Appendix F

Geotechnical Site Evaluation

Kimley»Horn

Geotechnical Site Evaluation
Ladyface Vista Business Center
29555 Canwood Street
Agoura Hills, California

prepared for

Martin Teitelbaum Construction, Inc.
569 Constitution Avenue, Suite H
Camarillo, California 93012

GORIAN
& ASSOCIATES, INC.

Table of Contents

Attachments:

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Work Order: 3187-0-0-100

Subject: **Geotechnical Site Evaluation, Canwood Office Center, 29541 Canwood Street (APN 2053001008), Agoura Hills, California**

1. INTRODUCTION

The following report contains the findings, conclusions, and recommendations of our geotechnical site evaluation addressing design and construction of the Canwood Office Center at 29541 Canwood Street in Agoura Hills, California (Site). Based on a review of the 30 scale Site Plan by Delane Engineering (plot dated April 27, 2021), the development will include five detached buildings surrounded by surface parking and drive areas on the existing gently sloping terrain, as well as retaining walls in several locations. The Site Plan (Sheet 2 of 2) serves as a base for the attached Geotechnical Map (Plate 1).

Borings were used to obtain data on the subsurface consisting of Miocene-age Topanga formation bedrock overlain topsoil and local areas of thin fill soils as described herein. The field exploration was supplemented with laboratory testing to determine mechanical properties of the encountered soils. In addition, research was performed that indicated the Site is not within Earthquake Fault, Liquefaction, or Earthquake Induced Landslide Zones (CGS, *Earthquake Zones of Required Investigation* website). Based on our site evaluation, the Site is suitable for the proposed construction from a geotechnical standpoint provided recommendations presented herein are implemented in the project design and construction. Descriptions of the Site and geologic units along with our conclusions and recommendations are presented within the text of this report.

2. PROPOSED DEVELOPMENT

Based on a review of the Site Plan attached as Plate 1, the development will include five, single story, above grade (no basements) buildings with a total of 20239 square feet. The development will also include 55,189 square feet of hardscape including 110 surface parking stalls and drive areas. Conventional grading operations consisting of cut / fill grading along with retaining wall construction are anticipated to achieve design grades. Retaining walls are proposed on the northern, eastern and southern sides of the Site. Shallow fills on the order of 6 feet are anticipated along the southern edge of the front parking area and cuts will be on the order of 20 feet (northern portion, retaining walls). Access to the property will be from Canwood Street. The buildings are anticipated to be of wood construction supported on lightly loaded conventional footings.

3. SCOPE OF GEOTECHNICAL SERVICES

Our site evaluation was performed in accordance with generally accepted geotechnical engineering practices under the direction of a California state registered geotechnical engineer and certified engineering geologist. These services included the following:

3.1 ARCHIVAL REVIEW

Pertinent geologic/geotechnical data in our files was reviewed including regional geologic maps, geotechnical/geologic hazard maps, and Alquist-Priolo earthquake fault-rupture hazard zone maps.

3.2 GEOTECHNICAL SUBSURFACE EXPLORATION

Three borings were drilled, sampled, and logged ranging in depth from 41 feet (B-1) to 19.5 feet (B-2) below the existing ground surface (bgs). The borings were drilled utilizing a subcontractor supplied and operated truck-mounted bucket auger drill rig equipped with a 24-inch bucket. The borings were observed by a geologist from this office, who logged the underlying materials and obtained bulk and relatively undisturbed drive soil and bedrock samples for laboratory analyses and to characterize the subsurface soil and bedrock conditions. At the conclusion of drilling and sampling, the geologist entered the borings for direct downhole observation and logging of encountered lithologies and geologic structural elements to be utilized in the site evaluation.

At the conclusion of drilling, logging and sampling, the excavations were backfilled with spoils from the borings and tamped with the Kelly bar. The backfilled materials may settle with time, therefore, the owner or his representative should periodically observe the boring locations and fill any depressions that may occur.

3.3 LABORATORY TESTING

A program of laboratory testing was performed on selected soil and bedrock samples obtained from the field during the subsurface exploration. Testing included in-situ moisture and density determinations, compaction characteristics, shear strength parameters, expansion, and consolidation potential. In addition, corrosion testing of a selected sample was performed by an independent subcontracted laboratory.

3.4 GEOTECHNICAL ENGINEERING ANALYSIS AND REPORT PREPARATION

The results of our archival review, field exploration, laboratory testing programs and engineering analyses were used to develop geotechnical recommendations for design and construction of the office center. The findings, conclusions, and recommendations of our evaluation are provided in this report that includes:

a) A description of soil, bedrock and groundwater conditions, as encountered during the subsurface exploration, including Logs of Subsurface Data (Appendix A) and a Geotechnical Map (Plate 1). Geotechnical Cross Section A-A’ (Plate 2) was prepared illustrating subsurface conditions and for use in slope stability / soil nail wall analyses.

b) A description of the laboratory testing program, including test results (Appendix B).

c) Discussion and geotechnical recommendations regarding:
   - i. Geologic hazards including seismic setting of the site and faulting;
   - ii. Seismic design criteria;
   - iii. Soil expansion potential;
   - iv. Slope stability;
   - v. Site preparation and remedial grading;
   - vi. Conventional foundation design and construction;
   - vii. Retaining wall design parameters including backfill recommendations;
   - viii. Soil nail retaining walls;

ix. Lateral earth pressures;
x. Slab-on-grade and hardscape design;
xi. Preliminary pavement recommendations; and
xii. Soil chemistry analysis, by subcontract.

4. SITE DESCRIPTION

The site is at 29541 Canwood Street, which is on the north side of the street roughly centered between Forest Cove Lane and Kanan Road in Agoura Hills, California. This portion of Canwood Street runs roughly east and west directly on the north side of the Ventura Freeway (Route 101). The property consists of approximately 3.25 acres of gently sloping hillside terrain (see Figure 1). The property is bounded by Los Angeles County Fire Station 89 to the west (29575 Canwood Street), a medical building (29525 Canwood Street) to the east, single family residential housing Tract 23760 to the north, and Canwood Street on the south side. Improvements along Canwood Street include curb and gutter as well as a sidewalk. The southerly facing hillside is characterized by a moderate growth of seasonal weeds and grasses, previously disced for periodic weed abatement, and several mature oak trees. Drainage of the site is by sheet flow towards the south where it is in turn captured by Canwood Street storm drain improvements. Total relief of the site is on the order of 76 feet.

5. BACKGROUND

This office prepared geotechnical reports for design and construction of two projects on the northern side of Canwood Street (29501 and 29353), which are to the east and in close proximity to the subject Site. The following provides a description of previously conducted site evaluations in this area by this office as well as other consultants.

<u>TRACT 23760</u> - North adjacent the Site
Preliminary geotechnical site evaluations and grading compaction reports for the residential Tract 23760 (located north of the subject property) were both prepared by Robert Stone and Associates (RSA) in 1969 and 1970 (Gorian, 2005). The RSA reports indicate the Topanga Formation and Conejo Volcanics underlie the area just north of the Site. RSA characterized the Topanga Formation as being discontinuous and found as irregular patches within the Conejo Volcanics and as large masses containing scattered volcanic beds. The Conejo Volcanics generally consist of intrusions, lava flows, and massive agglomerate and breccia beds of andesitic and basaltic composition. RSA indicated the bedding within the Topanga Formations is inclined northward at moderate to locally steep angles because the area is on the south flank of an east-west trending syncline.

<u>29525 CANWOOD STREET</u> - East adjacent the Site
In 1977 Geolabs Westlake Village, Inc. (Geolabs) performed a geotechnical evaluation for a skateboard park (which was subsequently re-developed into a medical office building 29525 Canwood Street) and indicated the site was underlain 2-3 feet of brown to grey clay and silty clay soils overlying bedrock of the Miocene-age Topanga Formation based on four test pits. Their test pit logs indicate the bedrock is generally inclined to the north at moderate angles (32° to 36°), however, locally the bedding was observed to dip steeply (51° to 55°) to the southwest.

In 1981 Kovacs-Byer and Associates, Inc. (KBA) performed an additional geotechnical evaluation within the skateboard park for a medical office building at 29525 Canwood Street (Gorian, 2005). Based on their six exploratory test pits, besides local fills from the construction and demolition of the skatepark, native residual soil/alluvium on the order of 2-3 feet thick and consisting of dark brown silty clays were observed overlying the bedrock. They concurred with Geolabs (1977) that the site was underlain by the Topanga Formation with some localized areas of basaltic intrusions. Structurally, they indicated the bedrock was inclined to the north at moderate to steep angles (27° to 70°), however, small folds were found and interpreted to be near the basalt contact.

<u>29501 CANWOOD STREET</u> - Second parcel East of Site
In 1998, this office prepared a geotechnical investigation report for Lot 54 (29501 Canwood Street) on the northern side of Canwood Street (Gorian, 1998). The field exploration included excavating two 24-inch diameter bucket auger borings to depths ranging from 21 to 42 feet and three backhoe trenches ranging in depth from 3.5 to 6 feet below the existing ground surface. In 2005, this office prepared a G*eotechnical Update Investigation* report which included the drilling sampling and logging of three additional 24-inch diameter bucket auger borings to depths ranging from 16 to 31 feet below the existing ground surface (Gorian, 2005).

The following is reiterated for clarity from the Gorian 2005 report:

With the exception of Boring B-3, all five bucket borings performed by Gorian (B-1 through B-5) encountered bedrock of the Miocene-age Topanga Formation at depths ranging from 2 to 5 feet bgs. In boring B-3, Topanga Formation was encountered below the volcanic intrusion at an approximate depth of 6.5 feet bgs. The backhoe trenches also exposed Topanga Formation at depths ranging from 1 to 2.5 feet below the existing ground surface.

Bedrock of the Topanga Formation as encountered in the all of the exploratory borings and trenches consists predominately of brown to light olive brown siltstone and argillaceous shale with localized interbedded fine-grained sandstone. However, in boring B-1 the uppermost portion of the Topanga Formation consists of 1.5 feet of light olive brown volcanic conglomerate in a weathered yet indurated condition. At a depth of 24.5 and 14 feet below the ground surface in boring B-1 and B-2 respectively, the bedrock transitions into a dark gray siltstone in an indurated and damp condition. Generally, above this dark gray siltstone, the bedrock is slightly weathered and locally moderately fractured.

Based on our subsurface exploration, the bedrock is inclined to the northwest and northeast at moderate to steep angles (24° to 85°). Bedding attitudes presented in the referenced reports on adjacent properties are consistent with the bedrock orientation observed during this site evaluation (29555 Canwood Street). Regional geologic maps (Weber, 1984 and Dibblee, 1993) indicate the bedrock is inclined to the north at moderate to steep angles (35° to 65°). Structurally, Dibblee indicates a tightly folded syncline exists near the contact of the Topanga Formation and the Conejo Volcanics resulting in overturned bedding within the Topanga Formation. The contact between these two units was not encountered on-site but was reported to be approximately 180 feet north of the proposed development within Tract 23760 (RSA, 1969).

After stage grading operations (borrow area) on the site (29501 Canwood Street), additional field mapping was performed and revealed that a Conejo Volcanic intrusion does exist on the site. The Topanga Formation is tightly folded on the north side of the Conejo Volcanics intrusion. Just north of the contact with the Conejo Volcanics, the Topanga Formation is folded into a syncline that trends generally east-west with the southern limb inclined at 88° and the northern limb inclined at 61° to 65°. Just north of the syncline is an anticline also trending east-west, with the north limb inclined to the north at moderate angles (26° to 38°). Boring B-1 is located on the northern limb of this anticline with bedding attitudes inclined at moderate to steep angles.

As observed during previous field mapping, the Conejo Volcanics generally consist of yellowish brown to gray basalt in a slightly weathered and fractured condition. Based on field mapping, it appears the exposed Conejo Volcanics intrusion in contact with the Topanga Formation is steeply inclined to the north (88°).

In 2007, this office prepared the *Rough Grading Compaction Test Report* for 29501 Canwood Street (Gorian, 2007a). The exposed geologic conditions encountered during the grading operation are reiterated below for completeness.

Both cut slope and retaining wall backcut excavations expose bedrock of the Miocene-age Topanga Formation, which is locally intruded by dikes of Conejo Volcanics. The bedrock as exposed in these excavations generally consists of laminated to thinly bedded, medium brown to light gray silty shale and argillaceous siltstone with occasional beds of fine to medium-grained sandstone. The bedrock is moderately indurated and slightly to moderately weathered, fractured with some iron oxide staining on fracture surfaces.

Structurally, geologic mapping of the cut slopes and wall backcuts during grading revealed the Topanga Formation materials to be moderately folded yet generally inclined in a northerly direction at moderate to near vertical angles (44° to 75°). However, in the northeastern portion of the site a localized anticlinal fold trending generally east west was observed with bedding on the southern limb inclined towards the south at steep angles (70° to 71°). Locally, the Topanga Formation is intruded with volcanic rocks of the Conejo Volcanics. The volcanic intrusions appear as isolated patches and masses of yellowish brown to dark gray fine-grained basalt in a damp and fractured condition.

<u>29575 CANWOOD STREET</u> – West adjacent the Site
In 1999, Converse Consultants prepared a *Geotechnical Investigation Report* for a proposed fire station at 29575 Canwood Street (Converse, 1999). This site is adjacent and just west of the subject site. The evaluation included drilling sampling and logging seven hollow stem auger borings to depths ranging from 11 feet to 51 feet below the existing ground surface (bgs). The yellowish-brown fat clay alluvium encountered in six of the seven borings was reported to vary in thickness from 4 feet to 23 feet and the underlying bedrock was referred to as the Calabasas Formation of the upper Topanga Group. The alluvium was described as a fat clay with an expansion index of 132 and the bedrock was described as yellowish brown to gray siltstone interbedded with brown and gray claystone with some sandstone interbeds noted. Locally perched shallow groundwater was reported encountered during drilling in 3 of the 7 borings (within the Alluvium at 4' bgs BH-3 (el.872) and 18' bgs BH-4 (el. 878) as well as within the bedrock at 9' bgs BH-9 (el.878)). [There was no readily available geotechnical map attached to this report.]

In 2001, Converse Consultants prepared an updated geotechnical report for the fire station (Converse, 2001). The report indicates the site was unchanged since 1999.

In 2004, Mactec prepared a *Report of Supplementary Geotechnical Investigation for Los Angeles County Fire Station 89 at 29575 Canwood Street* (Mactec, 2004a) in response to unexpected groundwater seepage being encountered at or near design rough grades during grading operations. The investigation included drilling logging and sampling five borings ranging from 1.8 to 3 feet deep. This report indicated bedrock was much shallower than Converse reported, exposed at approximately rough grade elevations (el. 873 @northern portion to el. 870 @ southern portion of the site) at the time of their evaluation and geotechnical recommendations were presented for footings to be in bedrock. Recommendations to dewater the site due to the shallow groundwater conditions were also presented.

In September of 2004, the County of Los Angeles, Department of Public Works, Geotechnical and Materials Engineering Division (GMED) issued a Geologic Review Sheet and Soils Engineering Review Sheet dated September 16 (GMED, 2004a) regarding the Mactec (2004a) report. Mactec (2004b) presented geotechnical and geologic responses to the GMED comments. The project was subsequently approved as indicated in the GMED review letters dated October 5 and 6, 2004 (GMED, 2004b).

<u>29353 CANWOOD STREET</u> – Third parcel East of Site
In 2007, this office prepared a geotechnical report for design and construction of a fitness center at 29353 Canwood Street (Gorian, 2007b). The site evaluation included the drilling logging and sampling of four bucket auger borings. The encountered geologic conditions are reiterated below for completeness.

Bedrock of the Miocene-age Topanga Formation as encountered in each exploratory boring generally consists of yellowish brown to light olive brown argillaceous siltstone and gray shale with localized interbeds of yellow silty fine-grained sandstone and limy siltstone beds in a damp to very moist and moderately indurated condition. These sedimentary units are typically thinly bedded, fissile, and contain some manganese and iron oxide staining. At depth, the color of the bedrock commonly grades to brown (siltstone) and dark gray (shale) before becoming very dark gray. In boring B-2 as described below, an intrusive body of Conejo Volcanics basalt was encountered.

Structurally, based on our subsurface exploration, the bedrock is inclined to the northwest and northeast at moderate to steep angles (37° to 66°). Bedding attitudes presented in the referenced reports on adjacent properties are consistent with the bedrock orientation observed during this investigation. Regional geologic maps (Weber, 1984 and Dibblee, 1993) indicate the bedrock is inclined northward at moderate to steep angles (35° to 65°). Dibblee indicates a tightly folded syncline exists near the contact of the Topanga Formation and the Conejo Volcanics to the north of the proposed development resulting in overturned bedding within the Topanga Formation.

An isolated outcrop of Topanga Formation siltstone was observed within the surrounding Conejo Volcanics in the extreme northeastern portion of the site. The bedding within the sedimentary rock was mapped as being vertical. The approximate contact between the Topanga Formation and Conejo Volcanics is irregular and is considerably higher on the slope than regional geologic maps show. A large sandstone outcrop near the contact appears to be inclined towards the northeast at a steep angle (70°).

An intrusion of Miocene-age Conejo Volcanics was encountered during the investigation within boring B-2 at a depth of 2.5 feet and extended to a minimum depth of 11 feet before the drilling operation was stopped. The intrusion generally consists of dark gray fine-grained basalt in a damp and indurated to highly indurated condition. Similar intrusions were found to the west (Gorian 2005).

The volcanic bedrock as observed in boring B-2 is typically fractured yet indurated. Conventional grading equipment was able to cut the volcanic intrusions encountered during grading of the property directly to the west. However, difficult drilling was encountered in boring B-2 in the volcanic rock, therefore hard bedrock should be anticipated on-site.

Based on field mapping operations, the upper reaches of the site are underlain by volcanic rock that outcrops in many locations. Based on a review of regional geologic maps (Weber, 1984, Dibblee, 1993), the inferred contact between the Conejo Volcanic and the Topanga Formation on this site is at about elevation 900.0. However, based on our recent boring B-1 at elevation 925.0 and the field mapping, this contact is higher on the hillside and is irregular. An isolated outcropping of the Topanga Formation was observed within the larger volcanic body in the extreme northeastern portion of the site.

6. SITE GEOLOGY

The subject site at 29541 Canwood Street is underlain at depth by sedimentary bedrock referred to the Topanga Formation mantled with topsoil, local artificial fill, and presumably Alluvial deposits in the flatter areas adjacent Canwood Street. Descriptions of these units are presented below and in the attached Logs of Subsurface Data (Appendix A). The interpreted geologic structure is illustrated on the attached Geotechnical Cross Section A-A' (Plate 2).

