GEOTECHNICAL SITE EVALUATION UPDATE REPORT
and
RESPONSE TO CITY OF AGOURA HILLS REVIEW SHEET

SENIOR HOUSING COMMUNITY
VESTING TENTATIVE TRACT NUMBER 71742
APN# 2061-001-025
30800 AGOURA ROAD
AGOURA HILLS, CALIFORNIA

prepared for

Agoura Hills Center Properties, LLC
2985 E Hillcrest Drive #107
Thousand Oaks, CA 91362

GORIAN & ASSOCIATES, INC.

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January 30, 2014

Agoura Hills Center Properties, LLC
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Work Order: 2272-1-0-101

Attention: Mr. Steve Rice

Subject: **GEOTECHNICAL SITE EVALUATION UPDATE REPORT AND RESPONSE TO CITY OF AGOURA HILLS REVIEW SHEET DATED NOVEMBER 11, 2011, SENIOR HOUSING COMMUNITY, VESTING TENTATIVE TRACT NUMBER 71742 (APN# 2061-001-025), 30800 AGOURA ROAD, AGOURA HILLS, CALIFORNIA.**

1. INTRODUCTION

This geotechnical site evaluation update was performed to review the current grading plan and observe the present site conditions for the proposed senior housing community at 30800 Agoura Road in Agoura Hills, California. This report is intended as a stand alone update report and includes the results and recommendations from previous studies of the property by Gorian and Associates, Inc. (see attached reference list). Also, included in this report are responses to the City of Agoura Hills - Geotechnical Review Sheet (GeoDynamics, 2011).

Our current and previous scopes of services included archival research, field exploration and surficial mapping, laboratory testing, and geotechnical analyses as discussed herein. Our understanding of the current proposed development is based a review of the civil engineering plans by HMK Engineering, Inc. The proposed development shown on this plan, which is the basis for our attached Plate 1, was significantly changed since our previous report (Gorian, 2007). The current plan shows development of two residential buildings with subterranean parking, associated surface infrastructure improvements, and widening of Agoura Road. Based on the scope of services performed, the site is considered suitable for the proposed development as outlined herein.

Conditions of the property as encountered during our subsurface exploration and field reconnaissance are described herein and within the attached logs of excavation presented in Appendix A. The area of the proposed development is mainly underlain by a relatively thick sequence of Older Alluvial soils overlying Calabasas Formation bedrock and in low gradient areas residual soils and colluvial / younger alluvial soils mantle the Older Alluvium. Hard volcanic bedrock of the Conejo Volcanics Formation is exposed along the south easternmost site boundary and steeper hillside to the south.

2. PROPOSED DEVELOPMENT

The project currently proposed for the site consists of two multi-unit senior residential buildings with subterranean parking garages at the lower levels. These buildings will be constructed in the eastern and western portions of the 7.1-acre site (refer to Plate 1). A previously proposed community building in the central portion of the development has been deleted. Separate drives to each building with surface parking will extend from Agoura Road to provide vehicular access to the buildings. Agoura Road is to be widened along the northern property line.

The development area will be graded using cut and fill grading. Manufactured slopes (both cut and fill) will be at a 2(h):1(v) gradient. Retaining walls with maximum heights of 27 feet and 10 feet (exterior and internal to buildings, respectively) are also proposed at various locations within the developed area. An inlet structure will be constructed at the northern limit of the western drainage course. An existing debris basin at the eastern limit of the property would be reconfigured as part of the Agoura Road widening.

3. SCOPE OF SERVICES

In accordance with our Proposal Number 5802-10 (dated December 2, 2013) Gorian and Associates, Inc. conducted the following scope of services for our geotechnical site update evaluation. This evaluation was conducted by or under the supervision of a State licensed geotechnical engineer and certified engineering geologist. This evaluation included the following:

3.1 ARCHIVAL REVIEW

A review was performed of the previous referenced geologic and geotechnical engineering reports in our office addressing the site and vicinity. A list of the reports reviewed for this evaluation is included on the attached References section.

3.2 SITE RECONNAISSANCE AND PRIOR EXPLORATION

A site reconnaissance was conducted by a geologist from our office to observe the current site conditions and performance of the natural slopes in the areas of proposed cuts. Previously a series of borings were excavated on site, of which the logs of subsurface exploration are presented in Appendix A.

3.3 LABORATORY TESTING

The results of previously performed laboratory tests from our prior study of the site are presented in Appendix B.

3.4 GEOLOGIC AND GEOTECHNICAL ENGINEERING ANALYSIS AND REPORT PREPARATION

The results of the archival review and prior field exploration programs and laboratory testing programs were used to evaluate the geotechnical engineering factors affecting the current development plan. The *Vesting Tentative Tract Map Number 71742, Located in a Corporated Territory of the County of Los Angeles, State of California* prepared by HMK Engineering, Inc. (Scale: 1”=40’ and dated September 2012) was used in our evaluation of the proposed development and serves as the base map of our revised Geotechnical Map, Plate 1.

This geotechnical report was prepared to summarize the site’s geologic setting and geotechnical conclusions and recommendations for revised site development and construction. In addition, provided herein are responses to City of Agoura Hills - Geotechnical Review Sheet (GeoDynamics 2011).

4. SITE DESCRIPTION AND CURRENT SITE CONDITIONS

The approximately 7.1 acre parcel is south and adjacent Agoura Road, south of the Ventura Freeway (U.S. 101), between the Lindero Canyon Road and Reyes Adobe Road exits (Figure 1). Situated in the western part of the city of Agoura Hills, the site is east of the Oak Ridge Apartments (located at 30856 Agoura Road) and across the street from the Teradyne campus (located at 30801 Agoura Road).

The hillside site is along the north base of Ladyface ridge, in the central Santa Monica Mountains, between an elevation of approximately 955 and 1030 feet above sea level. Low gradient areas characterize the northern part of the site with slopes less than 5:1 (horizontal to vertical). Slopes are steeper in the southern part of the property. Here slopes are typically 3:1 (horizontal to vertical) or less, but limited areas along the southern property line are as steep as 1½:1 (horizontal to vertical).

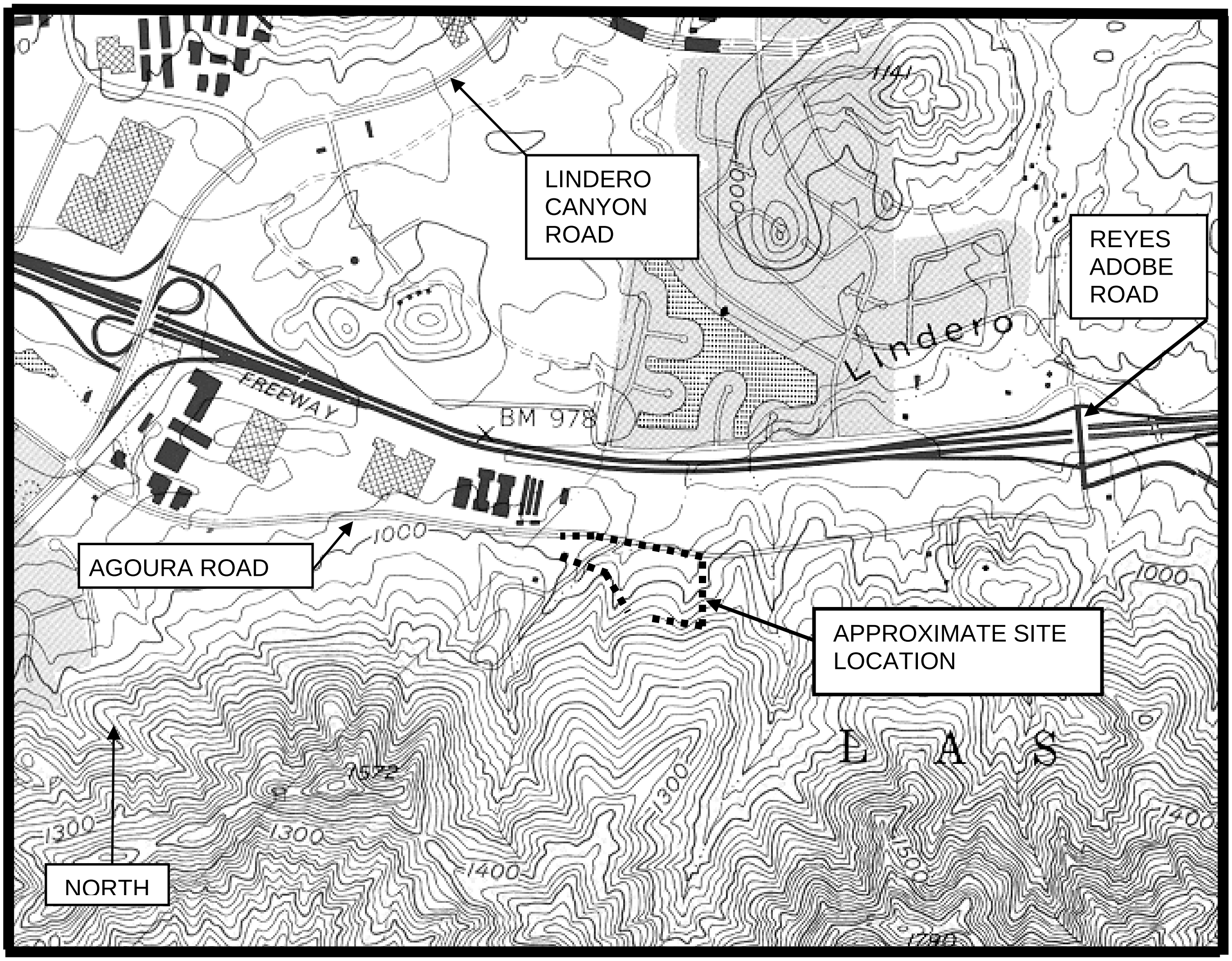

Figure 1. Portion of Thousand Oaks Quadrangle (7.5-minute series topographic) illustrating the approximate location of the site. Scale 1" ≈ 3,100 ft. (use quadrangle map for accurate scale).

Three drainage courses flow northerly across the site. The western drainage course is shown as a blueline stream on the USGS quadrangle map (Figure 1). The stream courses drain to inlets along Agoura Road, which is on a fill berm along much of the site's northern property line. These drainages are tributary to Lindero Canyon creek.

On December 30, 2013, the undersigned engineering geologist performed a site and photo reconnaissance to document current site conditions relative to observed conditions documented in previous evaluations/reports. The following observations of current site conditions were made:

Other than localized minor trash (landscape debris, plastic/paper litter, and occasional trash bags) and dead tree falls/branch trimming debris, the site remains as previously described in our prior reports.

The lower slopes of the property are currently covered by a low growth of seasonal weeds and grasses with occasional clusters of native oaks and chaparral. Valley Oaks are relatively common in this area. In the canyons and steeper slopes of the southern part of the site, coastal live oaks, scrub brush, and chaparral plants are present. Willow and a cottonwood line the stream in the western part of the site. The natural slopes continue to perform well and no signs of surficial instability were observed. At the time of our site visit, all the drainage courses on the property were dry.

In the approximate center of the property, at the north end of an existing north-south drainage is a body of non-certified artificial fill (Plate 1). Where this fill crosses the active drainage channel, an approximately 4 foot deep channel has been eroded. Maps and aerial photographs in our files indicate a residence was previously present in the western part of the site.

5. REGIONAL GEOLOGIC SETTING

The site is in the Santa Monica Mountains an east-west trending mountain range along the southern edge of the Transverse Ranges geomorphic province. This geomorphic province is dominated by active compressional tectonics and characterized by roughly east-west trending ranges and ridges with intervening canyons and valleys. The Santa Monica Mountains consist of a west plunging anticline (a convex upward-shaped fold) and the site is on the northern limb of this anticline along the northern base of Ladyface ridge. This anticline of the Santa Monica Mountains generally consists of Cretaceous and Tertiary rocks with a core of Jurassic metasediments and Cretaceous granitic rocks.

Ladyface ridge is a hogback composed of an interlayered sequence of volcanic and volcaniclastic rocks that are grouped in the Conejo Volcanics, which are of Miocene age. The layers of rock dip to the north at moderate angle (~40 to 60 degrees). North of Ladyface is an area of low relief and rolling hills composed of marine sedimentary rocks and volcanic rocks. These rocks are complexly folded and faulted. Figure 2 is a portion of a portion of a geologic map by Weber (1984) that includes the site.

6. SITE GEOLOGY

Based on our archival review, previous surficial mapping, and subsurface exploration programs, the area of the proposed development is mainly underlain by a relatively thick sequence of Older Alluvial soils. Marine sedimentary rocks assigned to the Calabasas Formation underlie the Older Alluvium and in low gradient areas of the site residual soils and colluvial / younger alluvial soils mantle the Older Alluvium. Along the south easternmost site boundary and the steeper hillside to the south, hard volcanic bedrock of the Conejo Volcanics Formation is exposed. General descriptions of these earth units are presented in the following sections. The areal distribution and spatial relationships of these earth units (except for topsoil / colluvium) are shown on the attached Geotechnical Map, Plate 1 and Cross Sections, Plate 2.

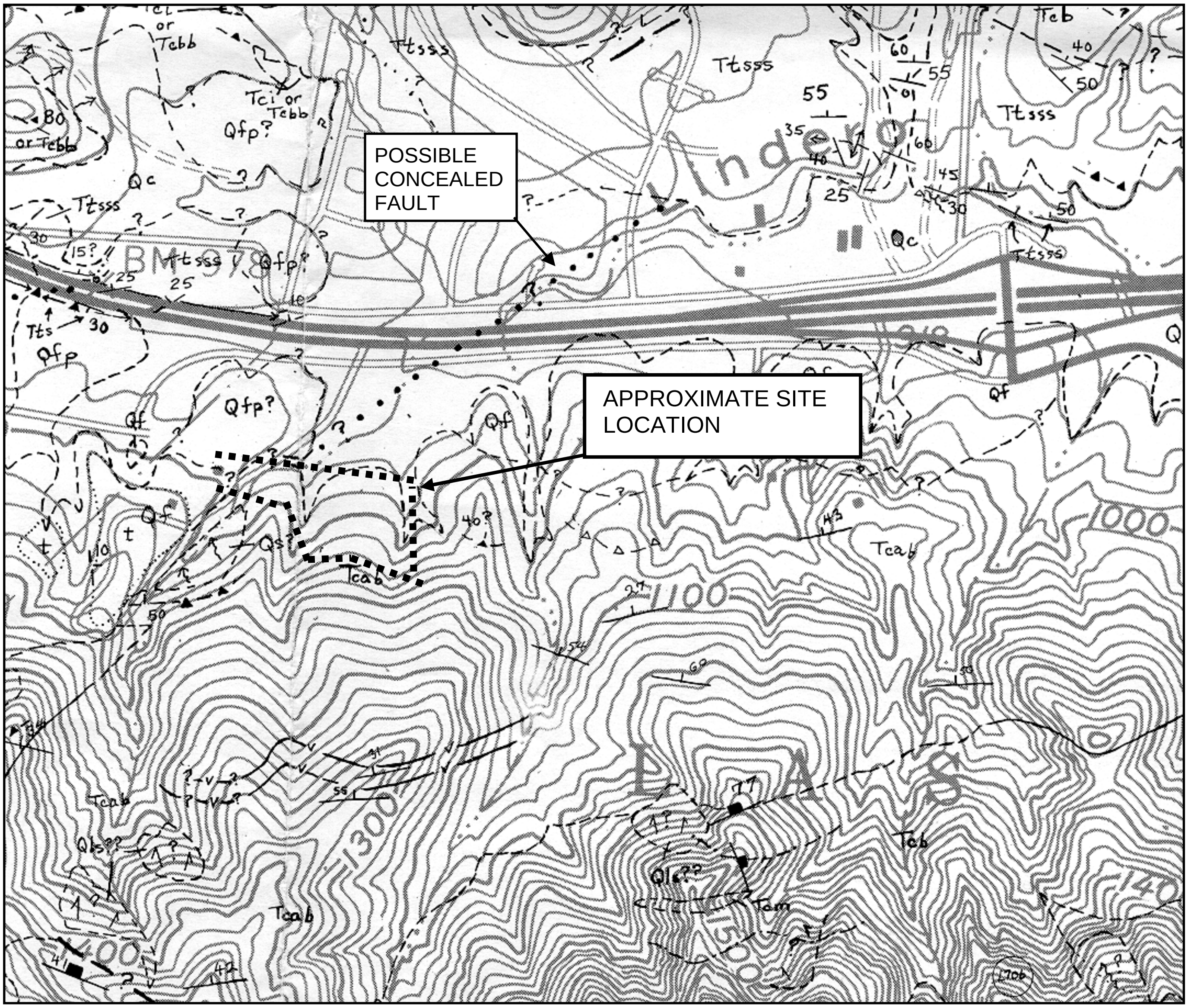

Figure 2. Portion of the geologic map of S½, Thousand Oaks Quadrangle, Ventura and Los Angeles counties, California by Weber and Blackerby (Weber, 1984). Symbols are as follows: Tcab, andesite to dacite flow breccia and agglomerate; Tcb, basalt; Tcbb, basaltic breccia; Tts, siltstone and shale; Ttsss, interlayered siltstone, sandstone, and shale; Qf, fanglomerate; Qfp, flood plain deposits; Qc, colluvium, Qs, bedrock slide.

6.1 CONEJO VOLCANICS (Tcv)

Representing the oldest bedrock unit exposed on and adjacent the site, the Miocene–age Conejo Volcanics underlies the southernmost edge of the site and adjacent steeper hillside ascending Ladyface ridge. As observed in outcrop, the bedrock generally consists of andesitic agglomerate that dips at a moderate angle (27-55 degrees) to the north. Typically, this volcanic bedrock is indurated and stable.

6.2 CALABASAS FORMATION (Tc)

Miocene-age Calabasas Formation underlies the major portion of the property. Although not exposed in outcrop, (being mantled by the surficial Older / Younger Alluvial deposits) this bedrock formation was encountered in all of our exploratory borings, except B-5, at depths ranging from 42.5 feet (B-1) to 10 feet (B-4) below the existing ground surface. As observed in our exploratory borings, the Calabasas Formation generally consist of pale olive to light olive gray to light olive brown silty claystone to claystone occasionally interbedded with very pale brown clayey siltstone and fine grained sandstone. Bedding within the Calabasas Formation bedrock is commonly massive to poorly defined and non-fissile. At depth, the Calabasas Formation becomes dark gray to black in color. The bedrock is typically tightly fractured with manganese and iron oxide staining yet is in a hard and moist condition.

Structurally, the Calabasas Formation in this area is plastically deformed with complexly folded, multi-directionally oriented bedding. Bedding orientations noted during downhole logging in boring B-2 were inclined to the northwest at low angles (10 to 12 degrees) and to the southeast at steep angles (37 degrees). Bedding observed in boring B-3 were inclined to the southwest at moderate to steep angles (24 to 88 degrees) before becoming vertical at 34.5 feet below the ground surface.

6.3 OLDER ALLUVIUM (Qoal)
As mentioned previously, Quaternary-age Older Alluvium mantles the underlying Calabasas Formation over most of the site, (refer to Plate 1). This relatively thick sequence of Older Alluvial soils forms the ridge east of the site (being well exposed on the Agoura Road cut) and is expected to cap the spur (minor ridge) in the western part of the site.

As observed in our exploratory borings the thickness of the older alluvium varies from 35.5 feet (B-1) to 6 feet (B-6). The Older Alluvium generally consists of brownish yellow silty clay interbedded with silty fine to coarse sand and clayey fine sand grading downward to pale brown silty clay and clayey fine to coarse sand. This deposit is typically in a hard to dense and moist condition. The base of the Older Alluvium is generally denoted with fine to coarse sand and gravel with some cobbles of volcanic rock.

The contact with the underlying bedrock is abrupt with an irregular and undulatory to planar surface. In boring B-3, the contact with the underlying bedrock was inclined at 13 degrees to the northeast.

6.4 TOPSOIL / COLLUVIUM AND YOUNGER ALLUVIUM (Qal)
Low gradient areas of the northern part of the site are mantled by residual soils and colluvial soils while minor alluvial deposits are present were the canyon stream courses run out onto the low gradient areas of the northern part of the site. As encountered in the borings the Topsoil / Colluvium mantling the Older Alluvium varies in thickness from 7 feet (B-1) to 2.5 feet (B-3). The colluvium generally consists of very dark grayish brown to grayish brown sandy silty clay to silt with subangular to subrounded gravel to cobbles sized clasts of volcanic rock in a hard and damp to moist condition. Typically, the upper portion of these materials is porous with scattered roots. The Younger Alluvial deposits consist of unconsolidated sand, silts, and clays with scattered to locally abundant gravel to cobble size volcanic clasts.

6.5 ARTIFICIAL FILL
Mechanically placed fill is locally present associated with graded roads and with the previous building pads. A fill berm was constructed for Agoura Road along the northern edge of the property with the southern slope extending onto the site. Near surface soils are disturbed in the northern part of the site as a result of plowing for “weed abatement. Artificial fill, 1 foot in thickness, was encountered in boring B-5 mantling the colluvium. As encountered, the artificial fill generally consists of dark grayish brown very silty clay with roots and some rock. Additional areas of concealed deeper fill deposits may exist on the property and will need to be removed to underlying suitable materials within the limits of the proposed construction.

