not to disturb or damage the existing utility.

Proofroll and Compact Subgrade

After site clearing and lowering of site grades where necessary, the subgrades within the proposed new building, pavement and drive thru areas should be proofrolled in the presence of the geotechnical engineer with appropriate rubber-tire mounted heavy construction equipment or a loaded truck to detect very loose/soft yielding soil which should be removed to a stable subgrade. Following proofrolling and completion of any necessary overexcavation, the subgrades should be scarified to a minimum depth of 12 inches, moisture conditioned or air dried, and recompacted to at least 90 percent of the Modified Proctor (ASTM D1557-00) maximum density. The upper 1 foot of the pavement subgrade should have minimum in-place density of at least 95% of the maximum dry density. Low areas and excavations may then be backfilled in lifts with suitable very low to medium expansion (EI less than 91) structural compacted fill. The selection, placement and compaction of structural fill should be performed in accordance with the project specifications.

The Guide Specifications included in Appendix D (Modified Proctor) of this report are recommended to be used, at a minimum, as an aid in developing the project specifications. The floor slab subgrade may need to be recompacted prior to slab construction due to weather and equipment traffic effects on the previously compacted soil.

Reuse of On-site Soil

On-site medium expansive soils may be reused as structural compacted fill provided they do not contain oversized materials and/or significant quantities of organic matter or other deleterious materials. Due to the moisture sensitivity of the site soils, care should be used in controlling the moisture content of the soils to achieve proper compaction for load bearing and pavement support. Some drying of the site soils is expected to be necessary prior to their use as engineered fill, based on the in-situ moisture contents of these soils. During inclement weather, drying is not expected to be feasible and use of a select fill may be necessary. All subgrade soil compaction as well as the selection, placement and compaction of new fill soils should be performed in accordance with the project specifications under engineering controlled conditions.

Subgrade Protection

The near surface soils that are expected to comprise the subgrade are sensitive to water and disturbance from construction activities. Unstable soil conditions will develop if the soils are exposed to moisture increases or are disturbed (rutted) by construction traffic. The site should be graded to prevent water from ponding within construction areas and/or flowing into excavations. Accumulated water must be removed immediately along with any unstable soil. Foundation concrete should be placed and excavations backfilled as soon as possible to protect the bearing grade. The degree of subgrade instability and associated remedial construction is dependent, in part, upon precautions taken by the contractor to protect the subgrade during site development.

Silt fences or other appropriate erosion control devices should be installed in accordance with local, state and federal requirements at the perimeter of the development areas to control sediment from erosion. Since silt fences or other erosion control measures are temporary structures, careful and continuous monitoring and periodic maintenance to remove accumulated soil and/or replacement should be anticipated.

Fill Placement

Material for engineered fill should be free of organic material, debris, and other deleterious substances, and should not contain fragments greater than 3 inches in maximum dimension. Fill soils should possess a very low to medium expansive potential (EI<91). On-site excavated soils that meet these requirements may be used to backfill the excavated new building and pavement areas.

All fill should be placed in 8-inch-thick maximum loose lifts, moisture conditioned and then compacted to at least 90 percent of the Modified Proctor maximum density. A representative of the project geotechnical consultant should be present on-site during grading operations to document proper placement and compaction of all fill, as well as to verify compliance with the other geotechnical recommendations presented herein.

Import Structural Fill

Any soils imported to the site for use as structural fill should consist of very low to low expansive (EI less than 51) soils. Materials designated for import should be submitted to the project geotechnical engineer no less than three working days for evaluation. In addition to expansion criteria, soils imported to the site should exhibit adequate shear strength characteristics for the recommended allowable soil bearing pressure, soluble sulfate content and corrosivity and pavement support characteristics.

7.3 Construction Considerations

Construction Dewatering

Groundwater was encountered at a depth of about 30 feet during our field exploration. However, shallower perched water conditions may occur due to seasonal precipitation and runoff characteristics of the site. Conventional filtered sump pumps placed in excavations are expected to be suitable for dewatering within shallow excavations should any excess water conditions be observed. Deeper excavations that extend into the water table may require a more elaborate dewatering system.

Soil Excavation

Some localized slope stability problems may be encountered in steep, unbraced excavations considering the nature of the subsoils. All excavations must be performed in accordance with CAL-OSHA requirements, which is the responsibility of the contractor. Shallow excavations may be adequately sloped for bank stability while deeper excavations or excavations where adequate back sloping cannot be performed may require some form of external support such as shoring or bracing.

7.4 Foundation Recommendations

Vertical Load Capacity

Upon completion of the recommended building pad proof rolling, scarification and recompaction, it is our opinion the proposed structure may be supported by a shallow foundation system. Foundations may be designed for a maximum, net, allowable soil-bearing pressure of 3,000 pounds per square foot (psf). Minimum foundation widths for walls and columns should be 16 and 24 inches, respectively, for bearing considerations, regardless of actual soil pressure. The maximum bearing value applies to combined dead and sustained live loads. This allowable soil bearing pressure may be increased by one-third for short term wind and/or seismic loads.

Reinforcing

The recommended minimum quantity of longitudinal reinforcing within continuous strip footings for geotechnical considerations is four No. 5 bars (2 top and 2 bottom) continuous through any intermittent column pad footings. The recommended quantity of reinforcing pertains to a minimum 12-inch thick and a maximum 24-inch wide footing; additional reinforcing may be necessary if a thinner or wider footing is used to develop equivalent rigidity. The reinforcing recommendation is intended to provide greater rigidity due to the presence of medium expansive onsite soils. A qualified structural engineer should determine the actual reinforcing details.

Lateral Load Resistance

Lateral load resistance will be developed by a combination of friction acting at the base of foundations and slabs and the passive earth pressure developed by footings below grade. Passive pressure and friction may be used in combination, without reduction, in determining the total resistance to lateral loads. A one-third increase in the passive pressure value may be used for short duration wind or seismic loads.

A coefficient of friction of 0.30 may be used with dead load forces for footings placed on compacted fill soil. An allowable passive earth pressure of 250 psf per foot of footing depth (pcf) below the lowest adjacent grade may be used for the sides of footings placed against structural fill. The maximum recommended allowable passive pressure is 2,000 psf.

Bearing Material Criteria

Evaluation of the foundation bearing soils is recommended to be performed by the geotechnical engineer at the time of foundation construction prior to placement of reinforcing steel. Soil suitable to serve as subgrade for support of foundations should exhibit at least a stiff comparative consistency (qu ≥ 1.5 tsf) for cohesive soils and/or a firm relative density (N-value of least 10) for non-cohesive soils for the recommended 3,000 psf allowable soil bearing pressure.. The depth of evaluation should be determined by the geotechnical engineer. If unsuitable bearing soils are encountered, they should be recompacted in-place if feasible, or excavated to a suitable bearing soil subgrade and to a lateral extent as defined by Item No. 3 of the enclosed Guide Specifications, with the excavation backfilled with structural compacted fill to develop a uniform bearing grade. Alternatively, footings may be locally stepped through any unsuitable bearing soil and be founded entirely upon competent materials. If stepping is desired, foundation details should be provided by the structural engineer.

Foundation Embedment

The California Building Code (CBC) requires a minimum 12-inch foundation embedment depth. However, it is recommended that exterior foundations extend at least 18 inches below the adjacent exterior grade for bearing capacity consideration. Interior footings may be supported at nominal depth below the floor. All footings must be protected against weather and water damage during and after construction, and must be supported within suitable bearing materials.

Estimated Foundation Movement

Post-construction total and differential settlement of a shallow foundation system designed and constructed in accordance with the recommendations provided in this report are estimated to be less than 1 and ½ inch, respectively, for static conditions. The estimated differential movement is anticipated to result in an angular distortion of about 0.002 inches per inch on the basis of a minimum clear span of 20 feet. The maximum estimated total and differential movement is considered within tolerable limits for the proposed structures provided it is considered in the structural design.

7.5 Floor Slab Recommendations

Subgrade

The floor slab subgrade should be prepared in accordance with the appropriate recommendations presented in the Site Development Recommendations section of this report. Foundation, utility trenches and other below-slab excavations should be backfilled with structural compacted fill in accordance with the project specifications. Due to the expansive nature of the subgrade soils, these soils must be maintained at a moisture content of about 2 to 4% above the soil's optimum moisture content (per ASTM D-1557) to a depth of 12 inches prior to concrete placement. Testing by the geotechnical engineer is recommended within 24 hours of concrete placement to document proper soil moisture conditioning.

Design

The floor of the proposed building may be designed and constructed as a slab-on-grade supported on a properly prepared subgrade. If desired, the floor slab may be constructed monolithically with foundations where the foundations consist of thickened sections thereby using a turned-down slab construction technique. Minimum slab reinforcing, for geotechnical considerations, is recommended to consist of No. 3 rebars at 18 inches on center, each way. Based on the recommended reinforcing, assumed live loading and medium expansion potential of the near surface soils, the slab is recommended to possess a minimum thickness of 4 inches. A qualified structural engineer should perform the actual design of the slab to ensure proper thickness and reinforcing.

The floor slab is recommended to be underlain by a 4-inch thick layer of granular material. A minimum 10-mil synthetic sheet should be placed below the floor slab to serve as a vapor retarder where required to protect moisture sensitive floor coverings (i.e. tile, or carpet, etc.). The sheets of the vapor retarder material should be evaluated for holes and/or punctures prior to placement and the edges overlapped and taped. If materials underlying the synthetic sheet contain sharp, angular particles, a layer of sand approximately 2 inches thick or a geotextile should be provided to protect it from puncture. An additional 2-inch thick layer of sand is recommended between the slab and the vapor retarder to promote proper curing. The sand layers above and below the synthetic sheeting may be used as a substitute for the granular material below the slab. Proper curing techniques are recommended to reduce the potential for shrinkage cracking and slab curling.

Estimated Settlement

Post-construction total and differential movement (settlement and/or heave) of the floor slab designed and constructed in accordance with the recommendations provided in this report are estimated to be less than ½ and ⅓ inch, respectively. The estimated differential movement is anticipated to occur across the short dimension of the structure. The maximum total and differential movement is considered within tolerable limits for the proposed structure, provided that the structural design adequately considers this distortion.

7.6 Retaining Wall Recommendations

Due to the existing site grades, it is possible that retaining walls may be needed for this site. The retaining wall(s) may be supported by conventional shallow spread footings designed for an allowable soil bearing pressure of 3,000 psf. A higher allowable soil bearing pressure may be possible but that determination should be based on a review of the locations and details of the planned wall.

Retaining walls may be designed for an allowable passive earth pressure of 250 pounds per square foot, per foot of depth, to a maximum value of 3,000 pounds per square foot. In addition, a coefficient of friction of 0.30 may be used with dead load forces for footings placed on competent soil, as determined by the geotechnical engineer. The recommended allowable soil bearing pressure and passive pressure may be increased by one-third for short term wind and/or seismic loads.

Design of walls should incorporate an adequate factor-of-safety against both over-turning and sliding (FS=1.5). The overturning resultant should also fall within the center third (kern) of the retaining wall footing for stability, or the design must be re-evaluated with a reduced bearing area.

Static Lateral Earth Pressures

Retaining walls should be designed to resist the applicable lateral earth pressures. On-site expansive soils are **not** recommended for use as backfill behind walls. Retaining wall backfill should consist of very low to low expansive soils and allow for a drainage layer as discussed in subsequent paragraphs. For very low to low expansive soils (EI less than 51) to be used as backfill materials, an active earth pressure of 40 pounds per cubic foot (equivalent fluid pressure) should be used assuming a level adjacent backfill and drained conditions. For walls to be restrained at the top, an at-rest pressure of 60 pcf should be used for design. All retaining walls should be supplied with a proper subdrain system. All walls should be designed to support any adjacent structural surcharge loads imposed by other nearby walls or footings and vehicles in addition to the above recommended active earth pressure.

Pea gravel, crushed rock or clean sand exhibiting a sand equivalent of 30 or greater may also be used for retaining wall backfill. If these materials are used as backfill, the retaining wall may be designed for active and at-rest earth pressures of 30 and 45 pounds per cubic foot (equivalent fluid pressure), respectively.

Drainage and Damp-proofing

Retaining walls are recommended to be designed for drained earth pressures and therefore, adequate drainage should be provided behind the walls. This can be accomplished by installing subdrains at the base of the walls. Wall footing-drains should consist of a system of filter material and perforated pipe. The perforated pipe system should consist of 4-inch diameter, schedule 40, PVC pipe or equivalent, embedded in 1 cubic foot of Class II Permeable Material (CALTRANS Standard Specifications, latest edition) or equivalent per lineal foot of pipe. Alternatively, ¾-inch open graded gravel or crushed rock enveloped in Mirafi 140 geofabric or equivalent may be used instead of the Class II Permeable Material. The pipe should be placed at the base of the wall, and then routed to a suitable area for discharge of accumulated water.

Wall backfill should be protected against infiltration of surface water. Backfill adjacent to walls should be sloped so that surface water drains freely away from the wall and will not pond. Damp-proofing of walls below-grade is recommended especially where moisture control is required by an approved waterproofing compound or covered with similar material to inhibit infiltration of moisture through the walls.

Wall Backfill

Retaining wall backfill behind the drainage layers should consist of very low to low-expansive soils with an E.I. less than 51, as determined by the ASTM D 4829-03 method. Wall backfill should not contain organic material, rubble, debris, and rocks or cemented fragments larger than 3 inches in greatest dimension. A 1 foot thick low-expansive cohesive layer, or pavement, should be placed at the surface to help prevent surface water intrusion. A geotextile or filter fabric should be placed between the granular drainage layers and adjacent soils (excavated face or compacted materials) to prevent fines from migrating into the drainage layers.