6.1 ARTIFICIAL FILL

Artificial fill deposits were encountered in boring B-3 with a thickness of 1 foot. The artificial fill generally consists of brown silty clay mottled with dark brown silty clay in a wet and stiff condition. Other apparently artificial fill deposits were encountered in the western half of the southern-most portion of the site as well as adjacent the existing sidewalk. Here, the surface soil is mixed with gravel and concrete debris indicated additional artificial fill deposits may exist but do not appear to be more than surficial deposits.

Based on Google Earth air photo imagery (12/21/2005 and 3/15/2006) the southwestern area was used as a staging/parking/construction office area for the construction of the adjacent Fire Station 89. Regardless, the artificial fill is not considered suitable for structural support and should be removed to competent underlying materials prior to structural fill placement.

6.2 TOPSOIL

Surficial soils mantling the bedrock are referred to as topsoil. The upper portion of this surficial deposit has been disturbed by the periodic discing of the Site for weed abatement. As encountered in the borings, the topsoil zone varies in thickness from 2 feet (B-3) to 3.5 feet (B-1) and generally, consists of brown silty clay with some sand and shale fragments in a moist and stiff to very stiff condition below the disturbed disced zones. These soils are considered to be very expansive.

6.3 ALLUVIUM

Although not encountered in the subsurface exploration program, regional mapping indicates alluvial deposits in the lowland, flat portion of the Site adjacent Canwood Street. If present on Site these deposits are masked by fill and the discing operations. On the adjacent site to the east, Geolabs (1977) and KBA (1981) reported two to three feet dark brown to brown clay and silty clay alluvial-native residual/alluvium encountered in their respective exploratory programs. To the west of the Site, although Converse (1999) reported four to as much as twenty-two feet of yellowish-brown fat clay alluvial soils, the subsequent Mactec (2004a and b) reports indicated a much thinner section of alluvial soil with bedrock being encountered during grading operations at elevations 870 to 873. Consequently, it is our opinion that any alluvial soils under the mantle of fill adjacent Canwood Street would likely be of minor thickness (<5'±).

6.4 TOPANGA FORMATION

Bedrock of the Miocene-age Topanga formation underlies the site at depth and was encountered in all of the exploratory borings. As encountered near the ground surface, the sedimentary bedrock generally consists of yellowish-brown clayey siltstone interbedded with yellowish brown to olive gray to gray claystone in a moist condition. Locally these fine-grained sediments are interbedded with yellowish brown silty fine-grained sandstone and limy siltstone. The limy siltstone in B-2 was indurated and difficult to drill resulting in refusal conditions. The bedrock is typically thinly bedded, fissile and fractured yet tight. Some iron oxide and manganese oxide staining were noted.

At depth, the bedrock grades to dark gray in color and generally consists of unoxidized clayey siltstone interbedded with claystone with thin (1" to 2" thick) gray to light yellowish brown silty fine-grained sandstone beds in a hard and damp condition. No critically expansive clay seams (Bentonite) was observed.

Structurally, the bedrock is inclined towards the northeast and northwest at moderate to near vertical angles (37º to 86º). The site appears to be on the southern limb of a generally east west trending synclinal fold as previously described in the *Background* section herein.

6.5 GROUNDWATER

In our current exploration, groundwater was not encountered to the maximum depth explored, 41 feet (B-1; el. 903) below the existing ground surface or to the lowest elevation explored, el. 850 (B-3 el.871). However, perched groundwater conditions have been reported at a variety of elevations in exploratory excavations and construction for surrounding developments; most notably for the adjacent Fire Station 89 construction where significant seepages were encountered during construction at an elevation of 870-873 (Mactec, 2004a and b). Seepages were noted both in bedrock and alluvial soils and often within explorations/construction proximal to topographic drainages / swales. However, it should be noted that groundwater, particularly with the perched water level conditions, will fluctuate seasonally in response to precipitation and often with area irrigation practices.

6.6 LANDSLIDES

No landslides are present within or near the site nor are any shown on regional geologic maps.

6.7 FAULTING AND SEISMICITY

The Site, like any in the southern California area, is in a seismically active region prone to occasional damaging earthquakes. The destructive power of earthquakes can be grouped into fault-rupture, ground shaking (strong motion), and secondary effects of ground shaking such as tsunami, liquefaction, settlement, mass wasting, and flooding from dam failures.

The hazard of fault-rupture is generally thought to be associated with a relatively narrow zone along well-defined, pre-existing active faults. No doubt there is and will be exceptions to this, because it is not possible to predict the precise location of a new fault where none existed before (CDMG, 1975). No Holocene-active faults are known to cross the Site nor is the project site currently located within an Alquist-Priolo (A-P) Earthquake Fault Zone as defined by the State Geologist (CGS 2018). The closest Holocene-active faults are the Malibu Coast fault located approximately 7⅔ miles southeast of the Site and the Simi-Santa Rosa Fault Zone, approximately 9¼ miles northeast of the Site. The potential for ground rupture on-site due to faulting during the time period of concern is considered remote.

Although no active or potentially active faults are known to exist within or adjacent the site, the area will be subject to strong ground motion from occasional earthquakes in the region. Four significant earthquakes have occurred epicentered within a 50± mile radius of the site within the last eight decades; the March 11, 1933 Long Beach earthquake (6.4 magnitude), the February 9, 1971 San Fernando earthquake (6.6 magnitude), the October 1, 1987 Whittier Narrows earthquake (5.9 magnitude) and the January 17, 1994 Northridge earthquake (6.7 magnitude). Significant earthquakes will likely occur in this area within the life expectancy of the project and the site will experience strong ground shaking from these events.

Based on the latest United States Geological Survey (USGS) interactive web application, Unified Hazard Tool https://earthquake.usgs.gov/hazards/interactive/ probabilistic seismic hazard analyses (PSHA) predict the Design Basis Earthquake (475-year return period) peak horizontal ground acceleration will be on the order of 0.39g for the bedrock conditions of the class C site. The mean magnitude from this PSHA is 6.64 (Mw) with a mean distance of approximately 18.0 km from the property. Utilizing a 2% chance of being exceeded in 50 years (2475-year return period) peak horizontal ground acceleration will be on the order of 0.68g for the soil conditions on site. The mean magnitude from this PSHA is 6.73 (Mw) with a mean distance of approximately 13.8 km from the property.

Secondary effects of strong ground motion include tsunami, seiche, liquefaction, seismic settlement, earthquake triggered landslides, and flooding from dam failures. Tsunamis are impulsively generated water waves that can cause damage to shoreline areas. A seiche is an oscillation wave within an enclosed body of water. The site is not near the ocean or adjacent a body of water and, therefore, is not subject to tsunami and seiche hazards. Furthermore, the site is not prone to earthquake triggered landslides due to the relatively low relief in the area, nor is the site in the vicinity of a dam failure inundation zone. The site is not within a State designated seismic hazard zone for liquefaction or earthquake induced landslides (CGS 2000, *Earthquake Zones of Required Investigation* website).

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 GENERAL

The site was evaluated from a geotechnical site standpoint and is considered suitable for the proposed office building project as described herein at 29541 Canwood Street in Agoura Hills, California. The bedrock deposits underlying the site are suitable for support of the structure. However, remedial grading is proposed to prepare the Site as discussed hater herein. Differential settlement should be negligible.

The project may be developed as described earlier in this report provided recommendations presented herein are followed and incorporated into the project design and construction.

7.2 GEOTECHNICAL SEISMIC DESIGN

As previously discussed, active faults identified by the State are not onsite nor is the site within an Alquist-Priolo Earthquake Fault Zone. Nevertheless, the site is within a seismically active region prone to occasional damaging earthquakes.

Structures within the site may be designed using procedures for seismic design presented in ASCE/SEI 7-16. Mapped acceleration parameters are initially determined for sites having a shear wave velocity of 2,500 feet per second (Section C11.4.4). The S_s and S_1 values are adjusted to obtain the maximum considered earthquake (MCE) spectral acceleration values for the site based on its site class of C. The seismic design parameters for the site's coordinates (latitude 34.1479 N and longitude 118.7687 W) were obtained from the web based SEAOC/OSHPD Seismic Design Maps: https://seismicmaps.org/. The parameters are presented on the following page.

The purpose of the building code earthquake provisions is primarily to safeguard against major structural failures and loss of life, not to limit damage nor maintain function. Therefore, values provided in the building code should be considered minimum design values and should be used with the understanding site acceleration could be higher than addressed by code-based parameters. Cracking of walls and possible structural damage should be anticipated in a significant seismic event.

SEISMIC PARAMETER	VALUE PER CBC
Short Period Mapped Acceleration (S_s)	1.455g
Long Period Mapped Acceleration (S_1)	0.514g
Site Class Definition	C
Site Coefficient (F_a)	1.2
Site Coefficient (F_v)	1.486
$S_{MS} = F_aS_s$	1.746g
$S_{M1} = F_vS_1$	0.764g
$S_{DS} = 2/3S_{MS}$	1.164g
$S_{D1} = 2/3S_{M1}$	0.509g
PGA_M	0.728g
Seismic Design Category	D

7.3 SITE PREPARATION AND GRADING

7.3.1 General

The building pads will be graded using a combination of cut and fill grading and retaining wall construction. Remedial grading will consist of the removal of the upper soils and undercutting the bedrock in transition areas from cut to fill. The recommendations herein are for the preparation of the site for the proposed construction. Grading including site preparation, excavation, and fill placement should be per the city of Agoura Hills Grading Ordinance.

7.3.2 Site Cleanup

Deleterious surface materials, including trash, debris, vegetation, rocks, and organic materials on-site should be removed from the areas of grading and construction should be removed prior to grading.

7.3.3 Soil and Bedrock Removals
Within areas of grading and construction and 5 feet beyond, soil and bedrock removals should extend to firm in-place bedrock. In some areas, this may be 2 to 3 feet below the soil and bedrock contact. However, soil removals should not extend below a 2(horizontal)1(vertical) line extending down from the property lines or the Canwood Street right of way. The removal bottom should be observed by this office to evaluate if local areas exist where deeper removals are necessary.

Where soil removal may not be feasible such as along the property lines, e.g., property line retaining walls, it may be necessary to deepen the retaining wall footings or provide compaction of the bottom of the retaining wall footings. The need for deepened footings or in footing compaction should be determined based on observation of the exposed footing excavations by this firm.

Conventional grading equipment should be capable of performing the excavations necessary to achieve design grades. However, due to the presence of locally indurated volcanic intrusions it is possible that production of oversize rock will occur if large indurated rock bodies are encountered.

7.3.4 Building Area Undercuts
In addition to the removals indicated above, the building areas and within the building foundation influence zones, overexcavation should extend to a depth of at least 5 feet below the existing or proposed grade or 3 feet below foundations, whichever is the deeper overexcavation. The bottom of the removal should extend at least 5 foot outside the perimeter of the building or foundation, whichever is greater. The undercut area should be observed by this office prior to fill placement.

7.3.5 Preparation of Fill Areas
Areas to receive fill should be processed before placing fill. Processing should consist of surface scarification to a minimum depth of 8 inches, moisture conditioning to slightly over the optimum moisture content, and recompaction to a minimum of 90% relative compaction.

7.3.6 Relative Compaction
Relative compaction is the ratio of the in-place dry soil density to the maximum dry soil density determined in general accordance with ASTM test method D 1557.

7.3.7 Keying and Benching
Fills placed on slopes steeper than 5(horizontal):1(vertical) should be keyed and benched (horizontal benches) into firm competent in-place bedrock (after required removals are made). A minor slope is shown along Canwood Street that will require removals and possibly a keyway. Keyways should be a minimum of 15 feet wide and cut a minimum depth of 2 feet at the toe into firm competent in-place bedrock. Keyways should be tilted into the slope and should be at least 3 feet deep at the heel (measured from below the slope toe elevation). A representative of this office should observe the keyways and benches prior to placing fill. Horizontal benches should be a minimum of 5 feet wide, i.e., a minimum 5 feet of competent material. The vertical portion of the bench in competent bedrock should not exceed 5 feet.

7.3.8 Fill Placement
On-site materials obtained from excavations may be used as fill soils. Where possible, the higher expansive soils encountered should be exported from the site or placed outside the building area. Fill soils should be free of deleterious materials including trash, debris, and organic matter.

Fill containing excavated rock up to 8-inch size may be used for general engineered compacted fill, however, rock within three feet of the footings should be maintained at less than 6 inches. It may be desirable to keep rock larger than 3 inches outside of the building area. Rock should not be permitted to nest with unfilled voids. The fill should contain less than 30% of material from 6 to 8-inch maximum diameters. The fills should be placed in thin lifts, at slightly over optimum moisture content, and compacted to 90 percent relative compaction.

7.3.9 Temporary Excavations
During construction, excavation and maintenance of safe and stable slope angles are the responsibility of the contractor, who should consider the subsurface conditions and the method of operation. All subsurface construction should conform to the requirements of OSHA. Surcharge loads should be setback from the top of temporary excavations a minimum horizontal distance equal to the depth of the cut or 10 feet, whichever is more. All excavated backfill should be properly placed and compacted. Vertical trench excavation should be 4 foot or less in height unless shored. Vertical retaining wall cuts may be made to 5 feet in height (such as needed for soil nail wall construction).

7.3.10 Utility Trenches
Backfill of utility trenches within building, parking, and drive areas should be compacted to a minimum of 90% relative compaction.

7.3.11 Shrinkage/Bulking
Shrinkage or bulking is the volume loss or gain respectively of soils excavated and recompacted. Shrinkage of the upper 5 feet of soil and bedrock from cut to fill is estimated to be approximately 5 to 10 percent; i.e., 1 cubic yard of cut will yield approximately 0.9 to 0.95 cubic yards of engineered compacted fill. Bulking is the volume expansion of the earth materials from cut to fill. The amount of volume change will depend on the material in situ density, the final compacted density achieved, etc. For excavations below 5 feet, the bedrock is expected to bulk 5 to 10 percent, i.e., 1 cubic yard of cut will yield 1.05 to 1.10 cubic yards of engineered compacted fill. In addition to the shrinkage/bulking values, subsidence or a loss of 0.1 to 0.2 feet should be considered for stripping of vegetation and densification of the surface soils.

Shrinkage / bulking values presented are based on an assumption that fills will be compacted to an average of 93% of the maximum dry soil density. The actual in-place compacted density can vary with the type of soil compacted, the compacting effort applied to the soil, and the in-situ moisture content. These values are provided for gross estimating purposes only. If quantities are critical, it is recommended that test strips be performed and monitored at the site using the actual grading equipment to be utilized for the grading operations.

7.4 SLOPE CONSTRUCTION

7.4.1 General
Manufactured fill and cut slopes may be constructed at maximum gradient of 2(horizontal):1(vertical). At this time cut slopes are not planned and cuts within the northern portion of the development will be supported by retaining walls.

7.4.2 Fill Slopes
Fill slopes should be keyed and benched into competent bedrock materials, as previously recommended. Select grading will be required when placing fill materials within 20 feet of permanent slope faces. Fill soils near slope faces should average at least 250 psf cohesive shear strength and 25 friction.

Where possible, the outer slope faces should be overfilled and trimmed back to provide for firm, well-compacted surfaces. The slope faces should be sheep footed and/or grid rolled if the slopes are not trimmed back. The slope faces should be tested and reworked as necessary to achieve the required compaction.

7.4.3 Cut (Retaining Wall) Slopes
Though cut slopes are not anticipated as this time, bedding within the site is inclined favorably to the north, into the retaining wall excavation and will be grossly stable. Cuts should be evaluated by an engineering geologist from our office.

Depending on the time of year and precipitation, seepage could be encountered at the toe of a cut or retaining wall. At these locations, a toe of cut subdrain should be installed to remove subsurface water migrating towards the toe. This toe of slope drain may be omitted where a retaining wall with proper back drainage is constructed at the toe of slope.

The drain should be a minimum of 2 to 3 feet below the toe of slope and should consist of a 4-inch diameter perforated Schedule 40 PVC or equivalent. The pipe should be placed with perforations down approximately 3 to 6 inches from the bottom of the excavation. The pipe should be contained in a minimum 2 cubic feet of ¾ inch crushed rock. The rock should be wrapped in filter fabric with joints overlapped 12 inch minimum. The rock should be covered by 1 foot of compacted soil backfill.

The outlet pipe should be non-perforated 4-inch diameter PVC. A concrete cutoff wall should be installed at the transition from perforated to non-perforated pipe. The subdrain excavation should be observed this office prior to backfilling.

7.4.4 Slope Maintenance
Slopes within the site will require maintenance or protection to reduce the risk of erosion and degradation with time due to natural or other conditions. Slope (requiring planting) planting should consist of dense, deep rooting, drought resistant groundcover with shrubs and trees. A reliable irrigation system should be installed, adjusted so that over watering does not occur, and periodically checked for leakage. Over watering of slopes should be avoided because it can cause expansion, erosion, and surficial failures. A uniform, near optimum moisture content should be maintained below the slope surface. Slopes should not be over watered and should not be watered before forecasted rain. Drainage structures should be kept in good condition and clean. Burrowing animals (e.g., ground squirrels) can destroy slopes; therefore, where present, immediate measures should be taken to eliminate them.

7.4.5 Slope Stability
Slope stability was analyzed for Cross Section a-A' (Plate 2) to demonstrate the stability of the slope and proposed (recommended) soil nail retaining wall. Our analyses considered non-circular type and circular failures with the use of the computer program Slide2D by Rocscience. Static and pseudo-static analyses were performed using the Spencer Method. Pseudo-static analyses were completed using a horizontal acceleration coefficient of 0.15g. The stability of slopes is commonly stated in terms of the slope's calculated factor of safety. The generally accepted lower limit for factor of safety is 1.5 and 1.1 for static and pseudo-static conditions, respectively. Acceptable factors of safety were obtained; the results are presented in Appendix C. The material strengths used in the analysis for the bedrock are a Ø of 28 degrees and a cohesion of 410 pounds per square foot. The analyses were performed to demonstrate the suitability of using soil nails to support the proposed cut along the northern edge of the development. The soil nail or pile supported wall design should be by an engineer specializing in these types of wall design.

7.5 SOIL EXPANSIVENESS
A soil expansion test was performed on a representative soil sample obtained from the site. Test results indicate the underlying materials have a high expansion potential, in the 91-130 Expansion Index range. Additional expansion tests should be performed at the conclusion of the recommended remedial grading.

Expansive soils contain clay particles that change in volume (shrink or swell) due to a change in the soil moisture content. The amount of volume change depends upon the soil swell potential (amount of expansive clay in the soil), availability of water to the soil, and the soil confining pressure. Swelling occurs when soils containing clay become wet due to excessive water from poor surface drainage, over-irrigation of lawns and planters, and sprinkler or plumbing leaks. Swelling clay soils can cause distress to structures, walks, drains, and patio slabs.