6.6 LANDSLIDES
No landslides were evident in our reconnaissance of the site nor are any shown to exist on-site in the regional geologic literature. However, we are aware a landslide occurred along Agoura Road northeast

of the site. A significant rotational failure occurred near the contact between clayey siltstone and the overlying saturated Older Alluvial Deposits. A landslide was mapped by Weber and Blackerby (Weber, 1984) southwest of the site in terrain underlain by volcanic bedrock (see Figure 2). Landslides are relatively uncommon in areas underlain by Conejo Volcanics and generally, irregular topographic expressions due to resistant rocks have been misinterpreted as landslides. Bedrock of the Conejo Volcanics is generally the most stable rock unit within the area.

6.7 GROUNDWATER

During the subsurface exploration program performed in 2000 (Gorian 200a) groundwater was encountered in boring B-1 at 24 feet below the ground surface in a silty fine to coarse sand layer within the Older Alluvium and as seepage in boring B-3 from 15.3 feet to 16.9 feet below the ground surface. The seepage was observed just above the contact with the underlying bedrock. Also, the Seismic Hazards Zone Report indicates groundwater at a depth of 10 feet along Agoura Road. Groundwater levels can fluctuate seasonally in response to precipitation and area irrigation practices.

6.8 FAULTING AND SEISMICITY

The site is within a seismically active region that will experience 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, landslides). The hazard of fault-rupture is generally thought to be associated with a relatively narrow zone along well-defined pre-existing active or potentially active faults. No doubt there are 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).

The hazard of fault-rupture is generally thought to be associated with a relatively narrow zone along well-defined pre-existing active or potentially active faults. No active faults are known to cross the site and the project site is not within an Alquist-Priolo Earthquake Fault Zone (Bryant and Hall, 2007). The Malibu Coast fault is the nearest active fault about 7 miles south of the site (Figure 3). As depicted on the geologic map, Figure 2, by Weber (1984), a northeast trending fault is interpreted to cross the western part of the site. This fault, if indeed present, is a minor local feature. Some geologists have suggested the contact between the Conejo Volcanics and Calabasas Formation in this area may be a fault contact. While this may account for the complex folding (plastic deformation) observed in the Calabasas Formation on site, this relationship has not been demonstrated. It appears the contact between these two bedrock units is beyond the area of proposed construction; probably south of site, past borings B-2 and B-3. Therefore, the potential for on-site ground rupture within the area of proposed construction due to faulting is considered remote during the life expectancy of the project.

Nevertheless, the site will be subjected to ground motion from occasional earthquakes in the region. Significant earthquakes have occurred within a 40-mile radius of the site within the last 40 years. The 1994 Northridge earthquake produced strong ground motion at the site and a peak horizontal acceleration of approximately 25 to 30 percent the acceleration of gravity (0.25 to 0.30g) for the stiff soil/soft bedrock site (Chang, et al., 1994). It is likely significant earthquakes will occur in this region within the life expectancy of the proposed project and the site will experience strong ground shaking from these events.

Based on the latest United States Geological Survey (USGS) interactive web application, 2008 Interactive Deaggregations, <https://geohazards.usgs.gov/deaggint/2008/> probabilistic seismic hazard analyses (PSHA) predict the Design Basis Earthquake peak horizontal ground acceleration will be on the order of 0.41g for the site (latitude 33.1442ºN and longitude 118.7923ºW) assuming a shear wave velocity, V_s^{30} of 350 meters/second. The Design Basis Ground Motion is defined as having a 10% chance of being exceeded in 50 years is based on probabilistic analyses. The mean magnitude from this PSHA is 6.8 (Mw) with a mean distance of 20.3 km from the property and a modal magnitude of 7.0 (Mw) with a modal distance of 25.7 km from the property.

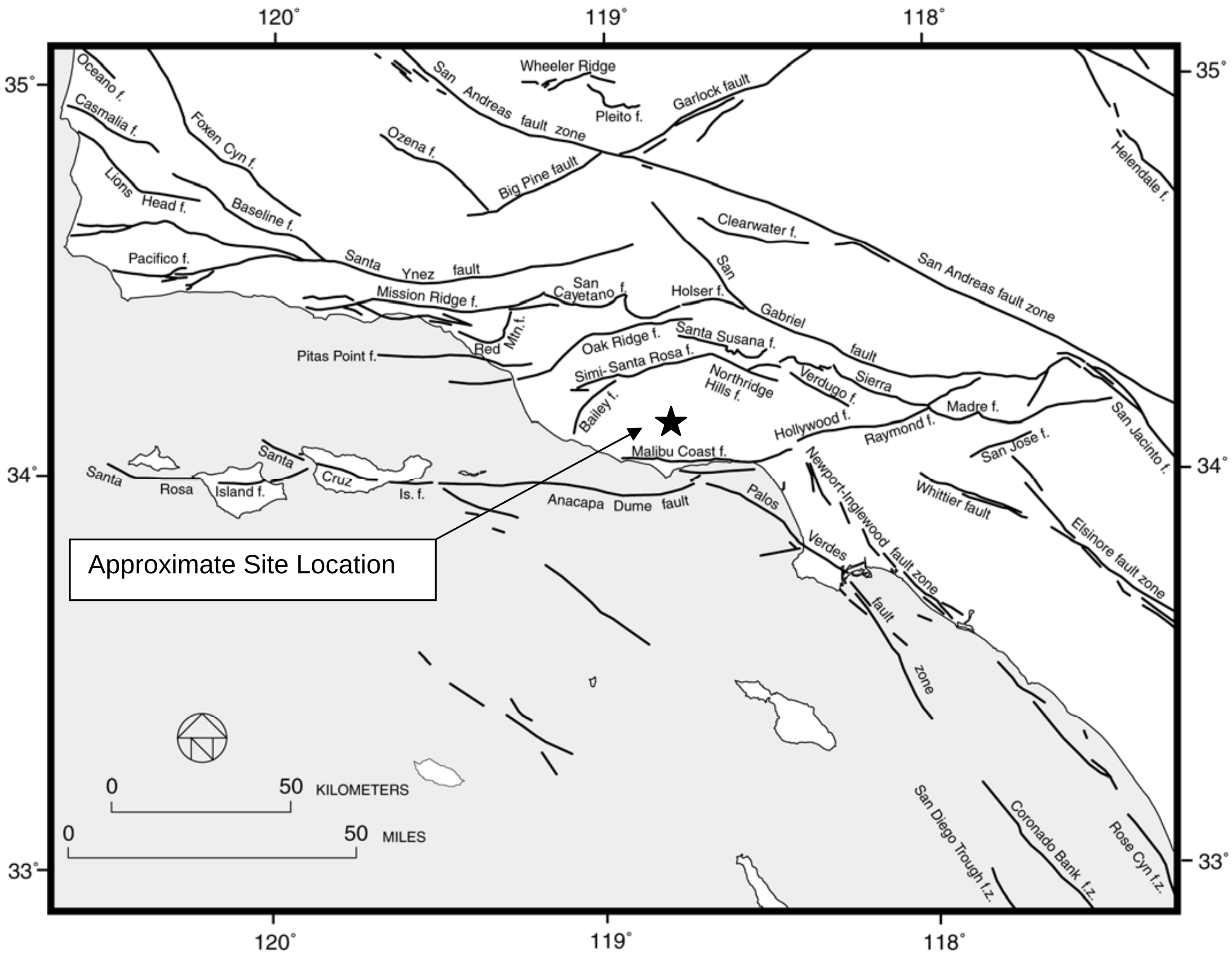

Figure 3. Map showing approximate site location in relationship to Holocene and late Quaternary faults of the Los Angeles region after Jennings (1992).

Secondary effects of strong ground motion include such phenomena as tsunami, seiche, liquefaction, settlement, flooding from dam failure, landslides, etc. Inundation by a tsunami (seismic sea wave) or seiches (standing wave) are not hazards inherent to the site due its distance from the ocean or any inland large body of water. The site is also not in an area susceptible to flooding due to seismic related dam failure. Slope stability analyses indicate the existing and proposed manufactured slopes are not susceptible to seismically induced landsliding as long as proper slope maintenance recommendations are followed. The site is not within an area shown to have a potential for secondary seismic settlement / liquefaction on the Seismic Hazards Zones map.

6.9 ROCK HARDNESS

Three shallow seismic refraction traverse surveys were preformed to provide data for the evaluation of rock hardness and rippability of the areas of the deepest proposed cuts. The locations of our traverses are shown on the attached Geotechnical Map, Plate 1.

The excavation characteristics of rock material are a function of lithology, seismic velocity, geologic structure, ripping equipment capacity, and operation. Shallow seismic refraction survey traverses can provide data to compute compressional wave velocities (p-wave) traveling through the underlying earth materials. These velocities can be roughly correlated with the rippability of these materials by conventional grading equipment. These correlations are not precise but rather, are intended to represent a generalized means of indicating relative excavation characteristics.

Based on our experience with full-scale rippability tests at other sites in the area, thick to massively bedded Conejo Volcanics Formation bedrock materials can be ripped to a maximum compression wave velocity of approximately 7500 to 9500 feet per second (ft/sec). The rippability tests were performed utilizing a D9R Caterpillar tractor or equivalent bulldozer in good condition with a single shank, variable pitch ripper. Although rippable, oversized rock (i.e., rock greater than 8 inch diameter) can be generated in materials above 5000 ft/sec. Other tests with a Caterpillar D-10N bulldozer equipped with a single shank variable pitch ripper indicated the D-10N was able to rip bedrock at production rates to within the 8,500 to 10,500 ft/sec range. At higher velocities, however, very difficult ripping was encountered and considerable quantities of oversized rock were generated.

The average (and rounded) results of our (two direction) shallow seismic refraction survey traverse is presented in Table 1. Comments regarding rock rippability reflect usage of Caterpillar D9R bulldozer or equivalent, and are based on local experience and on rippability curves published by Caterpillar, Inc. (1995).

TABLE 1

SHALLOW SEISMIC REFRACTION TRAVERSE SURVEY RESULTS

Traverse Number	Layer	Depth (ft)	Average Velocity (ft/sec)	Comments
ST-1	1	surface to 5	1550	Easy ripping
	2	5 to at least 32	2150	Moderate ripping
	3	>32	7000*	Possible blasting
ST-2	1	surface to 6½	1310	Easy ripping
	2	6 ½ to at least 43	2690	Moderate ripping
	3	>43	7000*	Possible blasting
ST-3	1	surface to 5	1520	Easy ripping
	2	5 to at least 38	2520	Moderate ripping
	3	>38	7000*	Possible blasting

*Assumed velocity of layer 3 (used to calculate depth to layer 3)

The seismic traverses indicate the surficial soil is easily rippable. At a depth of about 5 feet below ground surface (bgs) the earth material is moderately rippable to at least 32 feet. Consequently, the proposed design grades should be able to be obtained without blasting or difficult ripping. The results suggest the material underlying the site in the area of the proposed cuts is not composed of hard rock and may not be underlain in the shallow subsurface by volcanic rock as depicted on regional geologic map.

As a matter of completeness, we quote from the *Caterpillar Performance Handbook*:

> "*Use of Seismic Velocity Charts*
>
> *The charts of ripper performance estimated by seismic wave velocities have been developed from field tests conducted in a variety of materials. Considering the extreme variations among*

materials and even among rocks of a specific classification, the charts must be recognized as being at best only one indicator of rippability.

Accordingly, consider the following precautions when evaluating the feasibility of ripping a given formation:

-- Tooth penetration is often the key to ripping success, regardless of seismic velocity. This is particularly true in homogeneous materials such as mudstone and claystone and the fine-grained caliches. It is also true in tightly cemented formations such as conglomerate, some glacial tills, and caliches containing rock fragments.

-- Low seismic velocities of sedimentaries can indicate probable rippability. However, if the fractures and bedding joints do not allow tooth penetration, the material may not be ripped effectively.

-- Pre-blasting or "popping" may induce sufficient fracturing to permit tooth entry, particularly in the caliches, conglomerates and some other rock; but the economics should be checked carefully when considering popping in the higher grades of sandstones, limestones, and granites.

Ripping is still more art than science, and much will depend on the skill and experience of the tractor operator. Ripping for scraper loading may call for different techniques than if the same material is to be dozed away. If cross-ripping is called for, it, too, requires a change in approach. The number of shanks used, length and depth of shank and tooth angle, direction, throttle position--all must be adjusted according to field conditions encountered. Ripping success may well depend on the operator finding the proper combination for those conditions."

7. SLOPE STABILITY

7.1 GENERAL

Manufactured slopes will be constructed at a maximum gradient of 2(h):1(v) or flatter within the proposed development area. Cross Section A-A' has been drawn to depict the deepest cut within the project. Stability analyses were conducted using this cross section to evaluate the gross stability of the retaining wall below the 2:1 (horizontal to vertical) slope. In addition, a generalized slope was analyzed to represent the overall area to the west of cross section A-A'. Surficial stability of the existing slope was also evaluated.

The computer program GSTABL7 utilizing Bishop's simplified method of slices for rotational failures was used to evaluate gross slope stability of the proposed slopes discussed above. The results of the gross and surficial stability analyses are presented in Appendix C.

7.2 SHEAR STRENGTH PARAMETERS

The shear strength parameters used in the slope stability analyses were derived from a series of direct shear test results. Shear strength parameters for Conejo Volcanics bedrock were derived from direct shear testing conducted on relatively undisturbed bedrock samples from a nearby site with bedrock of the same formation (Gorian, 1979). The resulting shear strength parameters are as follows:

SOIL TYPE	UNIT WEIGHT	COHESION	FRICTION ANGLE
Engineered Fill	115 pcf	400 psf	21.5º
Older Alluvium	125 pcf	200 psf	35º
Calabasas Formation	125 pcf	560 psf	27.5º
Conejo Volcanics	125 pcf	1000 psf	26º

Shear strength for the contorted bedding within the Calabasas Formation was rationalized as being roughly 50 percent cross bedding and 50 percent along bedding. For the along bedding, a default strength of 10 degrees friction and 200 psf cohesion was used in the analyses. The resulting shear strength used for the contorted Calabasas Formation is 19 degrees friction and 380 psf cohesion.

7.3 GROUNDWATER

In the analyses, two types of water input were used to model the conditions at the site. In general, a piezometric groundwater surface was input and applied to the Older Alluvium and Calabasas Formation. The surface was modeled at the contact between the Older Alluvium and artificial fill placed for Agoura Road near the base of the section transitioning to the contact between the proposed fill and Calabasas Formation beneath the proposed building. The transition was modeled to account for the drainage that will be installed at the toes of proposed retaining walls. In addition to the piezometric surface, a constant pore pressure of 312 psf was applied to the Conejo Volcanics to account for possible seeps that may occur within this formation. This value is equivalent to having a water level 5 feet above each failure plane evaluated within the Conejo Volcanics.

7.4 GLOBAL ANALYSES

Global static and pseudostatic stability analyses were conducted to evaluate the stability of the proposed retaining wall shown in Cross Section A-A' and a generalized cut slope to the west of Cross Section A-A'. The retaining wall shown in Cross Section A-A' was analyzed as a soil nail wall with 4 foot vertical spacing on the nails, which have lengths of 25 to 35 feet.

Rotational failure paths were evaluated with varying toe and exit paths to find the critical failure surface. Pseudostatic analyses were conducted using a horizontal acceleration coefficient of 0.15g. The results of the analyses indicate the proposed development has satisfactory factors of safety against global rotational failures. The output and plot files from the stability analyses are contained in Appendix C, herein.

7.5 SURFICIAL STABILITY

The proposed 2(h):1(v) cut and fill slopes have a minimum factor of safety of 1.5 as demonstrated by the surficial stability calculations presented in Appendix C.

8. RESPONSES TO GEODYNAMICS REVIEW LETTER DATED NOVEMBER 11, 2011

PLANNING/FEASIBILITY COMMENTS

COMMENT 1

The latest report is over four years old and addresses an earlier plan. The consultant should provide a stand-alone update report. The update report should address changes to geotechnical conditions at the site, and should be based on the most current plan. The update report should also include new or updated cross sections (including orthogonal sections through buildings) and recommendations as necessary to adequately depict relationships between the existing geologic conditions and the various aspects of the proposed development/grading plan (example: pedestrian bridge, debris basins, etc.). Additional mitigation measures should be recommended as necessary.

RESPONSE

The requested stand-alone update report is contained herein.

COMMENT 2

New fills are proposed to derive support from existing fill along Agoura Road. Sufficient exploration and testing appears warranted to verify the adequacy of this existing fill to support the proposed improvements. If the fill needs to be removed, the consultant should provide specific recommendations to support the existing road during the fill removal.

RESPONSE
Based on the proposed grades, Agoura Road will be widened southward beyond the existing slope supporting the southern roadway edge. The proposed road widening though shown on the grading plans will be a City of Agoura Hills project and is not part of this project. The widening will extend from the western edge of the project and will extend eastward past the project. The section of Agoura road along the northern boundary of the site is performing well without evidence of settlement/creep along the roadway edge.

Construction of the building pad for Building A will require a fill over the existing slope that ascends from Agoura Road. In addition, the drive to Building A is rough at grade near Agoura Road. The fill between the two drives will be placed to widen Agoura Road roughly 30 feet in this area. Therefore, the toe of the new slope will be well beyond the toe of the existing slope and removals will only be of the topsoil colluvium in this area.

Construction of Building B will create an ascending fill slope up from inside the tract boundary and outside of the proposed widening of Agoura Road. This slope for the building pad will not derive support from fills for Agoura Road. The driveway to Building B will have fills placed to gain access to Agoura Road.

Benching of new fills will not extend significantly into the slope descending from the southern edge of Agoura Road and the benches should only be wide enough to remove weathered/deleterious surficial soils on the slope and provide for a flat surface on which to place the new fills. Therefore, undermining of the existing road should not occur.

Consequently from a geometric standpoint the proposed fill will buttress the existing roadway slope. Additional exploration within the road right of way is not considered needed due to the buttressing affect of the proposed fill, the anticipated quality of the existing fill, and the existing roadway fill has no affect on the proposed building pad stability.

COMMENT 3
The consultant should provide a more detailed discussion of stability issues where contorted Calabasas Formation will be exposed in cut-slopes and retaining walls. Cut-slopes and retaining walls depicted on the current plan appear likely to expose Calabasas Formation with bedding planes at least locally inclined northerly at low angles. The consultant should provide analyses to verify the recommended equipment width stability fill will be adequate to mitigate the potential for translational failures along unsupported sections of bedding in the Calabasas Formation. Continuity of bedding should be assumed in critical areas unless sufficient field exploration is provided to demonstrate a lack of continuity. Mitigation measures should be recommended as necessary.

RESPONSE
Contorted Calabasas Formation bedrock is anticipated only in the deepest cuts that are deeper than 20 feet. The only area of contorted bedding anticipated to be encountered is illustrated in Cross Section A-A' on Plate 2. In this area, a high retaining wall shown on Plate 1 and cut for this wall will extend into the underlying Calabasas Formation. This wall was analyzed and it was determined best approach is to construct this wall as a soil nail retaining wall. The analysis of this wall is discussed further in the Slope Stability Section of this report.

COMMENT 4
The contact between the Older Alluvium and the underlying Calabasas Formation is reported to be inclined northerly at an overall gradient of about 13 degrees, with variable material conditions. At some locations the contact was found to be abrupt. Other locations encountered residual soil of gray clay (B-1), or plastic clay seams within the uppermost part of the Calabasas Formation inclined roughly parallel

to the contact (B-3). The consultant should discuss and evaluate as necessary the potential for translational deformation where this contact will be exposed in future cut-slopes or retaining wall back-cuts. Mitigation measures should be recommended as necessary.

RESPONSE
The reviewer is referred to our response to Comment 3 and the Slope Stability Section of this report.

COMMENT 5
Jarosite is noted in the weathered section of the Calabasas Formation in Boring B-4. Jarosite is commonly associated with the oxidation of sulfide minerals known to exist in the Calabasas Formation. This process can lead to severe expansion and produces a highly acidic environment that can be very corrosive to concrete and steel. The consultant should discuss the potential for sulfide expansion and the related potential for highly corrosive soil resulting from the oxidation reaction of sulfide minerals where relatively unoxidized Calabasas Formation will be exposed below the future structures or utilized in future fill materials. Mitigation measures should be recommended as necessary.

RESPONSE
Of the five borings excavated into Calabasas Formation, only in boring B-4 was a minor amount of Jarosite noted. In the area of boring B-4, the pad will be constructed by placing fill over the Older Alluvium capping the bedrock. In addition, the adjacent subterranean parking garage finished floor should be supported on fill over a thick sequence of Older Alluvium. Where the garage will extend into the underlying Calabasas Formation in the ridge area illustrated in Cross Section A-A', no Jarosite was observed in either borings B-2 or -6. Therefore, the presence of Jarosite is not anticipated to affect the current planned development. Nevertheless, as in any over excavation extending into bedrock, the removal bottom should be observed by an engineering geologist from this office for the presence of adverse geologic conditions.