Backfill should be placed in lifts not exceeding 8 inches in thickness, moisture conditioned to slightly above optimum moisture content, and mechanically compacted throughout to at least 90 percent of the maximum dry density as determined by Modified Proctor (ASTM D 1557). Retaining walls should be properly braced prior to placement and compaction of backfill should be performed with extreme care not to damage the walls.

7.7 New Pavement

The following recommendations for the new pavement are intended for vehicular traffic associated with the restaurant development within the subject property.

New Pavement Subgrades

Following completion of the recommended subgrade preparation procedures, the subgrade in areas of new pavement construction are expected to consist of medium expansive soil. The anticipated subgrade soils are classified as a poor subgrade material with estimated R-value of 5-10 when properly prepared based on the Unified Soil Classification System designation of CL/CH. An R-value of 5 has been assumed in the preparation of the pavement design. It should however, be recognized that the City of Livermore may require a specific R-value test to verify the use of the following design. It is recommended that this testing, if required, be conducted following completion of rough grading in the proposed pavement areas so that the R-value test results are indicative of the actual pavement subgrade soils. Alternatively, a minimum code pavement section may be required if a specific R-value test is not performed. To use this R-value, all fill added to the pavement subgrade must have pavement support characteristics at least equivalent to the existing soils, and must be placed and compacted in accordance with the project specifications.

Asphalt Pavements

The following table presents recommended thicknesses for a new flexible pavement structure consisting of asphaltic concrete over a granular base, along with the appropriate CALTRANS specifications for proper materials and placement procedures. An alternate pavement section has been provided for use in parking stall areas due to the anticipated lower traffic intensity in these areas.

However, care must be used so that truck traffic is excluded from areas where the thinner pavement section is used, since premature pavement distress may occur. In the event that heavy vehicle traffic cannot be excluded from the specific areas, the pavement section recommended for drive lanes should be used throughout the parking lot.

ASPHALT PAVEMENTS

Materials	**Thickness (inches)**		**CALTRANS Specifications**
	Parking Stalls (TI=4.0)	**Drive Lanes (TI=5.0)**	
Asphaltic Concrete Surface Course (b)	1	1	Section 39, (a)
Asphaltic Concrete Binder Course (b)	2	2	Section 39, (a)
Crushed Aggregate Base Course	7	10	Section 26, Class 2 (R-value at least 78)

NOTES:
(a) Compaction to density between 95 and 100 percent of the 50-Blow Marshall Density
(b) The surface and binder course may be combined as a single layer placed in one lift if similar materials are utilized.

Pavement recommendations are based upon CALTRANS design parameters for a twenty-year design period and assume proper drainage and construction monitoring. It is, therefore, recommended that the geotechnical engineer monitors and tests subgrade preparation, and that the subgrade be evaluated immediately before pavement construction.

Portland Concrete Pavements

Portland Cement Concrete pavements are recommended in areas where traffic is concentrated such as the entrance/exit aprons as well as areas subjected to heavy loads such as the trash enclosure loading zone. The preparation of the subgrade soils within concrete pavement areas should be performed as previously described in this report. Portland Cement Concrete pavements in high stress areas are recommended to be at least 6 inches thick containing No. 3 bars at 18-inch on-center both ways placed at mid-height. The pavement should be constructed in accordance with Section 40 of the CALTRANS Standard Specifications. A minimum 4-inch thick layer of base course (CALTRANS Class 2) is recommended below the concrete pavement. This base course should be compacted to at least 95% of the material's maximum dry density.

The maximum joint spacing within all of the Portland Cement Concrete pavements is recommended to be 15 feet or less to control shrinkage cracking. Load transfer reinforcing is recommended at construction joints perpendicular to traffic flow if construction joints are not properly keyed. In this event, ¾-inch diameter smooth dowel bars, 18 inches in length placed at 12 inches on-center are recommended where joints are perpendicular to the anticipated traffic flow. Expansion joints are recommended only where the pavement abuts fixed objects such as light standard foundations. Tie bars are recommended at the first joint within the perimeter of the concrete pavement area. Tie bars are recommended to be No. 4 bars at 42-inch on-center spacings and at least 48 inches in length.

General Considerations

Pavement recommendations assume proper drainage and construction monitoring and are based on traffic loads as indicated previously. Pavement designs are based on either PCA or CALTRANS design parameters for twenty (20) year design period. However, these designs are also based on a routine pavement maintenance program and significant asphalt concrete pavement rehabilitation after about 8 to 10 years, in order to obtain a reasonable pavement service life. Due to the presence of expansive soils, some increased pavement maintenance should be expected.

7.8 Recommended Construction Materials Testing Services

The report was prepared assuming that Giles will perform Construction Materials Testing (CMT) services during construction of the proposed development. In general, CMT services are recommended (and expected) to at least include observation and testing of: foundation and pavement support soil and other construction materials. It might be necessary for Giles to provide supplemental geotechnical recommendations based on the results of CMT services and specific details of the project not known at this time.

7.9 Basis of Report

This report is based on Giles' proposal, which is dated June 21, 2016 and is referenced by Giles' proposal number 2GEP-1606025. The actual services for the project varied somewhat from those described in the proposal because of the conditions that were encountered while performing the services and in consideration of the proposed project.

This report is strictly based on the project description given earlier in this report. Giles must be notified if any parts of the project description or our assumptions are not accurate so that this report can be amended, if needed. This report is based on the assumption that the facility will be designed and constructed according to the codes that govern construction at the site.

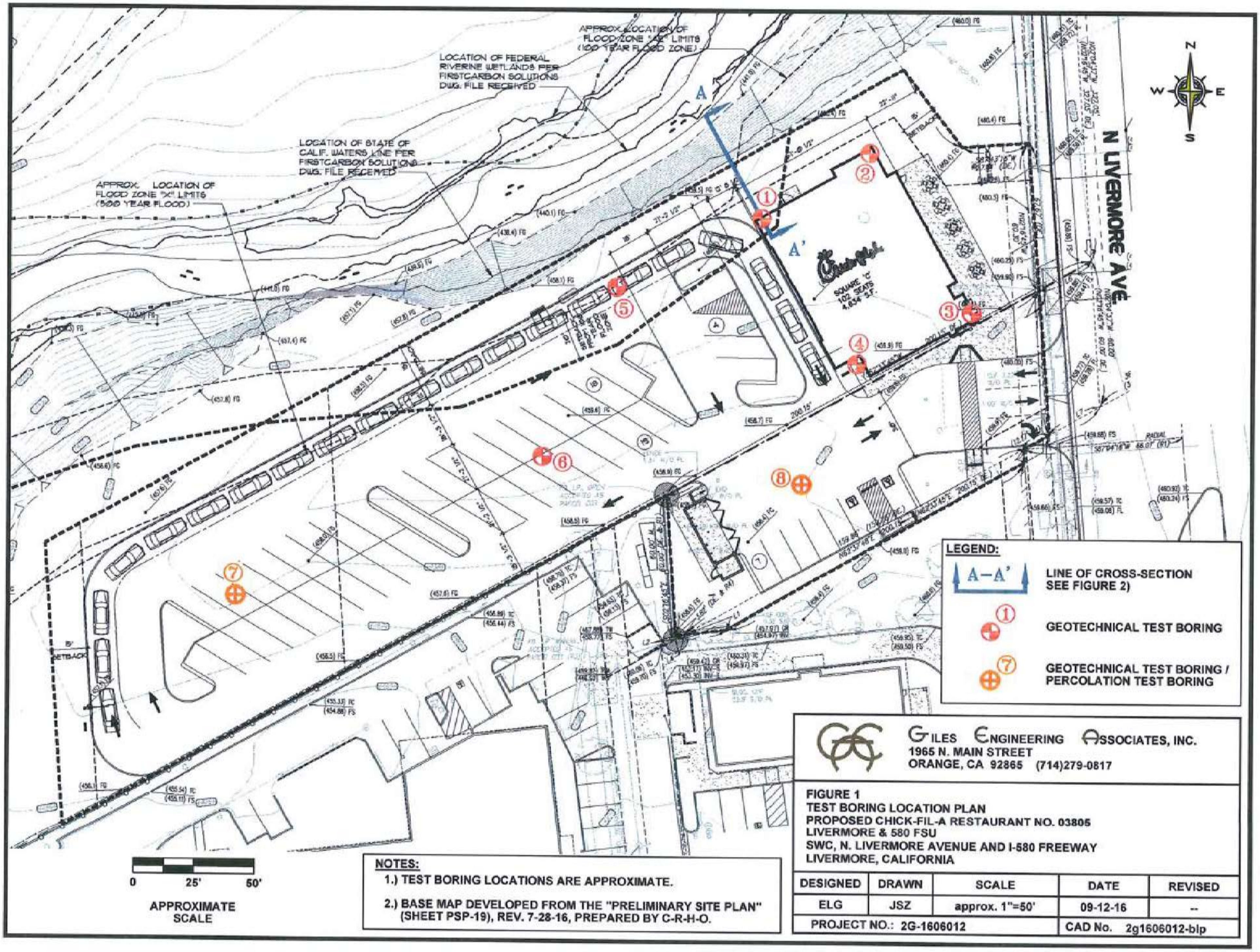
The conclusions and recommendations in this report are based on estimated subsurface conditions as shown on the *Records of Subsurface Exploration*. Giles must be notified if the subsurface conditions that are encountered during construction of the proposed development differ from those shown on the *Records of Subsurface Exploration* because this report will likely need to be revised. General comments and limitations of this report are given in the appendix.

APPENDIX A

FIGURES AND TEST BORING LOGS

The Test Boring Location Plan contained herein was prepared based upon information supplied by *Giles'* client, or others, along with *Giles'* field measurements and observations. The diagram is presented for conceptual purposes only and is intended to assist the reader in report interpretation.

The Test Boring Logs and related information enclosed herein depict the subsurface (soil and water) conditions encountered at the specific boring locations on the date that the exploration was performed. Subsurface conditions may differ between boring locations and within areas of the site that were not explored with test borings. The subsurface conditions may also change at the boring locations over the passage of time.



LEGEND:

A–A'	LINE OF CROSS-SECTION SEE FIGURE 2)
①	GEOTECHNICAL TEST BORING
⑦	GEOTECHNICAL TEST BORING / PERCOLATION TEST BORING

NOTES:

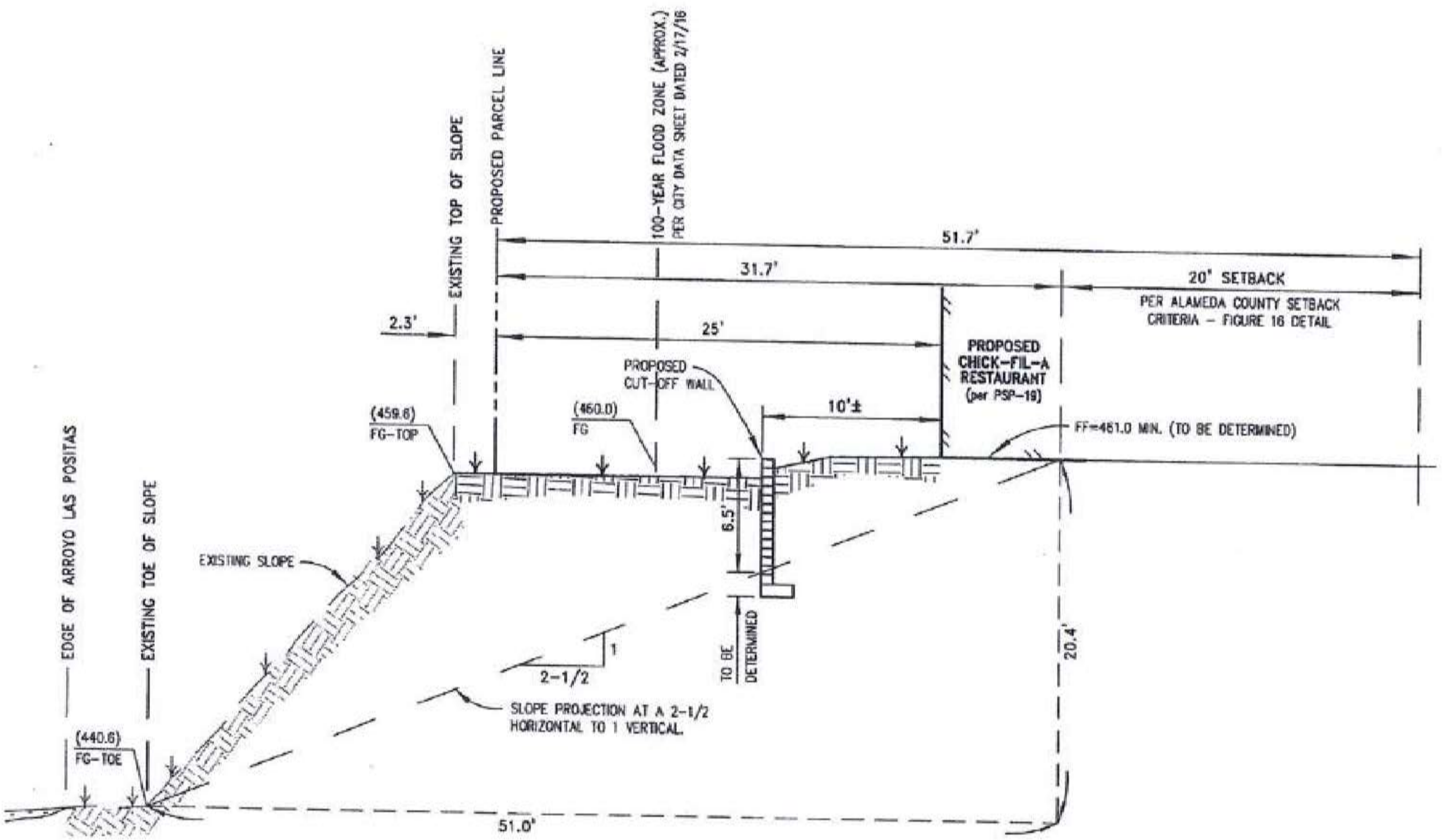
1.) TEST BORING LOCATIONS ARE APPROXIMATE.