Swelling clay soils can cause distress to construction (generally as uplift). Construction on expansive soil has an inherent risk that should be acknowledged and understood by the developer/property owner. The geotechnical recommendations presented herein are intended to reduce the potential for expansive soil action. However, these recommendations are not intended, nor designed to provide complete and full mitigation of expansive soil conditions.

7.6 FOUNDATION RECOMMENDATIONS

7.6.1 Conventional Footings

The structures will be supported on continuous or isolated footings underlain by engineered compacted soil as addressed above and may be designed for an allowable bearing pressure of 2,500 pounds per square foot (psf). The allowable net bearing pressure may be increased by one-third when considering wind or seismic loads. The weight of concrete below grade may be excluded from the footing load.

Footings should be embedded a minimum of 24 inches below the interior pad grade (not top of slab) or 36 inches below the exterior grade, whichever provides the deeper embedment. The exterior grade should be the lowest adjacent rough grade or permanent lowest grade, whichever is deeper.

The footing width should be a minimum of eighteen inches for continuous footings and twenty-four inches for isolated footings. Footing reinforcement should be per the structural engineer's recommendations. However, minimum continuous footing reinforcement should consist of two number five bars in the top and bottom (total of 4 bars). Perimeter isolated footings should be tied together with a grade beam extending 36 inches deep below the lowest adjacent grade.

Shallow footings adjacent retaining walls, should be included in the design of the wall or stepped down below a 2(horizonal):1(vertical) plane projecting upward from the bottom of wall footings.

7.6.2 Lateral Resistance

Lateral forces on foundations may be resisted by lateral passive earth pressure and base friction. Passive earth pressure may be assumed equal to an equivalent fluid pressure of 300 pounds per cubic foot for level ground, however should not exceed 2,000 pounds per square foot. This allowable passive pressure may be used adjacent a descending slope provided the footing has the appropriate setback to slope face. A coefficient of friction of 0.30 may be assumed along the base of concrete elements cast directly against the subgrade. Passive earth pressure and friction may be combined with no reductions.

7.6.3 Mat Slab Design Data

Mat slabs may be designed using an allowable soil bearing pressure of 1,500 pounds per square foot or a modulus of subgrade reaction "K" of 125 pounds per cubic inch (pci) at the surface of a properly prepared building pad. The project structural engineer should determine the steel reinforcement and concrete compressive strength. The slabs supporting interior steel stud walls should be a minimum of 8 inches thick.

7.6.4 Estimated Foundation Settlements

Static settlement of footings should be evaluated once building footing locations and structural loads are known. However, as a preliminary estimate, footing settlement for static loading is anticipated on the order of 1 inch or less, with a maximum differential settlement of 1± inch over a span of approximately 30 feet or between adjacent individual footings. This is provided building construction is started directly after footing excavation, footings are cast soon after the footing excavation, and construction is completed in a timely manner. Settlements due to static loading are expected to occur rapidly as the loads are applied.

All structures settle during construction and some minor settlement of structures can occur after construction during the life of the project. Minor wall cracking could occur within the structure associated with expansion and contraction of the structural members. In addition, wall or slab cracking may be associated with settlement or expansive soil movement. Additional settlement/soil movement could occur if the soils dry or become saturated due to excessive water infiltration generally caused by excessive irrigation, poor drainage, etc.

7.6.5 Footing and Beam Excavations

Footing and grade beam excavation should be cut square and level; and cleaned of slough and soils silted into the excavations during the premoistening operations. Soil excavated from the footing trenches should not be spread over areas of construction unless properly compacted. A representative of this office should observe the footing excavations prior to placing reinforcing steel. The footings should be cast as soon as possible to avoid deep desiccation of the footing subsoils.

7.6.6 Premoistening

Footing subsoils should be premoistened to 3% over the optimum moisture content for a depth of 18 inches. Saturated soils or soils silted into the footing excavations should be removed prior to concrete placement.

7.7 SLABS-ON-GRADE

7.7.1 Site Preparation

Concrete slabs on-grade not used for structural support may be supported on compacted engineered fill soils. Slab subgrade soils should be recompacted prior to placing the aggregate subbase, if the soils were disturbed during footing or utility construction.

7.8 Design Data

Interior concrete slabs on-grade not used for structural support should be 5 inches thick and underlain by 6-inch-thick layer of ½ inch or larger clean aggregate or per applicable building codes, whichever is the more restrictive. The slab should be reinforced with a minimum of number 4 bars at 18-inch centers in each direction. The reinforcement should be placed and kept at slab mid-depth.

7.9 Premoistening

Soils under lightly loaded slabs on-grade should be premoistened to 3% over the optimum moisture content for a depth of 18 inches.

7.9.1 Moisture Vapor Retarder

A moisture vapor retarder layer should be incorporated into the slab on-grade design within the building interior. The water vapor retarder should be one that is specifically designed as a vapor retarder and consist of a minimum 15 mil extruded polyolefin plastic and comply with Class A requirements under ASTM E1745 (*Standard Specification for Plastic Water Vapor Retarders Used in Contact with Soil or Granular Fill under Concrete Slabs*). The vapor retarder should be installed in accordance with ASTM E1643. The water vapor retarder should be installed in direct contact with the concrete slab along with a concrete mix design to control bleeding, shrinkage, and curling (ACI 302.2R). The vapor retarder shall be installed over a minimum 6-inch-thick layer of ½ inch or larger clean aggregate or per applicable building codes, whichever is the more restrictive. The vapor retarder should be placed per ASTM E1643-98(2005) *Standard Practice for Installation of Water Vapor Retarders Used in Contact with Earth or Granular Fill Under Concrete Slabs*. All joints should be lapped and sealed along with proper sealing of perforations such as for plumbing. In addition, various trades and the concrete contractor should be required to protect the moisture retarder during construction.

Perforations through the moisture vapor retarder such as at pipes, conduits, columns, grade beams, and wall footing penetrations should be sealed per the manufacture's specifications or ASTM E1643. Proper construction practices should be followed during construction of slabs on-grade. Repair and seal tears or punctures in the moisture barrier that may result from the construction process prior to concrete placement.

Minimizing shrinkage cracks in the slab on-grade can further minimize moisture vapor emissions. A properly cured slab utilizing low-slump concrete will reduce the risk of shrinkage cracks in the slab as described herein.

The concrete contractor should make the necessary changes in the concrete placement and curing for concrete placed directly over the retarder. Placing the concrete directly on top of the moisture vapor retarder layer allows the layer to be observed for damage directly prior to concrete placement.

The slabs should be tested for moisture content prior to the selection of the flooring and adhesives. Moisture in the slabs should not exceed the flooring manufacture's specifications. The concrete surface should be sealed per the manufacture's specifications if the moisture readings are excessive. It may be necessary to select floor coverings that are applicable to high moisture conditions.

7.9.2 Concrete Placement and Cracking

Minor cracking of concrete slabs is common and generally the result of concrete shrinkage continuing after construction. Concrete shrinks as it cures resulting in shrinkage tension within the concrete mass. Since concrete is weak in tension, development of tension results in cracks within the concrete. Concrete should be placed using procedures to minimize the cracking within the slab. Shrinkage cracks can become excessive if water is added to the concrete above the allowable limit and proper finishing and curing practices are not followed. Concrete mixing, placement, finishing, and curing should be performed per the American Concrete Institute Guide for Concrete Floor and Slab Construction (ACI 302.1). Concrete slump during concrete placement should not exceed the design slump specified by the structural engineer. Concrete slabs on grade should be provided with tooled or saw cut (saw cuts should be made the same day a maximum within few hours of the pour or per the structural engineer's recommendations) crack control joints at 10–15-foot centers or as specified by the structural engineer.

7.10 EXTERIOR SLABS AND WALKWAYS (Hardscape)

Lightly loaded exterior concrete hardscape (non-auto traffic) and walkways should be a minimum of 4 inches thick and underlain by a minimum of 4 inches of sand. Slabs should be reinforced with a minimum of #3 bars on 24-inch centers in each direction placed at mid-height in the slab. Slabs should have crack control joints at intervals of 10 to 15 feet or per the structural engineer's recommendation. Sidewalks may be constructed of non-reinforced concrete provided they are cut into square panels (i.e., 4-foot-wide walks should be cut into 4 foot by 4-foot squares).

Concrete slab subgrade soils should be properly placed and compacted for support of concrete flatwork. Prior to placing concrete, subgrade soils should be premoistened to a minimum of 3% over the optimum moisture content for a minimum depth of 18 inches. Proper premoistening can reduce the risk of slab subgrade expansion, if used in addition to other preventive measures.

7.11 TOP OF SLOPE DEEPENED EDGE

Exterior slabs at or near the top of slope should have a reinforced 12-inch-wide deepened edge extending a minimum of 24 inches below the slab. The edge should be reinforced with a minimum of 2 number 4 bars in the top and bottom.

Where a driveway will be at the top of a slope it should be constructed with a deepened edge. The bottom of the edge should have sufficient depth to provide a bottom of edge to slope setback of at least 10 feet (discounting the outer 2 feet of the slope). The edge should be constructed with two number five bars in the top and bottom. Vertical reinforcement of #4 bars should be installed on 24-inch centers. The vertical steel should extend to the bottom edge reinforcement and extended a minimum of 36 inches into the slab.

7.12 SOIL CORROSION

The results are presented herein of analytical laboratory testing to evaluate the potential for corrosion of materials in contact with the onsite soils. Testing was performed by Project X Corrosion Engineering on a soil sample considered to represent the onsite soils (the test results are attached hereto in Appendix B). From ACI Table 19.3.1.1, the evaluated soil is categorized as Class S0. The required concrete design requirements for this exposure class can be obtained from ACI Table 19.3.2.1. The potential for corrosion of metals in contact with the onsite soils is very severely corrosive as determined from Table 1. (The tables are presented in Appendix B.) For specific recommendations, a corrosion engineer should be consulted.

7.13 RETAINING WALL DESIGN

7.13.1 Foundations

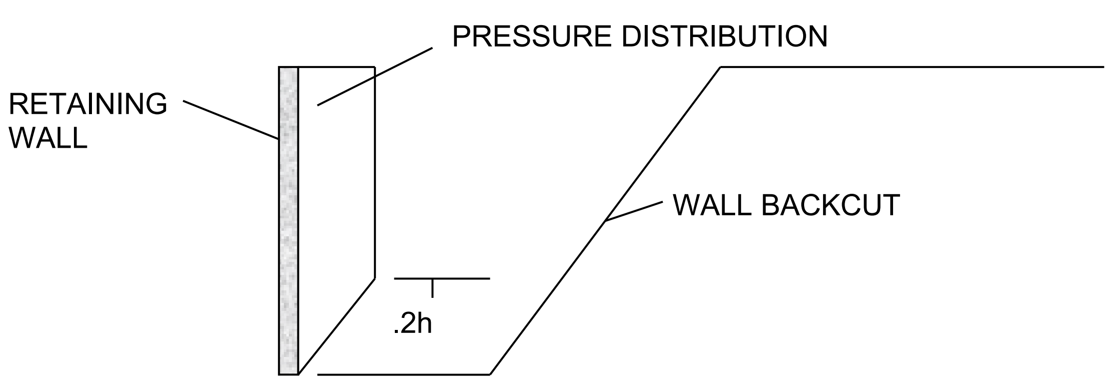
Retaining wall footings should be design in accordance with foundation design recommendations previously provided herein for bearing capacity, lateral resistance, embedment, etc.

7.13.2 Active Pressures

Retaining walls should be designed to resist an active pressure exerted by compacted backfill or retained soil/bedrock. Retaining walls that may yield at the top may be designed for an equivalent fluid pressure equal to 45 and 65 psf for a level or 2(horizontal):1(vertical) sloped backfill, respectively. The pressures may be used for walls supporting either cut or certified compacted fill consisting of on-site soils. To prevent saturation of backfill and to reduce problems with expansive soil pressures against the back of the retaining wall, the walls should be equipped with a drainage system as described below.

Permanent braced retaining walls should be designed for a pressure of 40H (psf) where H is the height of the retained soil. The pressure distribution should be over the area shown below. A surface surcharge of 300 pounds per square foot (psf) should be included in the design where the shoring is near traffic zones. Surcharge on the wall from loads directly adjacent the wall can be evaluated by this office on an individual basis. The backdrain should be designed as described below.

A representative of this office should observe retaining wall backcuts in bedrock for adverse geologic conditions.



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7.13.3 Seismic Pressure
Lateral seismic soil pressure is not required for retaining walls under 6 feet high. Walls over 6 feet high should be designed for a total seismic load of the static and dynamic load increments:

$$P_{ae} = P_{static} + \Delta P_{ae} = F_1 + F_2$$

P_{static} is determined based on active or at-rest conditions. The dynamic load increment, ΔP_{ae} (F2), shall be determined using the following equations for different wall type and backfill conditions (after Agusti and Sitar, 2013):

Basement (restrained) walls with level backfill: ΔPae = 40 pcf

Cantilever (unrestrained) wall with level backfill: ΔPae = 25 pcf

Cantilever (unrestrained) wall with sloping backfill*: ΔPae = 42 pcf

*Applicable for sloping backfill that is no steeper than 2:1 (horizontal: vertical).

7.13.4 Pile Foundation
Where necessary retaining walls along the perimeter of the site may be supported on a pile system consisting of drilled cast in-place piles. Axial load carrying capacity of a drilled pile is based on the skin friction of 750 pounds per square foot between the drilled shaft and surrounding soils. Uplift may be taken as 2/3 of the downward capacity. The weight of the pile and pile cap may be ignored in determining the column loads. Pile capacities may be increased by one-third for short term wind and seismic loading for downward capacities only. The weight of the drilled pile may be added to the uplift resistance. No friction may be assumed below a pile cap.

Drilled piles should have a minimum center to center spacing of 3 times the pile diameter to avoid vertical interaction between adjacent piles. No lateral capacity should be assumed in the upper portion of a pile near a utility or in the direction of the adjacent slope. Exterior grade beams should extend to the depths recommended for footings the lowest adjacent grade.

A passive earth pressure equal of 300 pounds per cubic foot (maximum pressure of 2000 psf may be used to resist lateral forces on pile caps and piles 24 inch diameter or greater). Base friction should not be used in calculating the passive resistance of pile caps. The pile spacing should be at least 3 diameters center to center and pile diameter may be double in the determination of pile lateral resistance. Pile caps should have a thickness of at least 24 inches below grade. The piles should be embedded a minimum of 10 feet below the pile cap.

7.13.5 Pile Settlement
The maximum expected settlement of drilled piles should be minor, on the order of 1/4 inch or less. Differential settlement between similarly loaded piles is expected to be less than 1/4 inch. All structures settle during construction and some minor settlement of the structures can occur after construction during the life of the project.

7.13.6 Pile Construction
Piles constructed within the site should encounter soils/bedrock as previously described herein. Conditions within the site could vary and some caving and or hard drilling should be anticipated as within any subsurface excavation. Piles should be drilled and cast in the same day and the holes should not be left open overnight.

Care should be exercised when casting adjacent pile borings to avoid blowout from one excavation into the other. From an engineering standpoint, the preferred method would be to excavate, cast, and let the concrete achieve initial set prior to excavating the adjacent drilled shaft. Drilled pile excavations should be observed by this office prior to setting reinforcing steel to verify the anticipated geotechnical conditions.

7.13.7 Soil Nail Retaining Walls

Soil nail walls consist of steel bar inserted and grouted in to holes drilled at an approximate angle shown in the detail be low to provide a reinforced soil mass to support the ground behind the wall. Normally, soil nail walls are constructed in vertical segments with each segment being 5 to 6 foot high. The nails are installed after which the backcut face is covered with shotcrete. Therefore, backfill is not necessary for a soil nail wall. See the typical section on the following page excerpted from the United States Department of Transportation (USDOT) Federal Highway Administration *Soil Nail Walls Reference Manual* in which can be found design procedures. The walls may be designed using the soil parameters used in the slope stability analyses presented in this report. The soil nail wall should be designed by an engineer specializing in soil nail wall design and construction.

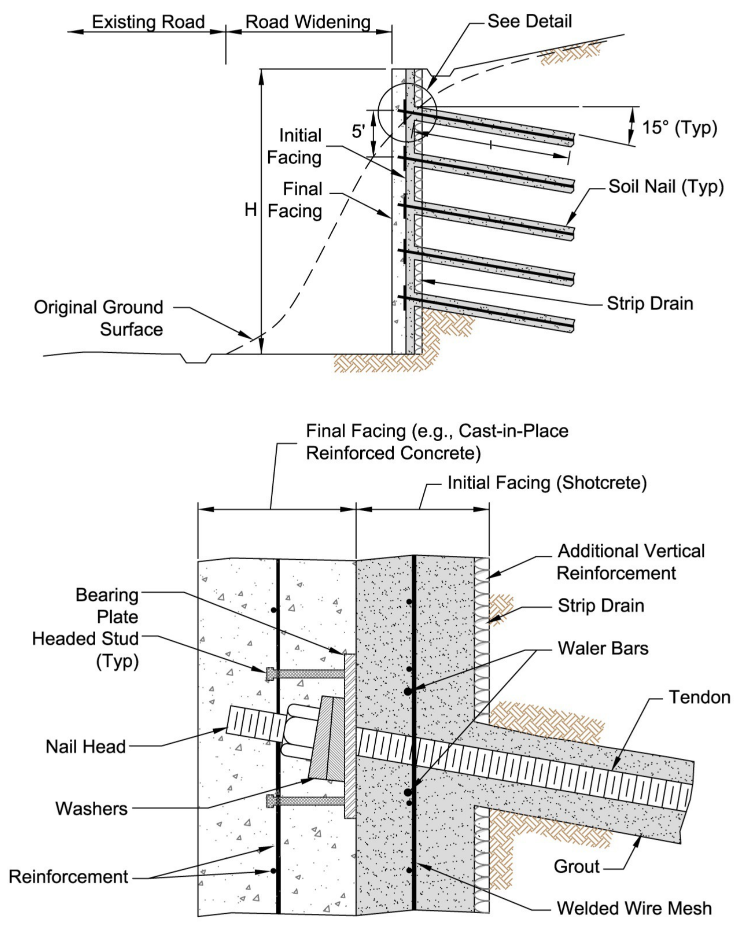

Figure 2.1: Typical cross-section of a soil nail wall. (Federal Highway Administration, February 2015, Soil Nail Walls Reference Manual)

7.13.8 Retaining Wall Drainage and Backfill

A drainage system should be constructed behind the retaining walls to relieve buildup of hydrostatic pressures. In addition, the back of the walls should be waterproofed. The drainage system may consist of either a drainage composite or granular drain consisting of a minimum 12 inch wide zone of clean sand and #4 rock at a 1:1 ratio. The drainage system should extend to within 2 feet of finish grade with the upper 2 feet backfilled with native material. A layer of filter cloth should be placed to separate the granular drainage material from the native backfill. The drainage system should be hydraulically connected to a perimeter pipe drain consisting of a minimum 4 inch diameter perforated PVC (Schedule 40) pipe or equivalent. Drainpipe may be laid horizontally on the footing however, the pipe invert should be at least 6 inches below the top of slab-on-grade. The outlet pipe from the perimeter drain should be a non-perforated 4 inch diameter PVC (Schedule 40) pipe that is sloped to and connected to a storm drain system or sump. An as-built plan should be prepared detailing the location of the wall drainage system.