Corrosion testing of the underlying soils/bedrock should be performed during the grading phase of development to evaluate the potential for corrosion of concrete or metals in contact with the on site soil/bedrock. However, based on experience the soils/bedrock should be considered corrosive to metals and all metal including copper pipe should be protected from contact with the on site soils/bedrock. Also where possible, copper piping should be run overhead and not below concrete slabs on grade.

COMMENT 6
Review of currently proposed grades and previous cross sections indicates the final pad grades will be underlain by and likely transition between Conejo Volcanics, Calabasas Formation, Older Alluvium, and artificial fill. The consultant recommends overexcavation to at least 3 ft below the footings within building pad areas. Hence, the consultant should discuss and substantiate the adequacy of the recommended depth of overexcavation to mitigate the potential for differential expansion. Mitigation measures should be recommended as necessary.

RESPONSE
The proposed grading of the pads for Buildings A and B is not expected to extend into the underlying Conejo Volcanic bedrock. The grading for Building A will consist of a removal to the Older Alluvial soils and should not extend to either bedrock unit. Therefore within Building A, the resulting pad will consist of compacted fill over Older Alluvium, which should not create a significant soil expansion differential.

The over excavation for Building B will extend into the underlying Calabasas Formation for a limited distance mainly in cuts of the ridge extending into the building area shown in Cross Section A-A'. The upper portion of the Calabasas is not anticipated to be highly expansive based on an expansion test from Boring B-6 at the northeast corner of Building B. We have previously recommended and continue to recommend select grading be performed within the buildings to eliminate very high or critically expansive soils from below the buildings. Therefore, a significant soil expansion differential is not anticipated in Building B.

COMMENT 7
The City of Agoura Hills requirements for slope to structure/foundations setback should be complied with.

Please note the city of Agoura Hills has more stringent setback requirements than the California Building Code as recommended on page 17 of the October 12, 2000 report.

RESPONSE
The building to slope setback should be per the City of Agoura Hills amendments (Ordinance No. 10-381) to the California Building Code.

REPORT REVIEW COMMENTS

COMMENT 1
The consultant should review final development/grading plans when they become available and provide additional geotechnical recommendations as necessary.

RESPONSE
Acknowledged and will be complied with as the entitlement process proceeds.

COMMENT 2
The consultant recommends that fill keyways should extend a minimum of two feet into "firm competent in-place soil." *The consultant should clarify which units are acceptable for keyway support.*

RESPONSE
As provided in Section 10.3.7 of Gorian, 2000, firm competent in-place soil is what is exposed after all required soil removals are made. The reviewer is referred to Section 9.4.3 of this report for recommended soil removals.

PLAN-CHECK COMMENTS

COMMENT 1
The name, address, and phone number of the Consultant and a list of all the applicable geotechnical reports shall be included on the building/grading plans.

RESPONSE
Acknowledged and the information should be provided by the project design structural and civil engineers the as the entitlement process proceeds.

COMMENT 2
*The following note must appear on the grading and foundation plans: "*All retaining wall excavations shall be reviewed by the project engineering geologist for the presence of adversely oriented joint surfaces. Adverse surfaces shall be evaluated and supported in accordance with recommendations of the project geotechnical engineer.*"*

RESPONSE
Acknowledged and the note should be provided by the project design civil engineer the as the entitlement process proceeds.

COMMENT 3
The grading plan should include the limits and depths of overexcavation for the swimming pool, road, and flatwork areas as recommended by the Consultant.

RESPONSE
Acknowledged and requested information should be provided by the project design civil engineer the as the entitlement process proceeds.

COMMENT 4
The following note must appear on the grading and foundation plans: "Excavations shall be made in compliance with CAL/OSHA Regulations."

RESPONSE
Acknowledged and the note should be provided by the project design civil engineer as the entitlement process proceeds.

COMMENT 5
*The following note must appear on the foundation plans: "*All foundation excavations must be observed and approved, in writing, by the Project Geotechnical Consultant prior to placement of reinforcing steel*."*

RESPONSE
Acknowledged and should be provided by the project structural engineer as the entitlement process proceeds.

COMMENT 6
Foundation plans and foundation details shall clearly depict the embedment material and minimum depth of embedment for the foundations.

RESPONSE
Acknowledged and should be provided by the project structural engineer as the entitlement process proceeds.

COMMENT 7
Drainage plans depicting all surface and subsurface non-erosive drainage devices, flow lines, and catch basins shall be included on the building plans.

RESPONSE
Acknowledged and should be provided by the project design civil engineer as the entitlement process proceeds.

COMMENT 8
Final grading, drainage, and foundation plans shall be reviewed, signed, and wet stamped by the consultant.

RESPONSE
Acknowledged and will be provided as the entitlement process proceeds.

COMMENT 9
*Provide a note on the grading and foundation plans that states: "*An as-built report shall be submitted to the City for review. This report prepared by the Geotechnical Consultant must include the results of all compaction tests as well as a map depicting the limits of fill, locations of all density tests, outline and elevations of all removal bottoms, keyway locations and bottom elevations, locations of all subdrains

and flow line elevations, and location and elevation of all retaining wall backdrains and outlets. Geologic conditions exposed during grading must be depicted on an as-built geologic map."

RESPONSE
Acknowledged and should be complied with by the project design structural and civil engineers as the entitlement process proceeds.

9. CONCLUSIONS AND RECOMMENDATIONS

9.1 GENERAL
The site of the proposed senior housing community at 30800 Agoura Road in Agoura Hills, California as addressed herein has been evaluated by this firm from a geotechnical standpoint. The proposed project is feasible from a geotechnical standpoint providing construction is performed in accordance with the recommendations contained herein. Recommendations are presented in the following sections regarding design and construction of the project based on the recommendations in our prior reports (see Gorian 2003 and 2007). However, the recommendations have been updated to reflect the current development, building codes, and construction practice.

9.2 PROJECT DESIGN CONSIDERATIONS
The project will consist of the development of two building areas separated by natural drainages. The most significant aspect of the project is the larger cut of the ridge extending into the Building B area. The highest of the cut will be supported by a retaining wall. This wall is recommended to be constructed as a soil nail retaining wall.

9.3 SEISMIC DESIGN PARAMETERS
Seismic ground motion parameters were evaluated using a simplified code based approach and ground motion procedures for seismic design. The simplified code based approach follows procedures based on ASCE/SEI 7-05 Section 11.4. The 2010 CBC is based on the 2009 IBC which references the Minimum Design Loads for Buildings and Other Structures (ASCE/SEI 7-05) as indicated under Effective use of the IBC/CBC on page ix of the 2010 CBC. In addition, seismic parameters based on ASCE/SEI 7-10 are proved herein in consideration of anticipated near future adoption of new building code requirements.

Parameters presented herein should be used with the understanding site acceleration could be higher than addressed by code based parameters. The purpose of the building code earthquake provisions is to primarily 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. Cracking of walls and possible structural damage should be anticipated in a significant seismic event.

Seismic ground motion values are initially determined based on site class B (rock) conditions. The values are adjusted to obtain the maximum considered earthquake (MCE) spectral acceleration values for the site based on its site class of D. The following parameters for site's coordinates (latitude 33.1442ºN and longitude 118.7923ºW) were obtained from the USGS web based spectral acceleration response maps and calculator (http://geohazards.usgs.gov/designmaps/us/application.php).

Seismic Parameters based on ASCE/SEI 7-05

2010 CBC CHAPTER 16 TABLE/FIGURE NO.	SEISMIC PARAMETER	VALUE PER CALIFORNIA BUILDING CODE
Figure 1613.5 (3)	Short Period Mapped Acceleration (S_s)	1.705g
Figure 1613.5 (4)	Long Period Mapped Acceleration (S_1)	0.715 g
Table 1613.5.2	Site Class Definition	D
Table 1613.5.3 (1)	Site Coefficient (F_a)	1.0
Table 1613.5.3 (2)	Site Coefficient (F_v)	1.5
Equation 16-37	$S_{MS} = F_aS_s$	1.705 g
Equation 16-38	$S_{M1} = F_vS_1$	1.072 g
Equation 16-39	$S_{DS} = 2/3S_{MS}$	1.137 g
Equation 16-40	$S_{D1} = 2/3S_{M1}$	0.715 g

Seismic Parameters based on ASCE/SEI 7-10

2010 CBC CHAPTER 16 TABLE/FIGURE NO.	SEISMIC PARAMETER	VALUE PER CALIFORNIA BUILDING CODE
Figure 1613.5 (3)	Short Period Mapped Acceleration (S_s)	1.548 g
Figure 1613.5 (4)	Long Period Mapped Acceleration (S_1)	0.600 g
Table 1613.5.2	Site Class Definition	D
Table 1613.5.3 (1)	Site Coefficient (F_a)	1.0
Table 1613.5.3 (2)	Site Coefficient (F_v)	1.5
Equation 16-37	$S_{MS} = F_aS_s$	1.548 g
Equation 16-38	$S_{M1} = F_vS_1$	0.900 g
Equation 16-39	$S_{DS} = 2/3S_{MS}$	1.032 g
Equation 16-40	$S_{D1} = 2/3S_{M1}$	0.600 g

9.4 SITE PREPARATION AND GRADING

9.4.1 General

Site preparation and grading recommendations presented below are for preparation of the development area for support of residential structures and related site improvements. All aspects of grading including site preparation, grading, and fill placement should be per the City of Agoura Hills Building Code. Fill placement and bottom preparation for fill placement, backfill placement, utility trench backfill, and sub-grade should be observed (and tested when appropriate) by this firm during construction.

9.4.2 Site Clearing

Prior to starting earthwork, areas to be graded should be stripped of vegetation, trash, and debris. Minor vegetation may be blended with the soils during processing until it is not discernable from the fill. Roots over one-half inch in diameter should be included with removal of brush or trees.

9.4.3 Soil Removal

Removal of the upper soils will be necessary in all areas of grading. The removal should include non-engineered fill, recent alluvium, and colluvium. Additionally, the artificial fill body at the north end of the central drainage channel should be remediated in the same fashion as described for treatment of non-certified fills in our previous report. This fill body is outside the area of proposed grading. However, continued erosion of this fill could adversely affect future development.

Additionally, soil removals should extend to competent soil having a minimum relative compaction of 85% or bedrock, whichever is shallower. However, within the building area and five feet beyond, the soil removal should extend to in-place soils having a minimum relative compaction of 90% or competent bedrock, whichever is the lesser removal. Alluvial removals should be on the order of 3 to 10 feet and 7 to 15 feet in parking areas and the building pad, respectively.

After the removals are completed as addressed above, the exposed soils should be observed by a representative of this office to evaluate if additional removals are necessary. Fill soils should not be placed until completion of the geotechnical observation. Observation of the removal areas by Gorian and Associates, Inc. does not waive inspections that may be required by the local grading department inspector. The City of Agoura Hills may require a representative from the City observe the bottom of the soil removal areas.

9.4.4 Soil Removal On Slopes

Cut slopes in the alluvial areas or areas of recently deposited soils will possibly expose unsuitable soils. These soils should be removed from the slope to firm in-place Older Alluvium or bedrock. The slope may be reconstructed as a stabilization fill having the design slope grade. This condition may be encountered at the rear of Building B in the north facing cut slope. Therefore, all cut slopes should be observed by an engineering geologist from this office. The tops of all cut slopes exposing topsoil/colluvium should be rounded.

9.4.5 Building Pad Removals

As previously stated within the building area, soil removal should extend to in-place soils having a minimum relative compaction of 90% or competent bedrock, whichever is the lesser removal. In addition, removals will be necessary where transitions between contrasting materials (such as bedrock/alluvium) cross the footprint of a structure.

For transition pads which incorporate both cut and fill materials, the cut portions within building areas should be undercut to a minimum of five feet below proposed pad grade or one-third of the maximum fill thickness, whichever is deeper. In addition, the undercut should be a minimum of 3 feet below the footings. A construction level foundation plan will be necessary to provide the foundation depths and locations.

The purpose of the undercut is to reduce the potential for significant differential settlement or uplift between these contrasting materials. For reference a Building Pad Over-excavation Detail is attached in Appendix D.

The undercut should extend past the building footprint or outer perimeter of the exterior footings (whichever is greater) at least 5 feet, equal to the depth of soil removal, or as directed by this firm, whichever is the greater distance. Removal limits will need to be reviewed in the field due to possible constraints such as property lines and existing improvement to remain in place.

9.4.6 Construction Dewatering Considerations

Groundwater seepages were encountered in the borings for previous exploration as discussed under Section 6.7 Groundwater. The groundwater data available indicates groundwater could possibly be encountered at or near the contact with bedrock. Fluctuations in the groundwater elevation should always be anticipated in dewatering or below grade excavations. Therefore, localized dewatering may be necessary depending upon the volume of water encountered. Dewatering should be provided by the grading contractor. Handling of collected water should be in conformance with permits/requirements for the Site.

9.4.7 Shrinkage and Subsidence

Shrinkage or bulking is the volume loss or gain respectively of soils excavated and recompacted. Shrinkage of the recent alluvium and artificial fill is expected to range from 5 to 15 percent. Colluvium and Older Alluvial soils are expected to shrink on the order of 5 to 10 percent and shrinkage of bedrock removed and recompacted should range from 0 to 5 percent. For example, 1 cubic yard of cut in Older Alluvium will yield approximately 0.9 to 0.95 cubic yards of engineered compacted fill. In addition to the shrinkage/bulking values presented above, subsidence or a loss of 0.1 to 0.2 feet should be considered for stripping of vegetation and densification of the surface soils. These values are estimates only and if a more accurate determination of estimated shrinkage amount is critical for the balance of cut and fill quantities, values can be reevaluated during the early stages of site grading.

9.4.8 Hard Rock

Seismic traverses ST-1, -2, and -3 were performed in the area of the proposed deepest excavation. The results of the surveys are presented in Section 6.9 of this report. The survey results indicated the bedrock should be rippable to the proposed excavation depths. However, the conditions can vary within bedrock with depth or location and some additional effort may be required to excavate the bedrock.

9.4.9 Bottom Stabilization

Seepages and groundwater were encountered within the bedrock as discussed in Section 6.7 Groundwater. Therefore, high moisture contents above the optimum value may be encountered at the bottom of the removals or deep cuts. Therefore, some type of stabilization of the removal bottom may be necessary prior to placement of compacted fill depending upon the encountered condition. Stabilization methods include the use of geotextile fabric, lime treatment, placement of gravel/rock or other approved alternative.

9.4.10 Subdrain

Typically, subdrains are placed within drainages or canyons prior to the placement of fill. However, per the grading shown in Plate 1, the proposed grading will not extend into the natural drainages except directly adjacent Agoura Road where it will be widened. Therefore, the need for subdrains is not anticipated at this time. Backdrains will be required behind retaining walls and fill slope keyways as discussed later herein.

9.4.11 Processing

Once the soil removals and undercutting recommended above have been completed, the bottoms of the removal and undercut areas should be observed by this office. Deeper removals may be required if uncertified fill or loose or soft zones are encountered. Prior to placing fill, the exposed surfaces should be processed. Processing consists of scarifying to a depth of 6 to 8 inches, conditioning to near optimum moisture content, and compacting to at least 90 percent relative compaction. Hard in-place bedrock will not require scarification.

9.4.12 Fill Placement

Fill soils should be cleaned of deleterious materials including trash, debris, and organic matter. Fill soils should be placed in thin uniform lifts, moisture conditioned to slightly above the optimum moisture content, and compacted. Material exceeding 12 inches in maximum dimension should be excluded from the fill. In addition, it may be desirable to keep rock larger than 3 inches outside of the upper 5 feet of the building area. Fill placed within building pad areas should be mixed and blended.

Soils excavated on-site may be used as fill. However, clayey soils having expansion indices greater than 130 should not be placed within the building footprint and five feet beyond or within 10 feet of the slope faces. Very highly expansive clays were found within the Older Alluvium units during the subsurface exploration (Gorian 2000a). Therefore, selected grading will be necessary within the building and slope areas. The expansion potential of the very highly expansive on-site soils (EI > 130) could possibly be reduced by blending very highly expansive soils with the more granular soils. If the soils are blended, the soils should be disked to provide thorough mixing. Frequent expansion index tests should be performed during grading to determine if the resulting expansion indices are below 130 within the building and slope areas. Additionally, select grading will be required within a 1(h):1(v) wedge, projected up from the toe, behind retaining walls and within 10 feet of any fill slopes.

Very highly expansive soils (EI > 130) whenever possible should be placed at the bottom of the parking and drive fill areas. Near the parking and drive finished grades, the expansive clayey soils may be used if lime treated. Parking and drive subgrade soils may be lime treated using 4% to 5% lime, measured by weight, to a minimum depth of 8 inches. Subgrade preparation, lime spreading, mixing, and compacting should be completed per the current Greenbook (Standard Specifications for Public Works Construction) specifications.

If import fill is required, this firm should approve the sources of the fill. The shear strength parameters and the expansion indices of the fill soils should be determined by this office prior to importing to the site.

9.4.13 Fill Compaction

Fill soils should be compacted to a minimum of 90 percent relative compaction. Relative compaction is the ratio of the in-place dry soil density to the maximum dry soil density as determined in general accordance with ASTM test method D 1557.

9.4.14 Keying and Benching

Fills placed on slopes steeper than a 5(horizontal):1(vertical) gradient should be keyed and benched (horizontal benches) into competent in-place soil (after the required removals are made as discussed in Section 9.4.3) or bedrock. All keyways should be a minimum of 15 feet wide measured from the design toe of slope and cut a minimum depth of 2 feet at the toe into firm competent in-place soil or bedrock. Soil removal at the toe of slope may be required to extend past the design toe of slope as shown in Removals Beyond the Toe of Proposed Fill Slopes Detail attached in Appendix D. Also presented in Appendix D is a Canyon Cleanout and Benching Typical Detail.

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 before placing any fill. Horizontal benches should be a minimum of 5 feet wide, i.e. a minimum 5 feet of competent material. A representative of this office should observe benching before placing any fill soils.

9.4.15 Utility Trenches

Utility trench backfill within building, parking, and drive areas should be compacted to a minimum 90% relative compaction.

9.4.16 Temporary Excavations

Temporary excavations should conform to the requirements of CAL/OSHA. Excavations deeper than 4 feet should be shored or sloped. Surcharge loads should be setback sufficient distances from the tops of temporary excavations. Temporary excavations adjacent property line constraints should be shored.

During construction, the contractor is responsible for the excavation and maintenance of safe and stable slope angles considering the subsurface conditions and the methods of operation. Surcharge loads should be set back from the top of temporary excavations a minimum horizontal distance of 15 feet.

9.5 MANUFACTURED SLOPE CONSTRUCTION AND MAINTENANCE

9.5.1 General

Manufactured cut and fill slopes are shown on Plate 1 at a maximum gradient of 2(horizontal):1(vertical). The following sections contain general recommendations for cut and fill slopes for the site development.

9.5.2 Cut Slopes

Cut slopes may be constructed at a maximum gradient of 2(horizontal):1(vertical). The highest cut slope will roughly 20 feet high to the east of the retaining wall along the rear of Building B. This slope should exposed colluvium over Older Alluvium. The Older Alluvium was at least 20 feet deep in boring B-2. Therefore, only a possible minor exposure if any of bedrock is anticipated in the deepest cut slopes. Where topsoil/colluvium is present at the top of a cut slope, the top of the slope should be laid back or rounded. All cut slopes should be observed by an engineering geologist from this office for adverse geologic conditions.

9.5.3 Fill Slopes

Fill slopes may be constructed at a maximum gradient of 2(horizontal):1(vertical). Fills placed on slopes steeper than a 5(horizontal):1(vertical) gradient should be keyed and benched (horizontal benches) into competent in-place soil (after the required removals are made as discussed in Section 9.4.3) or bedrock. All keyways should be a minimum of 15 feet wide measured from the design toe of slope and cut a minimum depth of 2 feet at the toe into firm competent in-place soil or bedrock. Soil removal at the slope toe may be required to extend past the toe of slope as shown in Removals Beyond the Toe of Proposed Fill Slopes Detail attached in Appendix D.

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 before placing any fill. Horizontal benches should be a minimum of 5 feet wide, i.e. a minimum 5 feet of competent material. A representative of this office should observe benching before placing any fill soils.

Fill slopes over 10 feet high or depending on the conditions encountered during keying and benching operations should be constructed with a backdrain constructed at the heel of the slope keyway. The drain should consist of a 24 inch square section of rock (3/4 to 1 inch) wrapped in filter cloth having an equivalent screen opening size of 70± to 100. A perforated 4 inch diameter PVC schedule 40 pipe should be installed at the base of the gravel material with non-perforated outlet pipes. The outlets should be roughly 12 inches above the toe of slope or tied to a suitable drain system. The outlets at the surface should be protected with a concrete monument and the ends covered with a slotted cap to prevent rodent entry. The backdrain should be observed by a representative of this office prior fill placement. A Stabilization Fill Typical Detail showing recommended backdrain is attached in Appendix D for reference

Where possible, the outer slope faces should be overfilled and trimmed back to provide for firm, well-compacted surfaces. If the slopes are not overfilled and trimmed, it will be necessary to sheepsfoot and/or grid-roll the slopes. Slope faces should be tested and reworked as necessary to achieve the required 90 percent relative compaction. Select grading may be necessary so that fill slopes are constructed with materials with adequate surficial stability. The outer portions of slopes should be constructed with material having at least 250 psf of cohesion and a friction angle of 30 degrees.