2.) BASE MAP DEVELOPED FROM THE "PRELIMINARY SITE PLAN" (SHEET PSP-19), REV. 7-28-16, PREPARED BY C-R-H-O.

GILES ENGINEERING ASSOCIATES, INC.
1965 N. MAIN STREET
ORANGE, CA 92865 (714)279-0817

FIGURE 1
TEST BORING LOCATION PLAN
PROPOSED CHICK-FIL-A RESTAURANT NO. 03805
LIVERMORE & 580 FSU
SWC, N. LIVERMORE AVENUE AND I-580 FREEWAY
LIVERMORE, CALIFORNIA

DESIGNED	DRAWN	SCALE	DATE	REVISED
ELG	JSZ	approx. 1"=50'	09-12-16	--
PROJECT NO.: 2G-1606012			CAD No. 2g1606012-blp	



GILES ENGINEERING ASSOCIATES, INC.
1965 N. MAIN STREET
ORANGE, CA 92865 (714)279-0817

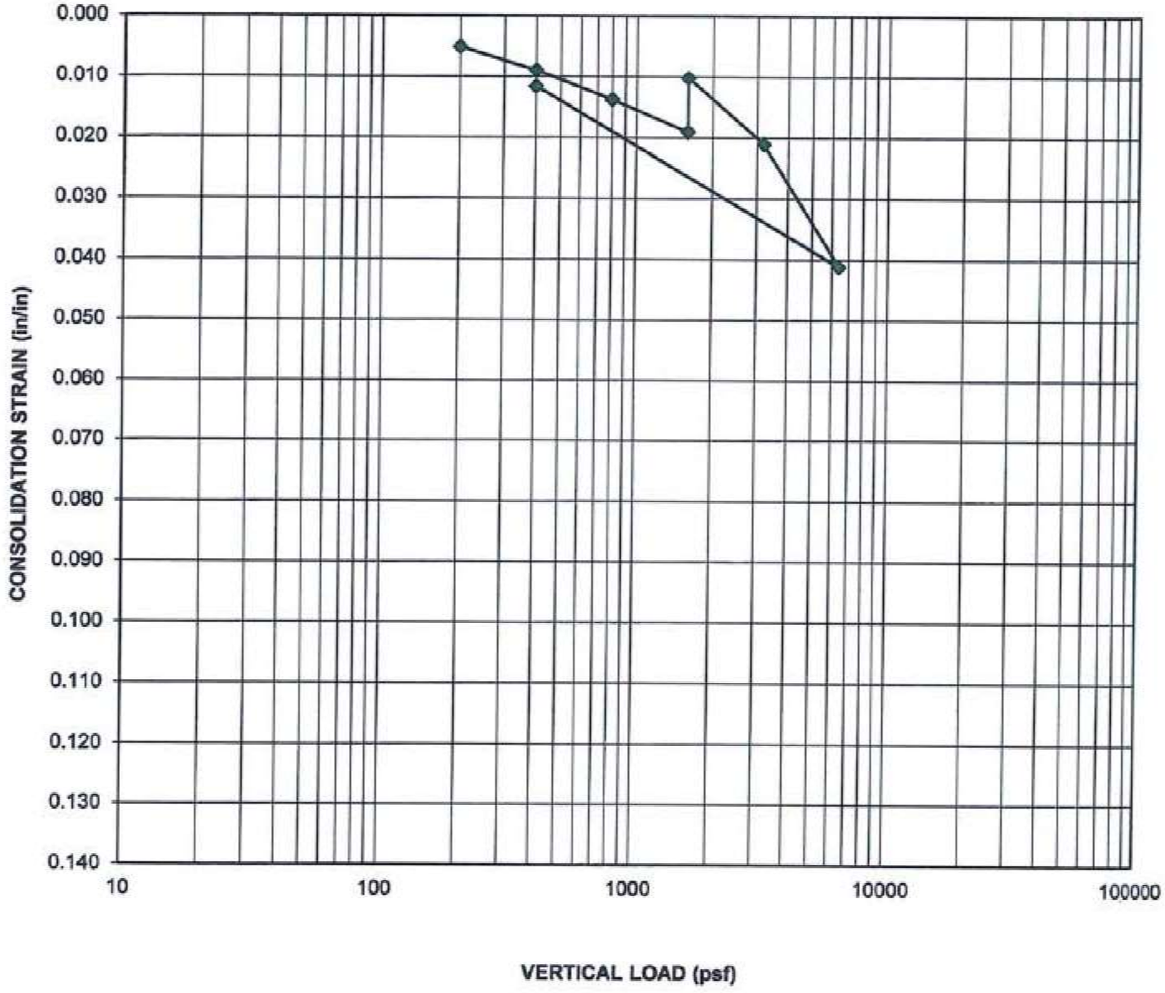
FIGURE 2
CROSS-SECTION A-A'
PROPOSED CHICK-FIL-A RESTAURANT NO. 03805
LIVERMORE & 580 FSU
SWC, N. LIVERMORE AVENUE AND I-580 FREEWAY
LIVERMORE, CALIFORNIA

DESIGNED	DRAWN	SCALE	DATE	REVISED
ELG	JSZ	approx. 1"=10'	09-12-16	--
PROJECT NO.: 2G-1606012			CAD No. 2g1606012-cs1	

NOTES:

1.) BASE MAP DEVELOPED FROM THE "SECTION A-A" PRINT DATE 8-26-16, PREPARED BY JOSEPH C. TRUXAW AND ASSOCIATES.

CONSOLIDATION / COLLAPSE TEST ASTM D2435/ASTM D5333



Classification	Sandy Clay
Boring No.	B-4
Sample No.	2-CS
Depth (ft.)	3.5 - 5.0
Elevation	
Liquid Limit	
Plastic Limit	
Specimen Diameter (in.)	2.42
Initial Specimen Thickness (in.)	1.00

Initial Moisture Content (%)	16.1
Final Moisture Content (%)	23.7
Natural Density (pcf)	121.0
Initial Dry Density (pcf)	104.2
Final Dry Density (pcf)	105.4
Swell at 1600 psf	0.89%

Sample inundated at 1600 psf pressure

Project:	CFA Livermore
Client:	Chick-fil-a Inc.
Project No.:	2G-1606012
Figure No.:	3

GILES ENGINEERING ASSOCIATES, INC.

-GEOTECHNICAL, ENVIRONMENTAL, AND CONSTRUCTION MATERIALS-
1965 NORTH MAIN STREET, ORANGE, CALIFORNIA
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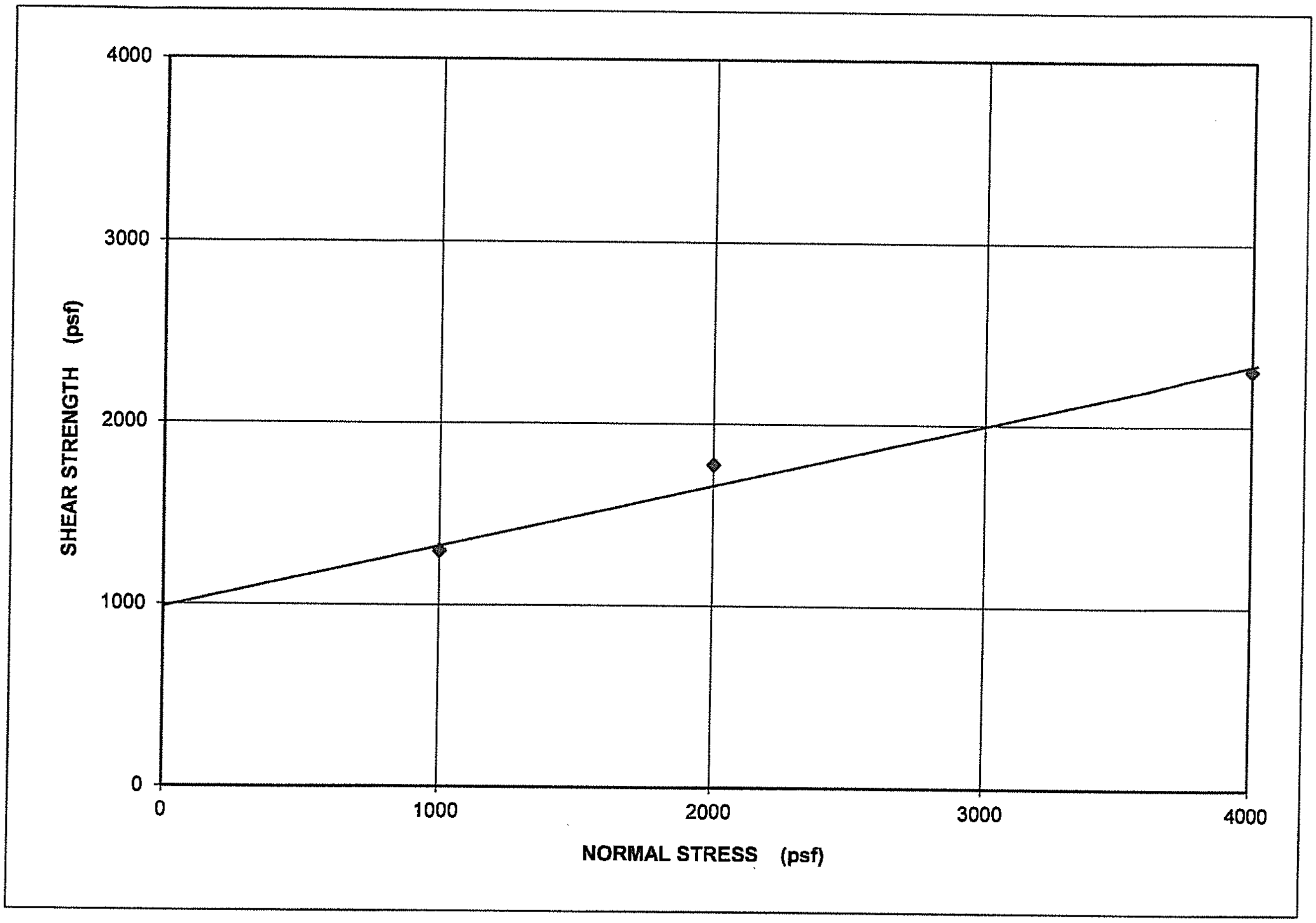
Giles Engineering Associates, Inc.

GEOTECHNICAL, ENVIRONMENTAL AND CONSTRUCTION MATERIALS CONSULTANTS

1965 N. MAIN STREET, ORANGE, CA 92865
OFFICE: 714-279-0817 FAX: 714-279-9687

UNDISTURBED DIRECT SHEAR TEST (ASTM D3080)



Classification Sandy Clay

Boring No. B-2 Depth 3.5 - 5.0 ft

		Initial Specimen Properties:	
		Diameter (in.)	2.4
		Height (in.)	1.0
PROJECT:	CFA#3805, Livermore	Moisture Content (%)	12.3
		Remolded Dry Density (pcf)	
CLIENT:	chick-fil-A	Dry Density (pcf)	111.3
		LL	
PROJECT NO.:	2G-1606012	PL	
		Ultimate - C (psi/psf)	1000
FIGURE NO.	4	Ultimate - PHI (degrees)	18