Wall backfill should be compacted to a minimum of 90% of the maximum soil density using light equipment. Walls at the toe of slopes should have a concrete drainage swale placed behind the wall at the toe of slope to collect surface run off from the slope face.

7.14 PRELIMINARY PAVEMENT DESIGN

For preliminary planning based on an estimated “R” Value of 5 and a Traffic Index of 5, assume 3 inches of A/C over 10 inches of aggregate base for drive areas and 3 inches of A/C over 7 inches of aggregate base for parking stalls. The structural sections should be confirmed after conclusion of grading. The upper 6 inches of subgrade, and the base material, should be compacted to at least 90 and 95 percent of the maximum dry density, respectively, just prior to placing the asphalt.

A preliminary structural section for the widening of Canwood Street may consist of 4 inches of asphalt concrete on 17 inches of aggregate base. This preliminary section is based on a design traffic index of 7 and an assumed R-value of less than 10.

Concrete curbs at the top of a slope should be deepened so that the bottom outside edge of the curb has a setback of a least 5 feet to the slope face. This may require deepening of the outside edge of the curb.

Concrete pavement should be considered in driveways that will receive high abrasion loads, and in areas subject to repeated heavy truck loads, such as trash pickup areas. The concrete pavement in these areas should be a minimum 7-inch thick with No. 3 bars at 18 inches on centers in both directions or per the structural engineer's design. The slab should be underlain by 4 inches of Class 2 aggregate base compacted to a minimum 95% relative compaction. Concrete should have a minimum 28 day compressive strength of 3500 psi. Concrete pavement subgrade soils should be premoistened to a minimum of 3% above the optimum moisture content for a minimum depth of 18 inches.

Planter areas should be graded and constructed so that excess water collected by an area drain system or drained onto and not beneath the adjacent AC pavement. Consideration should be given to deepening the curbs adjacent to planters so that water is prevented from entering the pavement base and saturating the pavement subgrade. Concrete curbs near the top of descending slopes should be embedded so the bottom of the curb has a setback of at least 5 feet to the slope face.

SITE DRAINAGE

Positive drainage should be provided away from structures during and after construction per the grading plan or applicable building codes. Water should not be allowed to gather or pond against foundations. In addition, planters near a structure should be constructed so that irrigation water will not saturate footing and slab subgrade soils.

PLAN REVIEW

As the design process continues, this office should review the grading, building, and foundation plans prior to starting site grading.

CLOSURE

This report was prepared under the direction of a registered geotechnical engineer and certified engineering geologist. No warranty, express or implied, is made as to conclusions and professional advice included in this report. Gorian and Associates, Inc. disclaim responsibility and liability for problems that may occur if the recommendations presented in this report are not followed.

This report was prepared for Martin Teitelbaum Construction, Inc. and design consultants solely for design and construction of the development described herein. This report may not contain sufficient information for other uses or the purposes of other parties. These recommendations should not be extrapolated to areas not covered by this report or used for other development without consulting Gorian and Associates, Inc.

The recommendations are based on interpretations of the subsurface conditions concluded from information gained from previous grading observations and a surficial site reconnaissance. The interpretations may differ from actual subsurface conditions, which can vary horizontally and vertically across the site. Persons using this report for bidding or construction purposes should perform such independent investigations as they deem necessary. This office should observe all aspects of field construction addressed in this report.

Services of Gorian and Associates, Inc. or this report should not be construed to relieve the owner or any construction contractor from their responsibility or liabilities, or for maintaining a safe jobsite. Neither the professional activities of Gorian and Associates, Inc. nor the presence of our employees shall be construed to imply Gorian and Associates, Inc. has responsibility for methods of work performance, superintendence, sequencing of construction, or safety in, on, or about the jobsite.

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Please contact our office if you have questions regarding the information or recommendations contained in this report, or require additional consultation.

Respectfully,

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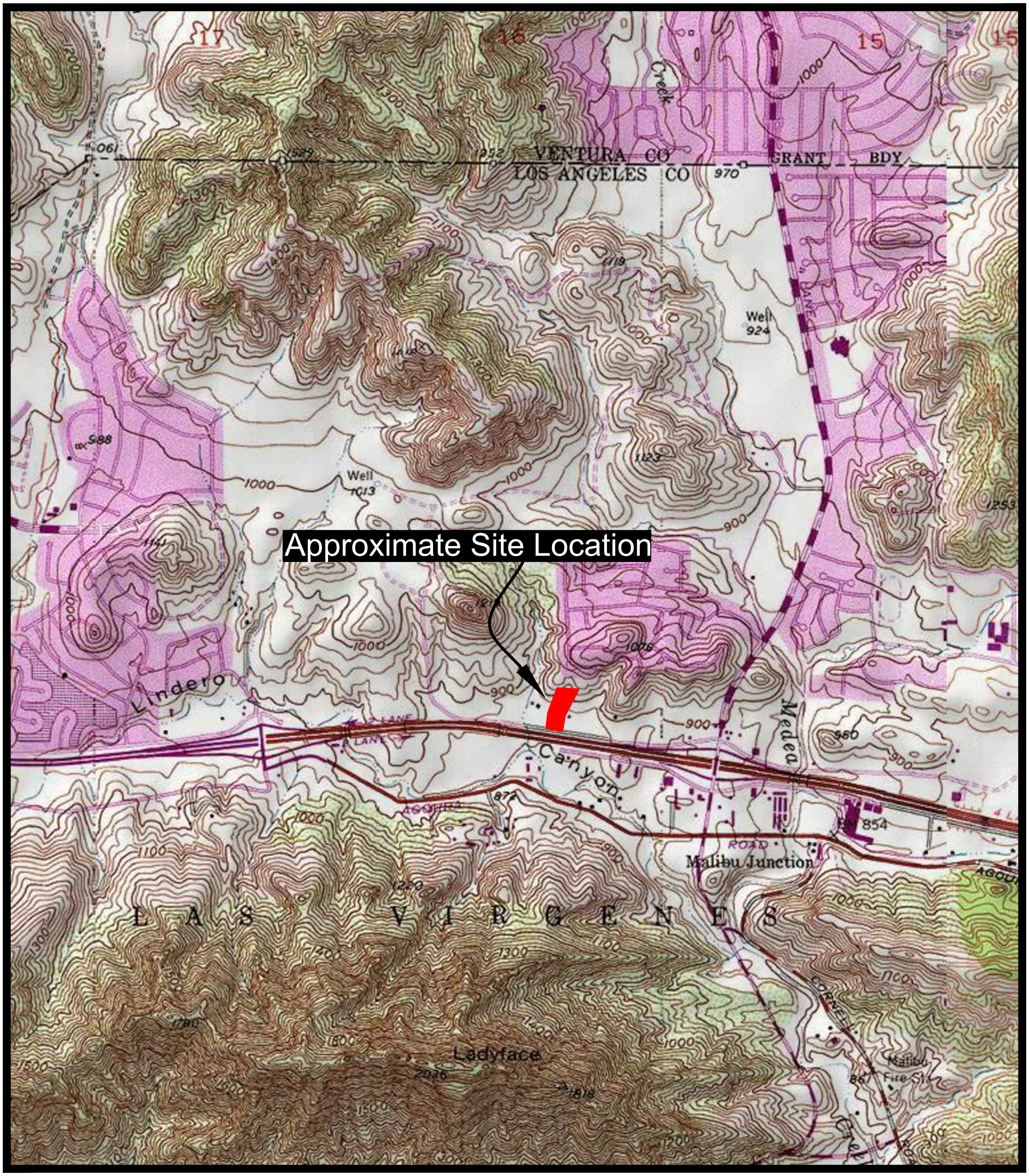
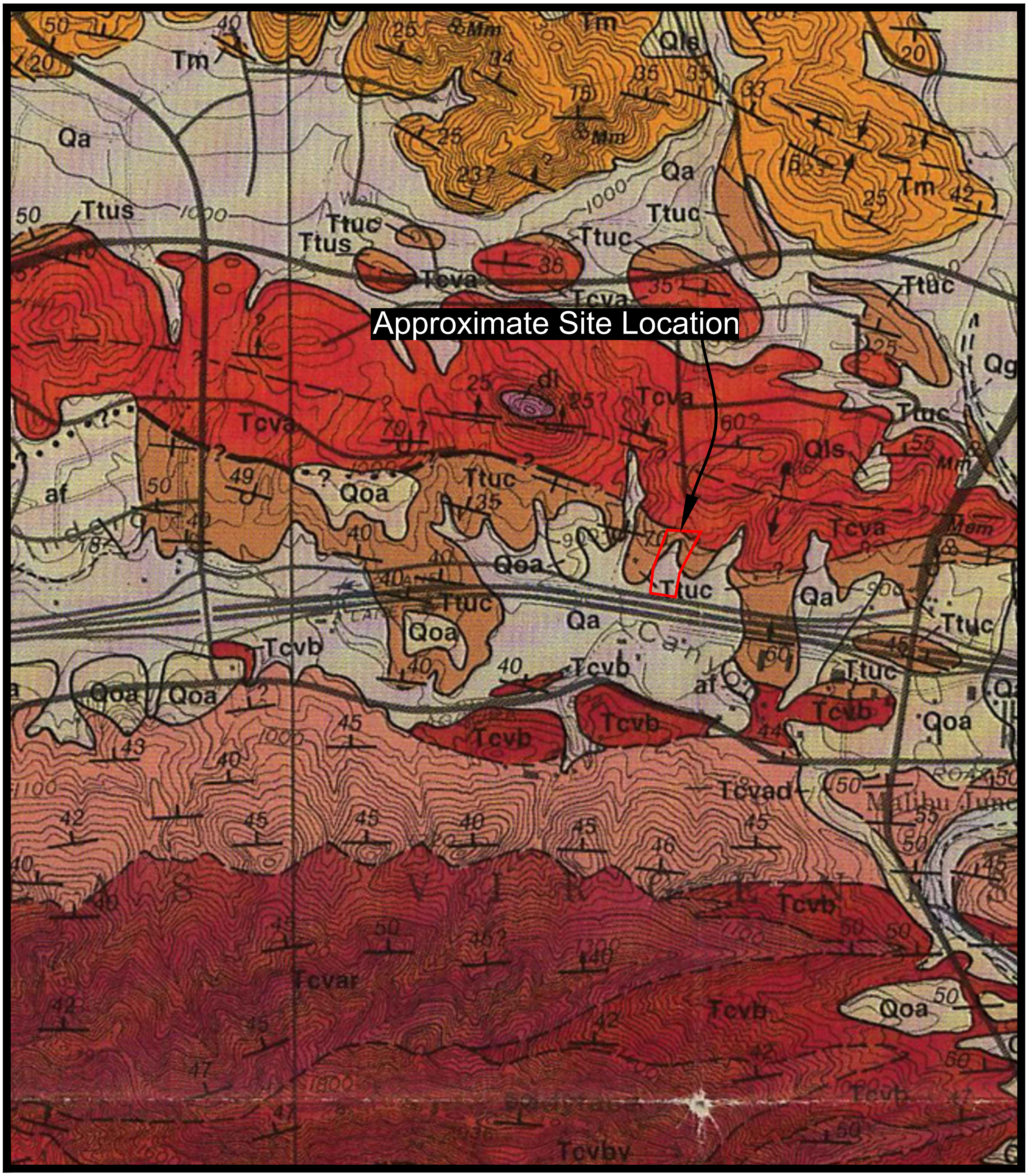


Source
United States Geological Survey, Thousand Oaks Quadrangle, California-Ventura County and Los Angeles County. 7.5 Minute Series (Topographic)

SITE VICINITY MAP

29555 Canwood Street,
Unit F Agoura Hills, California 91301

Gorian & Associates, Inc. *Applied Earth Sciences*		
Job No: 3187-0-0-100		Date: February 2021
Scale: NTS	Drawn by: Approved by:	Figure 1



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EXPLANTATION

Qa - Surficial Sediments. Alluvial gravel, sand and clay of valley areas. (Holocene)
Qoa - Older Surficial Sediments. Unconsolidated to weakly
Ttuc - Upper Topanga Formation. Locally contains calcareous concretions or lenses includes few thin sandstone strata
Tcva - Conejo Volcanics. Andesitic flows and breccias

REGIONAL GEOLOGIC MAP

29555 Canwood Street,
Unit F Agoura Hills, California 91301

Gorian & Associates, Inc.
Applied Earth Sciences

Job No: 3187-0-0-100		Date: May 2021
Scale: NTS	Drawn by: Approved by:	Figure 2

APPENDIX A

LOGS OF SUBSURFACE DATA

Project: MTC, 29555 Canwood Street, Agoura Hills

Work Order: 3187-0-0-100

SUBSURFACE LOG

Excavation Number: B-1

Page Number: 1

Date(s) Excavated 2/3/21	Logged By CHD	Excavation Location See Geotechnical Map	Approximate Surface Elevation 903½'±
Excavation Dimension 24" Dia.	Equipment Contractor Tri-Valley Drilling	Equipment Type Rig #7	Hammer Data SEE NOTE*

Elevation / Depth (ft.)	Blow Counts	Moisture Content (% dry weight)	Dry Density (pcf)	USCS	Description	Remarks
0				CL	TOPSOIL: Brown silty CLAY, trace sand (moist, stiff). Some shale fragments.	
	5	20.5	104		At 2'; becoming very stiff to hard. Expansive.	
900						
5	8	--	--		TOPANGA FORMATION: Yellowish brown clayey SILTSTONE (damp). Interbedded with yellowish brown silty fine-grained SANDSTONE from 5' to 7'. Weathered.	
	5	30.6	93		At 7'; becoming interbedded with olive gray silty Claystone. Some iron oxide staining. (Moist).	ATTITUDE ON BEDDING @ 7½'; N10°E/86°N
895						
10	6	32.0	91		At 10'; becoming interbedded with gray clayey Siltstone and yellowish brown Claystone.	@ 10'; N60°E/43°NW
890						
15	6	21.8	101			@ 14'; E-W/53°NW
					Dark gray clayey SILTSTONE (damp). Locally interbedded with Claystone and light gray silty fine-grained Sandstone.	@ 17'; N70°W/62°NE
885						
20	15	18.3	110		At 20'; 1" thick gray silty fine-grained Sandstone interbed.	
					At 22'; 2" thick gray silty fine-grained Sandstone interbed.	@ 22'; N65°W/63°NE
880						
25	11	16.7	115		At 25'; 2" thick gray silty fine-grained Sandstone interbed.	@ 25'; N65°W/55°NE
875						
30	21	17.6	112		At 30'; 1" thick gray silty fine-grained Sandstone interbed.	@ 30'; N70°W/50°NE
870						
					At 33'; 1" thick gray silty fine-grained Sandstone interbed.	@ 33'; N65°W/60°NE
35	24	17.1	111			
865						

Project: MTC, 29555 Canwood Street, Agoura Hills

Work Order: 3187-0-0-100

SUBSURFACE LOG

Excavation Number: B-1

Page Number: 2

Elevation / Depth (ft.)	Bulk	Sample Type	Blow Counts	Moisture Content (% dry weight)	Dry Density (pcf)	USCS	Soil / Lithology	Description	Remarks
40		■	21	16.0	110		▨		
								Total Depth 41' No Caving Observed No Groundwater Encountered Downhole logged to 37' Backfilled with cuttings and tamped. *NOTE: KELLY WEIGHTS 0-26' 3390# 26'-52' 2280#	

Elevation scale: 860, 855, 850, 845, 840, 835, 830, 825

Depth scale: 40, 45, 50, 55, 60, 65, 70, 75, 80

GORIAN & ASSOCIATES, INC.

Project: MTC, 29555 Canwood Street, Agoura Hills

Work Order: 3187-0-0-100

SUBSURFACE LOG

Excavation Number: B-2

Page Number: 1

Date(s) Excavated 2/3/21	Logged By CHD	Excavation Location See Geotechnical Map	Approximate Surface Elevation 887'±
Excavation Dimension 24" Dia.	Equipment Contractor Tri-Valley Drilling	Equipment Type Rig #7	Hammer Data SEE NOTE*

Elevation / Depth (ft.)	Bulk	Sample Type	Blow Counts	Moisture Content (% dry weight)	Dry Density (pcf)	USCS	Soil / Lithology	Description	Remarks
0						CL		TOPSOIL: Brown silty CLAY, trace sand (moist, stiff to very stiff). Some shale fragments. Expansive.	
885			1	27.4	93				
								TOPANGA FORMATION: Yellowish brown SILTSTONE (damp), interbedded with clayey SILTSTONE. Weathered. (Damp). Fractured. Manganese and iron oxide staining At 4'; thin Claystone interbed.	ATTITUDE ON BEDDING @ 4'; N80°E/42°NW
5			5	21.3	100				
880			7	25.2	95			At 7½'; becoming interbedded with olive brown to light gray Siltstone and clayey Siltstone.	
10			9	16.6	104			At 10'; locally interbedded with very dark gray clayey Siltstone (moist).	
875								At 12'; thin lamination.	@ 12'; N65°W/55°NE
15			8/6"	15.8	93.8			Below 15'; indurated yellowish brown limy Siltstone (damp).	
870								At 17'; crowd used. Slow drilling.	
20								Total Depth 19½' (Practical Refusal) No Caving Observed No Groundwater Encountered Downhole logged to 16' Backfilled with cuttings and tamped. *NOTE: KELLY WEIGHTS 0-26' 3390# 26'-52' 2280#	
865									
25									
860									
30									
855									
35									
850									

GORIAN & ASSOCIATES, INC.