9.5.4 Stabilization Fill Slope

The north facing cut slope along the rear of Building B may require reconstruction as a stabilization fill if a conventional retaining wall is to be used in this area. The stabilization fill will require the complete removal of the Calabasas Formation bedrock horizontally to the contact with Conejo Volcanics. The slope will need to be reconstructed using engineered fill per the recommendations for fill slopes.

9.5.5 Slope Maintenance

Slopes will require maintenance to reduce the risk of erosion and degradation with time due to natural or man-made conditions. Future performance of the slopes will depend on the control of the burrowing animals and maintenance of the brow ditches, drainage structures, and the slope vegetation as discussed below.

All graded or exposed natural slopes should be maintained with dense, deep rooting (minimum 2± feet deep), drought resistant ground cover and shrubs or trees. A reliable irrigation system should be installed on the slopes where necessary, adjusted so over watering does not occur, and periodically checked for leakage. Care should be taken to maintain a uniform, near optimum moisture content in the slopes, and to avoid over drying, or excess irrigation. Excess watering of slopes should be avoided to reduce the risk of erosion and surficial failures. Slopes should not be watered before forecasted rain.

All drainage structures (including those at the surface and buried) should be kept in good condition and clean the entire length to the outlet. Final grading of the site should provide positive drainage away from slopes, and water should not be allowed to pond or gather in a slope area. Burrowing animals, particularly ground squirrels, can destroy slopes; therefore, where present, immediate measures should be taken to evict them.

9.6 SOIL CORROSIVITY

Soils within the building pads should be analyzed during site rough grading to determine the corrosion potential of concrete and metals in contact with the on-site soils. In addition, generally fine grading soils are corrosive to ferrous and copper metals, which should be protected from contact with the on-site soils.

9.7 SOIL EXPANSIVENESS

Expansion tests previously performed on representative samples of the upper soil profile and bedrock resulted in expansion indexes of 80 and 177, which are in the moderately and critically high range, respectively. The recommended grading is intended to reduce the expansion potential within the building area to a soil expansion of less than 130. However, additional expansion tests should be performed within the finished pads to determine the appropriate final expansion to be used for final foundation design. For planning purposes, foundation design recommendations for a highly expansive soil are presented in the foundation section of this report.

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, availability of water, and the soil restraining pressure. Swelling occurs when clay soils become wet due to excessive water. Excessive water can be caused by poor surface drainage, over-irrigation of lawns and planters, and sprinkler or plumbing leaks.

Expansive clay soils can cause distress both as uplift and shrinkage or settlement. Construction on expansive soil has an inherent risk that should be acknowledged and understood by the builder and property owner. Recommendations presented in this report 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. Additional recommendations can be provided to further reduce the risk of expansive soil movement however additional costs will be incurred to implement these recommendations. Expansive soil movement can be on the order of 1 to 2 inches when exposed to excessive water or drying out. Therefore, the following should be maintained within the site.

a) Positive drainage should be continually provided and maintained away from structures and should not be changed creating an adverse drainage condition. Ponding or trapping of water adjacent foundations can cause differential moisture levels in subsurface soils. Plumbing leaks should be immediately repaired so the subgrade soils underlying the structure do not become saturated.

b) Initial landscaping should be undertaken in unpaved areas adjacent to structures. However, trees and shrubbery should not be planted where roots can grow under foundations and hardscape when they mature.

c) Landscape watering should be held to a minimum; however, landscaped areas should be maintained in a uniformly moist condition and not allowed to dry out.

9.8 FOUNDATION DESIGN

9.8.1 General

Foundations for the proposed structures should be supported entirely on engineered compacted fill prepared in accordance with the recommendations of the previous Site Preparation and Grading section. The foundations and slabs-on-grade should be designed by a structural engineer in accordance with the current applicable building code and following recommendations. A final expansion test(s) should be performed at the conclusion of the proposed rough grading to determine the expansions of the finished building pad. Soils with an EI of greater than 130 should not be placed within the building footprint or 5 feet beyond.

9.8.2 Design Data

The proposed construction may be supported on continuous and spread footings embedded in properly compacted fill. Continuous and isolated footings, a minimum of 12 and 24 inches wide respectively, may be designed to impose an allowable net bearing pressure of 2000 pounds per square foot (psf). This value may be increased by 250 psf for each foot of increased footing width. The bearing value may also be increased by one third for temporary wind and seismic loading. The provided allowable bearing capacity has a minimum factor of safety of 3.

Embedment depth should be a minimum of 36 inches. The embedment for exterior perimeter footings should be measured from the lowest adjacent rough grade or permanent lowest grade, whichever is deeper. Interior footing embedment may be measured from the top of the interior slab-on-grade. Embedment of basement footings may be measured from the top of slab provided the perimeter footing has an embedment of 36 inches below the perimeter grade. These dimensions apply for either continuous or individual spread (isolated) footings.

The footing reinforcement should be per the structural engineer's design. However, continuous footings should be reinforced with a minimum of two #5 bars in the top and bottom (total of four bars). The footings should be tied to the slabs by extending the slab reinforcement to within three inches of the footing bottom.

9.8.3 Lateral Soil Resistance

Lateral forces on foundations may be resisted by passive earth pressure and base friction. For the sides of footings bearing against engineered compacted fill or competent native soils, the lateral passive earth pressure may be considered equal to that exerted by an equivalent fluid having a density of 300 pounds per cubic foot (pcf) with a maximum pressure of 2,000 psf. Base friction may be computed at 0.3 times the normal load. Where the footing is adjacent a descending slope the passive pressure should be reduced to 25o pcf. Passive pressure may be increased by one third for short term loading. Lateral passive earth pressure should be reduced by one third when combined with base friction. A factor of safety of 1 is provided for these values.

9.8.4 Footings on or Adjacent Slopes

Footings on or near the top or toe of slopes should be deepened or setback to provide footing support and to reduce the impact of changes that can occur on slope faces. Changes to the slope, such as erosion, slumping, over watering and expansive soil action can affect the support of footings on or near descending slopes. Therefore, deepened footings or setbacks should be used for buildings and accessory structures sensitive to differential movement. The building to slope setback should be per the City of Agoura Hills amendments (Ordinance No. 10-381) to the California Building Code or a minimum setback of 5 feet, whichever is greater.

9.8.5 Estimated Settlements

Potential settlement of continuous and isolated footings is expected to be minor for static loading on the order of 1 inch or less, with a maximum differential settlement of 1/2± inch over a span of approximately 30 feet or between adjacent individual footings. This is provided building construction is started directly after 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. This is provided building construction is started directly after excavation, footings are cast soon after the footing excavation, and construction is completed in a timely manner. Footing movement could occur due to expansive soil movement if extreme moisture changes are allowed to occur under the foundations.

Settlements due to static loading are expected to occur rapidly as the loads are applied. All structures settle during construction and minor structure settlement can occur after construction during the life of the project. Minor wall or slab cracking may also be associated with settlement or soil movement such as due to seismic shaking. Settlement or soil movement could occur if the soils become saturated due to excessive water infiltration generally caused by excessive irrigation, poor drainage, etc.

9.8.6 Footing Excavations

Footings should be cut square and level, and cleaned of slough prior to casting concrete. Soil excavated from the footing trenches should not be spread over areas of construction unless properly placed and compacted. Footing excavations should be observed by a representative of this office prior to placing reinforcing steel. Footings should be cast as soon as possible to avoid deterioration of the footing subsoils. All cavities around footings and columns should be backfilled with soil compacted to 90% of the maximum dry soil density or concrete; crushed rock or gravel may be used for interior footings.

9.8.7 Premoistening

Footing subgrade soils should be premoistened to 3% over the optimum moisture content for a depth of 18 inches prior to concrete placement. All saturated soils should be removed from the footing excavation prior to casting the footings. This office should observe the subgrade soil moisture prior to placing concrete.

9.9 SLABS-ON-GRADE

9.9.1 Site Preparation

The subgrade for all slabs-on-grade, if disturbed during foundation and utility construction, should be conditioned prior to placement of aggregate materials. Loose soils should be removed to firm in-place material, the exposed subgrade processed, and the material replaced as engineered compacted fill or aggregate material.

9.9.2 Slab-on-Grade Design Data

Concrete slabs on-grade within the basement area for auto loading should be a minimum 6 inches thick and within the building interior in general should be a minimum 5 inches thick. The basement (subterranean garage) slab should be underlain by 6 inches of 3/4± rock. In non auto areas, the slabs may be underlain by 6 inches of clean sand. Slabs should be reinforced with a minimum of No. 4 bars on 18 inch centers in each direction placed at mid-depth of the slab. Also, interior slabs should be tied to the footings using No. 4 bars at 18 inch centers extending to within 3 inches of the base of the footing. These recommendations are for geotechnical concerns and the project structural engineer should evaluate the slab from a structural standpoint. In addition, the project structural engineer should determine the concrete compressive strength. Recommendations for basement slab under drains are provided below.

All exterior concrete slabs-on-grade (non-auto traffic) and walkways should be a minimum of 4 inches thick and underlain by a minimum of 6 inches of sand or sand-gravel base. In areas of heavy loading for truck traffic (including trash pickup areas and loading docks) the slab thickness should be increased to a minimum of 7 inches thick.

Concrete slabs (excluding sidewalks) should be reinforced with a minimum #4 bars at a spacing of 24 or 18 inches or less in both directions, respectively for the 51-90 and 91-130 soil expansion ranges. In either case, reinforcement should be placed at mid-depth of the slab. The recommendations for slab design should be revised if the underlying soils have an EI of greater than 130. Reinforced (1- #4 bar top and bottom) deepened edges of 18 inches should be constructed on all exterior (non-auto traffic) slabs that are adjacent landscape areas to prevent water from entering the sand base.

The slabs should be constructed with a 6 inch deep deepened edge where adjacent landscaped areas or slopes. All planter areas should be constructed so excess water drains away from concrete hardscape.

Interior and exterior concrete slabs on grade should be provided with tooled crack control joints at 10-15 foot centers or as specified by the structural engineer. Sidewalks should be scored (tooled crack control joints) into square panels (that is a 5-foot wide sidewalk should be scored every 5 feet). Concrete placement should be performed per the recommendations provided in the Concrete Placement and Cracking section of this report.

9.9.3 Slab Under Drain System

A drain system should be designed and constructed below the basement slab on-grade. Below slab drains are intended to provide drainage of groundwater if it occurs from below the basement floor. However, drains will not drain water naturally held by the soils or stop vapor migration.

The interior basement (subterranean garage) slab should be constructed on 6 inches of 3/4± rock. An acceptable gradation would be as specified in the Standard Specifications for Public Works Construction (Greenbook) Table 200-1.2, Crushed Rock and Rock Dust for 3/4 inch rock. The rock should be placed on a properly prepared subgrade as addressed herein and should be separated from the subgrade by a single layer of filter cloth. The filter cloth having a maximum equivalent opening of 0.212 mm (70 U.S. sieve size) should be lapped at least 12 inches at the seams and the seams sealed per the manufacturer's specifications.

Directly above the filter cloth within the rock, rows of 4 inch PVC (Schedule 40) perforated pipes should be placed with holes down at a maximum pipe spacing of 30 feet and may be placed horizontally on the filter cloth. Piping should be routed around footings and grade beams wherever possible however should not extend below any footing. Where piping must cross a structural element, a sleeve should be constructed per the structural engineer's design. The slab under drain system is in addition to the perimeter retaining wall backdrain.

Drainpipes should be connected to a single outlet pipe prior to exiting the building. Connector pipes should be placed preferably with a slight slope to drain (or horizontal if necessary). Rock should be carefully placed over the piping so as not to disturb the pipe layout or distort the piping. Manifold piping or solid piping connecting the drains may be 4 inch or larger PVC (Schedule 40) with glued connections.

The drains should be hydraulically connected to a sump and pump system or allowed to flow to suitable drainage area such as the natural drainages. The design of the slab under drain system should be reviewed by this office as part of the foundation review. Generally, the slab under drain and pipe is shown on the plumbing plans, which should be provided to this office as part of the foundation review.

Elevator pits should also be constructed with perimeter drains that extend a minimum of 6 inches below the top of the pit floor. As an alternative, elevator pits may be constructed watertight. The drains may be constructed similar to the perimeter basement wall drains.

9.9.4 Concrete Placement and Cracking

Minor cracking of concrete slabs is common and is 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. Therefore, the concrete should be placed using procedures to minimize the cracking within the slab. Shrinkage cracks can become excessive if water is added to the concrete above the allowable limit and proper finishing and curing practices are not followed. Concrete mixing, placement, finishing, and curing should be performed per the American Concrete Institute Guide for Concrete Floor and Slab Construction (ACI 302.1R). 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 crack control joints at 10-15 foot centers or as specified by the structural engineer.

9.9.5 Premoistening

Soils under lightly loaded slabs on-grade (including exterior slabs and walkways) should be pre-moistened to 3% over the optimum moisture content for a depth of 18 inches. This office should observe the subgrade soil moisture prior to pouring the concrete.

9.9.6 Tile Flooring

Tile flooring can crack, reflecting cracks in the on grade concrete slab below the tile. Therefore, if tile flooring is used, the slab designer should consider this in the design of concrete slabs on grade where tile will be placed. The tile installer should consider installation methods to reduce possible tile cracking. Placement of a vinyl crack isolation membrane between tile and concrete slabs on grade (utilizing approved materials and techniques per Tile Council of America/Ceramic Tile Institute guidelines) is one such method to reduce possible cracking of tile.

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 manufacturer's specifications. The concrete surface should be sealed per the manufacturer's specifications if the moisture readings are excessive. It may be necessary to select floor coverings that are applicable with high moisture conditions.

9.9.7 Moisture Vapor Retarder Layer

An appropriate moisture vapor retarder layer should be installed and maintained below interior slabs on grade. However, a moisture vapor retarder layer may be eliminated in the auto parking areas. The intent of the moisture vapor retarder layer is to reduce moisture vapor transmission through the slab.

Ten-mil polyethylene plastic sheeting may be used as a minimum moisture vapor retarder layer placed mid-height in the sand below the slab. Edges of the sheeting should overlap at least 12 inches onto an adjacent sheet. Where necessary or where a heavier moisture vapor retarder layer is desired to reduce possible water vapor transmission, products specifically manufactured as moisture retarders per ASTM E 1745-97 Standard Specification for Plastic Water Vapor Retarders Used in Contact with Soil or Granular Fill under Concrete Slabs should be considered below the interior concrete slabs on-grade. The retarder should be installed per ASTM E1643-98(2005) Standard Practice for Installation of Water Vapor Retarders Used in Contact with Earth or Granular Fill Under Concrete Slabs.

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 E1643-98(2005) Standard Practice for Installation of Water Vapor Retarders Used in Contact with Earth or Granular Fill Under Concrete Slabs. Proper construction practices should be followed during construction of the slab on-grade. Repair and seal tears or punctures in the moisture barrier resulting 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 be made aware of the moisture vapor retarder and requirement to protect the layer. Perforations made in the layer by the concrete contractor should be properly sealed prior to concrete placement. In addition, if the concrete is placed directly on top of the layer the concrete contractor should make the necessary changes in the concrete placement and curing. Placing concrete directly on top of the moisture vapor retarder layer allows the layer to be observed for damage directly prior to concrete placement.

The grade of the project should be kept as high as practical and the interior slabs should be maintained as high as practical above the exterior grades. Drainage should be maintained away from the structures. Provide proper drainage and elevation of ground adjacent the slab (that is the ground surface should be at least 6 inches below the wall plate). In addition, the landscaping should not be over-watered resulting in excess moisture below the slab

9.10 RETAINING WALL DESIGN

9.10.1 Foundations

Retaining wall footings should be design in accordance with the foundation design recommendations previously provided herein under Foundation Design for bearing capacity, lateral resistance, and embedment.

9.10.2 Active Pressures

Retaining walls should be designed to resist active pressure exerted by compacted backfill or retained soil. Retaining walls that may yield at the top may be designed for an equivalent fluid pressure of 45 and 60 pounds per cubic foot (pcf) for a level and 2(horizontal):1(vertical) sloping backfill, respectively.

The large retaining wall at the rear of Building B should be designed as a soil nail retaining wall. If a conventional retaining wall must be used, substantial regrading of the backcut will be required to remove the contorted Calabasas Formation.

Retaining walls restrained at the top should be designed for a lateral earth pressure of 40H where H is the supported height of the wall. The lateral earth pressures should be applied with a trapezoidal distribution. The lateral earth pressure at the ground surface may be taken as zero. The pressure will then increase with depth to the design pressure at a depth of 0.2H below the ground surface. The pressure would then extend uniformly to a depth of 0.8H and then decrease uniformly to zero at the base of the excavation. The resultant of the wall pressure is in units of psf. The wall loads are for static loading on the walls.

Areal surcharge may be treated as additional height of backfill where one foot of additional height is assumed for each 125 psf of areal surcharge. An areal surcharge of 300 psf should be included in the design where the retaining wall supports street traffic.

The above active pressures are not designed to resist expansion of the backfill. Therefore, if water is allowed to saturate backfill or backcut materials consisting of clayey soils, the expansion pressure could exceed the active pressures provided. An engineering geologist from this office should observe retaining wall backcuts in bedrock for adverse geologic conditions. The above active pressures are not designed to retain an adverse geologic condition.

9.10.3 Seismic Pressures

Walls greater than 6 feet in height should be designed for a temporary seismic lateral pressure equal to an inverted triangular pressure of $18H^2$. The resultant of the seismic pressure should be considered to act at 0.67H from the base of the wall, where H is the height of the wall measured from the base of the footing to the top of the backfill.

9.10.4 Retaining Wall Drainage and Backfill

A drainage system should be constructed behind the retaining / basement walls. In addition, retaining walls should be waterproofed.

The drainage system may consist of a prefabricated drainage composite consisting of a filter fabric bonded to a corrugated panel. For the basement walls the composite drain may be tied to the interior piping system using connectors as provided by the drain manufacturer.

Aggregate drains should consist of a minimum 1 foot wide continuous section of clean gravel (½ to ¾ inch) or equivalent drain material wrapped in filter fabric. The drain material should extend from the base of the wall to within 2 feet of the top of wall. The upper 2 feet of exterior wall backfill should consist of compacted native soils. The drain material should be drained by a perforated drainpipe placed holes down on a maximum of 2 inches of drain material. The invert of the pipe should be a minimum of 6 inches below the lowest adjacent grade.

The basement 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 an adjacent slab-on-grade. An as-built plan should be prepared detailing the location of the wall drainage system.

9.10.5 Backfilling

Retaining walls should be backfilled with soils having a soil expansion of less than 50, therefore, select grading or import may be needed for the desired backfill. The backfill should be placed in 6-inch lifts at slightly over optimum moisture content and compacted to at least 90 percent relative compaction. If the backcut is flatter than ½(h):1(v), the backfill should be benched into the backcut slope. Light equipment should be used immediately behind the walls to prevent possible over-stressing. Any bracing needed to resist wall movement should be in-place prior to placing the backfill.

9.10.6 Soil Nail Wall Design

A soil nail retaining wall is recommended for the large retaining wall along the rear of Building B. Within the Calabasas Formation, shear strength of 19 degrees friction and 380 pounds per cubic foot cohesion should be used to account for the contorted nature of the Calabasas Formation. The Calabasas Formation is at a depth of 20 feet below the ground surface as indicated in boring B-2. Shear strength of 35 degrees of friction and cohesion of 200 pounds per cubic foot may be used in the alluvial soils overlying the bedrock. A unit weight of 125 pounds per cubic foot may be used for both units.

9.11 SITE DRAINAGE

Positive drainage should be provided away from slopes and structures during and after construction. Planters near a structure should be constructed so irrigation water will not saturate the soils underlying the building footings and slabs. Site drainage should conform to the grading plan or applicable building codes. Landscape planting and trees should be kept away from foundations or flatwork to avoid roots extending beneath foundations and slabs. Irrigation watering should not saturate soils below or adjacent foundations.

9.12 GUTTERS AND DOWNSPOUTS

Gutters and downspouts should be installed to collect roof water that may otherwise infiltrate soils adjacent the structures. The downspouts should be drained into non-perforated PVC collector pipes that will carry the water away from the structure.

9.13 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% relative compaction, respectively, just prior to placing the asphalt.

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 center 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 is either collected by an area drain system or is drained onto and not beneath the adjacent AC pavement. Consideration should be given to deepening the curbs adjacent planters to minimize water 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.

9.14 PLAN REVIEW

As the development process continues and finalized grading/foundation/pool plans and specifications are developed, they should be reviewed by Gorian and Associates, Inc. Additional geotechnical recommendations may be warranted at that time.

9.15 CONSTRUCTION OBSERVATIONS AND TESTING

All aspects of the construction addressed from a geotechnical standpoint (i.e., subgrades, fill placement, backfill, and footings) should be observed (and tested when appropriate) by this firm.