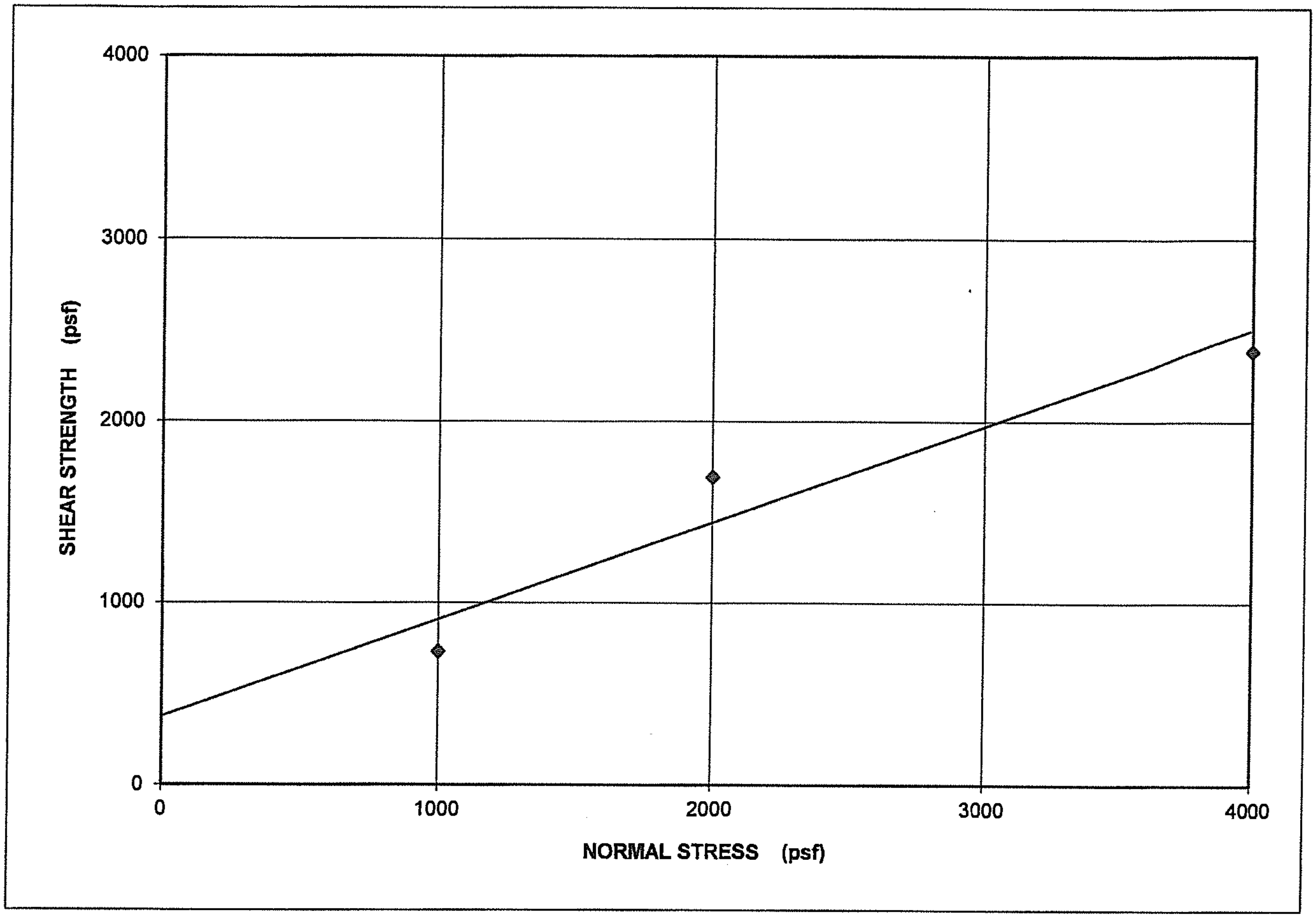
Giles Engineering Associates, Inc.

GEOTECHNICAL, ENVIRONMENTAL AND CONSTRUCTION MATERIALS CONSULTANTS

1965 N. MAIN STREET, ORANGE, CA 92865

OFFICE: 714-279-0817 FAX: 714-279-9687

UNDISTURBED DIRECT SHEAR TEST (ASTM D3080)



Classification Silty Sand

Boring No. B-4 Depth 10.0 - 11.5 feet

Initial Specimen Properties:

Diameter (in.) 2.4

Height (in.) 1.0

PROJECT: CFA#3805, Livermore

Moisture Content (%) 4.6

Remolded Dry Density (pcf)

CLIENT: chick-fil-A

Dry Density (pcf) 120.6

LL

PROJECT NO.: 2G-1606012

PL

Ultimate - C (psi/psf) 390

FIGURE NO. 5

Ultimate - PHI (degrees) 28

BORING NO. & LOCATION:
B-1

SURFACE ELEVATION:
459.5 feet

COMPLETION DATE:
08/17/16

FIELD REP:
JOSEPH HUYNH

TEST BORING LOG

PROPOSED CHICK-FIL-A RESTAURANT #3805

I-580 & LIVERMORE AVENUE
LIVERMORE, CA

PROJECT NO: 2G-1606012

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Light to Dark Sandy Clay to Clayey fine Sand - Moist			1 SS	15				18		LL=80 PL=30 PI=48
			2 SS	11				11		EI=54 (Medium)
Brown Silty fine to medium Sand to Light Gray Clayey Sand - Damp	10	450	3 SS	29				6		P_{200}=15%
			4 SS	30				6		
Bluish Gray Clayey fine to coarse Sand to Light Brown Clayey Sand to Sandy Clay, some Gravel, Possible Cobbles - Moist	20	440	5 SS	41				14		P_{200}=24%
			6 SS	74				12		
Bluish Gray Silty fine to coarse Sand to Clayey Sand, some Gravel, Possible Cobbles - Moist	▽ 30	430	7 SS	40/3"*				14		P_{200}=15%
			8 SS	50/2.5"*				15		
	40	420	9 SS	50/2"*				17		P_{200}=39%
			10 SS	50/6"*				9		
	50	410	11 SS	50/6"*				10		P_{200}=13%

Groundwater encountered at 30 feet
Boring Terminated at about 51.5 feet (EL. 408')

	Water Observation Data
▽	Water Encountered During Drilling: 30 ft.
▼	Water Level At End of Drilling:
░	Cave Depth At End of Drilling:
▼	Water Level After Drilling:
▒	Cave Depth After Drilling:

Remarks:

SS = Standard Penetration Test

*N-Value Possibly influenced by Possible Cobbles

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION: B-2	# TEST BORING LOG	**GILES ENGINEERING ASSOCIATES, INC.**
SURFACE ELEVATION: 460 feet	PROPOSED CHICK-FIL-A RESTAURANT #3805	
COMPLETION DATE: 08/17/16	I-580 & LIVERMORE AVENUE LIVERMORE, CA	
FIELD REP: JOSEPH HUYNH	PROJECT NO: 2G-1606012	

MATERIAL DESCRIPTION	**Depth (ft)**	**Elevation**	**Sample No. & Type**	**N**	**Q_u (tsf)**	**Q_p (tsf)**	**Q_s (tsf)**	**W (%)**	**PID**	**NOTES**
Light Brown to Gray Silty fine to medium Sand to Clayey Sand - Dry to Moist			1 SS	12				18		
Brown Silty fine to medium Sand, some Clay, some Gravel - Moist to Damp			2 CS	34				12		Dd=111.3 pcf
	5	455								
			3 CS	68				6		Dd=112.1 pcf
	10	450								
Light Brown fine to medium Sand, little Silt and Clay, some Gravel - Damp			4 CS	62				3		Dd=114.6 pcf P_{200}=9%
	15	445								
			5 SS	46				4		

No groundwater encountered
Boring Terminated at about 16.5 feet (EL. 443.5')

Water Observation Data	**Remarks:**
▽ Water Encountered During Drilling: None	CS = California Split Spoon
▼ Water Level At End of Drilling:	SS = Standard Penetration Test
░ Cave Depth At End of Drilling:	
▼ Water Level After Drilling:	
▒ Cave Depth After Drilling:	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION: B-3	# TEST BORING LOG	**GILES ENGINEERING ASSOCIATES, INC.**
SURFACE ELEVATION: 459.5 feet	PROPOSED CHICK-FIL-A RESTAURANT #3805	
COMPLETION DATE: 08/17/16	I-580 & LIVERMORE AVENUE LIVERMORE, CA	
FIELD REP: JOSEPH HUYNH	PROJECT NO: 2G-1606012	

MATERIAL DESCRIPTION	**Depth (ft)**	**Elevation**	**Sample No. & Type**	**N**	**Q_u (tsf)**	**Q_p (tsf)**	**Q_s (tsf)**	**W (%)**	**PID**	**NOTES**
Light Brown Silty to Clayey fine to medium Sand - Moist			1 SS	12				25		
	5	455	2 SS	12				16		
Yellowish Brown Silty fine Sand, some Clay - Moist			3 SS	17				10		
Brown Gray Silty fine Sand to fine Sand with some Gravel, Possible Cobbles - Damp	10	450	4 SS	60*				3		
	15	445	5 SS	50/6"*				6		

No groundwater encountered
Boring Terminated at about 16.5 feet (EL. 443')

	Water Observation Data	**Remarks:**
▽	Water Encountered During Drilling: None	SS = Standard Penetration Test
▼	Water Level At End of Drilling:	*N-Value Possibly influenced by Possible Cobbles
▨	Cave Depth At End of Drilling:	
▼	Water Level After Drilling:	
▨	Cave Depth After Drilling:	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION:
B-4

SURFACE ELEVATION:
460.2 feet

COMPLETION DATE:
08/17/16

FIELD REP:
JOSEPH HUYNH

TEST BORING LOG

PROPOSED CHICK-FIL-A RESTAURANT #3805

I-580 & LIVERMORE AVENUE
LIVERMORE, CA

PROJECT NO: 2G-1606012

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Dark Brown Silty fine Sand to Clayey fine Sand - Damp to Moist		460								
			1 SS	12				10		
Light Brown Sandy Clay to Clayey fine Sand - Moist			2 CS	35				16		Dd=104.2 pcf
	5	455								
Light Brown to Gray Silty fine Sand, some Clay - Moist			3 CS	49				8		Dd=113.0 pcf
Brown fine to medium Sand, little Silt, some Gravel, Possible Cobbles - Damp	10	450	4 CS	50/4"*				5		Dd=120.6 pcf P_{200}=9%
	15	445	5 SS	50/6"*				4		

No groundwater encountered
Boring Terminated at about 16.5 feet (EL. 443.7')

Water Observation Data

▽ Water Encountered During Drilling: None
▼ Water Level At End of Drilling:
░ Cave Depth At End of Drilling:
▼ Water Level After Drilling:
▒ Cave Depth After Drilling:

Remarks:

CS = California Split Spoon

SS = Standard Penetration Test

*N-Value Possibly influenced by Possible Cobbles

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION:
B-5

SURFACE ELEVATION:
459 feet

COMPLETION DATE:
08/17/16

FIELD REP:
JOSEPH HUYNH

TEST BORING LOG

PROPOSED CHICK-FIL-A RESTAURANT #3805

I-580 & LIVERMORE AVENUE
LIVERMORE, CA

PROJECT NO: 2G-1606012

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Light Gray to Black Clayey fine Sand to Sandy Clay - Moist		457.5	1 SS	8				17		
	2.5									
Light Brown Clayey fine Sand - Moist		455.0	2 SS	8				18		
	5.0									

No groundwater encountered
Boring Terminated at about 5 feet (EL. 454')

Water Observation Data

▽ Water Encountered During Drilling:
▼ Water Level At End of Drilling:
░ Cave Depth At End of Drilling:
▼ Water Level After Drilling:
▒ Cave Depth After Drilling:

Remarks:

SS = Standard Penetration Test

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION: B-6	
SURFACE ELEVATION: 459.5 feet	
COMPLETION DATE: 08/17/16	
FIELD REP: JOSEPH HUYNH	

TEST BORING LOG

PROPOSED CHICK-FIL-A RESTAURANT #3805

I-580 & LIVERMORE AVENUE
LIVERMORE, CA

PROJECT NO: 2G-1606012

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION		Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Black Silty Clay, some fine Sand - Dry to Moist	▨										
	▨		457.5	1 SS	10				16		
	▨	2.5									
Light Brown fine Sandy Clay - Very Moist	▨										
	▨		455.0	2 SS	11				30		
		5.0									

No groundwater encountered
Boring Terminated at about 5 feet (EL. 454.5')

	Water Observation Data
▽	Water Encountered During Drilling: None
▼	Water Level At End of Drilling:
▒	Cave Depth At End of Drilling:
▼	Water Level After Drilling:
▒	Cave Depth After Drilling:

Remarks:

SS = Standard Penetration Test

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION:
B-7

SURFACE ELEVATION:
459.2 feet

COMPLETION DATE:
08/17/16

FIELD REP:
JOSEPH HUYNH

TEST BORING LOG

PROPOSED CHICK-FIL-A RESTAURANT #3805

I-580 & LIVERMORE AVENUE
LIVERMORE, CA

PROJECT NO: 2G-1606012

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION		Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Dark Brown fine Sandy Clay to Clayey fine Sand, some Gravel - Dry to Moist	▨		457.5	1 SS	11				10		
	▨	2.5									
Dark Brown fine Sandy Clay to Silty Clay - Very Moist	▨		455.0	2 SS	9				37		
		5.0									

No groundwater encountered
Boring Terminated at about 5 feet (EL. 454.2')

	Water Observation Data
▽	Water Encountered During Drilling: None
▼	Water Level At End of Drilling:
░	Cave Depth At End of Drilling:
▼	Water Level After Drilling:
▒	Cave Depth After Drilling:

Remarks:

SS = Standard Penetration Test

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION:
B-8

SURFACE ELEVATION:
459.8 feet

COMPLETION DATE:
08/17/16

FIELD REP:
JOSEPH HUYNH

TEST BORING LOG

PROPOSED CHICK-FIL-A RESTAURANT #3805

I-580 & LIVERMORE AVENUE
LIVERMORE, CA

PROJECT NO: 2G-1606012

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION		**Depth (ft)**	**Elevation**	**Sample No. & Type**	**N**	**Q_u (tsf)**	**Q_p (tsf)**	**Q_s (tsf)**	**W (%)**	**PID**	**NOTES**
Dark Gray Clayey fine Sand to fine Sandy Clay - Moist to Very Moist	▨			1 SS	9				23		
		2.5	457.5								
Light Yellowish Brown Clayey fine Sand, some pockets of Sandy Clay - Moist	▨			2 SS	14				13		
		5.0	455.0								