Project: MTC; 29555 Canwood Street, Agoura Hills

Work Order: 3187-0-0-100

SUBSURFACE LOG

Excavation Number: B-3

Page Number: 1

Date(s) Excavated 2/3/21	Logged By CHD	Excavation Location See Geotechnical Map	Approximate Surface Elevation 871'±
Excavation Dimension 24" Dia.	Equipment Contractor Tri-Valley Drilling	Equipment Type Rig #7	Hammer Data SEE NOTE*

Elevation / Depth (ft.)	Bulk	Sample Type	Blow Counts	Moisture Content (% dry weight)	Dry Density (pcf)	USCS	Soil / Lithology	Description	Remarks
0						CL		ARTIFICIAL FILL: Brown silty CLAY mottled with dark brown silty CLAY (wet, stiff).	
870	■	■	1	22.2	101	CL		TOPSOIL: Brown silty CLAY (moist, stiff to very stiff).	
5	■	■	3	25.4	98			TOPANGA FORMATION: Yellowish brown clayey SILTSTONE interbedded with yellowish silty fine-grained SANDSTONE (locally). (Moist). Weathered. Fractured. At 5'; becoming interbedded with brown to gray Claystone.	
865		■	3	28.9	93				ATTITUDE ON BEDDING @ 7'; E-W/40°N
10		■	3	26.4	97				@10'; N80°W/48°NE
860								Gray to dark gray clayey SILTSTONE to CLAYSTONE (moist). Locally light yellowish brown silty fine-grained Sandstone interbeds and limy Siltstone.	@ 13'; N75°W/37°NE
15		■	9	21.7	108				@ 15'; N70°W/45°NE
855									
20		■	10	21.0	108				
850								Total Depth 21' No Caving Observed No Groundwater Encountered Downhole logged to 17' Backfilled with cuttings and tamped. *NOTE: KELLY WEIGHTS 0-26' 3390# 26'-52' 2280#	
25									
845									
30									
840									
35									
835									

APPENDIX B

LABORATORY TESTING

General

Laboratory test results on selected relatively undisturbed and bulk samples are presented below. Tests were performed to evaluate the physical and engineering properties of the encountered earth materials, including in-situ moisture content and dry density, optimum moisture-maximum dry density relationships, expansion potential, and shear strength parameters. In addition, a near surface sample of the onsite soils was tested for corrosion potential by an independent laboratory.

Field Density and Moisture Tests

In situ dry density and moisture content were determined from the relatively undisturbed drive samples obtained during exploratory operations. The test results and a detailed description of the soils encountered are shown on the attached Logs of Subsurface Data, Appendix A.

Maximum Density-Optimum Moisture

Maximum density/optimum moisture tests (compaction characteristics) were performed on two selected bulk samples of the encountered materials. The tests were performed in general accordance with ASTM D 1557 test method. The results are as follows:

Sample	Visual Soil Classification	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
B-1 @ 0-1''	Brown silty CLAY, trace sand	107.2	19.2
B-2 @ 4'	Yellowish brown Siltstone	98.5	21.2

Soil Expansion Test

Two representative samples of the encountered earth materials were tested for expansiveness using the Expansion Index Test method (ASTM D4829). The results are as follows:

Sample	Visual Soil Classification	Expansion Index	Expansion Range
B-1 @ 0-1'	Brown silty CLAY, trace sand	126	91 - 130
B-2 @ 4'	Yellowish brown Siltstone	67	51 - 90

Direct Shear Test

Strain controlled direct shear testing was performed on two undisturbed drive samples and one remolded sample. The sample sets were saturated prior to shearing under axial loads ranging from 920 to 3,680 psf. The shear strength results are presented as graphic summaries. Also, attached are the shear tests performed for Gorian, 2007.

Soil Corrosivity

The results of the analytical laboratory testing to evaluate the potential for soil corrosion are presented in this Appendix. The testing was performed on a soil sample considered to represent the onsite soils. From ACI Table 19.3.1.1 the evaluated soil is categorized as Class S0. The required concrete design requirements for this exposure class can be obtained from ACI Table 19.3.2.1. The site soils are considered very severely corrosive to metals as determined from Table 1. For specific recommendations a corrosion engineer should be consulted.

ACI Table 19.3.1.1 – Exposure Categories and Classes

Category	Class	Water-soluble sulfate (SO_4^{2-}) in soil, percent by mass	Dissolved sulfate (SO_4^{2-}) in water, ppm[1]
Sulfate (S)	S0	$SO_4^{2-} < 0.10$	$SO_4^{2-} < 150$
	S1	$0.10 \le SO_4^{2-} < 0.20$	$150 \le SO_4^{2-} < 1500$ or seawater
	S2	$0.20 \le SO_4^{2-} < 2.00$	$1500 \le SO_4^{2-} < 10{,}000$
	S3	$SO_4^{2-} > 2.00$	$SO_4^{2-} > 10{,}000$

1 ppm (parts per million) = milligrams per kilogram mg/kg of dry soil weight

ACI Table 19.3.2.1 – Requirements for Concrete by Exposure Class

Exposure Class	Maximum *w/cm*	Minimum f_c', psi	Cementitious materials - Types			Calcium chloride admixture
			ASTM C150	ASTM C595	ASTM C1157	
S0	N/A	2500	No type restriction	No type restriction	No type restriction	No restriction
S1	0.50	4000	II	Types IP, IS, or IT with (MS) designation	MS	No restriction
S2	0.45	4500	V	Types IP, IS, or IT with (MS) designation	HS	Not permitted
S3	0.45	4500	V plus pozzolan or slag cement	Types IP, IS, or IT with (MS) designation plus pozzolan or slag cement	HS plus pozzolan or slab cement	Not permitted

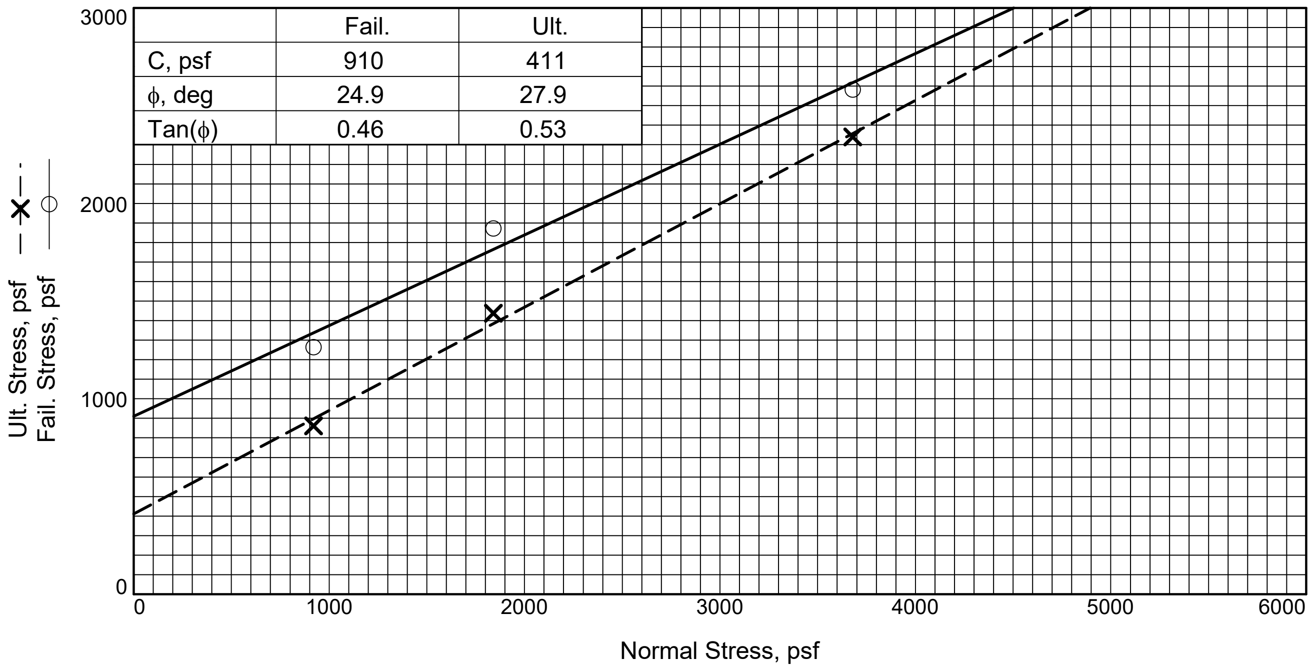
ACI Tables 19.3.1.1 and 19.3.2.1 - ACI 318-14 Building Code Requirements for Structural Concrete

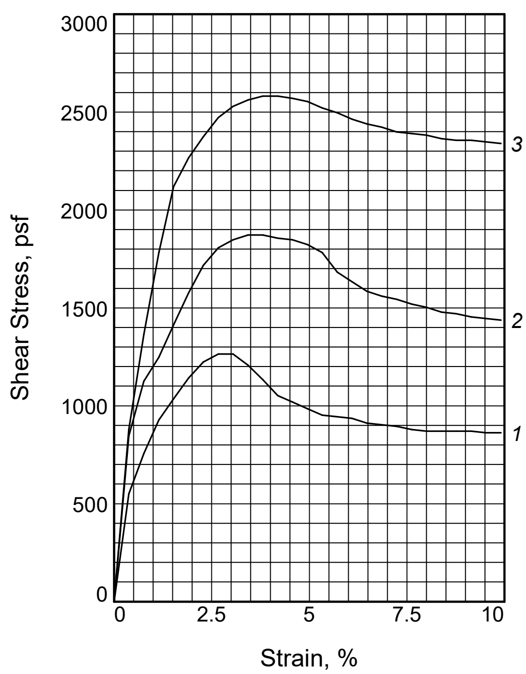
Table 1. Relationship Between Soil Resistivity and Soil Corrosivity

Soil Resistivity, ohm-cm	Classification of Soil Corrosiveness
0 to 900	Very severe corrosion
900 to 2,300	Severely corrosive
2,300 to 5,000	Moderately corrosive
5,000 to 10,000	Mildly corrosive
10,000 to >10,000	Very mildly corrosive

F. O. Waters, Soil Resistivity Measurements for Corrosion Control, Corrosion. 1952, Vol, No. 12, 1952, p. 407.

	Fail.	Ult.
C, psf	910	411
ϕ, deg	24.9	27.9
Tan(ϕ)	0.46	0.53





	Sample No.	1	2	3
Initial	Water Content, %	N/A	N/A	N/A
	Dry Density, pcf	N/A	N/A	N/A
	Saturation, %	N/A	N/A	N/A
	Void Ratio	N/A	N/A	N/A
	Diameter, in.	2.63	2.63	2.63
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	N/A	N/A	N/A
	Dry Density, pcf			
	Saturation, %			
	Void Ratio			
	Diameter, in.			
	Height, in.			
	Normal Stress, psf	920	1840	3680
	Fail. Stress, psf	1265	1872	2582
	Strain, %	3.0	3.8	4.2
	Ult. Stress, psf	862	1437	2340
	Strain, %	9.9	9.9	9.9
	Strain rate, in./min.	0.020	0.020	0.020

Sample Type: Relatively Undisturbed

Description: Clayey SILTSTONE interbedded w/ silty claystone

Specific Gravity=

Remarks:

Figure ________

Client: MTC

Project: MTC, 29555 Canwood Street, Agoura Hills

Location: B-1

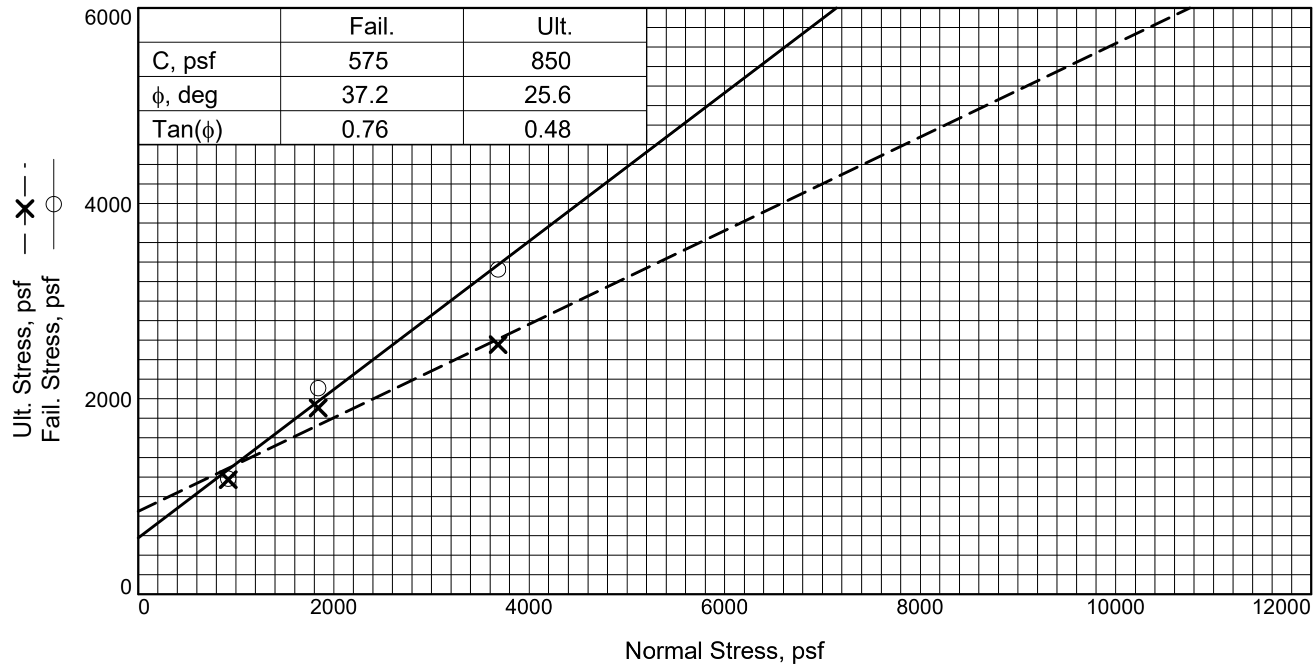
Depth: 7'

Proj. No.: 3187-0-0-100 **Date Sampled:**

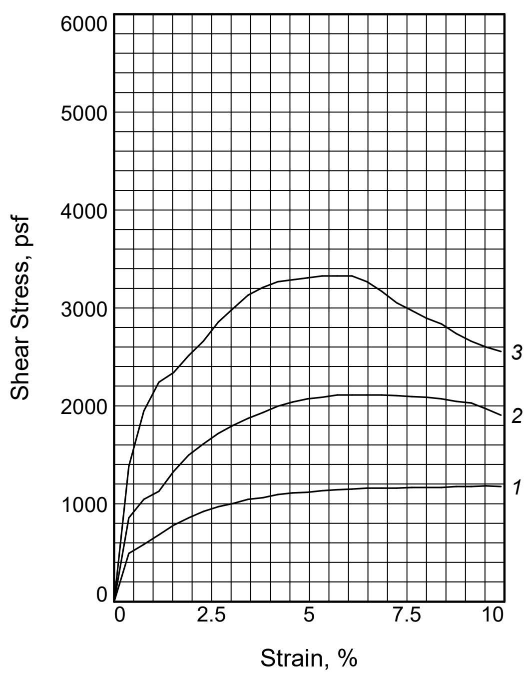
DIRECT SHEAR TEST REPORT

Gorian & Associates

Thousand Oaks, CA



	Fail.	Ult.
C, psf	575	850
ϕ, deg	37.2	25.6
Tan(ϕ)	0.76	0.48



	Sample No.	1	2	3
Initial	Water Content, %	N/A	N/A	N/A
	Dry Density, pcf	N/A	N/A	N/A
	Saturation, %	N/A	N/A	N/A
	Void Ratio	N/A	N/A	N/A
	Diameter, in.	2.63	2.63	2.63
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	N/A	N/A	N/A
	Dry Density, pcf			
	Saturation, %			
	Void Ratio			
	Diameter, in.			
	Height, in.			
	Normal Stress, psf	920	1840	3680
	Fail. Stress, psf	1182	2110	3325
	Strain, %	9.5	6.9	6.1
	Ult. Stress, psf	1174	1905	2554
	Strain, %	9.9	9.9	9.9
	Strain rate, in./min.	0.020	0.020	0.020

Sample Type: Relatively Undisturbed

Description: Clayey SILTSTONE interbedded w/ claystone and sandstone

Specific Gravity=

Remarks:

Figure ________

Client: MTC

Project: MTC, 29555 Canwood Street, Agoura Hills

Location: B-1

Depth: 15'

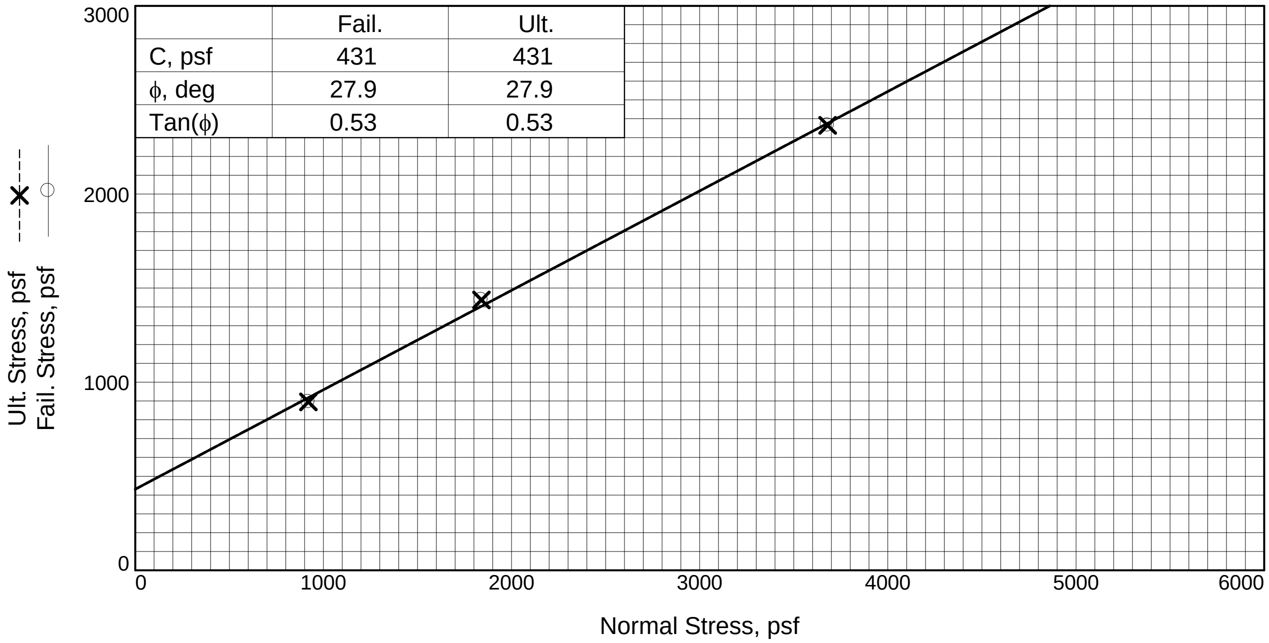
Proj. No.: 3187-0-0-100 **Date Sampled:**