9.16 SECTION 111

The opinion of this office is if the project is constructed in accordance with our recommendations and properly maintained, the proposed structures will be safe against hazard from landslide, settlement, or slippage, and the proposed building or grading construction will have no adverse effect on the geologic stability of property outside of the building site. The nature and extent of tests conducted for purposes of this declaration are, in the opinion of the undersigned, in conformance with generally accepted practice in the area. Test findings and statements of professional opinion do not constitute a guarantee or warranty, express or implied.

10. CLOSURE

This report was prepared under the direction of a State 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 Agoura Hills Center Properties, LLC and their design consultants solely for design and construction of the project as described herein. It may not contain sufficient information for other uses or the purposes of other parties. These recommendations should not be extrapolated to other areas or used for other facilities without consulting Gorian and Associates, Inc.

The scope of the services provided by Gorian and Associates, Inc. and its staff, excludes responsibility and/or liability for work conducted by others. Such work includes, but is not limited to, means and methods of work performance, quality control of the work, superintendence, sequencing of construction and safety in, on, or about the jobsite.

Recommendations are based on interpretations of the subsurface conditions concluded from information gained from subsurface exploration and a surficial reconnaissance. Interpretations may differ from actual subsurface conditions, which can vary horizontally and vertically across the property. Due to possible subsurface variations, this office should observe all aspects of field construction addressed in this report. Persons using this report for bidding or construction purposes should perform such independent evaluations, as they deem necessary.

-oOo-

Thank you for allowing our firm the opportunity to provide you geotechnical services on your project. Please contact us if you have questions concerning this geotechnical report or require additional information.

Respectfully,
Gorian and Associates, Inc.

By: Jerome J. Blunck, GE151
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Date: November 11, 2011
GDI #: 11.00103.0183

CITY OF AGOURA HILLS - GEOTECHNICAL REVIEW SHEET

To: Doug Hooper

Project Location: 30800 Agoura Road, Agoura Hills, California.

Building & Safety #: 08-CUP-001

Geotechnical Report: Gorian & Associates, Inc. (2007), "Geotechnical Update Study, Senior Housing Community, APN# 2061-001-025, 30800 Agoura Road, Agoura Hills, California," Log Number: 2272-1-0-100, dated September 7, 2007.

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Plans: HMK Engineering, Inc. (2003), "Preliminary Grading Plan, Tentative Tract Map No. 71742, County of Los Angeles," Scale 1"=40', W.O. 01-537, Plot Date August 18, 2003.

Previous Reviews: None.

FINDINGS

Planning/Feasibility Issues	Geotechnical Report
☐ Acceptable as Presented	☐ Acceptable as Presented
☒ Response Required	☒ Response Required

REMARKS

Gorian and Associates, Inc. (GAI; consultant) prepared the above-referenced reports for the proposed development at the site located at 30800 Agoura Road in the City of Agoura Hills, California. The proposed development includes the construction of 46-unit senior condominium project. The above-referenced preliminary grading plan shows that the project comprises two buildings with underground parking, with up to 12 ft high retaining walls and 1½(h):1(v) gradient cut slope. Other associated improvements include access and landscaping areas.

The City of Agoura Hills – Planning Department reviewed the referenced report from a geotechnical perspective for compliance with applicable codes, guidelines, and standards of practice. GeoDynamics, Inc. (GDI) performed the geotechnical review on behalf of the City. Based upon a review of the submitted reports, the consultant shall adequately respond to the following Planning/Feasibility comments prior to consideration by the Planning Commission of approval of Case # 08-CUP-001. The Consultant should respond to the following Report Review comments prior to Building Plan-Check Approval. Plan-Check comments should be addressed in Building & Safety Plan Check. A separate geotechnical submittal is not required for plan-check comments.

Note to City: *The grading plan show proposed retaining walls higher than 6 ft and a cut slope steeper than 2(h):1(v). The City code limits the height of retaining walls to 6 ft or less, and manufactured slope gradients to 2(h):1(v) or flatter. Variances for retaining wall heights and slope gradients may be required for approval of the grading plan. No justification for deviation from the code requirements were provided in the referenced reports.*

Planning/Feasibility Comments

1. The latest report is over four years old and addresses an earlier plan. The consultant should provide a stand-alone update report. The update report should address changes to geotechnical conditions at he site, and should be based on the most current plan. The update report should also include new or updated cross sections (including orthogonal sections through buildings) and recommendations as necessary to adequately depict relationships between the existing geologic conditions and the various aspects of the proposed development/grading plan (example: pedestrian bridge, debris basins, etc.). Additional mitigation measures should be recommended as necessary.

2. New fills are proposed to derive support from existing fill along Agoura Road. Sufficient exploration and testing appears warranted to verify the adequacy of this existing fill to support the proposed improvements. If the fill needs to be removed, the consultant should provide specific recommendations to support the existing road during the fill removal.

3. The consultant should provide a more detailed discussion of stability issues where contorted Calabasas Formation will be exposed in cut-slopes and retaining walls. Cut-slopes and retaining walls depicted on the current plan appear likely to expose Calabasas Formation with bedding planes at least locally inclined northerly at low angles. The consultant should provide analyses to verify that the recommended equipment width stability fill will be adequate to mitigate the potential for translational failures along unsupported sections of bedding in the Calabasas Formation. Continuity of bedding should be assumed in critical areas unless sufficient field exploration is provided to demonstrate a lack of continuity. Mitigation measures should be recommended as necessary.

4. The contact between the Older Alluvium and the underlying Calabasas Formation is reported to be inclined northerly at an overall gradient of about 13 degrees, with variable material conditions. At some locations the contact was found to be abrupt. Other locations encountered residual soil of gray clay (B-1), or plastic clay seams within the uppermost part of the Calabasas Formation inclined roughly parallel to the contact (B-3). The consultant should discuss and evaluate as necessary the potential for translational deformation where this contact will be exposed in future cut-slopes or retaining wall back-cuts. Mitigation measures should be recommended as necessary.

5. Jarosite is noted in the weathered section of the Calabasas Formation in Boring B-4. Jarosite is commonly associated with the oxidation of sulfide minerals known to exist in the Calabasas Formation. This process can lead to severe expansion and produces a highly acidic environment that can be very corrosive to concrete and steel. The consultant should discuss the potential for sulfide expansion and the related potential for highly corrosive soil resulting from the oxidation reaction of sulfide minerals where relatively unoxidized Calabasas Formation will be exposed below the future structures or utilized in future fill materials. Mitigation measures should be recommended as necessary.

6. Review of currently proposed grades and previous cross sections indicates that the final pad grades will be underlain by and likely transition between Conejo Volcanics, Calabasas Formation, Older Alluvium and artificial fill. The consultant recommends overexcavation to at least 3 ft below the footings within building pad areas. Hence, the consultant should discuss and substantiate the adequacy of the recommended depth of overexcavation to mitigate the potential for differential expansion. Mitigation measures should be recommended as necessary.

7. The City of Agoura Hills requirements for slope to structure/foundations setback should be complied with. Please note that the city of Agoura Hills has more stringent setback requirements than the California Building Code as recommended on page 17 of the October 12, 2000 report.

Report Review Comments

1. The consultant should review final development/grading plans when they become available and provide additional geotechnical recommendations as necessary.

2. The consultant recommends that fill keyways should extend a minimum of two feet into “firm competent in-place soil”. The consultant should clarify which units are acceptable for keyway support.

Plan-Check Comments

1. The name, address, and phone number of the Consultant and a list of all the applicable geotechnical reports shall be included on the building/grading plans.
2. The following note must appear on the grading and foundation plans: *"All retaining wall excavations shall be reviewed by the project engineering geologist for the presence of adversely oriented joint surfaces. Adverse surfaces shall be evaluated and supported in accordance with recommendations of the project geotechnical engineer."*
3. The grading plan should include the limits and depths of overexcavation for the swimming pool, the road and flatwork areas as recommended by the Consultant.
4. The following note must appear on the grading and foundation plans: "*Excavations shall be made in compliance with CAL/OSHA Regulations*."
5. The following note must appear on the foundation plans: "*All foundation excavations must be observed and approved, in writing, by the Project Geotechnical Consultant prior to placement of reinforcing steel*."
6. Foundation plans and foundation details shall clearly depict the embedment material and minimum depth of embedment for the foundations.
7. Drainage plans depicting all surface and subsurface non-erosive drainage devices, flow lines, and catch basins shall be included on the building plans.
8. Final grading, drainage, and foundation plans shall be reviewed, signed, and wet stamped by the consultant.
9. Provide a note on the grading and foundation plans that states: "*An as-built report shall be submitted to the City for review. This report prepared by the Geotechnical Consultant must include the results of all compaction tests as well as a map depicting the limits of fill, locations of all density tests, outline and elevations of all removal bottoms, keyway locations and bottom elevations, locations of all subdrains and flow line elevations, and location and elevation of all retaining wall backdrains and outlets. Geologic conditions exposed during grading must be depicted on an as-built geologic map*."

If you have any questions regarding this review letter, please contact GDI at (805) 496-1222.

Respectfully Submitted,

GeoDynamics, INC.

Ali A. Haq

Ali Abdel-Haq
Geotechnical Engineering Reviewer
GE 2308 (exp. 12/31/11)

Christopher J. Sexton
Engineering Geologic Reviewer
CEG 1441 (exp. 11/30/12)

APPENDIX A

LOGS OF SUBSURFACE EXPLORATION

The following logs of subsurface exploration are as previously presented in the Gorian and Associates, Inc. report dated October 12, 2000.

GORIAN & ASSOCIATES, INC.
Applied Earth Sciences

Project: Khantzis, 30800 Block of Agoura Rd.
Drill Co. and Rig Type: TriValley, 24" Bucket Auger
Hammer: 3450# 0-27', 2050# 27-57'
Boring Diameter: 24" Surface Elevation: 964'±

Work Order: 2272-1-0-11
Report Log No.: 20524
Logged by: CHD Date: 08/03/00

Depth (ft)	Undisturbed	Bulk	Blow Counts	Moisture Content (% dry weight)	Dry Density (pcf)	Penetrometer (tsf)	USCS	Soil/ Lithology	Description	Remarks
0							CL		COLLUVIUM: AT 0'-1.5'; Very dark grayish brown (10YR 3/2) sandy clay, some cobbles and gravel (damp, hard). Basalt clasts common.	
			4/ 12"	14.7	104		CL		: AT 1.5'-4.5'; Dark grayish brown (10YR 4/2) to grayish brown (10YR 5/2) sandy silty clay (damp to moist, hard). Some gravel. Few cobbles. At 4'; becoming very moist to wet.	
			4/ 12"	18.8	105					
5			6/ 12"	8.2	107		CL		: AT 4.5'-7'; Grayish brown (2.5Y 5/2) sandy clay with gravel (damp-moist, hard). Some cobbles. Below 5', hard drilling.	
							CL		OLDER ALLUVIUM: AT 7'-24'; Brownish yellow (10YR 6/6) silty clay mottled with light gray (5Y 7/2) (very moist, very stiff). Seepage at 7', Minor caving. At 15'; becoming stiff, very moist. At 23'; trace fine sand.	
10			2/ 12"	36.2	85					
15			1/ 12"	29.9	94					
20			1/ 12"	18.6	100					
							SM		: AT 24'-25'; Brownish yellow (10YR 6/6) silty fine to coarse sand, trace clay. Groundwater at 24'.	
25			1/ 12"	20.2	98		SC		: AT25'-27'; Brownish yellow (10YR 6/6) clayey fine sand (very moist, medium dense).	
							CL		: AT27'-27.5'; Brownish yellow (10YR 6/6) sandy clay (very moist, stiff).	
							CL		: AT 27.5'-31'; Yellowish brown (10YR 5/4) sandy clay (very moist, hard). Some gravel.	
30			11/ 12"	11.4	119					
							SC		: AT 31'-35'; Yellowish brown (10YR 5/4) clayey fine to coarse sand (saturated, dense). Common gravel, some cobbles.	

Gorian & Associates, Inc.
Applied Earth Sciences

Project: Khantzis, 30800 Block of Agoura Rd.
Drill Co. and Rig Type: TriValley, 24" Bucket Auger
Hammer: 3450# 0-27', 2050# 27-57'
Boring Diameter: 24" Surface Elevation: 964'±

Work Order: 2272-1-0-11
Report Log No.: 20524
Logged by: CHD Date: 08/03/00

Depth (ft)	Undisturbed	Bulk	Blow Counts	Moisture Content (% dry weight)	Dry Density (pcf)	Penetrometer (tsf)	USCS	Soil/ Lithology	Description	Remarks
35	■		7/ 12"	17.6	109		CL		: AT 35'-36.5'; Pale brown (10YR 6/3) silty clay, some coarse sand (moist, hard).	
							GC/ SC		: AT 36.5'-40.5'; Pale brown (10YR 6/3) clayey fine to coarse sand, common gravel. Interbedded with sandy clay.	
40	■		7/ 12"	26.3	100		CL		RESIDUAL SOIL: AT 40.5'-42.5'; Grayish brown (10YR 5/2) clay. Some coarse grains of sand (moist, hard). At 41'; crowd used to "get a bite". Few gravel.	
									CALABASAS FORMATION: AT 42.5'-46'; Olive gray (5Y 5/2) claystone. Fractured with iron oxide staining. At 45'; becoming greenish gray (10Y 5/1).	
45	■		6/ 12"	32.8	88					
									Total depth 46': Groundwater at 24', Caving from 5'-7', No Downhole.	
50										

GORIAN & ASSOCIATES, INC.
Applied Earth Sciences

Project: Khantzis, 30800 Block of Agoura Rd.
Drill Co. and Rig Type: TriValley, 24" Bucket Auger
Hammer: 3450# 0-27', 2050# 27-57'
Boring Diameter: 24" Surface Elevation: 1018'± Logged by: CHD Date: 08/03/00 & 08/04/

Work Order: 2272-1-0-11
Report Log No.: 20524

Depth (ft)	Undisturbed	Bulk	Blow Counts	Moisture Content (% dry weight)	Dry Density (pcf)	Penetrometer (tsf)	USCS	Soil/ Lithology	Description	Remarks
0							GM/ ML		COLLUVIUM: AT 0'-1'; Very dark grayish brown (10YR 3/2) silt (damp, stiff). Porous. Common gravel and cobbles of basalt and dacite.	
							CL		: AT 1'-3'; Very dark grayish brown (10YR 3/2) sandy clay (damp, stiff). Common gravel and cobbles. Some boulders.	
							GC/ SC		OLDER ALLUVIUM: AT 3'-20'; Brown (10YR 5/3) clayey fine to coarse sand (damp, very dense). Common gravel. Some cobbles. At 4'; core barrel used on large cobbles. At 4'; becoming yellowish brown (10YR 5/4) (moist, very dense). Very difficult drilling. Alternate core barrel and bucket auger. Rare shale fragments. At 16½'; crowd used. At 19'; common cobbles. Contact at 20'; is highly irregular, undulatory yet generally horizontal.	
5			12/ 12"	9.4	106					
10			9/ 10"							
15			14/ 9"	13.0	112					

GORIAN & ASSOCIATES, INC.
Applied Earth Sciences

Project: Khantzis, 30800 Block of Agoura Rd.
Drill Co. and Rig Type: TriValley, 24" Bucket Auger
Hammer: 3450# 0-27', 2050# 27-57'
Boring Diameter: 24"   Surface Elevation: 1018'±   Logged by: CHD
Work Order: 2272-1-0-11
Report Log No.: 20524
Date: 08/03/00 & 08/04/

Depth (ft)	Undisturbed	Bulk	Blow Counts	Moisture Content (% dry weight)	Dry Density (pcf)	Penetrometer (tsf)	USCS	Soil/ Lithology	Description	Remarks
20			6/ 12"	17.4	102				CALABASAS FORMATION: AT 20'-41'; Pale olive (5Y 6/3) claystone (moist, hard). Fractured with manganese and iron oxide staining. After sample at 20'; 24" bucket auger used. Generally massive. Plastic deformation. At 25'; becoming interbedded with light olive gray (5Y 6/2) occasionally interbedded with brownish yellow (10YR 6/8) claystone. At 29½'; 1/2" thick silty fine sand interbed. At 30'; becoming interbedded with brown (10YR 5/3) to gray (5Y 6/1) claystone. At 32'; becoming interbedded with light yellowish brown (2.5Y 6/4) claystone. At 35'; 1/4" thick silty fine sand interbed. At 36'; minor interbed of light yellowish brown (2.5Y 6/4) siltstone (indurated). Not continuous.	
25			4/ 12"	23.6	100					
										APPROXIMATE ATTITUDE ON BEDDING AT 28' N75°E/12°NW
										ATTITUDE ON BEDDING AT 29½' N30°E/10°NW
30			7/ 12"	15.7	108					
										APPROXIMATE ATTITUDE ON BEDDING AT 35' N80°E/37°SE
35			7/ 12"	23.5	101					

GORIAN & ASSOCIATES, INC.
Applied Earth Sciences

Project: Khantzis, 30800 Block of Agoura Rd.
Drill Co. and Rig Type: TriValley, 24" Bucket Auger
Hammer: 3450# 0-27', 2050# 27-57'
Boring Diameter: 24" Surface Elevation: 1018'± Logged by: CHD Date: 08/03/00 & 08/04/

Work Order: 2272-1-0-11
Report Log No.: 20524

Depth (ft)	Undisturbed	Bulk	Blow Counts	Moisture Content (% dry weight)	Dry Density (pcf)	Penetrometer (tsf)	USCS	Soil/ Lithology	Description	Remarks
40			7/ 12"	27.7	97					
									Total depth 41': No caving, No groundwater, Downhole logged to 36'	
45										
50										

GORIAN & ASSOCIATES, INC.
Applied Earth Sciences

Project: Khantzis, 30800 Block of Agoura Rd.
Drill Co. and Rig Type: TriValley, 24" Bucket Auger
Hammer: 3450# 0-30', 2050# 30-60'
Boring Diameter: 24" Surface Elevation: 998'±
Work Order: 2272-1-0-11
Report Log No.: 20524
Logged by: JPQ Date: 08/08/2000

Depth (ft)	Undisturbed	Bulk	Blow Counts	Moisture Content (% dry weight)	Dry Density (pcf)	Penetrometer (tsf)	USCS	Soil/ Lithology	Description	Remarks
0							ML		COLLUVIUM: AT 0'-1.2'; grayish brown (10YR 5/2) clayey silt with gravel. Clasts to 1', subangular volcanics. (Hard).	
							CL		: AT 1.2'-2.5'; grayish brown (10YR 5/2) silty clay with gravel. Clasts to 1', subangular volcanics. (Hard). Basalt contact gradual.	
							GM/ ML		OLDER ALLUVIUM: AT 2.5'-15.3'; light olive brown (2.5Y 5/3) grading to light yellowish brown (10YR 6/4) clayey silt and gravel. Clasts subangular to subrounded, gravel and cobbles to 6" and chiefly composed of volcanics. Local areas with heavy limonitic staining. Minor manganese oxide. Few rootlets. Soil is hard to dense and breaks along polished fractures. Clear basal contact.	
5			8/ 12"	19.3	92					
10			15/ 10"	23.0	86					APPROXIMATE ATTITUDE ON POLISHED SURFACE AT 10.9' N73°W/52°NE
15			11/ 12"	14.4	85		SW		: AT 15.3'-16.9'; light yellowish brown (10YR 6/4) silty fine to coarse sand with gravel. Basal contact abrupt, planar. seepage.	
									CALABASAS FORMATION: AT 16.9'+; olive gray (5Y 5/2) massive silty claystone. Very weathered and "sheared" to 17.3'; rootlets along plastic clay "seams" subparallel with upper contact. Below 17.3'; tightly fractured. Some fractures invaded by rootlets and calcium carbonate. Others stained by limonite. Thin plastic clay bed at 21', subparallel with limonite stained laminae of very fine-grained sandstone. Polished ? bedding surface at 29'. Bedding surface with subhorizontal striations, invaded by rootlets and calcium carbonate. Overall, bedding is poorly defined.	APPROXIMATE ATTITUDE ON CONTACT AT 16.9' N62°W/13°NE
			2/ 12"	20.8	102					
20										APPROXIMATE ATTITUDE ON CLAYBED AT 21' N30°W/24°SW
25			2/ 12"	25.3	98					
										APPROXIMATE ATTITUDE ON BEDDING AT 29' N81°W/88°SW
30			7/ 12"	25.3	99					

GORIAN & ASSOCIATES, INC.
Applied Earth Sciences

Project: Khantzis, 30800 Block of Agoura Rd.
Drill Co. and Rig Type: TriValley, 24" Bucket Auger
Hammer: 3450# 0-30', 2050# 30-60'
Boring Diameter: 24"   Surface Elevation: 998'±   Logged by: JPQ   Date: 08/08/2000
Work Order: 2272-1-0-11
Report Log No.: 20524

Depth (ft)	Undisturbed	Bulk	Blow Counts	Moisture Content (% dry weight)	Dry Density (pcf)	Penetrometer (tsf)	USCS	Soil/ Lithology	Description	Remarks
35			7/ 12"	19.0	104					APPROXIMATE ATTITUDE ON BEDDING AT 34½' N52°W/90°
40				23.8	102				: AT 40'-41'; dark gray to black silty claystone.	
									Total depth 41': No caving, Moderate seepage at 15.3' to 16.9', Downhole logged to 35'	
45										
50										