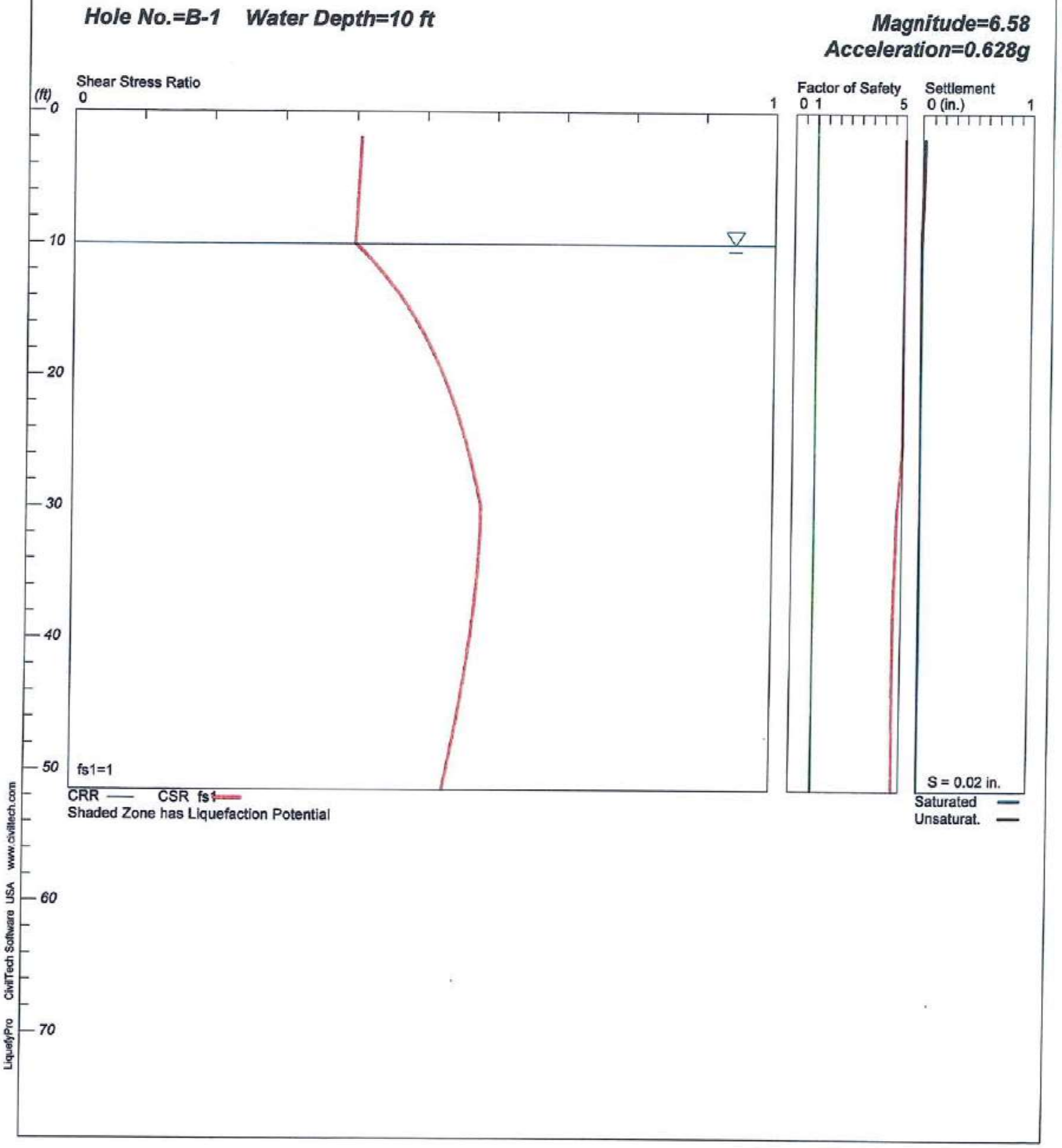
No groundwater encountered
Boring Terminated at about 5 feet (EL. 454.8')

	Water Observation Data	**Remarks:**
▽	Water Encountered During Drilling: None	SS = Standard Penetration Test
▼	Water Level At End of Drilling:	
░	Cave Depth At End of Drilling:	
▼	Water Level After Drilling:	
▒	Cave Depth After Drilling:	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

LIQUEFACTION ANALYSIS

CFA #3805, Livermore, CA



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************************************************************************************
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                                          LIQUEFACTION ANALYSIS SUMMARY

                                         Copyright by CivilTech Software
                                            www.civiltechsoftware.com

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        Font: Courier New, Regular, Size 8 is recommended for this report.
        Licensed to ,   9/1/2016        8:51:11 AM

        Input File Name: P:\Edgar Gatus\2G-1606012, CFA #3805, Livermore, CA\B-1.liq
        Title:  CFA #3805, Livermore, CA
        Subtitle:  2G-1606012

        Surface Elev.=
        Hole No.=B-1
        Depth of Hole= 51.50 ft
        Water Table during Earthquake= 10.00 ft
        Water Table during In-Situ Testing= 302.00 ft
        Max. Acceleration= 0.63 g
        Earthquake Magnitude= 6.58

Input Data:
        Surface Elev.=
        Hole No.=B-1
        Depth of Hole=51.50 ft
        Water Table during Earthquake= 10.00 ft
        Water Table during In-Situ Testing= 302.00 ft
        Max. Acceleration=0.63 g
        Earthquake Magnitude=6.58
        No-Liquefiable Soils:   CL, OL are Non-Liq. Soil

        1. SPT or BPT Calculation.
        2. Settlement Analysis Method: Tokimatsu/Seed
        3. Fines Correction for Liquefaction: Idriss/Seed
        4. Fine Correction for Settlement: During Liquefaction*
        5. Settlement Calculation in: All zones*
        6. Hammer Energy Ratio,                                   Ce = 1.25
        7. Borehole Diameter,                                         Cb= 1
        8. Sampling Method,                                          Cs= 1.2
        9. User request factor of safety (apply to CSR) ,   User= 1.1
           Plot one CSR curve (fs1=1)
        10. Use Curve Smoothing: Yes*
        * Recommended Options

        In-Situ Test Data:
        Depth   SPT     gamma   Fines
        ft              pcf     %
        ______________________________________
        2.00    15.00   120.00  80.00
        5.00    11.00   120.00  80.00
        10.00   29.00   120.00  15.00
        15.00   30.00   120.00  15.00
        20.00   41.00   120.00  24.00
        25.00   74.00   120.00  24.00
        30.00   40.00   120.00  15.00
        35.00   50.00   120.00  15.00
        40.00   50.00   120.00  39.00
        45.00   50.00   120.00  39.00
```

```
    50.00    50.00    120.00  13.00
    ____________________________________________

Output Results:
    Settlement of Saturated Sands=0.00 in.
    Settlement of Unsaturated Sands=0.02 in.
    Total Settlement of Saturated and Unsaturated Sands=0.02 in.
    Differential Settlement=0.010 to 0.014 in.

    Depth    CRRm     CSRfs    F.S.     S_sat.   S_dry    S_all
    ft                                  in.      in.      in.
    _______________________________________________________________
    2.00     2.79     0.41     5.00     0.00     0.02     0.02
    2.50     2.79     0.41     5.00     0.00     0.02     0.02
    3.00     2.79     0.41     5.00     0.00     0.02     0.02
    3.50     2.79     0.40     5.00     0.00     0.02     0.02
    4.00     2.79     0.40     5.00     0.00     0.02     0.02
    4.50     2.79     0.40     5.00     0.00     0.01     0.01
    5.00     2.79     0.40     5.00     0.00     0.01     0.01
    5.50     2.79     0.40     5.00     0.00     0.01     0.01
    6.00     2.79     0.40     5.00     0.00     0.01     0.01
    6.50     2.79     0.40     5.00     0.00     0.01     0.01
    7.00     2.79     0.40     5.00     0.00     0.01     0.01
    7.50     2.79     0.40     5.00     0.00     0.01     0.01
    8.00     2.79     0.40     5.00     0.00     0.00     0.00
    8.50     2.79     0.40     5.00     0.00     0.00     0.00
    9.00     2.79     0.40     5.00     0.00     0.00     0.00
    9.50     2.79     0.40     5.00     0.00     0.00     0.00
    10.00    2.79     0.40     5.00     0.00     0.00     0.00
    10.50    2.79     0.41     5.00     0.00     0.00     0.00
    11.00    2.79     0.42     5.00     0.00     0.00     0.00
    11.50    2.79     0.43     5.00     0.00     0.00     0.00
    12.00    2.79     0.43     5.00     0.00     0.00     0.00
    12.50    2.79     0.44     5.00     0.00     0.00     0.00
    13.00    2.79     0.45     5.00     0.00     0.00     0.00
    13.50    2.79     0.46     5.00     0.00     0.00     0.00
    14.00    2.79     0.46     5.00     0.00     0.00     0.00
    14.50    2.79     0.47     5.00     0.00     0.00     0.00
    15.00    2.79     0.48     5.00     0.00     0.00     0.00
    15.50    2.79     0.48     5.00     0.00     0.00     0.00
    16.00    2.79     0.49     5.00     0.00     0.00     0.00
    16.50    2.79     0.49     5.00     0.00     0.00     0.00
    17.00    2.79     0.50     5.00     0.00     0.00     0.00
    17.50    2.79     0.50     5.00     0.00     0.00     0.00
    18.00    2.79     0.51     5.00     0.00     0.00     0.00
    18.50    2.79     0.51     5.00     0.00     0.00     0.00
    19.00    2.79     0.52     5.00     0.00     0.00     0.00
    19.50    2.79     0.52     5.00     0.00     0.00     0.00
    20.00    2.79     0.53     5.00     0.00     0.00     0.00
    20.50    2.79     0.53     5.00     0.00     0.00     0.00
    21.00    2.79     0.53     5.00     0.00     0.00     0.00
    21.50    2.79     0.54     5.00     0.00     0.00     0.00
    22.00    2.79     0.54     5.00     0.00     0.00     0.00
    22.50    2.79     0.54     5.00     0.00     0.00     0.00
    23.00    2.79     0.55     5.00     0.00     0.00     0.00
    23.50    2.79     0.55     5.00     0.00     0.00     0.00
    24.00    2.79     0.55     5.00     0.00     0.00     0.00
    24.50    2.79     0.56     5.00     0.00     0.00     0.00
    25.00    2.79     0.56     5.00     0.00     0.00     0.00
    25.50    2.79     0.56     4.98     0.00     0.00     0.00
    26.00    2.79     0.56     4.96     0.00     0.00     0.00
    26.50    2.79     0.57     4.94     0.00     0.00     0.00
    27.00    2.79     0.57     4.91     0.00     0.00     0.00
```

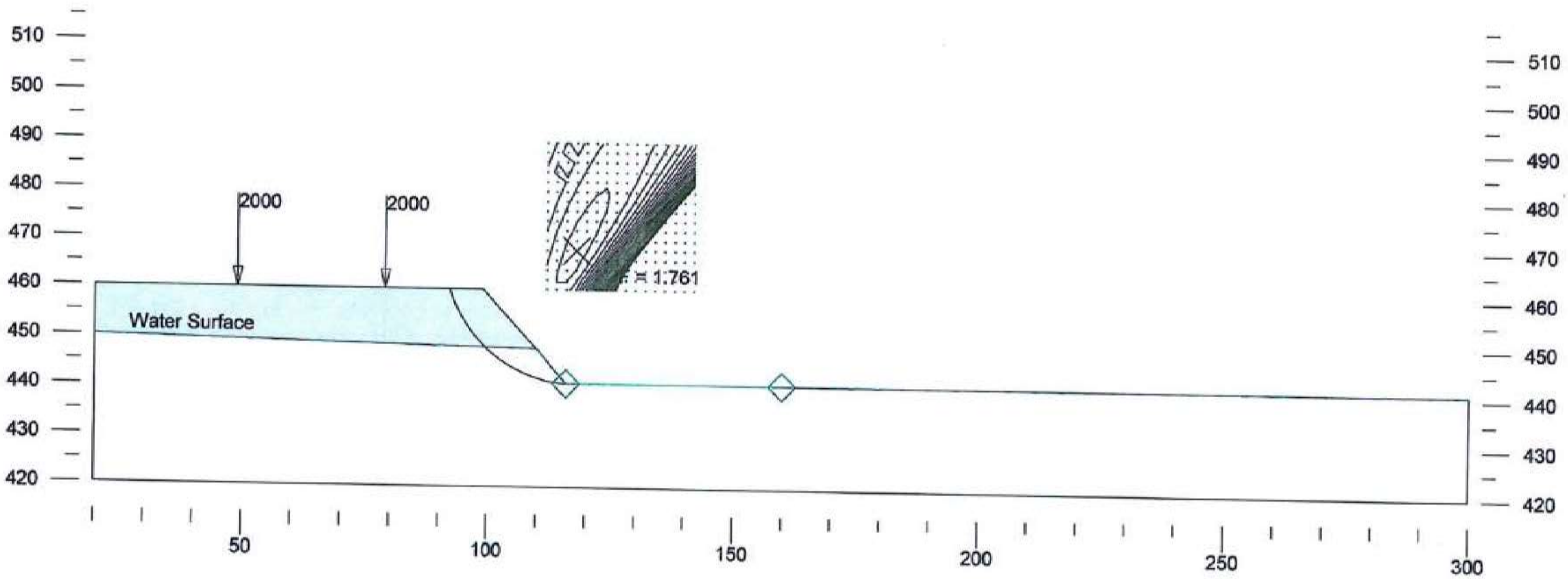
```
   27.50     2.81     0.57     4.92     0.00     0.00     0.00
   28.00     2.80     0.57     4.88     0.00     0.00     0.00
   28.50     2.79     0.58     4.85     0.00     0.00     0.00
   29.00     2.78     0.58     4.82     0.00     0.00     0.00
   29.50     2.77     0.58     4.79     0.00     0.00     0.00
   30.00     2.77     0.58     4.76     0.00     0.00     0.00
   30.50     2.76     0.58     4.75     0.00     0.00     0.00
   31.00     2.75     0.58     4.73     0.00     0.00     0.00
   31.50     2.74     0.58     4.72     0.00     0.00     0.00
   32.00     2.73     0.58     4.71     0.00     0.00     0.00
   32.50     2.73     0.58     4.70     0.00     0.00     0.00
   33.00     2.72     0.58     4.69     0.00     0.00     0.00
   33.50     2.71     0.58     4.68     0.00     0.00     0.00
   34.00     2.70     0.58     4.67     0.00     0.00     0.00
   34.50     2.70     0.58     4.66     0.00     0.00     0.00
   35.00     2.69     0.58     4.66     0.00     0.00     0.00
   35.50     2.68     0.58     4.65     0.00     0.00     0.00
   36.00     2.67     0.58     4.64     0.00     0.00     0.00
   36.50     2.67     0.58     4.64     0.00     0.00     0.00
   37.00     2.66     0.57     4.63     0.00     0.00     0.00
   37.50     2.65     0.57     4.63     0.00     0.00     0.00
   38.00     2.65     0.57     4.62     0.00     0.00     0.00
   38.50     2.64     0.57     4.62     0.00     0.00     0.00
   39.00     2.63     0.57     4.61     0.00     0.00     0.00
   39.50     2.62     0.57     4.61     0.00     0.00     0.00
   40.00     2.62     0.57     4.61     0.00     0.00     0.00
   40.50     2.61     0.57     4.61     0.00     0.00     0.00
   41.00     2.60     0.57     4.60     0.00     0.00     0.00
   41.50     2.60     0.56     4.60     0.00     0.00     0.00
   42.00     2.59     0.56     4.60     0.00     0.00     0.00
   42.50     2.58     0.56     4.60     0.00     0.00     0.00
   43.00     2.58     0.56     4.60     0.00     0.00     0.00
   43.50     2.57     0.56     4.60     0.00     0.00     0.00
   44.00     2.56     0.56     4.60     0.00     0.00     0.00
   44.50     2.56     0.56     4.60     0.00     0.00     0.00
   45.00     2.55     0.55     4.60     0.00     0.00     0.00
   45.50     2.54     0.55     4.61     0.00     0.00     0.00
   46.00     2.54     0.55     4.61     0.00     0.00     0.00
   46.50     2.53     0.55     4.61     0.00     0.00     0.00
   47.00     2.52     0.55     4.61     0.00     0.00     0.00
   47.50     2.52     0.55     4.62     0.00     0.00     0.00
   48.00     2.51     0.54     4.62     0.00     0.00     0.00
   48.50     2.51     0.54     4.62     0.00     0.00     0.00
   49.00     2.50     0.54     4.63     0.00     0.00     0.00
   49.50     2.49     0.54     4.63     0.00     0.00     0.00
   50.00     2.49     0.54     4.64     0.00     0.00     0.00
   50.50     2.48     0.53     4.64     0.00     0.00     0.00
   51.00     2.47     0.53     4.65     0.00     0.00     0.00
   51.50     2.47     0.53     4.65     0.00     0.00     0.00
-------------------------------------------------------------------
   * F.S.<1, Liquefaction Potential Zone
   (F.S. is limited to 5,  CRR is limited to 2,    CSR is limited to 2)