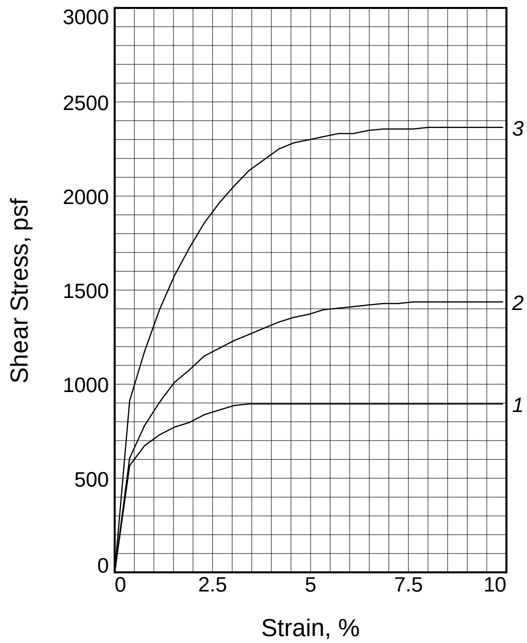
DIRECT SHEAR TEST REPORT

Gorian & Associates

Thousand Oaks, CA

	Fail.	Ult.
C, psf	431	431
ϕ, deg	27.9	27.9
Tan(ϕ)	0.53	0.53





Sample No.		1	2	3
Initial	Water Content, %	N/A	N/A	N/A
	Dry Density, pcf	N/A	N/A	N/A
	Saturation, %	N/A	N/A	N/A
	Void Ratio	N/A	N/A	N/A
	Diameter, in.	2.63	2.63	2.63
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	37.4	37.4	37.4
	Dry Density, pcf			
	Saturation, %			
	Void Ratio			
	Diameter, in.			
	Height, in.			
Normal Stress, psf		920	1840	3680
Fail. Stress, psf		895	1437	2365
Strain, %		9.9	9.9	9.9
Ult. Stress, psf		895	1437	2365
Strain, %		9.9	9.9	9.9
Strain rate, in./min.		0.020	0.020	0.020

Sample Type: Remolded
Description:

Specific Gravity=
Remarks: 2/18/21

Figure ________

Client: MTC

Project: MTC, 29555 Canwood Street, Agoura Hills

Location: B-2
Depth: 4.0'
Proj. No.: 3187-0-0-100 **Date Sampled:**

DIRECT SHEAR TEST REPORT
Gorian & Associates, Inc.
Thousand Oaks, CA

Tested By: CA

Soil Analysis Lab Results

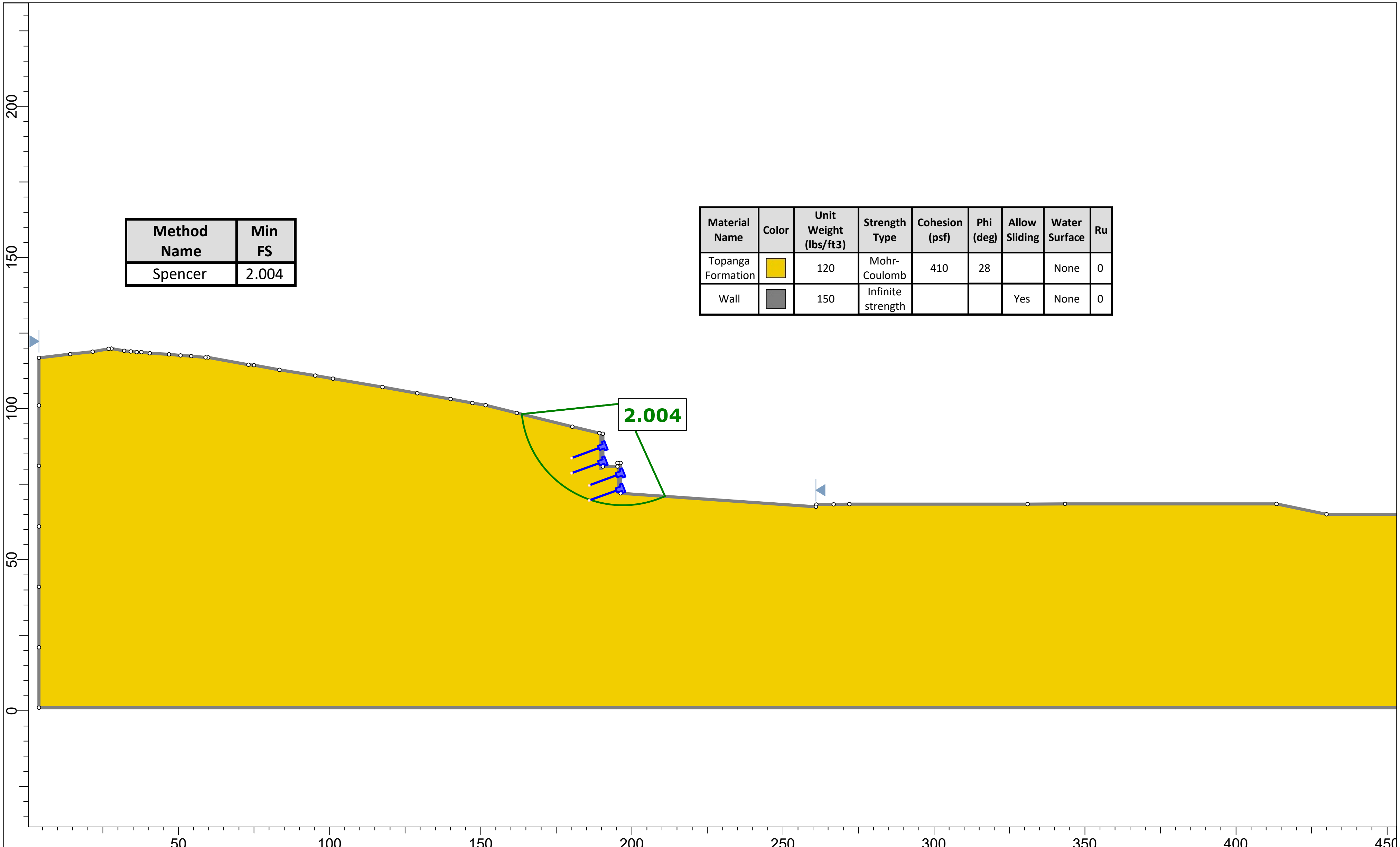
Client: Gorian & Associates, Inc.
Job Name: 29555 Canwool St
Client Job Number: 3187-0-0-100
Project X Job Number: S210208C
February 10, 2021

	Method	ASTM D4327		ASTM D4327		ASTM G187		ASTM D4972	ASTM G200	SM 4500-S2-D	ASTM D4327	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D4327	ASTM D4327
Bore# / Description	**Depth**	**Sulfates** SO_4^{2-}		**Chlorides** Cl^-		**Resistivity** As Rec'd	Minimum	**pH**	**Redox**	**Sulfide** S^{2-}	**Nitrate** NO_3^-	**Ammonium** NH_4^+	**Lithium** Li^+	**Sodium** Na^+	**Potassium** K^+	**Magnesium** Mg^{2+}	**Calcium** Ca^{2+}	**Fluoride** F_2^{--}	**Phosphate** PO_4^{3-}
	(ft)	(mg/kg)	(wt%)	(mg/kg)	(wt%)	(Ohm-cm)	(Ohm-cm)		(mV)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)
B-2	0-1	23.5	0.0023	9.7	0.0010	697	657	7.8	129	<0.01	31.6	9.2	0.02	26.4	3.2	75.1	277.3	2.7	7.0

Cations and Anions, except Sulfide and Bicarbonate, tested with Ion Chromatography
mg/kg = milligrams per kilogram (parts per million) of dry soil weight
ND = 0 = Not Detected | NT = Not Tested | Unk = Unknown
Chemical Analysis performed on 1:3 Soil-To-Water extract

APPENDIX C

SLOPE STABILITY ANALYSES



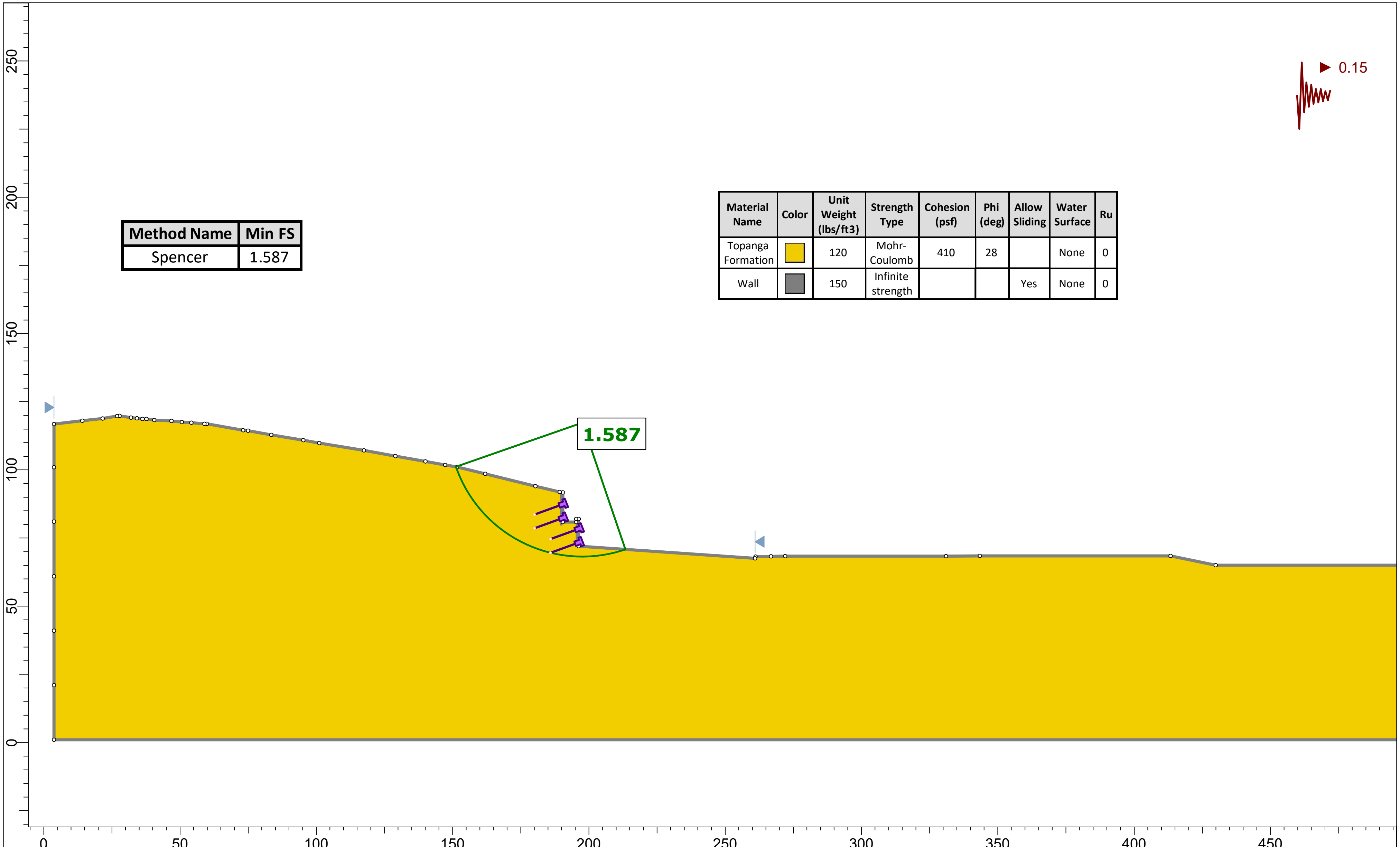
Method Name	Min FS
Spencer	2.004

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Allow Sliding	Water Surface	Ru
Topanga Formation		120	Mohr-Coulomb	410	28		None	0
Wall		150	Infinite strength			Yes	None	0

GORIAN & ASSOCIATES, INC.

SLIDEINTERPRET 9.012

Project	3187-0-0-100; 29555 Canwood Street		
Group	Group 1	*Scenario*	Master Scenario
Drawn By	DM	*Company*	Gorian & Associates, Inc.
Date	2/15/2021, 12:18:59 PM	*File Name*	Section A.slmd



Method Name	Min FS
Spencer	1.587

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Allow Sliding	Water Surface	Ru
Topanga Formation		120	Mohr-Coulomb	410	28		None	0
Wall		150	Infinite strength			Yes	None	0

SLIDEINTERPRET 9.012

GORIAN & ASSOCIATES, INC.

Project	3187-0-0-100; 29555 Canwood Street
Group Group 1	*Scenario* Pseudo-Static
Drawn By DM	*Company* Gorian & Associates, Inc.
Date 2/15/2021, 12:18:59 PM	*File Name* Section A.slmd

Analysis Options

All Open Scenarios

Slices Type:	Vertical
Analysis Methods Used	
	Spencer
Number of slices:	100
Tolerance:	0.005
Maximum number of iterations:	75
Check malpha < 0.2:	Yes
Create Interslice boundaries at intersections with water tables and piezos:	Yes
Initial trial value of FS:	1
Steffensen Iteration:	Yes

Surface Options

All Open Scenarios

Surface Type:	Circular
Search Method:	Auto Refine Search
Divisions along slope:	200
Circles per division:	10
Number of iterations:	10
Divisions to use in next iteration:	50%
Composite Surfaces:	Disabled
Minimum Elevation:	Not Defined
Minimum Depth:	Not Defined
Minimum Area:	Not Defined
Minimum Weight:	Not Defined

Materials

Topanga Formation	
Color	
Strength Type	Mohr-Coulomb
Unit Weight [lbs/ft3]	120
Cohesion [psf]	410
Friction Angle [deg]	28
Water Surface	Assigned per scenario
Ru Value	0
Wall	
Color	
Strength Type	Infinite strength
Unit Weight [lbs/ft3]	150
Allow Sliding Along Boundary	Yes
Water Surface	Assigned per scenario
Ru Value	0

Materials In Use

Material		Group 1	Pseudo-Static
Topanga Formation		✓	✓
Wall		✓	✓

Support

Static	
Color	
Support Type	Soil Nail
Force Application	Active
Force Orientation	Parallel to Reinforcement
Out-of-Plane Spacing [ft]	5
Tensile Capacity [lb]	32725
Plate Capacity [lb]	19000
Bond Strength [lb/ft]	2460
Material Dependent	No

Pseudo-Static	
Color	
Support Type	Soil Nail
Force Application	Active
Force Orientation	Parallel to Reinforcement
Out-of-Plane Spacing [ft]	5
Tensile Capacity [lb]	43630
Plate Capacity [lb]	25000
Bond Strength [lb/ft]	3280
Material Dependent	No

Global Minimums

Group 1 - Master Scenario

Method: spencer

FS	2.004470
Center:	197.119, 101.675
Radius:	33.701
Left Slip Surface Endpoint:	163.603, 98.143
Right Slip Surface Endpoint:	210.988, 70.961
Resisting Moment:	2.10936e+06 lb-ft
Driving Moment:	1.05233e+06 lb-ft
Resisting Horizontal Force:	51268.7 lb
Driving Horizontal Force:	25577.1 lb
Active Support Moment:	-6167.66 lb-ft
Active Horizontal Support Force:	-220.18 lb
Maximum Single Support Force:	234.311 lb
Total Support Force:	234.311 lb
Total Slice Area:	596.483 ft2
Surface Horizontal Width:	47.3845 ft
Surface Average Height:	12.5881 ft

Group 1 - Pseudo-Static

Method: spencer

FS	1.587240
Center:	197.566, 117.278
Radius:	49.105
Left Slip Surface Endpoint:	151.192, 101.128
Right Slip Surface Endpoint:	213.395, 70.794
Resisting Moment:	3.86874e+06 lb-ft
Driving Moment:	2.4374e+06 lb-ft
Resisting Horizontal Force:	68301.5 lb
Driving Horizontal Force:	43031.6 lb
Total Slice Area:	809.297 ft2
Surface Horizontal Width:	62.2025 ft
Surface Average Height:	13.0107 ft

Valid and Invalid Surfaces

Group 1 - Master Scenario

Method: spencer

Number of Valid Surfaces:	693618
Number of Invalid Surfaces:	0

Group 1 - Pseudo-Static

Method: spencer

Number of Valid Surfaces: 761263
Number of Invalid Surfaces: 0

Slice Data

Group 1 - Master Scenario

Global Minimum Query (spencer) - Safety Factor: 2.00447

Slice Number	Width [ft]	Weight [lbs]	Angle of Slice Base [deg]	Base Material	Base Cohesion [psf]	Base Friction Angle [deg]	Shear Stress [psf]	Shear Strength [psf]	Base Normal Stress [psf]	Pore Pressure [psf]	Effective Normal Stress [psf]	Base Vertical Stress [psf]	Effective Vertical Stress [psf]
1	0.473845	84.8467	-81.3155	Topanga Formation	410	28	136.573	273.756	-256.238	0	-256.238	637.888	637.888
2	0.473845	224.11	-76.873	Topanga Formation	410	28	176.663	354.116	-105.102	0	-105.102	652.447	652.447
3	0.473845	321.146	-73.6667	Topanga Formation	410	28	208.37	417.672	14.4281	0	14.4281	725.464	725.464
4	0.473845	399.533	-70.9971	Topanga Formation	410	28	236.91	474.88	122.021	0	122.021	809.946	809.946
5	0.473845	466.419	-68.6534	Topanga Formation	410	28	263.381	527.939	221.811	0	221.811	895.727	895.727
6	0.473845	525.221	-66.5351	Topanga Formation	410	28	288.265	577.819	315.622	0	315.622	979.699	979.699
7	0.473845	577.906	-64.5849	Topanga Formation	410	28	311.848	625.09	404.527	0	404.527	1060.83	1060.83
8	0.473845	625.73	-62.7661	Topanga Formation	410	28	334.322	670.139	489.249	0	489.249	1138.82	1138.82
9	0.473845	669.56	-61.0536	Topanga Formation	410	28	355.827	713.245	570.322	0	570.322	1213.67	1213.67
10	0.473845	710.021	-59.4293	Topanga Formation	410	28	376.473	754.628	648.151	0	648.151	1285.48	1285.48
11	0.473845	747.583	-57.8797	Topanga Formation	410	28	396.344	794.459	723.061	0	723.061	1354.39	1354.39
12	0.473845	782.612	-56.3943	Topanga Formation	410	28	415.51	832.878	795.317	0	795.317	1420.58	1420.58
13	0.473845	815.396	-54.9649	Topanga Formation	410	28	434.031	870.002	865.137	0	865.137	1484.19	1484.19
14	0.473845	846.17	-53.5847	Topanga Formation	410	28	451.954	905.929	932.707	0	932.707	1545.38	1545.38
15	0.473845	875.128	-52.2482	Topanga Formation	410	28	469.323	940.744	998.183	0	998.183	1604.28	1604.28
16	0.473845	902.43	-50.9509	Topanga Formation	410	28	486.173	974.519	1061.71	0	1061.71	1661.03	1661.03
17	0.473845	928.214	-49.6889	Topanga Formation	410	28	502.535	1007.32	1123.39	0	1123.39	1715.73	1715.73
18	0.473845	952.595	-48.4588	Topanga Formation	410	28	518.439	1039.2	1183.34	0	1183.34	1768.48	1768.48
19	0.473845	975.675	-47.2579	Topanga Formation	410	28	533.907	1070.2	1241.66	0	1241.66	1819.39	1819.39
20	0.473845	997.541	-46.0837	Topanga Formation	410	28	548.963	1100.38	1298.41	0	1298.41	1868.55	1868.55
21	0.473845	1018.27	-44.934	Topanga Formation	410	28	563.625	1129.77	1353.69	0	1353.69	1916.01	1916.01
22	0.473845	1037.93	-43.8068	Topanga Formation	410	28	577.911	1158.4	1407.55	0	1407.55	1961.87	1961.87
23	0.473845	1056.58	-42.7006	Topanga Formation	410	28	591.837	1186.32	1460.05	0	1460.05	2006.19	2006.19
24	0.473845	1074.28	-41.6137	Topanga Formation	410	28	605.419	1213.54	1511.25	0	1511.25	2049.02	2049.02
25	0.473845	1091.07	-40.5449	Topanga Formation	410	28	618.668	1240.1	1561.19	0	1561.19	2090.42	2090.42
26	0.473845	1106.99	-39.4928	Topanga Formation	410	28	631.596	1266.02	1609.93	0	1609.93	2130.45	2130.45
27	0.473845	1122.09	-38.4565	Topanga Formation	410	28	644.216	1291.31	1657.5	0	1657.5	2169.13	2169.13
28	0.473845	1136.4	-37.4348	Topanga Formation	410	28	656.537	1316.01	1703.95	0	1703.95	2206.54	2206.54