BORING: B-4

GORIAN & ASSOCIATES, INC.
Applied Earth Sciences

Project: Khantzis, 30800 Block of Agoura Rd.
Drill Co. and Rig Type: TriValley, 24" Bucket Auger
Hammer: 3450# 0-30', 2050# 30-60'
Boring Diameter: 24" Surface Elevation: 967'± Logged by: JPQ Date: 08/07/00

Work Order: 2272-1-0-11
Report Log No.: 20524

Depth (ft)	Undisturbed	Bulk	Blow Counts	Moisture Content (% dry weight)	Dry Density (pcf)	Penetrometer (tsf)	USCS	Soil/ Lithology	Description	Remarks
0							GM/ ML		COLLUVIUM: AT 0'-3½'; Grayish brown (10YR 5/2) silt with gravel and cobbles. Coarse crumb structure near surface. Rootlets and root filaments common. 1' diameter clast at approximately 3". Clasts chiefly gravel size, subangular to subround. Core barrel at 1' with crowd.	
5			15/ 10"	11.2	81		GM		OLDER ALLUVIUM: AT 3½'-6½'; Light yellowish brown to light olive brown (2.5Y 5-6/3) silty gravel. Possible self-supporting volcanic clasts to approximately 1'. Large boulder-size clasts at base (approximately 1').	
			6/ 12"	17.6	106		ML		: AT 6½'-10'; Light yellowish brown (10YR 6/4) with light greenish gray "veinlets" (10Y 7/1) very clayey silt with trace sand.	
			3/ 12"							
10									CALABASAS FORMATION: AT 10'+; Light olive brown (2.5Y 5/4) and greenish gray (10Y 5/1) silty claystone. Local calcareous "veinlets". Bedding inclined 15-20°, non-fissile. Minor jarosite.	
15			3/ 12"	25.3	96					
20			4/ 12"	26.8	97					
									Total depth 21': No groundwater observed, No observed caving, No downhole.	
25										

GORIAN & ASSOCIATES, INC.
Applied Earth Sciences

BORING: B-5

Project: Khantzis, 30800 Block of Agoura Rd.
Drill Co. and Rig Type: TriValley, 24" Bucket Auger
Hammer: 3450# 0-30', 2050# 30-60'
Boring Diameter: 24" Surface Elevation: 958'±
Work Order: 2272-1-0-11
Report Log No.: 20524
Logged by: JPQ Date: 08/08/00

Depth (ft)	Undisturbed	Bulk	Blow Counts	Moisture Content (% dry weight)	Dry Density (pcf)	Penetrometer (tsf)	USCS	Soil/ Lithology	Description	Remarks
0							ML		FILL: AT 0'-1'; Dark grayish brown (10YR 4/2) very silty clay. Coarse crumb structure. Numerous root filaments disturbed by disking. Core bucket at 1' due to rock.	
1							GM		COLLUVIUM: AT 1'-4½'; Dark grayish brown (10YR 4/2) silty gravel. Volcanic clasts to approximately 1'. (approximately 20% >6").	
5			7/ 12"	25.5	65		ML		OLDER ALLUVIUM: AT 4½'+; Light olive brown (2.5Y 5/3-4) to brownish yellow (10YR 6/6) with depth clayey silt with trace sand and gravel. Reduced adjacent to root traces. Thin interbed of wet fine sand in 15' sample (15'2").	
10			3/ 12"	27.1	97					
15			3/ 12"	33.2	90					
16									Total depth 16': No groundwater observed (after 10 minutes), but wet at 15', No caving, No downhole.	
20										

BORING: B-6

Page 1 of 1

GORIAN & ASSOCIATES, INC.

Applied Earth Sciences

Project: Khantzis, 30800 Block of Agoura Rd.

Drill Co. and Rig Type: TriValley, 24" Bucket Auger

Hammer: 3450# 0-30', 2050# 30-60'

Boring Diameter: 24" Surface Elevation: 977'±

Work Order: 2272-1-0-11

Report Log No.: 20524

Logged by: JPQ Date: 08/08/00

Depth (ft)	Undisturbed	Bulk	Blow Counts	Moisture Content (% dry weight)	Dry Density (pcf)	Penetrometer (tsf)	USCS	Soil/ Lithology	Description	Remarks
0							ML		COLLUVIUM: AT 0'-1'; Greyish brown (10YR 5/2) silt with gravel. Root filament common.	
							SM/ ML		: AT 1'-6½'; Very pale brown (10YR 7/3) silt with few sand and gravel, trace cobbles. Sandier with depth grading to very silty fine to coarse sand with gravel.	
			10/ 12"	16.1	99					
5			11/ 12"	17.2	102 -6 95					
			4/ 12"	18.9						
							ML/ CL		OLDER ALLUVIUM: AT 6½'-12½'; Brownish yellow (10YR 6/6) to yellowish brown (10YR 5/4) clayey silt with few gravel, grading to silty clay. Abundant calcium carbonate at 11'-12'.	
10			3/ 12"	23.3	102					
									CALABASAS FORMATION: At 12½'+; Pale olive (5Y 6/3) yellowish brown (10YR 5/6) and very pale brown (10YR 7/4) clayey siltstone (pale olive) with occasional thin interbed of fine- grained sandstone (very pale brown). Limonitic staining (yellowish brown) common. Bedding inclined at 10°-20°.	
15			4/ 12"	26.1	96					
									Total depth 16': No groundwater observed, No caving, No downhole.	
20										

APPENDIX B

LABORATORY TESTING

General
The following laboratory tests report are as previously presented in the Gorian and Associates, Inc. report dated October 12, 2000. Tests were performed to evaluate the physical and engineering properties of the encountered earth materials, including field moisture and density, compaction characteristics, expansion/consolidation potential, and shear strength.

Field Density and Moisture Tests
In situ dry density and moisture content were evaluated for relatively undisturbed samples obtained from the exploratory excavations. The test results and a detailed description of the soils encountered are shown on the attached logs in Appendix A.

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

Boring	Depth (feet)	Visual Soil Classification	Maximum Dry Density – pcf	Optimum Moisture Content - %
B-3	25	Olive gray silty clay	107	18
B-4	9	Light yellowish brown clayey silt and fine sand	116	14
B-5	1	Dark grayish brown silty gravel	116.5	12.5
B-6	9	Brownish yellow clayey silt	105	20

Expansion Test
Two expansion index tests were performed to evaluate expansion potential of the upper soils in general accordance with the Expansion Index Test method (UBC 29-2). The results are as follows:

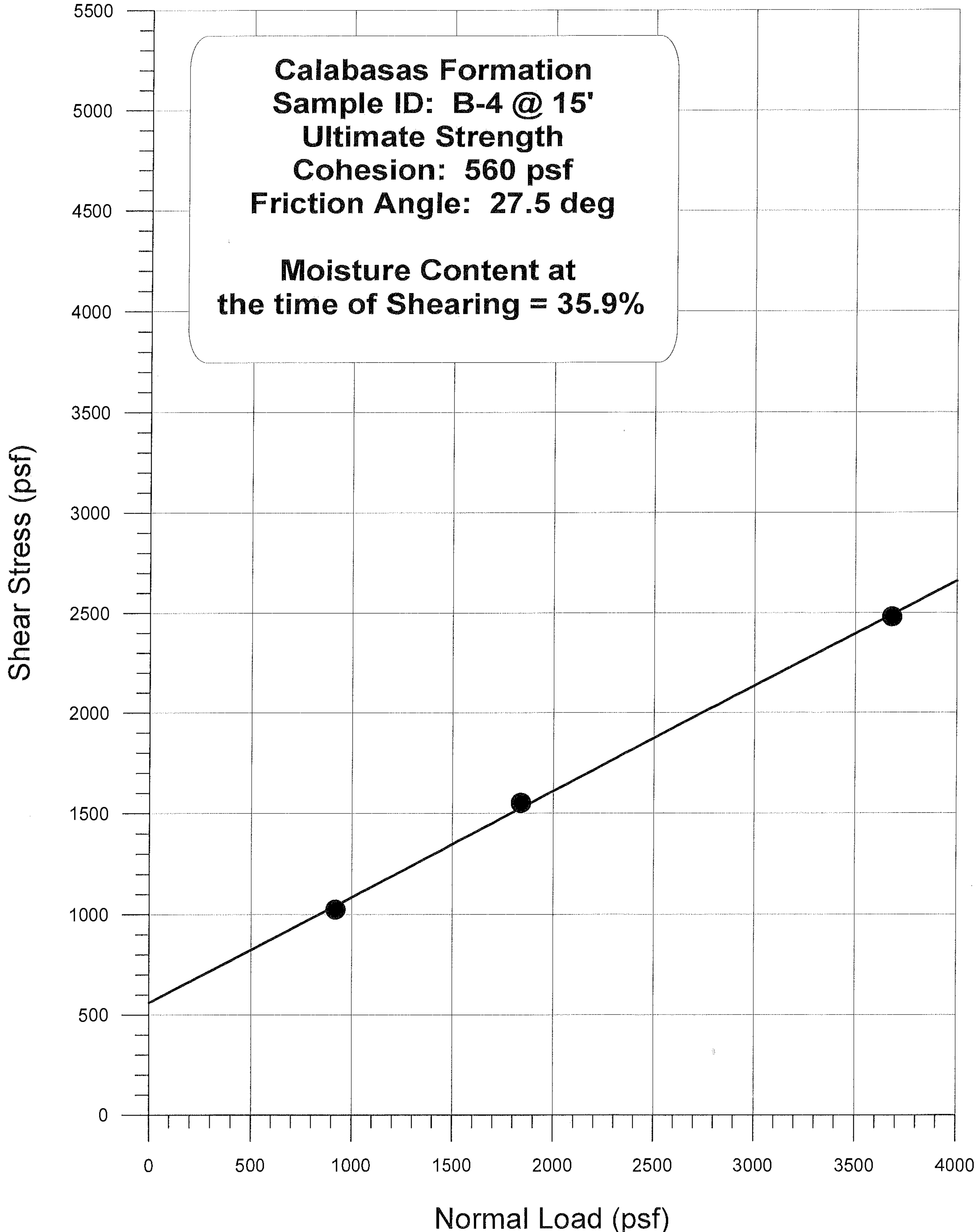
Boring	Depth (feet)	Visual Soil Classification	Expansion Index	Index Range
B-3	25	Olive gray silty clay	80	51-90
B-6	9	Brownish yellow clayey silt	177	130+

Direct Shear Tests
Strain controlled direct shear testing was performed on relatively undisturbed samples and remolded samples of the earth materials encountered during our exploratory program. Bulk samples were remolded to approximately 90% of the maximum density. The sample sets were saturated prior to shearing under axial loads ranging from 920 to 3,680 psf at a rate of 0.05 inches per minute. The shear strength results are attached as graphic summaries.

Load Consolidation Tests

Load consolidation tests were conducted on several relatively undisturbed soil samples. Test loads were added in increments to a maximum of 8,000 psf. Water was added at an axial load of 1,000 psf to study the effect of moisture infiltration on potential consolidation behavior. The results are attached as graphic summaries.

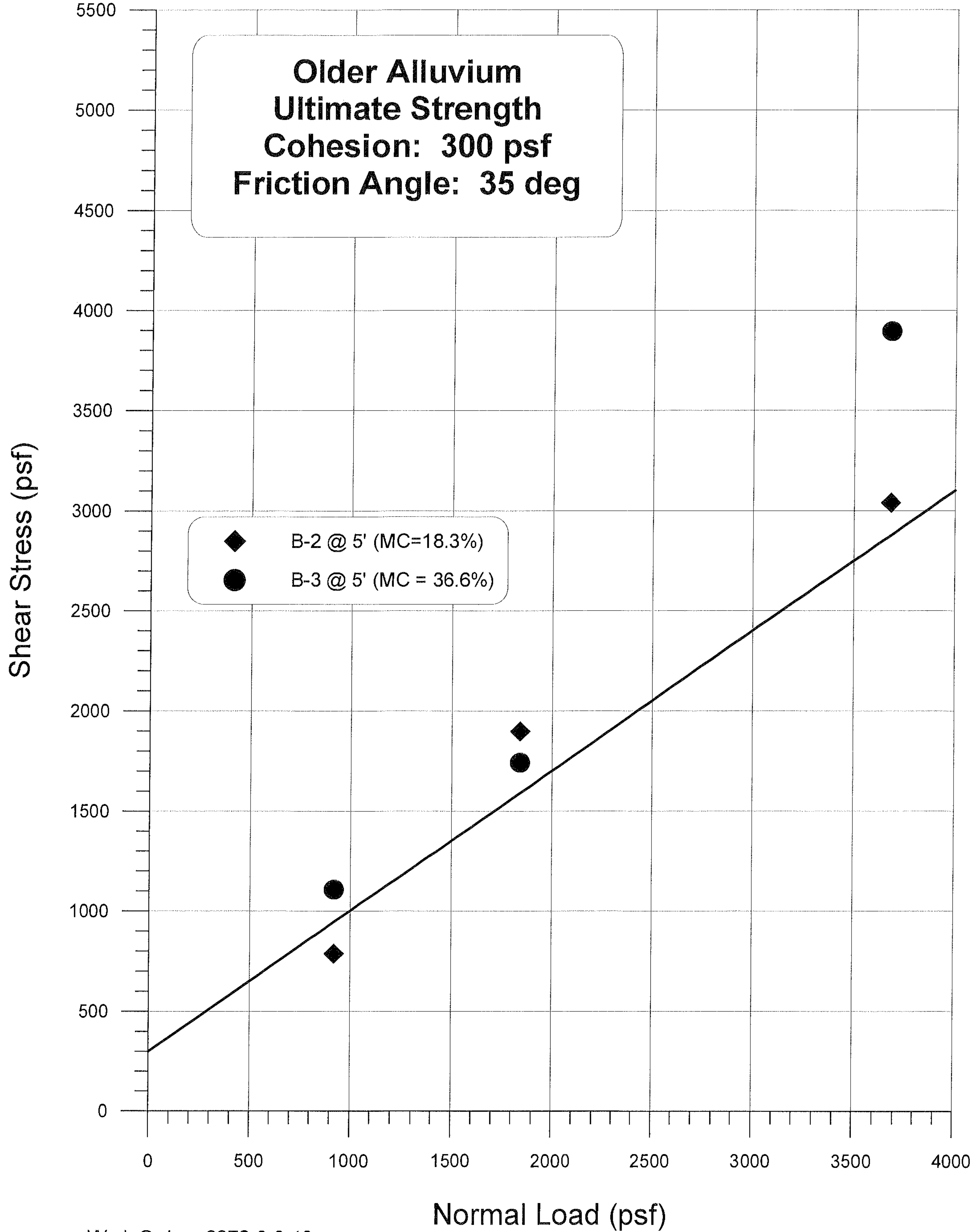
RESULTS OF SHEARING STRENGTH TESTS



Work Order: 2272-0-0-10
Log Number: 20524

GORIAN AND ASSOCIATES, INC.

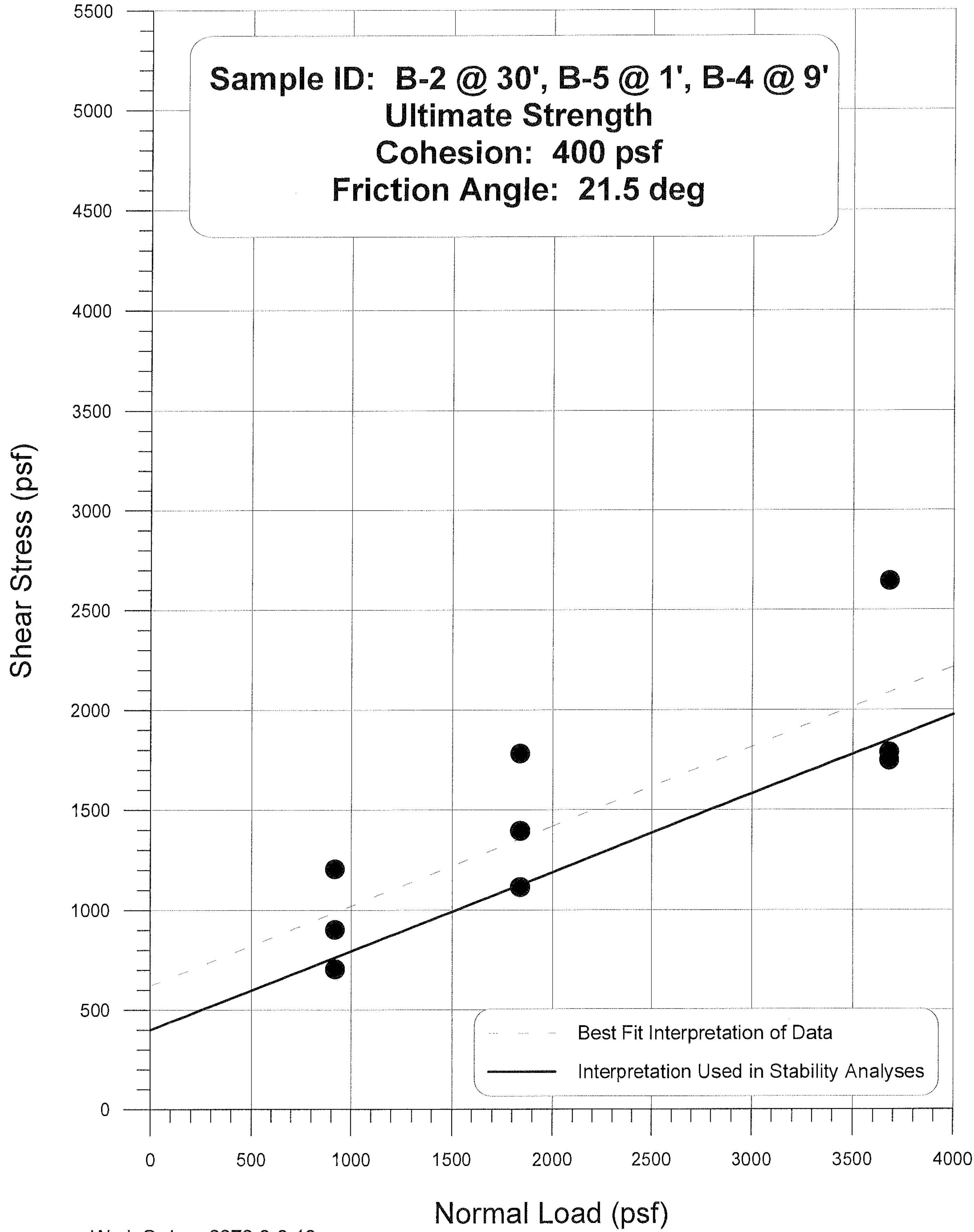
RESULTS OF SHEARING STRENGTH TESTS



Work Order: 2272-0-0-10
Log Number: 20524

GORIAN AND ASSOCIATES, INC.