        Units:  Unit: qc, fs, Stress or Pressure = atm (1.0581tsf); Unit Weight =
pcf; Depth = ft; Settlement = in.

-------------------------------------------------------------------------------------
        1 atm (atmosphere) = 1 tsf (ton/ft2)
        CRRm                Cyclic resistance ratio from soils
        CSRsf               Cyclic stress ratio induced by a given earthquake (with user
request factor of safety)
        F.S.                Factor of Safety against liquefaction, F.S.=CRRm/CSRsf
        S_sat               Settlement from saturated sands
```

S_dry	Settlement from Unsaturated Sands
S_all	Total Settlement from Saturated and Unsaturated Sands
NoLiq	No-Liquefy Soils

	Gamma	C	Phi	Piezo
	pcf	psf	deg	Surf.
Sandy Clay	125	500	18	0
Silty/Clayey Sand	130	300	28	0

Giles Engineering Associates Inc.

CFA Livermore-Preliminary Run
8/23/16



9/8/2016 2:42:37 PM P:\Bob Russell\Slope Info\CFA Livermore6.gsl Giles Engineering Associates Inc. F = 1.761

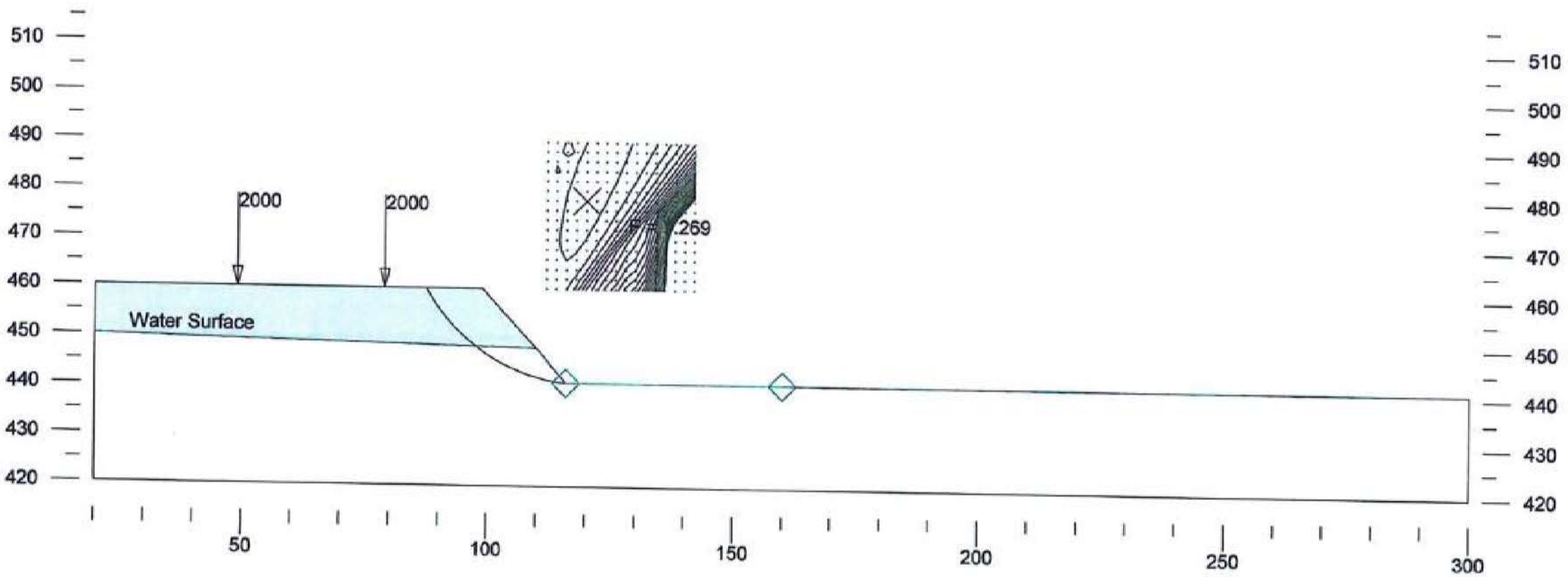
```
P:\Bob Russell\Slope Info\CFA Livermore6.gsl  09-08-2016 at 14:42:26


Minimum Bishop Factor of Safety this run:

    118.00     468.00     27.07     5      25        1.7606     0

Lowest results with no MAlpha value less than 0.3:          Min MAlpha

    118.00     468.00     27.07     5      25        1.7606   .5185
    120.00     472.00     31.26     5      26        1.7659   .5855
    118.00     470.00     29.07     5      28        1.7663   .5515
    120.00     470.00     29.27     5      25        1.7695   .5504
    116.00     464.00     23.00     5      25        1.7699   .4307
Lowest results with one MAlpha value less than 0.3:         Min MAlpha

    118.00     460.00     19.10     6      19        2.0083   .2141
    114.00     460.00     21.85     5      36        2.0426   .2966
    120.00     462.00     21.38     6      19        2.1165   .2925
    112.00     460.00     22.91     5      39        2.1380   .2821
    118.00     460.00     20.18     5      30        2.2306   .2656
Lowest results with two MAlpha values less than 0.3:        Min MAlpha

    130.00     464.00     31.28     4      30        5.1497   .1933
```

	Gamma pcf	C psf	Phi deg	Piezo Surf.
Sandy Clay	125	500	18	0
Silty/Clayey Sand	130	300	28	0

Seismic coefficient = 0.25

Giles Engineering Associates Inc.

CFA Livermore-Preliminary Run
8/23/16



9/8/2016 2:54:06 PM P:\Bob Russell\Slope Info\CFA Livermore6.gsl Giles Engineering Associates Inc. F = 1.269

```
P:\Bob Russell\Slope Info\CFA Livermore6.gsl  09-08-2016 at 14:43:47


Minimum Bishop Factor of Safety this run:

    120.00    478.00    37.22    3    31    1.2689    0

Lowest results with no MAlpha value less than 0.3:        Min MAlpha

    120.00    478.00    37.22    3    31    1.2689   .7259
    120.00    476.00    35.23    3    29    1.2700   .7039
    122.00    482.00    41.44    3    32    1.2703   .7618
    120.00    480.00    39.20    3    32    1.2704   .7468
    122.00    480.00    39.46    3    30    1.2709   .7439
Lowest results with one MAlpha value less than 0.3:       Min MAlpha

    118.00    460.00    19.10    5    19    1.5414   .2631
    116.00    460.00    32.53    4    33    1.9676   .2904
    124.00    460.00    26.45    4    48    2.1456   .2677
    116.00    462.00    40.99    3    41    2.1501   .2911
    124.00    460.00    33.18    4    34    2.1726   .2742
Lowest results with two MAlpha values less than 0.3:      Min MAlpha