29	0.473845	1149.95	-36.4269	Topanga Formation	410	28	668.568	1340.12	1749.3	0	1749.3	2242.7	2242.7
30	0.473845	1162.77	-35.4319	Topanga Formation	410	28	680.317	1363.68	1793.6	0	1793.6	2277.65	2277.65
31	0.473845	1174.9	-34.4491	Topanga Formation	410	28	691.794	1386.68	1836.88	0	1836.88	2311.43	2311.43
32	0.473845	1186.35	-33.4777	Topanga Formation	410	28	703.005	1409.15	1879.13	0	1879.13	2344.05	2344.05
33	0.473845	1197.14	-32.5171	Topanga Formation	410	28	713.959	1431.11	1920.42	0	1920.42	2375.56	2375.56
34	0.473845	1207.3	-31.5667	Topanga Formation	410	28	724.655	1452.55	1960.76	0	1960.76	2405.99	2405.99
35	0.473845	1216.85	-30.6258	Topanga Formation	410	28	735.107	1473.5	2000.16	0	2000.16	2435.35	2435.35
36	0.473845	1225.9	-29.694	Topanga Formation	410	28	745.359	1494.05	2038.81	0	2038.81	2463.85	2463.85
37	0.473845	1234.61	-28.7708	Topanga Formation	410	28	755.491	1514.36	2077	0	2077	2491.83	2491.83
38	0.473845	1242.78	-27.8557	Topanga Formation	410	28	765.399	1534.22	2114.35	0	2114.35	2518.85	2518.85
39	0.473845	1250.39	-26.9482	Topanga Formation	410	28	775.083	1553.63	2150.86	0	2150.86	2544.9	2544.9
40	0.473845	1257.48	-26.048	Topanga Formation	410	28	784.547	1572.6	2186.54	0	2186.54	2570.01	2570.01
41	0.473845	1264.04	-25.1547	Topanga Formation	410	28	793.796	1591.14	2221.41	0	2221.41	2594.18	2594.18
42	0.473845	1270.08	-24.2678	Topanga Formation	410	28	802.836	1609.26	2255.47	0	2255.47	2617.42	2617.42
43	0.473845	1275.63	-23.3871	Topanga Formation	410	28	811.666	1626.96	2288.75	0	2288.75	2639.78	2639.78
44	0.473845	1280.69	-22.5122	Topanga Formation	410	28	820.287	1644.24	2321.28	0	2321.28	2661.26	2661.26
45	0.473845	1285.27	-21.6428	Topanga Formation	410	28	828.713	1661.13	2353.03	0	2353.03	2681.86	2681.86
46	0.473845	1289.38	-20.7787	Topanga Formation	410	28	836.934	1677.61	2384.03	0	2384.03	2701.6	2701.6
47	0.473845	1293.02	-19.9194	Topanga Formation	410	28	844.962	1693.7	2414.27	0	2414.27	2720.47	2720.47
48	0.473845	1296.21	-19.0648	Topanga Formation	410	28	925.741	1855.62	2718.81	0	2718.81	3038.74	3038.74
49	0.473845	1298.95	-18.2146	Topanga Formation	410	28	860.432	1724.71	2472.61	0	2472.61	2755.75	2755.75
50	0.473845	1301.24	-17.3685	Topanga Formation	410	28	867.88	1739.64	2500.69	0	2500.69	2772.14	2772.14
51	0.473845	1303.1	-16.5263	Topanga Formation	410	28	875.144	1754.2	2528.08	0	2528.08	2787.75	2787.75
52	0.473845	1304.53	-15.6877	Topanga Formation	410	28	882.223	1768.39	2554.76	0	2554.76	2802.54	2802.54
53	0.473845	1305.54	-14.8526	Topanga Formation	410	28	889.118	1782.21	2580.76	0	2580.76	2816.54	2816.54
54	0.473845	1306.12	-14.0207	Topanga Formation	410	28	895.828	1795.66	2606.04	0	2606.04	2829.74	2829.74
55	0.473845	1399.84	-13.1918	Topanga Formation	410	28	952.102	1908.46	2818.19	0	2818.19	3041.36	3041.36
56	0.473845	1478.76	-12.3657	Topanga Formation	410	28	1001.29	2007.06	3003.63	0	3003.63	3223.15	3223.15
57	0.473845	1137.56	-11.5422	Topanga Formation	410	28	824.173	1652.03	2335.92	0	2335.92	2504.23	2504.23
58	0.473845	697.092	-10.7212	Topanga Formation	410	28	590.027	1182.69	1453.22	0	1453.22	1564.93	1564.93
59	0.473845	701.994	-9.90233	Topanga Formation	410	28	596.644	1195.96	1478.17	0	1478.17	1582.33	1582.33
60	0.473845	706.5	-9.08551	Topanga Formation	410	28	603.121	1208.94	1502.58	0	1502.58	1599.03	1599.03
61	0.473845	710.613	-8.27056	Topanga Formation	410	28	609.457	1221.64	1526.47	0	1526.47	1615.06	1615.06
62	0.473845	714.335	-7.45729	Topanga Formation	410	28	615.651	1234.05	1549.82	0	1549.82	1630.4	1630.4
63	0.473845	717.668	-6.64552	Topanga Formation	410	28	621.702	1246.18	1572.63	0	1572.63	1645.07	1645.07
64	0.473845	720.614	-5.8351	Topanga Formation	410	28	627.612	1258.03	1594.91	0	1594.91	1659.05	1659.05
65	0.473845	723.175	-5.02584	Topanga Formation	410	28	633.378	1269.59	1616.65	0	1616.65	1672.35	1672.35

66	0.473845	725.354	-4.21759	Topanga Formation	410	28	639.002	1280.86	1637.85	0	1637.85	1684.97	1684.97
67	0.473845	751.513	-3.41018	Topanga Formation	410	28	658.769	1320.48	1712.37	0	1712.37	1751.62	1751.62
68	0.473845	936.367	-2.60344	Topanga Formation	410	28	772.698	1548.85	2141.86	0	2141.86	2177	2177
69	0.473845	932.533	-1.79723	Topanga Formation	410	28	776.021	1555.51	2154.39	0	2154.39	2178.74	2178.74
70	0.473845	226.345	-0.991364	Topanga Formation	410	28	356.993	715.582	574.718	0	574.718	580.895	580.895
71	0.473845	224.757	-0.185699	Topanga Formation	410	28	358.164	717.928	579.129	0	579.129	580.29	580.29
72	0.473845	222.79	0.61993	Topanga Formation	410	28	359.122	719.85	582.743	0	582.743	578.857	578.857
73	0.473845	220.444	1.42568	Topanga Formation	410	28	359.865	721.338	585.541	0	585.541	576.585	576.585
74	0.473845	217.718	2.23172	Topanga Formation	410	28	360.387	722.384	587.51	0	587.51	573.465	573.465
75	0.473845	214.613	3.03819	Topanga Formation	410	28	360.683	722.978	588.625	0	588.625	569.482	569.482
76	0.473845	211.128	3.84527	Topanga Formation	410	28	360.748	723.109	588.872	0	588.872	564.625	564.625
77	0.473845	207.261	4.65311	Topanga Formation	410	28	360.578	722.767	588.228	0	588.228	558.881	558.881
78	0.473845	203.011	5.46189	Topanga Formation	410	28	360.165	721.94	586.673	0	586.673	552.234	552.234
79	0.473845	198.378	6.27175	Topanga Formation	410	28	359.504	720.614	584.181	0	584.181	544.671	544.671
80	0.473845	193.358	7.08288	Topanga Formation	410	28	358.588	718.778	580.727	0	580.727	536.172	536.172
81	0.473845	187.951	7.89543	Topanga Formation	410	28	357.409	716.416	576.284	0	576.284	526.719	526.719
82	0.473845	182.154	8.70959	Topanga Formation	410	28	355.961	713.513	570.825	0	570.825	516.294	516.294
83	0.473845	175.965	9.52552	Topanga Formation	410	28	354.234	710.052	564.316	0	564.316	504.875	504.875
84	0.473845	169.38	10.3434	Topanga Formation	410	28	352.221	706.016	556.724	0	556.724	492.439	492.439
85	0.473845	162.398	11.1634	Topanga Formation	410	28	349.91	701.385	548.014	0	548.014	478.962	478.962
86	0.473845	155.014	11.9858	Topanga Formation	410	28	347.293	696.139	538.148	0	538.148	464.419	464.419
87	0.473845	147.226	12.8107	Topanga Formation	410	28	344.358	690.255	527.082	0	527.082	448.779	448.779
88	0.473845	139.029	13.6382	Topanga Formation	410	28	341.093	683.71	514.773	0	514.773	432.013	432.013
89	0.473845	130.419	14.4687	Topanga Formation	410	28	337.484	676.477	501.171	0	501.171	414.088	414.088
90	0.473845	121.392	15.3023	Topanga Formation	410	28	333.519	668.529	486.223	0	486.223	394.968	394.968
91	0.473845	111.942	16.1392	Topanga Formation	410	28	329.182	659.835	469.871	0	469.871	374.613	374.613
92	0.473845	102.065	16.9797	Topanga Formation	410	28	324.455	650.36	452.052	0	452.052	352.982	352.982
93	0.473845	91.7554	17.824	Topanga Formation	410	28	319.321	640.07	432.699	0	432.699	330.029	330.029
94	0.473845	81.0062	18.6723	Topanga Formation	410	28	313.76	628.923	411.735	0	411.735	305.702	305.702
95	0.473845	69.8114	19.5249	Topanga Formation	410	28	307.751	616.877	389.078	0	389.078	279.947	279.947
96	0.473845	58.1639	20.3819	Topanga Formation	410	28	301.268	603.882	364.64	0	364.64	252.707	252.707
97	0.473845	46.0565	21.2438	Topanga Formation	410	28	294.285	589.886	338.318	0	338.318	223.913	223.913
98	0.473845	33.481	22.1108	Topanga Formation	410	28	286.775	574.831	310.001	0	310.001	193.492	193.492
99	0.473845	20.429	22.9831	Topanga Formation	410	28	278.703	558.651	279.571	0	279.571	161.366	161.366
100	0.473845	6.89141	23.8611	Topanga Formation	410	28	258.05	517.254	201.716	0	201.716	87.5733	87.5733

Group 1 - Pseudo-Static

Global Minimum Query (spencer) - Safety Factor: 1.58724

Slice Number	Width [ft]	Weight [lbs]	Angle of Slice Base [deg]	Base Material	Base Cohesion [psf]	Base Friction Angle [deg]	Shear Stress [psf]	Shear Strength [psf]	Base Normal Stress [psf]	Pore Pressure [psf]	Effective Normal Stress [psf]	Base Vertical Stress [psf]	Effective Vertical Stress [psf]
1	0.622025	58.9216	-69.7506	Topanga Formation	410	28	225.804	358.405	-97.0366	0	-97.0366	515.049	515.049
2	0.622025	168.291	-67.7438	Topanga Formation	410	28	240.246	381.327	-53.9251	0	-53.9251	533.133	533.133
3	0.622025	265.651	-65.897	Topanga Formation	410	28	257.743	409.1	-1.69181	0	-1.69181	574.42	574.42
4	0.622025	354.254	-64.1753	Topanga Formation	410	28	276.824	439.385	55.266	0	55.266	627.274	627.274
5	0.622025	435.666	-62.5549	Topanga Formation	410	28	296.683	470.906	114.548	0	114.548	685.805	685.805
6	0.622025	511.021	-61.0185	Topanga Formation	410	28	316.888	502.977	174.865	0	174.865	746.981	746.981
7	0.622025	581.172	-59.5534	Topanga Formation	410	28	337.197	535.212	235.491	0	235.491	809.158	809.158
8	0.622025	646.778	-58.1495	Topanga Formation	410	28	357.469	567.389	296.006	0	296.006	871.412	871.412
9	0.622025	708.364	-56.7991	Topanga Formation	410	28	377.621	599.375	356.163	0	356.163	933.209	933.209
10	0.622025	766.355	-55.4957	Topanga Formation	410	28	397.604	631.093	415.815	0	415.815	994.241	994.241
11	0.622025	821.099	-54.2342	Topanga Formation	410	28	417.388	662.495	474.875	0	474.875	1054.33	1054.33
12	0.622025	872.889	-53.0102	Topanga Formation	410	28	436.958	693.557	533.293	0	533.293	1113.37	1113.37
13	0.622025	921.973	-51.82	Topanga Formation	410	28	456.303	724.263	591.043	0	591.043	1171.32	1171.32
14	0.622025	968.561	-50.6604	Topanga Formation	410	28	475.421	754.608	648.114	0	648.114	1228.15	1228.15
15	0.622025	1012.84	-49.5288	Topanga Formation	410	28	494.312	784.591	704.502	0	704.502	1283.86	1283.86
16	0.622025	1054.96	-48.4229	Topanga Formation	410	28	512.973	814.212	760.212	0	760.212	1338.45	1338.45
17	0.622025	1095.06	-47.3405	Topanga Formation	410	28	531.411	843.477	815.251	0	815.251	1391.95	1391.95
18	0.622025	1133.2	-46.2799	Topanga Formation	410	28	549.607	872.359	869.571	0	869.571	1444.3	1444.3
19	0.622025	1169.35	-45.2395	Topanga Formation	410	28	567.525	900.799	923.059	0	923.059	1495.35	1495.35
20	0.622025	1203.8	-44.2178	Topanga Formation	410	28	585.222	928.888	975.885	0	975.885	1545.34	1545.34
21	0.622025	1236.65	-43.2135	Topanga Formation	410	28	602.705	956.637	1028.08	0	1028.08	1594.32	1594.32
22	0.622025	1267.98	-42.2255	Topanga Formation	410	28	619.977	984.053	1079.64	0	1079.64	1642.3	1642.3
23	0.622025	1297.86	-41.2528	Topanga Formation	410	28	637.044	1011.14	1130.58	0	1130.58	1689.31	1689.31
24	0.622025	1326.36	-40.2943	Topanga Formation	410	28	653.909	1037.91	1180.93	0	1180.93	1735.37	1735.37
25	0.622025	1353.52	-39.3492	Topanga Formation	410	28	670.575	1064.36	1230.68	0	1230.68	1780.5	1780.5
26	0.622025	1379.42	-38.4168	Topanga Formation	410	28	687.046	1090.51	1279.84	0	1279.84	1824.72	1824.72
27	0.622025	1404.09	-37.4962	Topanga Formation	410	28	703.325	1116.35	1328.45	0	1328.45	1868.05	1868.05
28	0.622025	1427.58	-36.5869	Topanga Formation	410	28	719.416	1141.89	1376.48	0	1376.48	1910.51	1910.51
29	0.622025	1449.94	-35.6882	Topanga Formation	410	28	735.321	1167.13	1423.95	0	1423.95	1952.1	1952.1
30	0.622025	1471.2	-34.7994	Topanga Formation	410	28	751.044	1192.09	1470.89	0	1470.89	1992.87	1992.87
31	0.622025	1491.4	-33.9202	Topanga Formation	410	28	766.587	1216.76	1517.29	0	1517.29	2032.81	2032.81
32	0.622025	1510.56	-33.0499	Topanga Formation	410	28	781.954	1241.15	1563.16	0	1563.16	2071.94	2071.94
33	0.622025	1528.73	-32.1882	Topanga Formation	410	28	797.145	1265.26	1608.51	0	1608.51	2110.27	2110.27
34	0.622025	1545.92	-31.3345	Topanga Formation	410	28	812.165	1289.1	1653.35	0	1653.35	2147.83	2147.83