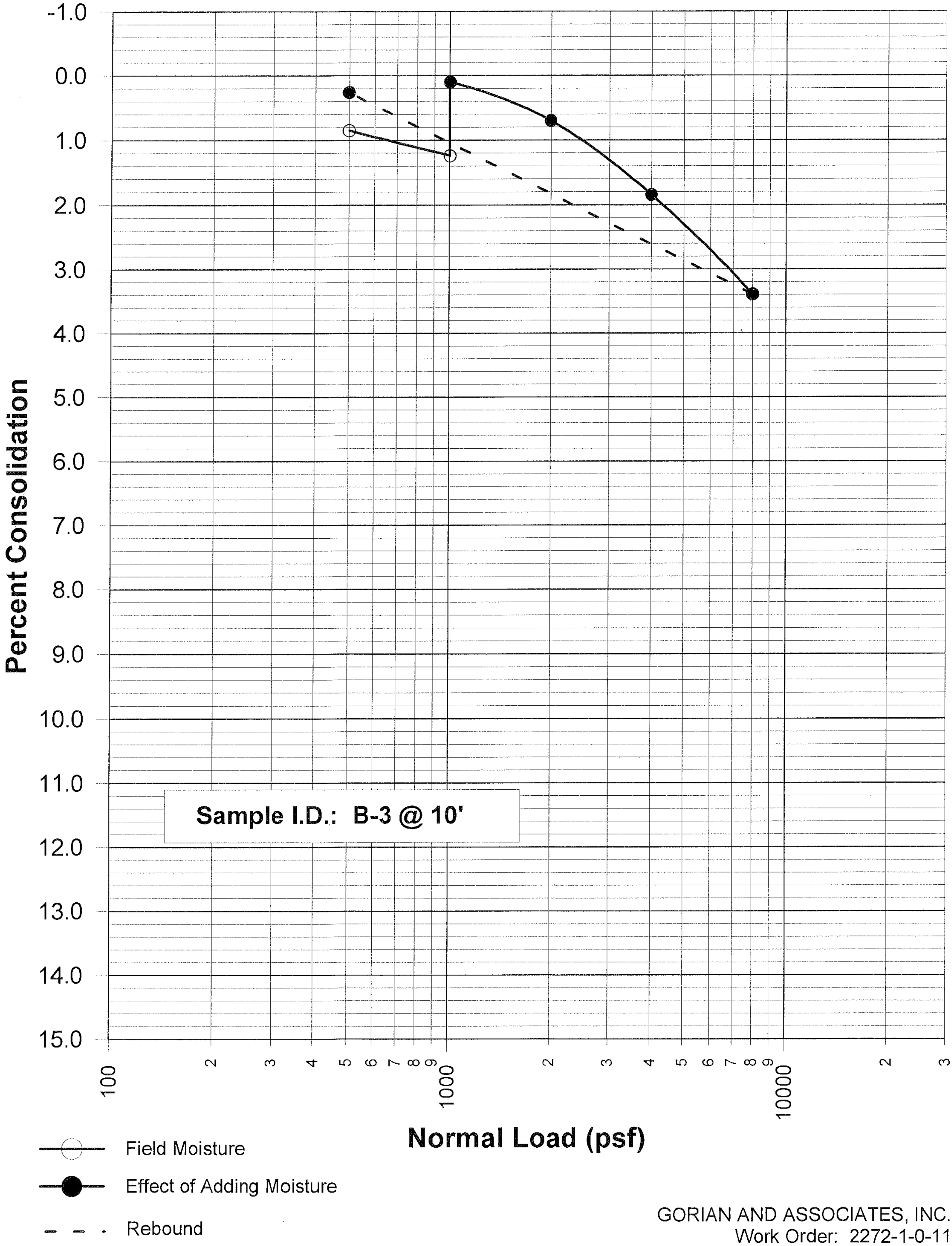
RESULTS OF SHEARING STRENGTH TESTS



Work Order: 2272-0-0-10
Log Number: 20524

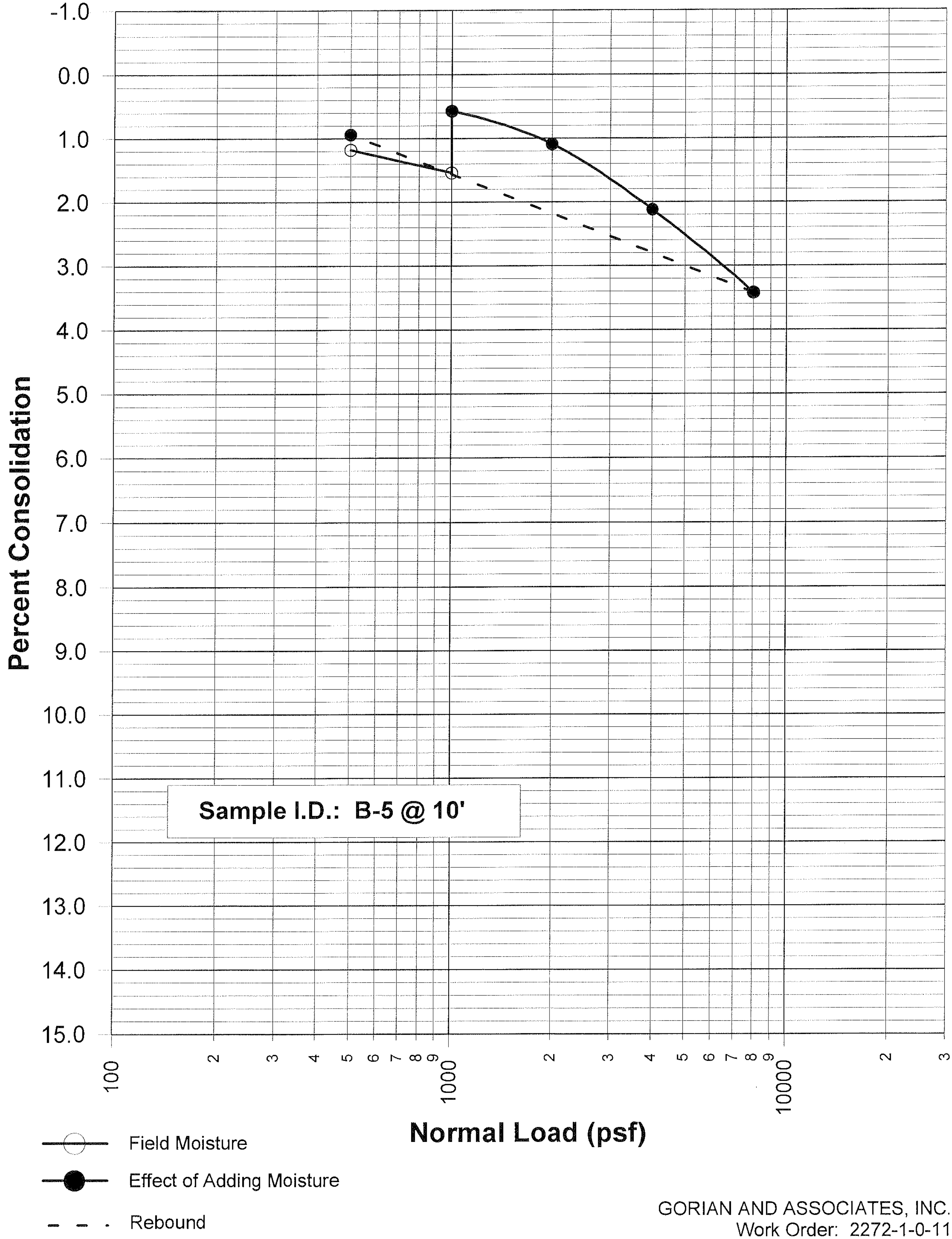
GORIAN AND ASSOCIATES, INC.

LOAD CONSOLIDATION TEST RESULTS



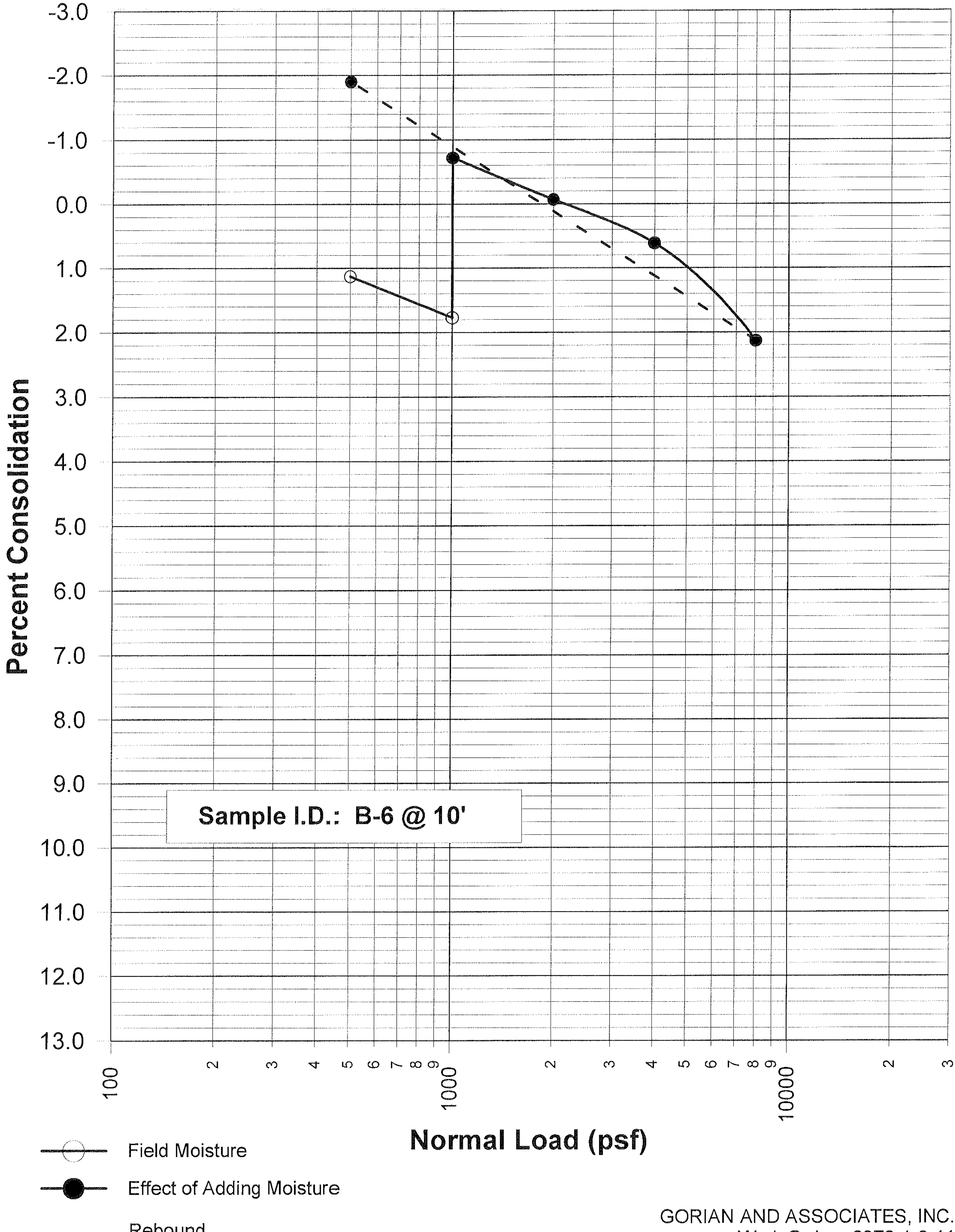
GORIAN AND ASSOCIATES, INC.
Work Order: 2272-1-0-11

LOAD CONSOLIDATION TEST RESULTS



GORIAN AND ASSOCIATES, INC.
Work Order: 2272-1-0-11

LOAD CONSOLIDATION TEST RESULTS



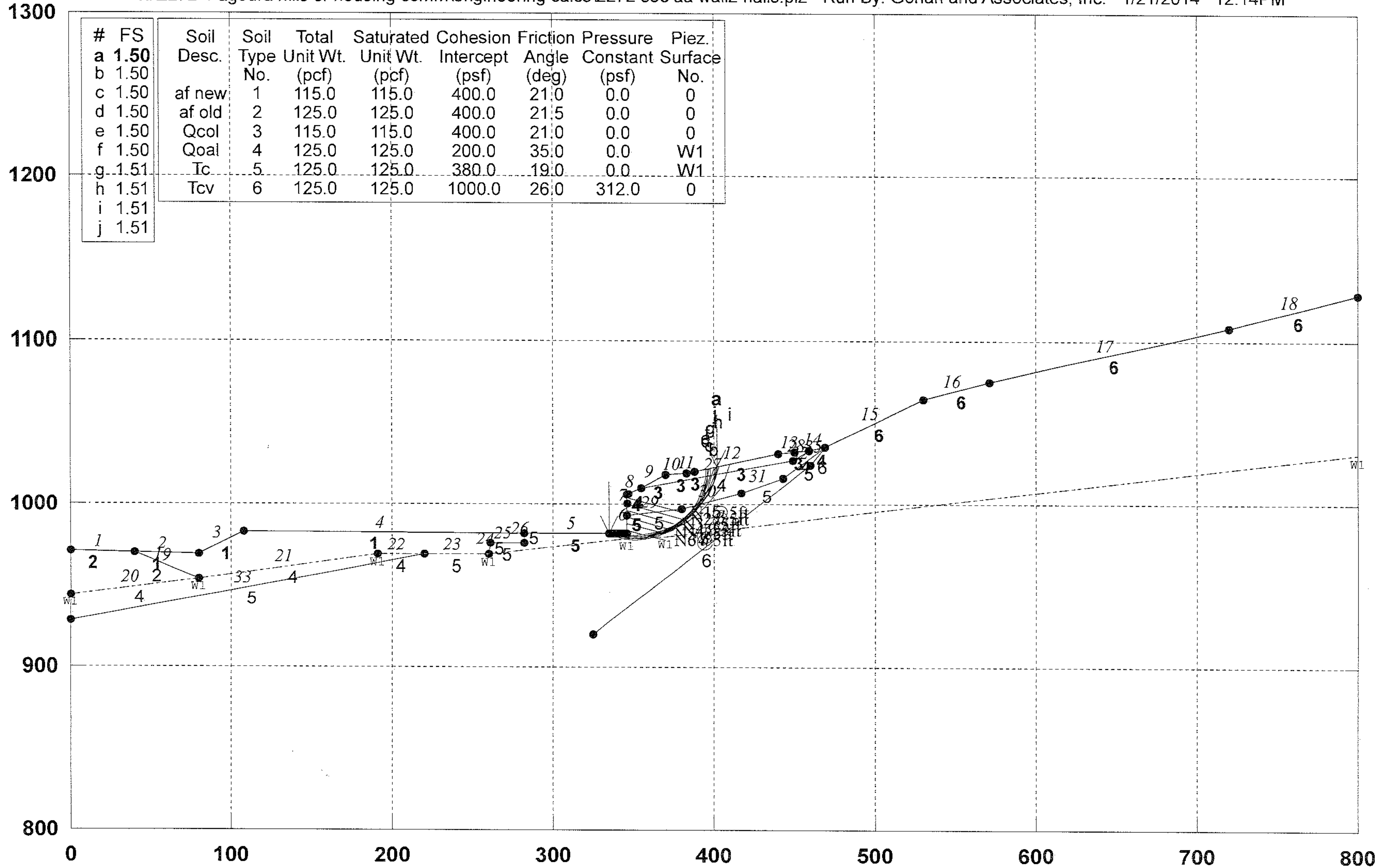
GORIAN AND ASSOCIATES, INC.
Work Order: 2272-1-0-11

APPENDIX C

SLOPE STABILITY ANALYSES

WO 2272-1-0-101 Section A-A' with soil nails

x:\2272-1 agoura hills sr housing comm\engineering calcs\2272 sec aa wall2 nails.pl2 Run By: Gorian and Associates, Inc. 1/21/2014 12:14PM



#	FS
a	**1.50**
b	1.50
c	1.50
d	1.50
e	1.50
f	1.50
g	1.51
h	1.51
i	1.51
j	1.51

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pressure Constant (psf)	Piez. Surface No.
af new	1	115.0	115.0	400.0	21.0	0.0	0
af old	2	125.0	125.0	400.0	21.5	0.0	0
Qcol	3	115.0	115.0	400.0	21.0	0.0	0
Qoal	4	125.0	125.0	200.0	35.0	0.0	W1
Tc	5	125.0	125.0	380.0	19.0	0.0	W1
Tcv	6	125.0	125.0	1000.0	26.0	312.0	0

GSTABL7 v.2 FSmin=1.50

Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7

***** GSTABL7 *****

** GSTABL7 by Garry H. Gregory, P.E. **
** Original Version 1.0, January 1996; Current Version 2.005, Sept. 2006 **

****
SLOPE STABILITY ANALYSIS SYSTEM
Modified Bishop, Simplified Janbu, or GLE Method of Slices.
(Includes Spencer & Morgenstern-Price Type Analysis)
Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
Nonlinear Undrained Shear Strength, Curved Phi Envelope,
Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
****

Analysis Run Date: 1/21/2014
Time of Run: 12:14PM
Run By: Gorian and Associates, Inc.
Input Data Filename: X:\2272-1 Agoura Hills Sr Housing Comm\engineering calcs\2272 sec aa wall2 nails.dat
Output Filename: X:\2272-1 Agoura Hills Sr Housing Comm\engineering calcs\2272 sec aa wall2 nails.OUT
Unit System: English
Plotted Output Filename: X:\2272-1 Agoura Hills Sr Housing Comm\engineering calcs\2272 sec aa wall2 nails.PLT
PROBLEM DESCRIPTION: WO 2272-1-0-101
Section A-A' with soil nails

BOUNDARY COORDINATES
18 Top Boundaries
35 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	971.00	40.00	970.00	2
2	40.00	970.00	80.00	969.00	1
3	80.00	969.00	108.00	983.00	1
4	108.00	983.00	282.00	982.00	1
5	282.00	982.00	346.00	982.00	5
6	346.00	982.00	346.20	993.00	5
7	346.20	993.00	346.50	1006.00	4
8	346.50	1006.00	355.00	1010.00	4
9	355.00	1010.00	370.00	1018.00	3
10	370.00	1018.00	383.00	1019.00	3
11	383.00	1019.00	388.00	1020.00	3
12	388.00	1020.00	440.00	1031.00	3
13	440.00	1031.00	459.00	1033.00	3
14	459.00	1033.00	469.00	1035.00	4
15	469.00	1035.00	530.00	1064.00	6
16	530.00	1064.00	571.00	1075.00	6
17	571.00	1075.00	720.00	1108.00	6
18	720.00	1108.00	800.00	1128.00	6
19	40.00	970.00	80.00	954.00	2
20	0.00	944.00	80.00	954.00	4
21	80.00	954.00	191.00	969.00	4
22	191.00	969.00	220.00	969.00	4
23	220.00	969.00	260.00	969.00	5
24	260.00	969.00	261.00	976.00	5
25	261.00	976.00	281.90	976.00	5
26	281.90	976.00	282.00	982.00	5
27	355.00	1010.00	449.00	1027.00	4
28	449.00	1027.00	459.00	1033.00	4
29	346.20	993.00	380.00	997.00	5
30	380.00	997.00	417.00	1007.00	5
31	417.00	1007.00	443.00	1016.00	5
32	443.00	1016.00	469.00	1035.00	5
33	0.00	928.00	220.00	969.00	5
34	325.00	920.00	460.00	1024.00	6
35	460.00	1024.00	469.00	1035.00	6

User Specified Y-Origin = 800.00(ft)
Default X-Plus Value = 0.00(ft)
Default Y-Plus Value = 0.00(ft)

ISOTROPIC SOIL PARAMETERS

6 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	115.0	115.0	400.0	21.0	0.00	0.0	0
2	125.0	125.0	400.0	21.5	0.00	0.0	0
3	115.0	115.0	400.0	21.0	0.00	0.0	0
4	125.0	125.0	200.0	35.0	0.00	0.0	1
5	125.0	125.0	380.0	19.0	0.00	0.0	1
6	125.0	125.0	1000.0	26.0	0.00	312.0	0

1 PIEZOMETRIC SURFACE(S) SPECIFIED

Unit Weight of Water = 62.40 (pcf)

Piezometric Surface No. 1 Specified by 7 Coordinate Points

Pore Pressure Inclination Factor = 0.50

Point No.	X-Water (ft)	Y-Water (ft)
1	0.00	944.00
2	80.00	954.00
3	191.00	969.00
4	260.00	969.00
5	346.50	979.00
6	370.00	980.00
7	800.00	1031.00

SOIL NAIL LOAD(S)

6 SOIL NAIL LOAD(S) SPECIFIED

Nail No.	X-Pos (ft)	Y-Pos (ft)	Nail Dia (in)	Tendon Dia (in)	Spacing (ft)	Inclin. (deg)	Length (ft)
1	346.45	1004.00	6.0	1.0	5.00	15.00	35.00
2	346.36	1000.00	6.0	1.0	5.00	15.00	35.00
3	346.27	996.00	6.0	1.0	5.00	15.00	30.00
4	346.18	992.00	6.0	1.0	5.00	15.00	30.00
5	346.11	988.00	6.0	1.0	5.00	15.00	25.00
6	346.04	984.00	6.0	1.0	5.00	15.00	25.00

SOIL NAIL LOAD DATA

Soil Nail No. 1 4 Load Points Apply to This Nail

Load Diagram Type = 1

POINT NO.	X-COORD.(ft)	Y-COORD.(ft)	FORCE(lbs)
1	346.45	1004.00	1400.00
2	359.54	1000.61	5654.87
3	364.07	999.44	5654.87
4	380.26	994.94	0.00

Allowable Pullout Stress = 1000.0(psf)

Allowable Tendon Stress = 36000.0(psi)

Allowable Nail Head Load = 7000.0(lbs)

Soil Nail No. 2 4 Load Points Apply to This Nail

Load Diagram Type = 1

POINT NO.	X-COORD.(ft)	Y-COORD.(ft)	FORCE(lbs)
1	346.36	1000.00	1400.00
2	359.44	996.61	5654.87
3	363.97	995.44	5654.87
4	380.17	990.94	0.00

Allowable Pullout Stress = 1000.0(psf)

Allowable Tendon Stress = 36000.0(psi)

Allowable Nail Head Load = 7000.0(lbs)

Soil Nail No. 3 4 Load Points Apply to This Nail

Load Diagram Type = 1

POINT NO.	X-COORD.(ft)	Y-COORD.(ft)	FORCE(lbs)
1	346.27	996.00	1400.00
2	354.99	993.74	5654.87
3	364.68	991.24	5654.87
4	375.25	988.24	0.00

Allowable Pullout Stress = 1500.0(psf)

Allowable Tendon Stress = 36000.0(psi)

Allowable Nail Head Load = 7000.0(lbs)