    114.00    460.00    41.77    3    42    2.3098   .2746
    118.00    462.00    46.96    3    48    2.3794   .2890
    112.00    460.00    43.56    4    44    2.3867   .2638
    122.00    460.00    42.49    4    43    2.4188   .2680
    120.00    460.00    44.28    4    45    2.4267   .2611
```

APPENDIX B

FIELD PROCEDURES

The field operations were conducted in general accordance with the procedures recommended by the American Society for Testing and Materials (ASTM) designation D

420 entitled "Standard Guide for Sampling Rock and Rock" and/or other relevant specifications. Soil samples were preserved and transported to *Giles'* laboratory in general accordance with the procedures recommended by ASTM designation D 4220 entitled "Standard Practice for Preserving and Transporting Soil Samples." Brief descriptions of the sampling, testing and field procedures commonly performed by *Giles* are provided herein.

GENERAL FIELD PROCEDURES

Test Boring Elevations

The ground surface elevations reported on the Test Boring Logs are referenced to the assumed benchmark shown on the Boring Location Plan (Figure 1). Unless otherwise noted, the elevations were determined with a conventional hand-level and are accurate to within about 1 foot.

Test Boring Locations

The test borings were located on-site based on the existing site features and/or apparent property lines. Dimensions illustrating the approximate boring locations are reported on the Boring Location Plan (Figure 1).

Water Level Measurement

The water levels reported on the Test Boring Logs represent the depth of "free" water encountered during drilling and/or after the drilling tools were removed from the borehole. Water levels measured within a granular (sand and gravel) soil profile are typically indicative of the water table elevation. It is usually not possible to accurately identify the water table elevation with cohesive (clayey) soils, since the rate of seepage is slow. The water table elevation within cohesive soils must therefore be determined over a period of time with groundwater observation wells.

It must be recognized that the water table may fluctuate seasonally and during periods of heavy precipitation. Depending on the subsurface conditions, water may also become perched above the water table, especially during wet periods.

Borehole Backfilling Procedures

Each borehole was backfilled upon completion of the field operations. If potential contamination was encountered, and/or if required by state or local regulations, boreholes were backfilled with an "impervious" material (such as bentonite slurry). Borings that penetrated pavements, sidewalks, etc. were "capped" with Portland Cement concrete, asphaltic concrete, or a similar surface material. It must, however, be recognized that the backfill material may settle, and the surface cap may subside, over a period of time. Further backfilling and/or re-surfacing by *Giles'* client or the property owner may be required.

FIELD SAMPLING AND TESTING PROCEDURES

Auger Sampling (AU)

Soil samples are removed from the auger flights as an auger is withdrawn above the ground surface. Such samples are used to determine general soil types and identify approximate soil stratifications. Auger samples are highly disturbed and are therefore not typically used for geotechnical strength testing.

Split-Barrel Sampling (SS) – (ASTM D-1586)

A split-barrel sampler with a 2-inch outside diameter is driven into the subsoil with a 140-pound hammer free-falling a vertical distance of 30 inches. The summation of hammer-blows required to drive the sampler the final 12-inches of an 18-inch sample interval is defined as the "Standard Penetration Resistance" or N-value is an index of the relative density of granular soils and the comparative consistency of cohesive soils. A soil sample is collected from each SPT interval.

Shelby Tube Sampling (ST) – (ASTM D-1587)

A relatively undisturbed soil sample is collected by hydraulically advancing a thin-walled Shelby Tube sampler into a soil mass. Shelby Tubes have a sharp cutting edge and are commonly 2 to 5 inches in diameter.

Bulk Sample (BS)

A relatively large volume of soils is collected with a shovel or other manually-operated tool. The sample is typically transported to *Giles'* materials laboratory in a sealed bag or bucket.

Dynamic Cone Penetration Test (DC) – (ASTM STP 399)

This test is conducted by driving a 1.5-inch-diameter cone into the subsoil using a 15-pound steel ring (hammer), free-falling a vertical distance of 20 inches. The number of hammer-blows required to drive the cone 1¾ inches is an indication of the soil strength and density, and is defined as "N". The Dynamic Cone Penetration test is commonly conducted in hand auger borings, test pits and within excavated trenches.

- Continued -

Ring-Lined Barrel Sampling – (ASTM D 3550)

In this procedure, a ring-lined barrel sampler is used to collect soil samples for classification and laboratory testing. This method provides samples that fit directly into laboratory test instruments without additional handling/disturbance.

Sampling and Testing Procedures

The field testing and sampling operations were conducted in general accordance with the procedures recommended by the American Society for Testing and Materials (ASTM) and/or other relevant specifications. Results of the field testing (i.e. N-values) are reported on the Test Boring Logs. Explanations of the terms and symbols shown on the logs are provided on the appendix enclosure entitled “General Notes”.

APPENDIX C

LABORATORY TESTING AND CLASSIFICATION

The laboratory testing was conducted under the supervision of a geotechnical engineer in accordance with the procedures recommended by the American Society for Testing and Materials (ASTM) and/or other relevant specifications. Brief descriptions of laboratory tests commonly performed by *Giles* are provided herein.

LABORATORY TESTING AND CLASSIFICATION

Photoionization Detector (PID)

In this procedure, soil samples are "scanned" in *Giles'* analytical laboratory using a Photoionization Detector (PID). The instrument is equipped with an 11.7 eV lamp calibrated to a Benzene Standard and is capable of detecting a minute concentration of **certain** Volatile Organic Compound (VOC) vapors, such as those commonly associated with petroleum products and some solvents. Results of the PID analysis are expressed in HNu (manufacturer's) units rather than actual concentration.

Moisture Content (w) (ASTM D 2216)

Moisture content is defined as the ratio of the weight of water contained within a soil sample to the weight of the dry solids within the sample. Moisture content is expressed as a percentage.

Unconfined Compressive Strength (qu) (ASTM D 2166)

An axial load is applied at a uniform rate to a cylindrical soil sample. The unconfined compressive strength is the maximum stress obtained or the stress when 15% axial strain is reached, whichever occurs first.

Calibrated Penetrometer Resistance (qp)

The small, cylindrical tip of a hand-held penetrometer is pressed into a soil sample to a prescribed depth to measure the soils capacity to resist penetration. This test is used to evaluate unconfined compressive strength.

Vane-Shear Strength (qs)

The blades of a vane are inserted into the flat surface of a soil sample and the vane is rotated until failure occurs. The maximum shear resistance measured immediately prior to failure is taken as the vane-shear strength.

Loss-on-Ignition (ASTM D 2974; Method C)

The Loss-on-Ignition (L.O.I.) test is used to determine the organic content of a soil sample. The procedure is conducted by heating a dry soil sample to 440°C in order to burn-off or "ash" organic matter present within the sample. The L.O.I. value is the ratio of the weight loss due to ignition compared to the initial weight of the dry sample. L.O.I. is expressed as a percentage.

Particle Size Distribution (ASTB D 421, D 422, and D 1140)

This test is performed to determine the distribution of specific particle sizes (diameters) within a soil sample. The distribution of coarse-grained soil particles (sand and gravel) is determined from a “sieve analysis,” which is conducted by passing the sample through a series of nested sieves. The distribution of fine-grained soil particles (silt and clay) is determined from a “hydrometer analysis” which is based on the sedimentation of particles suspended in water.

Consolidation Test (ASTM D 2435)

In this procedure, a series of cumulative vertical loads are applied to a small, laterally confined soil sample. During each load increment, vertical compression (consolidation) of the sample is measured over a period of time. Results of this test are used to estimate settlement and time rate of settlement.

Classification of Samples

Each soil sample was visually-manually classified, based on texture and plasticity, in general accordance with the Unified Soil Classification System (ASTM D-2488-75). The classifications are reported on the Test Boring Logs.

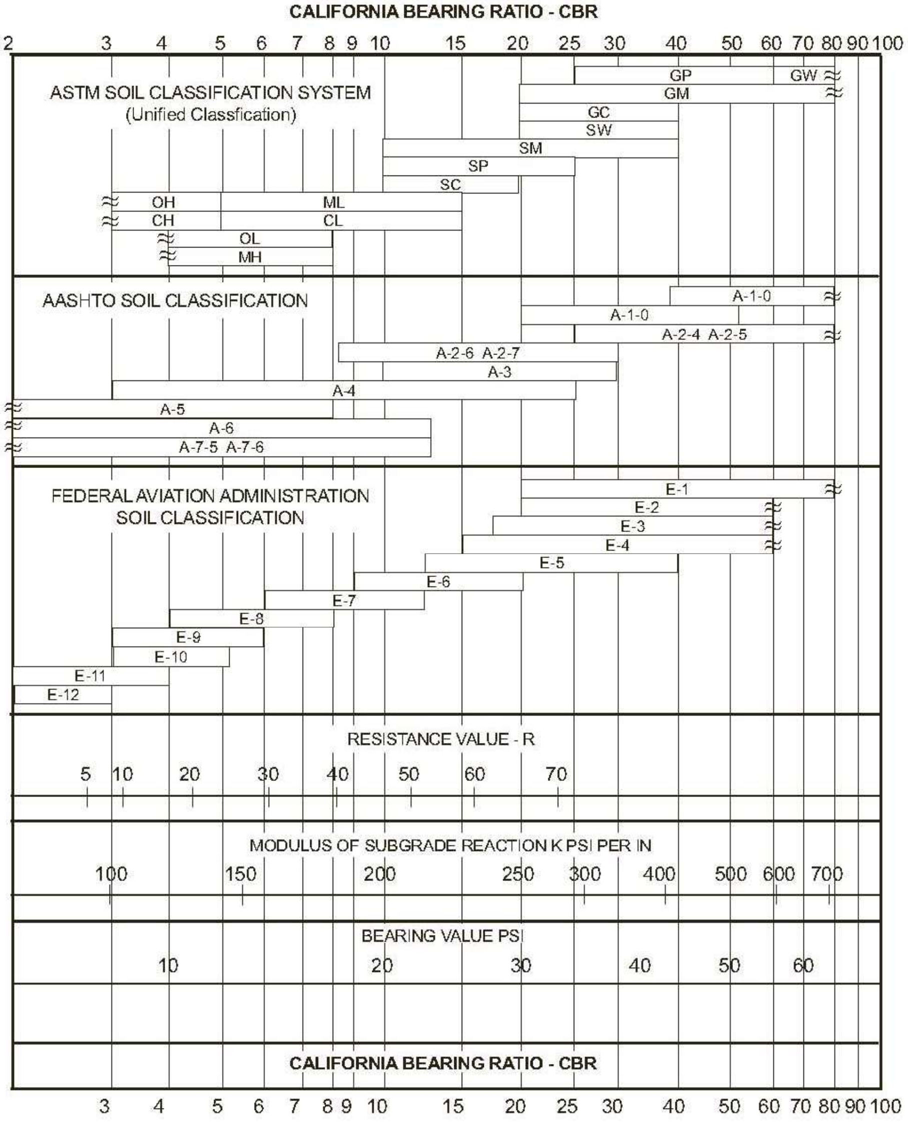
Laboratory Testing

The laboratory testing operations were conducted in general accordance with the procedures recommended by the American Society for Testing and Materials (ASTM) and/or other relevant specifications. Results of the laboratory tests are provided on the Test Boring Logs or other appendix enclosures. Explanation of the terms and symbols used on the logs is provided on the appendix enclosure entitled “General Notes.”

California Bearing Ratio (CBR) Test ASTM D-1833

The CBR test is used for evaluation of a soil subgrade for pavement design. The test consists of measuring the force required for a 3-square-inch cylindrical piston to penetrate 0.1 or 0.2 inch into a compacted soil sample. The result is expressed as a percent of force required to penetrate a standard compacted crushed stone.

Unless a CBR test has been specifically requested by the client, the CBR is estimated from published charts, based on soil classification and strength characteristics. A typical correlation chart is below.



APPENDIX D

GENERAL INFORMATION

GUIDE SPECIFICATIONS FOR SUBGRADE AND PREPARATION FOR FILL, FOUNDATION, FLOOR SLAB AND PAVEMENT SUPPORT; AND SELECTION, PLACEMENT AND COMPACTION OF FILL SOILS USING MODIFIED PROCTOR PROCEDURES

1. Construction monitoring and testing of subgrades and grades for fill, foundation, floor slab and pavement; and fill selection, placement and compaction shall be performed by an experienced soils engineer and/or his representatives.

2. All compacted fill, subgrades, and grades shall be (a) underlain by suitable bearing material, (b) free of all organic frozen, or other deleterious material, and (c) observed, tested and approved by qualified engineering personnel representing an experienced soils engineer. Preparation of subgrades after stripping vegetation, organic or other unsuitable materials shall consist of (a) proofrolling to detect soft, wet, yielding soils or other unstable materials that must be undercut, (b) scarifying top 6 to 8 inches, (c) moisture conditioning the soils as required, and (d) recompaction to same minimum in-situ density required for similar material indicated under Item 5. Note: Compaction requirements for pavement subgrade are higher than other areas. Weather and construction equipment may damage compacted fill surface and reworking and retesting may be necessary for proper performance.

3. In overexcavation and fill areas, the compacted fill must extend (a) a minimum 1 foot lateral distance beyond the exterior edge of the foundation at bearing grade or pavement at subgrade and down to compacted fill subgrade on a maximum 0.5(H):1(v) slope, (b) 1 foot above footing grade outside the building, and (c) to floor subgrade inside the building. Fill shall be placed and compacted on a 5(H):1(V) slope or must be stepped or benched as required to flatten if not specifically approved by qualified personnel under the direction of an experienced soils engineer.

4. The compacted fill materials shall be free of deleterious, organic, or frozen matter, shall contain no chemicals that may result in the material being classified as "contaminated", and shall be low-expansive with a maximum Liquid Limit (ASTM D-423) and Plasticity Index (ASTM D-424) of 30 and 15, respectively, unless specifically tested and found to have low expansive properties and approved by an experienced soils engineer. The top 12 inches of compacted fill should have a maximum 3 inch particle diameter and all underlying compacted fill a maximum 6 inch diameter unless specifically approved by an experienced soils engineer. All fill material must be tested and approved under the direction of an experienced soils engineer prior to placement. If the fill is to provide non-frost susceptible characteristics, it must be classified as a clean GW, GP, SW or SP per Unified Soils Classification System (ASTM D-2487).