35	0.622025	1562.18	-30.4885	Topanga Formation	410	28	827.015	1312.67	1697.68	0	1697.68	2184.61	2184.61
36	0.622025	1577.51	-29.6499	Topanga Formation	410	28	841.699	1335.98	1741.51	0	1741.51	2220.63	2220.63
37	0.622025	1591.95	-28.8181	Topanga Formation	410	28	856.218	1359.02	1784.85	0	1784.85	2255.91	2255.91
38	0.622025	1605.51	-27.9929	Topanga Formation	410	28	870.573	1381.81	1827.71	0	1827.71	2290.46	2290.46
39	0.622025	1618.21	-27.174	Topanga Formation	410	28	884.768	1404.34	1870.08	0	1870.08	2324.28	2324.28
40	0.622025	1630.09	-26.3611	Topanga Formation	410	28	898.805	1426.62	1911.98	0	1911.98	2357.39	2357.39
41	0.622025	1641.14	-25.5539	Topanga Formation	410	28	912.685	1448.65	1953.42	0	1953.42	2389.8	2389.8
42	0.622025	1651.39	-24.752	Topanga Formation	410	28	926.407	1470.43	1994.39	0	1994.39	2421.51	2421.51
43	0.622025	1660.86	-23.9553	Topanga Formation	410	28	939.984	1491.98	2034.89	0	2034.89	2452.52	2452.52
44	0.622025	1669.56	-23.1635	Topanga Formation	410	28	953.403	1513.28	2074.97	0	2074.97	2482.88	2482.88
45	0.622025	1677.5	-22.3764	Topanga Formation	410	28	966.672	1534.34	2114.59	0	2114.59	2512.55	2512.55
46	0.622025	1684.69	-21.5936	Topanga Formation	410	28	979.795	1555.17	2153.76	0	2153.76	2541.56	2541.56
47	0.622025	1691.16	-20.8151	Topanga Formation	410	28	992.78	1575.78	2192.51	0	2192.51	2569.93	2569.93
48	0.622025	1697.3	-20.0406	Topanga Formation	410	28	1005.78	1596.42	2231.33	0	2231.33	2598.21	2598.21
49	0.622025	1702.94	-19.2699	Topanga Formation	410	28	1018.74	1616.98	2270	0	2270	2626.16	2626.16
50	0.622025	1707.88	-18.5028	Topanga Formation	410	28	1031.56	1637.34	2308.29	0	2308.29	2653.5	2653.5
51	0.622025	1712.13	-17.7391	Topanga Formation	410	28	1044.25	1657.48	2346.17	0	2346.17	2680.22	2680.22
52	0.622025	1715.7	-16.9787	Topanga Formation	410	28	1056.82	1677.42	2383.67	0	2383.67	2706.34	2706.34
53	0.622025	1718.6	-16.2213	Topanga Formation	410	28	1069.25	1697.15	2420.77	0	2420.77	2731.85	2731.85
54	0.622025	1720.83	-15.4669	Topanga Formation	410	28	1081.55	1716.68	2457.51	0	2457.51	2756.77	2756.77
55	0.622025	1722.41	-14.7151	Topanga Formation	410	28	1093.73	1736.01	2493.86	0	2493.86	2781.11	2781.11
56	0.622025	1723.34	-13.966	Topanga Formation	410	28	1105.79	1755.15	2529.86	0	2529.86	2804.87	2804.87
57	0.622025	1723.62	-13.2193	Topanga Formation	410	28	1117.72	1774.09	2565.48	0	2565.48	2828.04	2828.04
58	0.622025	1723.27	-12.4748	Topanga Formation	410	28	1129.54	1792.85	2600.75	0	2600.75	2850.65	2850.65
59	0.622025	1722.28	-11.7325	Topanga Formation	410	28	1141.24	1811.42	2635.68	0	2635.68	2872.69	2872.69
60	0.622025	1720.67	-10.9922	Topanga Formation	410	28	1152.82	1829.8	2670.26	0	2670.26	2894.18	2894.18
61	0.622025	1718.43	-10.2538	Topanga Formation	410	28	1164.29	1848.01	2704.5	0	2704.5	2915.12	2915.12
62	0.622025	1842.88	-9.51704	Topanga Formation	410	28	1240.88	1969.58	2933.14	0	2933.14	3141.18	3141.18
63	0.622025	1939.6	-8.78189	Topanga Formation	410	28	1304.98	2071.31	3124.47	0	3124.47	3326.07	3326.07
64	0.622025	952.226	-8.04819	Topanga Formation	410	28	800.648	1270.82	1618.98	0	1618.98	1732.19	1732.19
65	0.622025	914.341	-7.31582	Topanga Formation	410	28	788.796	1252.01	1583.59	0	1583.59	1684.85	1684.85
66	0.622025	920.001	-6.58465	Topanga Formation	410	28	800.09	1269.93	1617.3	0	1617.3	1709.66	1709.66
67	0.622025	925.061	-5.85455	Topanga Formation	410	28	811.322	1287.76	1650.83	0	1650.83	1734.02	1734.02
68	0.622025	929.524	-5.12541	Topanga Formation	410	28	822.49	1305.49	1684.17	0	1684.17	1757.94	1757.94
69	0.622025	933.391	-4.3971	Topanga Formation	410	28	833.597	1323.12	1717.32	0	1717.32	1781.42	1781.42
70	0.622025	936.665	-3.6695	Topanga Formation	410	28	844.645	1340.65	1750.31	0	1750.31	1804.47	1804.47
71	0.622025	965.927	-2.94249	Topanga Formation	410	28	870.984	1382.46	1828.93	0	1828.93	1873.7	1873.7

72	0.622025	1214.22	-2.21596	Topanga Formation	410	28	1026.3	1628.98	2292.58	0	2292.58	2332.29	2332.29
73	0.622025	758.352	-1.48978	Topanga Formation	410	28	767.826	1218.72	1520.99	0	1520.99	1540.96	1540.96
74	0.622025	280.421	-0.763843	Topanga Formation	410	28	488.59	775.51	687.424	0	687.424	693.939	693.939
75	0.622025	277.532	-0.0380278	Topanga Formation	410	28	491.558	780.22	696.283	0	696.283	696.609	696.609
76	0.622025	274.055	0.687781	Topanga Formation	410	28	494.247	784.489	704.312	0	704.312	698.379	698.379
77	0.622025	269.99	1.4137	Topanga Formation	410	28	496.649	788.301	711.482	0	711.482	699.225	699.225
78	0.622025	265.335	2.13985	Topanga Formation	410	28	498.752	791.639	717.758	0	717.758	699.123	699.123
79	0.622025	260.092	2.86634	Topanga Formation	410	28	500.544	794.483	723.107	0	723.107	698.046	698.046
80	0.622025	254.258	3.59329	Topanga Formation	410	28	502.011	796.812	727.488	0	727.488	695.963	695.963
81	0.622025	247.832	4.32082	Topanga Formation	410	28	503.141	798.605	730.859	0	730.859	692.844	692.844
82	0.622025	240.813	5.04905	Topanga Formation	410	28	503.917	799.837	733.176	0	733.176	688.654	688.654
83	0.622025	233.199	5.7781	Topanga Formation	410	28	504.323	800.482	734.389	0	734.389	683.357	683.357
84	0.622025	224.988	6.50809	Topanga Formation	410	28	504.342	800.512	734.445	0	734.445	676.91	676.91
85	0.622025	216.177	7.23914	Topanga Formation	410	28	503.954	799.896	733.288	0	733.288	669.274	669.274
86	0.622025	206.764	7.97138	Topanga Formation	410	28	503.137	798.599	730.85	0	730.85	660.395	660.395
87	0.622025	196.745	8.70494	Topanga Formation	410	28	501.869	796.586	727.062	0	727.062	650.221	650.221
88	0.622025	186.117	9.43993	Topanga Formation	410	28	500.123	793.815	721.851	0	721.851	638.698	638.698
89	0.622025	174.876	10.1765	Topanga Formation	410	28	497.872	790.242	715.133	0	715.133	625.762	625.762
90	0.622025	163.018	10.9148	Topanga Formation	410	28	495.085	785.818	706.811	0	706.811	611.341	611.341
91	0.622025	150.539	11.6549	Topanga Formation	410	28	491.727	780.489	696.789	0	696.789	595.361	595.361
92	0.622025	137.434	12.397	Topanga Formation	410	28	487.761	774.193	684.948	0	684.948	577.734	577.734
93	0.622025	123.698	13.1412	Topanga Formation	410	28	483.144	766.866	671.167	0	671.167	558.37	558.37
94	0.622025	109.324	13.8876	Topanga Formation	410	28	477.83	758.431	655.303	0	655.303	537.162	537.162
95	0.622025	94.3078	14.6365	Topanga Formation	410	28	471.767	748.807	637.203	0	637.203	513.996	513.996
96	0.622025	78.642	15.388	Topanga Formation	410	28	464.896	737.901	616.691	0	616.691	488.743	488.743
97	0.622025	62.3199	16.1421	Topanga Formation	410	28	457.15	725.607	593.572	0	593.572	461.258	461.258
98	0.622025	45.3341	16.8992	Topanga Formation	410	28	448.458	711.81	567.623	0	567.623	431.378	431.378
99	0.622025	27.6768	17.6593	Topanga Formation	410	28	438.734	696.376	538.595	0	538.595	398.92	398.92
100	0.622025	9.33964	18.4226	Topanga Formation	410	28	442.454	702.28	549.699	0	549.699	402.32	402.32

Entity Information

Group 1

Shared Entities

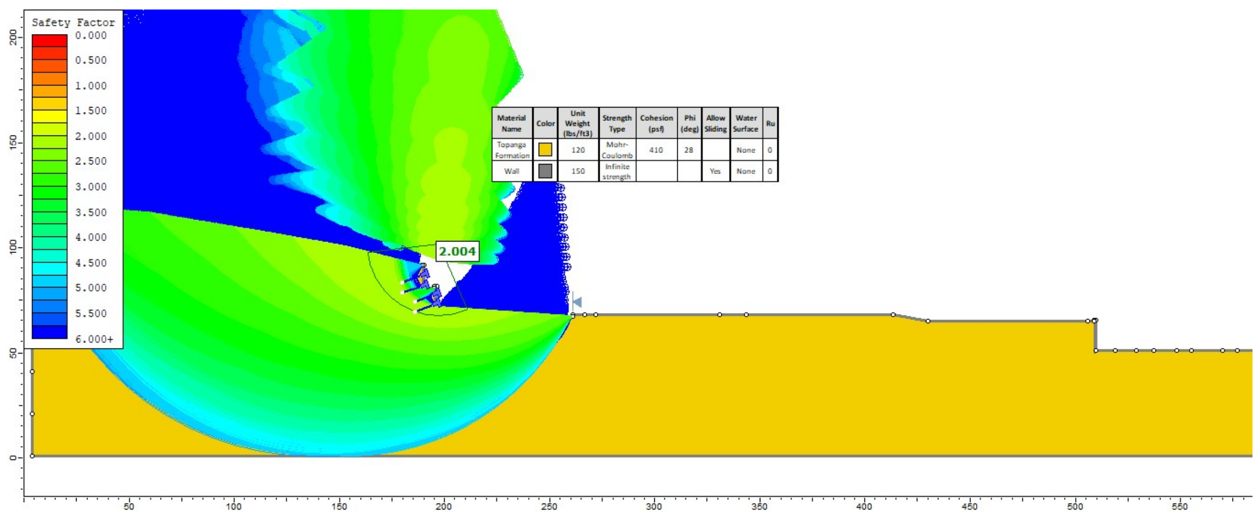
Type	Coordinates (x,y)
	603.795, 50.9923 577.248, 51.2556

External Boundary	570, 51.1846 555.424, 51.3004 548.421, 51.4141 537.448, 51.3731 528.964, 51.3095 519.189, 51.2298 509.92, 51.2298 509.92, 65.6409 508.92, 65.6409 508.92, 65.0014 505.906, 65.0014 429.906, 65.0014 413.366, 68.4393 343.366, 68.4393 331.006, 68.3511 272.006, 68.3511 266.792, 68.2546 261.091, 68.1948 260.952, 67.5022 196.295, 71.9777 196.295, 81.9777 195.295, 81.9777 195.295, 80.8231 190.406, 80.8231 190.406, 91.6758 189.295, 91.8692 180.327, 93.9832 161.963, 98.5515 151.658, 101.05 147.216, 101.796 140.038, 103.102 128.98, 105.055 117.481, 107.106 101.105, 109.846 95.2614, 110.9 83.4213, 112.829 74.9587, 114.319 73.115, 114.484 59.8448, 116.881 58.949, 116.864 54.1481, 117.304 50.6367, 117.537 46.836, 117.918 40.5097, 118.266 37.7254, 118.639 36.2039, 118.631 34.1864, 118.904 32.0271, 119.103 27.8076, 119.821 26.9083, 119.719 21.5982, 118.837 14.1164, 117.967 3.79546, 116.787 3.79546, 100.992 3.79546, 80.9923 3.79546, 60.9923 3.79546, 40.9923 3.79546, 20.9923 3.79546, 0.992306

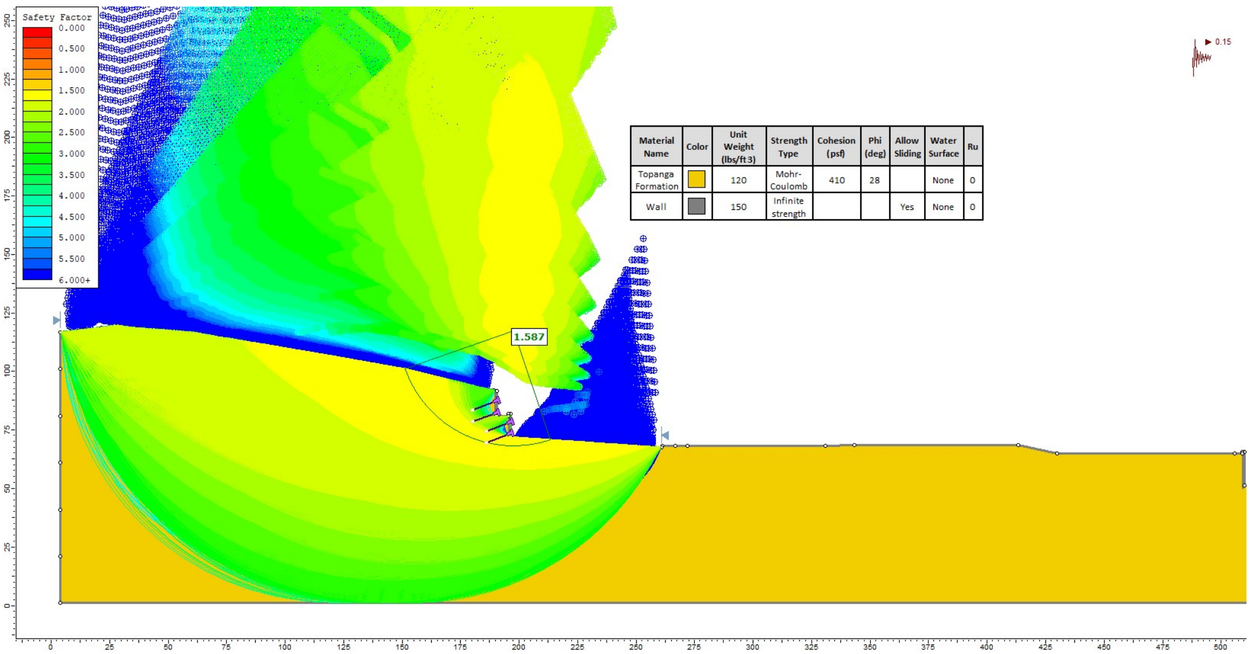
	603.795, 0.992306 603.795, 20.9923 603.795, 40.9923
Material Boundary	195.295, 80.8231 195.295, 78 195.295, 73 195.295, 71.9777 196.295, 71.9777
Material Boundary	190.406, 79.6758 189.406, 79.6758 189.406, 82 189.406, 87 189.406, 91.6758 190.406, 91.6758
Material Boundary	508.92, 65.0014 508.92, 49.6409 509.92, 49.6409 509.92, 51.2298
Material Boundary	190.406, 79.6758 190.406, 80.8231

Report Views

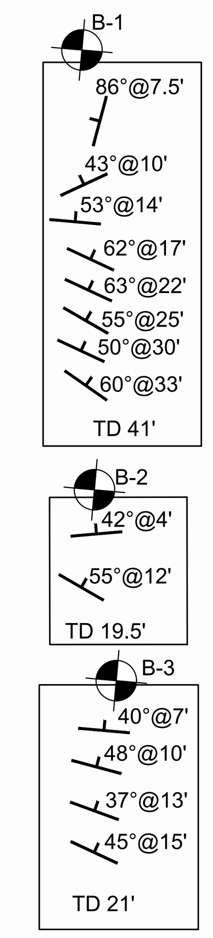
1: Group 1 - Master Scenario - spencer method



2: Group 1 - Pseudo-Static - spencer method



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Allow Sliding	Water Surface	Ru
Topanga Formation		120	Mohr-Coulomb	410	28		None	0
Wall		150	Infinite strength			Yes	None	0



CONSTRUCTION NOTES

2. CONCRETE CURB ONLY
3. CONCRETE CURB AND GUTTER
4. CONCRETE SIDEWALK
5. CONCRETE CURB RAMP
6. CONCRETE DRIVEWAY
7. AC PAVEMENT
8. CONCRETE PAVEMNT
9. STANDARD PARKING STALL STRIPING
10. ACCESSIBLE PARKING STALL STRIPING
11. CONCRETE RAMP
12. LANDSCAPING
13. CONRETE RETAINING WALL 6' MAX HEIGHT
14. SOIL NAIL WALL
15. CATCH BASIN DROP INLET FOR STORM WATER
16. CURB OPENING CATCH BASIN FOR STORM WATER
17. LONGITUDINAL GUTTER
18. LONGITUDINAL GUTTER TRANSITION FOR ADA STALLS

EXPLANATION

af	Artificial Fill
Qal	Alluvium
Tt	Topanga Formation
	Approximate Contact Between Geologic Units
45	Strike & Dip of Bedding
B-3	Approximate Location of 24' Diameter Exploratory boring (This Report)
A A'	Geotechnical Cross Section

SCALE: 1"=30'

GEOTECHNICAL MAP

Gorian & Associates, Inc.
Applied Earth Sciences

Job No: 3187-0-0-100	Date: May 2021
Scale: 1" = 30'	Drawn by: Approved by: / PLATE 1

REGISTERED PROFESSIONAL ENGINEER SCOTT DELANE UHLES No C 72391 CIVIL STATE OF CALIFORNIA

REVISION #	SYMBOL	DESCRIPTION OF CHANGE	APPROVED	DATE

PREPARED BY:
DELANE ENGINEERING
2812 SANTA MONICA BLVD, SUITE 206 SANTA MONICA CA 90404 PHONE: 310.546.5711 WWW.DELANEENGINEERING.COM
SCOTT D. UHLES DATE

CITY OF AGOURA HILLS APPROVAL
REVIEWED BY DATE
ROMIRO ADEVA CITY ENGINEER DATE RCE NO. EXP DATE

CITY OF AGOURA HILLS

SITE PLAN
FOR
CANWOOD OFFICE CAMPUS
SHEET 2 OF 2

SAVE DATE: 4/26/21 TNEWMAN

CITY OF AGOURA HILLS DWG. NO. ____________

Gorian & Associates, Inc. Applied Earth Sciences	
Job No: 3187-0-0-100	Date: May 2021
Scale: 1" = 20' / Drawn by: / Approved by:	PLATE 2

GEOTECHNICAL CROSS SECTION A-A'