5. For structural fill depths less than 20 feet, the density of the structural compacted fill and scarified subgrade and grades shall not be less than 90 percent of the maximum dry density as determined by Modified Proctor (ASTM D-1557) with the exception of the top 12 inches of pavement subgrade which shall have a minimum in-situ density of 95 percent of maximum dry density, or 5 percent higher than underlying structural fill materials. Where the structural fill depth is greater than 20 feet, the portion below 20 feet should have a minimum in-place density of 95 percent of its maximum dry density or 5 percent higher than the top 20 feet. Cohesive soils shall not vary by more than -1 to +3 percent moisture content and granular soil ±3 percent from the optimum when placed and compacted or recompacted, unless specifically recommended/approved by the soils engineer observing the placement and compaction. Cohesive soils with moderate to high expansion potentials (PI>15) should, however, be placed, compacted and maintained prior to construction at a 3±1 percent moisture content above optimum moisture content to limit future heave. Fill shall be placed in layers with a maximum loose thickness of 8 inches for foundations and 10 inches for floor slabs and pavements, unless specifically approved by the soils engineer taking into consideration the type of materials and compaction equipment being used. The compaction equipment should consist of suitable mechanical equipment specifically designed for soil compaction. Bulldozers or similar tracked vehicles are typically not suitable for compaction.

6. Excavation, filing, subgrade grade preparation shall be performed in a manner and sequence that will provide drainage at all times and proper control of erosion. Precipitation, springs, and seepage water encountered shall be pumped or drained to provide a suitable working platform. Springs or water seepage encountered during grade/foundation construction must be called to the soils engineer's attention immediately for possible construction procedure revision or inclusion of an underdrain system.

7. Non-structural fill adjacent to structural fill should typically be placed in unison to provide lateral support. Backfill along walls must be placed and compacted with care to ensure excessive unbalanced lateral pressures do not develop. The type of fill material placed adjacent to below grade walls (i.e. basement walls and retaining walls) must be properly tested and approved by an experienced soils engineer with consideration for the lateral pressure used in the wall design.

8. Wherever, in the opinion of the soils engineer or the Owner's Representatives, an unstable condition is being created either by cutting or filling, the work should not proceed into that area until an appropriate geotechnical exploration and analysis has been performed and the grading plan revised, if found necessary.

GENERAL COMMENTS

The soil samples obtained during the subsurface exploration will be retained for a period of thirty days. If no instructions are received, they will be disposed of at that time.

This report has been prepared exclusively for the client in order to aid in the evaluation of this property and to assist the architects and engineers in the design and preparation of the project plans and specifications. Copies of this report may be provided to contractor(s), with contract documents, to disclose information relative to this project. The report, however, has not been prepared to serve as the plans and specifications for actual construction without the appropriate interpretation by the project architect, structural engineer, and/or civil engineer. Reproduction and distribution of this report must be authorized by the client and *Giles*.

This report has been based on assumed conditions/characteristics of the proposed development where specific information was not available. It is recommended that the architect, civil engineer and structural engineer along with any other design professionals involved in this project carefully review these assumptions to ensure they are consistent with the actual planned development. When discrepancies exist, they should be brought to our attention to ensure they do not affect the conclusions and recommendations provided herein. The project plans and specifications may also be submitted to *Giles* for review to ensure that the geotechnical related conclusions and recommendations provided herein have been correctly interpreted.

The analysis of this site was based on a subsoil profile interpolated from a limited subsurface exploration. If the actual conditions encountered during construction vary from those indicated by the borings, *Giles* must be contacted immediately to determine if the conditions alter the recommendations contained herein.

The conclusions and recommendations presented in this report have been promulgated in accordance with generally accepted professional engineering practices in the field of geotechnical engineering. No other warranty is either expressed or implied.

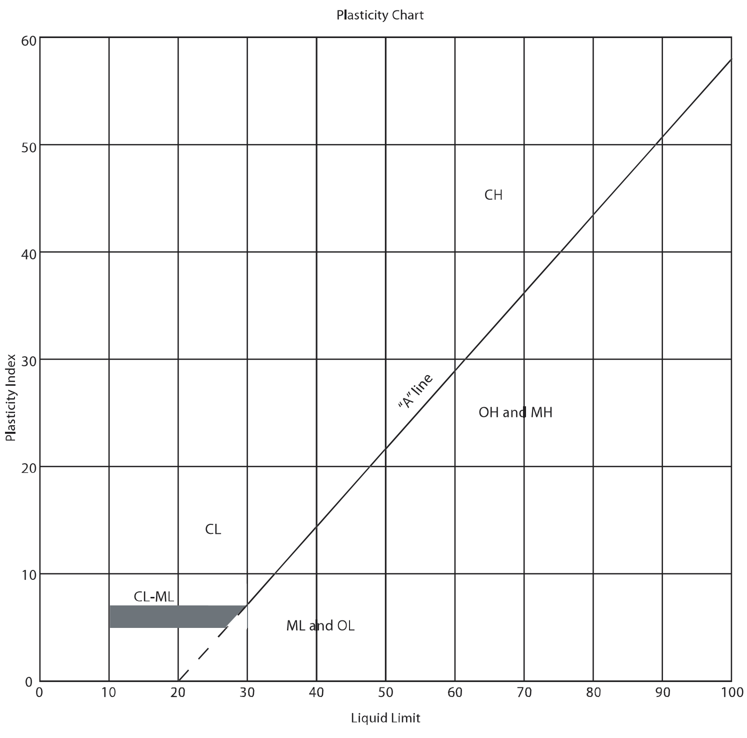
CHARACTERISTICS AND RATINGS OF UNIFIED SOIL SYSTEM CLASSES FOR SOIL CONSTRUCTION *

Class	Compaction Characteristics	Max. Dry Density Standard Proctor (pcf)	Compressibility and Expansion	Drainage and Permeability	Value as an Embankment Material	Value as Subgrade When Not Subject to Frost	Value as Base Course	Value as Temporary Pavement	
								With Dust Palliative	With Bituminous Treatment
GW	Good: tractor, rubber-tired, steel wheel or vibratory roller	125-135	Almost none	Good drainage, pervious	Very stable	Excellent	Good	Fair to poor	Excellent
GP	Good: tractor, rubber-tired, steel wheel or vibratory roller	115-125	Almost none	Good drainage, pervious	Reasonably stable	Excellent to good	Poor to fair	Poor	
GM	Good: rubber-tired or light sheepsfoot roller	120-135	Slight	Poor drainage, semipervious	Reasonably stable	Excellent to good	Fair to poor	Poor	Poor to fair
GC	Good to fair: rubber-tired or sheepsfoot roller	115-130	Slight	Poor drainage, impervious	Reasonably stable	Good	Good to fair **	Excellent	Excellent
SW	Good: tractor, rubber-tired or vibratory roller	110-130	Almost none	Good drainage, pervious	Very stable	Good	Fair to poor	Fair to poor	Good
SP	Good: tractor, rubber-tired or vibratory roller	100-120	Almost none	Good drainage, pervious	Reasonably stable when dense	Good to fair	Poor	Poor	Poor to fair
SM	Good: rubber-tired or sheepsfoot roller	110-125	Slight	Poor drainage, impervious	Reasonably stable when dense	Good to fair	Poor	Poor	Poor to fair
SC	Good to fair: rubber-tired or sheepsfoot roller	105-125	Slight to medium	Poor drainage, impervious	Reasonably stable	Good to fair	Fair to poor	Excellent	Excellent
ML	Good to poor: rubber-tired or sheepsfoot roller	95-120	Slight to medium	Poor drainage, impervious	Poor stability, high density required	Fair to poor	Not suitable	Poor	Poor
CL	Good to fair: sheepsfoot or rubber-tired roller	95-120	Medium	No drainage, impervious	Good stability	Fair to poor	Not suitable	Poor	Poor
OL	Fair to poor: sheepsfoot or rubber-tired roller	80-100	Medium to high	Poor drainage, impervious	Unstable, should not be used	Poor	Not suitable	Not suitable	Not suitable
MH	Fair to poor: sheepsfoot or rubber-tired roller	70-95	High	Poor drainage, impervious	Poor stability, should not be used	Poor	Not suitable	Very poor	Not suitable
CH	Fair to poor: sheepsfoot roller	80-105	Very high	No drainage, impervious	Fair stability, may soften on expansion	Poor to very poor	Not suitable	Very poor	Not suitable
OH	Fair to poor: sheepsfoot roller	65-100	High	No drainage, impervious	Unstable, should not be used	Very poor	Not suitable	Not suitable	Not suitable
Pt	Not suitable		Very high	Fair to poor drainage	Should not be used	Not suitable	Not suitable	Not suitable	Not suitable

* "The Unified Classification: Appendix A - Characteristics of Soil, Groups Pertaining to Roads and Airfields, and Appendix B - Characteristics of Soil Groups Pertaining to Embankments and Foundations," Technical Memorandum 357, U.S. Waterways Ixperiment Station, Vicksburg, 1953.

** Not suitable if subject to frost.

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

Major Divisions			*Group Symbols*		*Typical Names*	*Laboratory Classification Criteria*		
Coarse-grained soils (more than half of material is larger than No. 200 sieve size)	Gravels (More than half of coarse fraction is larger than No. 4 sieve size)	Clean gravels (little or no fines)	GW		Well-graded gravels, gravel-sand mixtures, little or no fines	Determine percentages of sand and gravel from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows: Less than 5 percent: GW, GP, SW, SP More than 12 percent: GM, GC, SM, SC 5 to 12 percent: *Borderline* cases requiring dual symbols[b]	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			GP		Poorly graded gravels, gravel-sand mixtrues, little or no fines		Not meeting all gradation requirements for GW	
		Gravels with fines (appreciable amount of fines)	GM[a]	d	Silty gravels, gravel-sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4	Limits plotting within shaded area, above "A" line with P.I. between 4 and 7 are *borderline* cases requiring use of dual symbols
				u				
			GC		Clayey gravels, gravel-sand-clay mixtures		Atterberg limits above "A" line or P.I. greater than 7	
	Sands (More than half of coarse fraction is smaller than No. 4 sieve size)	Clean sands (Little or no fines)	SW		Well-graded sands, gravelly sands, little or no fines		$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			SP		Poorly graded sands, gravelly sands, little or no fines		Not meeting all gradation requirements for SW	
		Sands with fines (Appreciable amount of fines)	SM[a]	d	Silty sands, sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4	Limits plotting within shaded area, above "A" line with P.I. between 4 and 7 are *borderline* cases requiring use of dual symbols
				u				
			SC		Clayey sands, sand-clay mixtures		Atterberg limits above "A" line or P.I. greater than 7	
Fine-grained soils (More than half 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			
			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 sandy or silty soils, elastic silts			
			CH		Inorganic clays of high plasticity, fat clays			
			OH		Organic clays of medium to high plasticity, organic silts			
	Highly organic soils		Pt		Peat and other highly organic soils			

[a] Division of GM and SM groups into subdivisions of d and u are for roads and airfields only. Subdivision is based on Atterberg limits, suffix d used when L.L. is 28 or less and the P.I. is 6 or less; the suffix u is used when L.L. is greater than 28.

[b] Borderline classifications, used for soils possessing characteristics of two groups, are designated by combinations of group sympols. For example GW-GC, well-graded gravel-sand mixture with clay binder.

GENERAL NOTES

SAMPLE IDENTIFICATION

All samples are visually classified in general accordance with the Unified Soil Classification System (ASTM D-2487-75 or D-2488-75)

DESCRIPTIVE TERM (% BY DRY WEIGHT)

Trace:	1-10%
Little:	11-20%
Some:	21-35%
And/Adjective	36-50%

PARTICLE SIZE (DIAMETER)

Boulders:	8 inch and larger
Cobbles:	3 inch to 8 inch
Gravel:	coarse - ¾ to 3 inch
	fine – No. 4 (4.76 mm) to ¾ inch
Sand:	coarse – No. 4 (4.76 mm) to No. 10 (2.0 mm)
	medium – No. 10 (2.0 mm) to No. 40 (0.42 mm)
	fine – No. 40 (0.42 mm) to No. 200 (0.074 mm)
Silt:	No. 200 (0.074 mm) and smaller (non-plastic)
Clay:	No 200 (0.074 mm) and smaller (plastic)

SOIL PROPERTY SYMBOLS

Dd:	Dry Density (pcf)
LL:	Liquid Limit, percent
PL:	Plastic Limit, percent
PI:	Plasticity Index (LL-PL)
LOI:	Loss on Ignition, percent
Gs:	Specific Gravity
K:	Coefficient of Permeability
w:	Moisture content, percent
qp:	Calibrated Penetrometer Resistance, tsf
qs:	Vane-Shear Strength, tsf
qu:	Unconfined Compressive Strength, tsf
qc:	Static Cone Penetrometer Resistance (correlated to Unconfined Compressive Strength, tsf)
PID:	Results of vapor analysis conducted on representative samples utilizing a Photoionization Detector calibrated to a benzene standard. Results expressed in HNU-Units. (BDL=Below Detection Limit)
N:	Penetration Resistance per 12 inch interval, or fraction thereof, for a standard 2 inch O.D. (1⅜ inch I.D.) split spoon sampler driven with a 140 pound weight free-falling 30 inches. Performed in general accordance with Standard Penetration Test Specifications (ASTM D-1586). N in blows per foot equals sum of N-Values where plus sign (+) is shown.
Nc:	Penetration Resistance per 1¾ inches of Dynamic Cone Penetrometer. Approximately equivalent to Standard Penetration Test N-Value in blows per foot.
Nr:	Penetration Resistance per 12 inch interval, or fraction thereof, for California Ring Sampler driven with a 140 pound weight free-falling 30 inches per ASTM D-3550. Not equivalent to Standard Penetration Test N-Value.

DRILLING AND SAMPLING SYMBOLS

SS:	Split-Spoon
ST:	Shelby Tube – 3 inch O.D. (except where noted)
CS:	3 inch O.D. California Ring Sampler
DC:	Dynamic Cone Penetrometer per ASTM Special Technical Publication No. 399
AU:	Auger Sample
DB:	Diamond Bit
CB:	Carbide Bit
WS:	Wash Sample
RB:	Rock-Roller Bit
BS:	Bulk Sample
Note:	Depth intervals for sampling shown on Record of Subsurface Exploration are not indicative of sample recovery, but position where sampling initiated

SOIL STRENGTH CHARACTERISTICS

COHESIVE (CLAYEY) SOILS

COMPARATIVE CONSISTENCY	BLOWS PER FOOT (N)	UNCONFINED COMPRESSIVE STRENGTH (TSF)
Very Soft	0 - 2	0 - 0.25
Soft	3 - 4	0.25 - 0.50
Medium Stiff	5 – 8	0.50 - 1.00
Stiff	9 – 15	1.00 - 2.00
Very Stiff	16 – 30	2.00 - 4.00
Hard	31+	4.00+

NON-COHESIVE (GRANULAR) SOILS

RELATIVE DENSITY	BLOWS PER FOOT (N)
Very Loose	0 - 4
Loose	5 - 10
Firm	11 - 30
Dense	31 - 50
Very Dense	51+

DEGREE OF PLASTICITY	PI
None to Slight	0 - 4
Slight	5 - 10
Medium	11 - 30
High to Very High	31+

DEGREE OF EXPANSIVE POTENTIAL	PI
Low	0 - 15
Medium	15 - 25
High	25+

Important Information About Your
Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

The following information is provided to help you manage your risks.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual

subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely, on Your ASFE-Member Geotechncial Engineer for Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you ASFE-member geotechnical engineer for more information.

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