Soil Nail No. 4 4 Load Points Apply to This Nail

Load Diagram Type = 1

```
POINT NO.     X-COORD.(ft)   Y-COORD.(ft)   FORCE(lbs)
   1             346.18         992.00        1400.00
   2             354.90         989.74        5654.87
   3             364.59         987.24        5654.87
   4             375.16         984.24           0.00
Allowable Pullout Stress =     1500.0(psf)
Allowable Tendon Stress  =    36000.0(psi)
Allowable Nail Head Load =     7000.0(lbs)
Soil Nail No.  5       4 Load Points Apply to This Nail
Load Diagram Type =   1
POINT NO.     X-COORD.(ft)   Y-COORD.(ft)   FORCE(lbs)
   1             346.11         988.00        1400.00
   2             352.65         986.31        5654.87
   3             362.42         983.78        5654.87
   4             370.26         981.53           0.00
Allowable Pullout Stress =     2000.0(psf)
Allowable Tendon Stress  =    36000.0(psi)
Allowable Nail Head Load =     7000.0(lbs)
Soil Nail No.  6       4 Load Points Apply to This Nail
Load Diagram Type =   1
POINT NO.     X-COORD.(ft)   Y-COORD.(ft)   FORCE(lbs)
   1             346.04         984.00        1400.00
   2             352.58         982.31        5654.87
   3             362.34         979.78        5654.87
   4             370.18         977.53           0.00
Allowable Pullout Stress =     2000.0(psf)
Allowable Tendon Stress  =    36000.0(psi)
Allowable Nail Head Load =     7000.0(lbs)
NOTE - An Equivalent Line Load Is Calculated For Each Row Of Soil Nails
       Assuming A Uniform Distribution Of Load Horizontally Between
       Individual Nails.
A Critical Failure Surface Searching Method, Using A Random
Technique For Generating Circular Surfaces, Has Been Specified.
2000 Trial Surfaces Have Been Generated.
 100 Surface(s) Initiate(s) From Each Of    20 Points Equally Spaced
Along The Ground Surface Between  X = 335.00(ft)
                             and  X = 347.00(ft)
Each Surface Terminates Between   X = 370.00(ft)
                             and  X = 450.00(ft)
Unless Further Limitations Were Imposed, The Minimum Elevation
At Which A Surface Extends Is  Y =       0.00(ft)
 8.00(ft) Line Segments Define Each Trial Failure Surface.
Following Are Displayed The Ten Most Critical Of The Trial
      Failure Surfaces Evaluated. They Are
      Ordered - Most Critical First.
      * * Safety Factors Are Calculated By The Modified Bishop Method * *
      Total Number of Trial Surfaces Attempted =  2000
      Number of Trial Surfaces With Valid FS = 2000
      Statistical Data On All Valid FS Values:
         FS Max =   6.553   FS Min =   1.496   FS Ave =   2.386
         Standard Deviation =    0.762   Coefficient of Variation =   31.92 %
      Failure Surface Specified By 13 Coordinate Points
        Point      X-Surf       Y-Surf
         No.        (ft)         (ft)
          1       335.000      982.000
          2       342.779      980.132
          3       350.752      979.475
          4       358.732      980.045
          5       366.530      981.828
          6       373.965      984.782
          7       380.860      988.838
          8       387.055      993.900
          9       392.402      999.851
         10       396.777     1006.548
         11       400.077     1013.836
         12       402.224     1021.543
         13       402.403     1023.047
```

Circle Center At X = 351.037 ; Y = 1031.649 ; and Radius = 52.175
Factor of Safety
*** 1.496 ***
Individual data on the 22 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	7.8	908.2	0.0	0.0	0.	0.	0.0	0.0	0.0
2	3.2	805.6	0.0	0.0	0.	0.	0.0	0.0	0.0
3	0.2	191.1	0.0	0.0	0.	0.	0.0	0.0	0.0
4	0.3	737.3	0.0	0.0	0.	0.	0.0	0.0	0.0
5	4.3	14536.0	0.0	0.0	0.	0.	0.0	0.0	0.0
6	4.2	15598.2	0.0	0.0	0.	0.	0.0	0.0	0.0
7	3.7	14474.0	0.0	0.0	0.	0.	0.0	0.0	0.0
8	7.8	32090.6	0.0	0.0	0.	0.	0.0	0.0	0.0
9	3.5	14825.9	0.0	0.0	0.	0.	0.0	0.0	0.0
10	4.0	16727.6	0.0	0.0	0.	0.	0.0	0.0	0.0
11	6.0	23850.5	0.0	0.0	0.	0.	0.0	0.0	0.0
12	0.9	3213.5	0.0	0.0	0.	0.	0.0	0.0	0.0
13	2.1	7724.6	0.0	0.0	0.	0.	0.0	0.0	0.0
14	4.1	13605.6	0.0	0.0	0.	0.	0.0	0.0	0.0
15	0.9	2972.6	0.0	0.0	0.	0.	0.0	0.0	0.0
16	4.4	12511.5	0.0	0.0	0.	0.	0.0	0.0	0.0
17	0.4	1018.6	0.0	0.0	0.	0.	0.0	0.0	0.0
18	4.0	8746.6	0.0	0.0	0.	0.	0.0	0.0	0.0
19	3.3	4811.6	0.0	0.0	0.	0.	0.0	0.0	0.0
20	1.3	985.5	0.0	0.0	0.	0.	0.0	0.0	0.0
21	0.9	299.1	0.0	0.0	0.	0.	0.0	0.0	0.0
22	0.2	15.1	0.0	0.0	0.	0.	0.0	0.0	0.0

Failure Surface Specified By 12 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	335.000	982.000
2	342.814	980.286
3	350.801	979.821
4	358.761	980.617
5	366.497	982.654
6	373.817	985.882
7	380.539	990.220
8	386.495	995.561
9	391.538	1001.771
10	395.542	1008.697
11	398.408	1016.166
12	399.745	1022.485

Circle Center At X = 349.762 ; Y = 1030.406 ; and Radius = 50.607
Factor of Safety
*** 1.497 ***
Failure Surface Specified By 12 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	335.000	982.000
2	342.763	980.067
3	350.741	979.477
4	358.704	980.249
5	366.420	982.360
6	373.667	985.748
7	380.235	990.316
8	385.933	995.931
9	390.597	1002.431
10	394.091	1009.628
11	396.315	1017.312
12	396.824	1021.867

Circle Center At X = 350.225 ; Y = 1026.259 ; and Radius = 46.805
Factor of Safety
*** 1.498 ***
Failure Surface Specified By 12 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	335.000	982.000
2	342.701	979.833
3	350.665	979.071
4	358.637	979.739
5	366.362	981.816
6	373.595	985.234
7	380.104	989.885
8	385.681	995.621
9	390.148	1002.258
10	393.362	1009.584
11	395.220	1017.365
12	395.455	1021.577

Circle Center At X = 350.947 ; Y = 1023.602 ; and Radius = 44.553

Factor of Safety

*** 1.501 ***

Failure Surface Specified By 12 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	335.000	982.000
2	342.664	979.704
3	350.617	978.846
4	358.594	979.453
5	366.326	981.505
6	373.554	984.935
7	380.035	989.625
8	385.550	995.420
9	389.916	1002.124
10	392.985	1009.512
11	394.654	1017.336
12	394.763	1021.431

Circle Center At X = 351.324 ; Y = 1022.264 ; and Radius = 43.447

Factor of Safety

*** 1.502 ***

Failure Surface Specified By 12 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	335.000	982.000
2	342.591	979.475
3	350.515	978.371
4	358.507	978.723
5	366.302	980.520
6	373.642	983.703
7	380.282	988.165
8	386.002	993.758
9	390.611	1000.297
10	393.957	1007.563
11	395.928	1015.317
12	396.357	1021.768

Circle Center At X = 352.593 ; Y = 1022.074 ; and Radius = 43.765

Factor of Safety

*** 1.504 ***

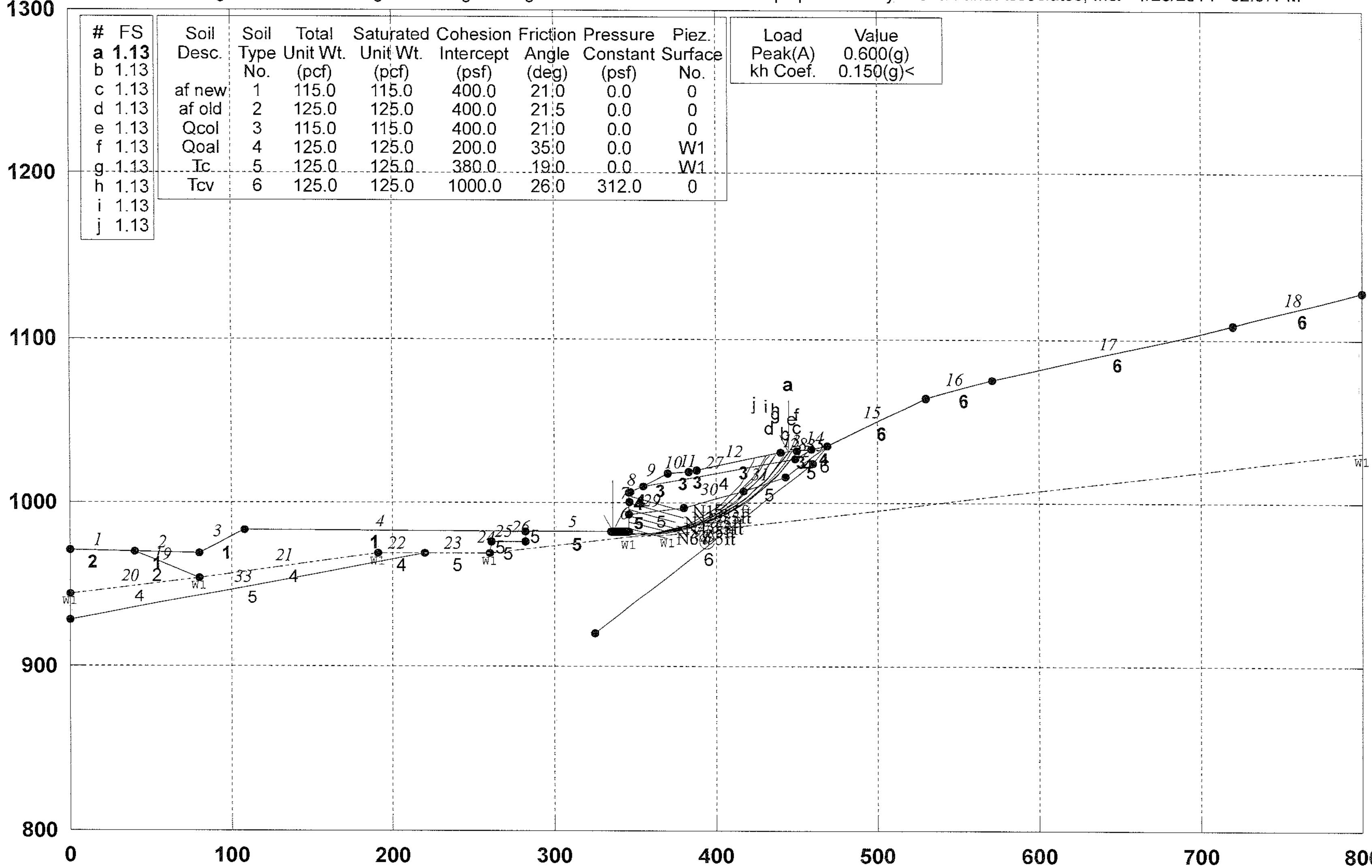
Failure Surface Specified By 12 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	336.263	982.000
2	344.068	980.245
3	352.056	979.808
4	360.006	980.700
5	367.699	982.897
6	374.921	986.339
7	381.472	990.930
8	387.173	996.543
9	391.864	1003.023
10	395.417	1010.191
11	397.733	1017.848

```
  12        398.286      1022.176
Circle Center At X =   350.710 ; Y =  1027.660 ; and Radius =    47.891
       Factor of Safety
      ***    1.506   ***
Failure Surface Specified By 12 Coordinate Points
  Point      X-Surf       Y-Surf
   No.        (ft)         (ft)
    1       336.263       982.000
    2       344.149       980.655
    3       352.147       980.472
    4       360.086       981.457
    5       367.797       983.587
    6       375.116       986.818
    7       381.886       991.081
    8       387.963       996.284
    9       393.217      1002.317
   10       397.536      1009.051
   11       400.828      1016.342
   12       402.764      1023.123
Circle Center At X =   349.406 ; Y =  1035.078 ; and Radius =    54.681
       Factor of Safety
      ***    1.508   ***
Failure Surface Specified By 13 Coordinate Points
  Point      X-Surf       Y-Surf
   No.        (ft)         (ft)
    1       336.263       982.000
    2       344.123       980.511
    3       352.111       980.068
    4       360.088       980.678
    5       367.915       982.331
    6       375.458       984.998
    7       382.584       988.633
    8       389.171       993.173
    9       395.105       998.539
   10       400.282      1004.637
   11       404.613      1011.364
   12       408.023      1018.601
   13       409.948      1024.643
Circle Center At X =   351.476 ; Y =  1040.821 ; and Radius =    60.756
       Factor of Safety
      ***    1.508   ***
Failure Surface Specified By 12 Coordinate Points
  Point      X-Surf       Y-Surf
   No.        (ft)         (ft)
    1       336.263       982.000
    2       344.162       980.734
    3       352.162       980.648
    4       360.086       981.744
    5       367.762       983.999
    6       375.021       987.362
    7       381.703       991.760
    8       387.663       997.097
    9       392.770      1003.255
   10       396.911      1010.100
   11       399.996      1017.481
   12       401.346      1022.823
Circle Center At X =   348.771 ; Y =  1034.351 ; and Radius =    53.825
       Factor of Safety
      ***    1.509   ***
          **** END OF GSTABL7 OUTPUT ****
```

WO 2272-1-0-101 Section A-A' with soil nails pseudo-stat

x:\2272-1 agoura hills sr housing comm\engineering calcs\2272 sec aa wall2 nails ps.pl2   Run By: Gorian and Associates, Inc.   1/28/2014   02:37PM



#	FS
a	**1.13**
b	1.13
c	1.13
d	1.13
e	1.13
f	1.13
g	1.13
h	1.13
i	1.13
j	1.13

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pressure Constant (psf)	Piez. Surface No.
af new	1	115.0	115.0	400.0	21.0	0.0	0
af old	2	125.0	125.0	400.0	21.5	0.0	0
Qcol	3	115.0	115.0	400.0	21.0	0.0	0
Qoal	4	125.0	125.0	200.0	35.0	0.0	W1
Tc	5	125.0	125.0	380.0	19.0	0.0	W1
Tcv	6	125.0	125.0	1000.0	26.0	312.0	0

Load	Value
Peak(A)	0.600(g)
kh Coef.	0.150(g)<

GSTABL7 v.2 FSmin=1.13

Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7

***** GSTABL7 *****

** GSTABL7 by Garry H. Gregory, P.E. **
** Original Version 1.0, January 1996; Current Version 2.005, Sept. 2006 **

****
SLOPE STABILITY ANALYSIS SYSTEM
Modified Bishop, Simplified Janbu, or GLE Method of Slices.
(Includes Spencer & Morgenstern-Price Type Analysis)
Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
Nonlinear Undrained Shear Strength, Curved Phi Envelope,
Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
****

Analysis Run Date: 1/28/2014
Time of Run: 02:37PM
Run By: Gorian and Associates, Inc.
Input Data Filename: X:\2272-1 Agoura Hills Sr Housing Comm\engineering calcs\2272 sec aa wall2 nails ps.dat
Output Filename: X:\2272-1 Agoura Hills Sr Housing Comm\engineering calcs\2272 sec aa wall2 nails ps.OUT
Unit System: English
Plotted Output Filename: X:\2272-1 Agoura Hills Sr Housing Comm\engineering calcs\2272 sec aa wall2 nails ps.PLT
PROBLEM DESCRIPTION: WO 2272-1-0-101
Section A-A' with soil nails pseudo-stat

BOUNDARY COORDINATES
18 Top Boundaries
35 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	971.00	40.00	970.00	2
2	40.00	970.00	80.00	969.00	1
3	80.00	969.00	108.00	983.00	1
4	108.00	983.00	282.00	982.00	1
5	282.00	982.00	346.00	982.00	5
6	346.00	982.00	346.20	993.00	5
7	346.20	993.00	346.50	1006.00	4
8	346.50	1006.00	355.00	1010.00	4
9	355.00	1010.00	370.00	1018.00	3
10	370.00	1018.00	383.00	1019.00	3
11	383.00	1019.00	388.00	1020.00	3
12	388.00	1020.00	440.00	1031.00	3
13	440.00	1031.00	459.00	1033.00	3
14	459.00	1033.00	469.00	1035.00	4
15	469.00	1035.00	530.00	1064.00	6
16	530.00	1064.00	571.00	1075.00	6
17	571.00	1075.00	720.00	1108.00	6
18	720.00	1108.00	800.00	1128.00	6
19	40.00	970.00	80.00	954.00	2
20	0.00	944.00	80.00	954.00	4
21	80.00	954.00	191.00	969.00	4
22	191.00	969.00	220.00	969.00	4
23	220.00	969.00	260.00	969.00	5
24	260.00	969.00	261.00	976.00	5
25	261.00	976.00	281.90	976.00	5
26	281.90	976.00	282.00	982.00	5
27	355.00	1010.00	449.00	1027.00	4
28	449.00	1027.00	459.00	1033.00	4
29	346.20	993.00	380.00	997.00	5
30	380.00	997.00	417.00	1007.00	5
31	417.00	1007.00	443.00	1016.00	5
32	443.00	1016.00	469.00	1035.00	5
33	0.00	928.00	220.00	969.00	5
34	325.00	920.00	460.00	1024.00	6
35	460.00	1024.00	469.00	1035.00	6

User Specified Y-Origin = 800.00(ft)
Default X-Plus Value = 0.00(ft)
Default Y-Plus Value = 0.00(ft)

ISOTROPIC SOIL PARAMETERS

6 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	115.0	115.0	400.0	21.0	0.00	0.0	0
2	125.0	125.0	400.0	21.5	0.00	0.0	0
3	115.0	115.0	400.0	21.0	0.00	0.0	0
4	125.0	125.0	200.0	35.0	0.00	0.0	1
5	125.0	125.0	380.0	19.0	0.00	0.0	1
6	125.0	125.0	1000.0	26.0	0.00	312.0	0

1 PIEZOMETRIC SURFACE(S) SPECIFIED

Unit Weight of Water = 62.40 (pcf)

Piezometric Surface No. 1 Specified by 7 Coordinate Points

Pore Pressure Inclination Factor = 0.50

Point No.	X-Water (ft)	Y-Water (ft)
1	0.00	944.00
2	80.00	954.00
3	191.00	969.00
4	260.00	969.00
5	346.50	979.00
6	370.00	980.00
7	800.00	1031.00

Specified Peak Ground Acceleration Coefficient (A) = 0.600(g)

Specified Horizontal Earthquake Coefficient (kh) = 0.150(g)

Specified Vertical Earthquake Coefficient (kv) = 0.000(g)

Specified Seismic Pore-Pressure Factor = 0.000

SOIL NAIL LOAD(S)

6 SOIL NAIL LOAD(S) SPECIFIED

Nail No.	X-Pos (ft)	Y-Pos (ft)	Nail Dia (in)	Tendon Dia (in)	Spacing (ft)	Inclin. (deg)	Length (ft)
1	346.45	1004.00	6.0	1.0	5.00	15.00	35.00
2	346.36	1000.00	6.0	1.0	5.00	15.00	35.00
3	346.27	996.00	6.0	1.0	5.00	15.00	30.00
4	346.18	992.00	6.0	1.0	5.00	15.00	30.00
5	346.11	988.00	6.0	1.0	5.00	15.00	25.00
6	346.04	984.00	6.0	1.0	5.00	15.00	25.00

SOIL NAIL LOAD DATA

Soil Nail No. 1 4 Load Points Apply to This Nail

Load Diagram Type = 1

POINT NO.	X-COORD.(ft)	Y-COORD.(ft)	FORCE(lbs)
1	346.45	1004.00	1400.00
2	359.54	1000.61	5654.87
3	364.07	999.44	5654.87
4	380.26	994.94	0.00

Allowable Pullout Stress = 1000.0(psf)

Allowable Tendon Stress = 36000.0(psi)

Allowable Nail Head Load = 7000.0(lbs)

Soil Nail No. 2 4 Load Points Apply to This Nail

Load Diagram Type = 1

POINT NO.	X-COORD.(ft)	Y-COORD.(ft)	FORCE(lbs)
1	346.36	1000.00	1400.00
2	359.44	996.61	5654.87
3	363.97	995.44	5654.87
4	380.17	990.94	0.00

Allowable Pullout Stress = 1000.0(psf)

Allowable Tendon Stress = 36000.0(psi)

Allowable Nail Head Load = 7000.0(lbs)

Soil Nail No. 3 4 Load Points Apply to This Nail

Load Diagram Type = 1

POINT NO.	X-COORD.(ft)	Y-COORD.(ft)	FORCE(lbs)
1	346.27	996.00	1400.00
2	354.99	993.74	5654.87
3	364.68	991.24	5654.87
4	375.25	988.24	0.00

Allowable Pullout Stress = 1500.0(psf)

```
Allowable Tendon Stress  =    36000.0(psi)
Allowable Nail Head Load =     7000.0(lbs)
Soil Nail No.  4       4 Load Points Apply to This Nail
Load Diagram Type =   1
POINT NO.     X-COORD.(ft)   Y-COORD.(ft)   FORCE(lbs)
   1            346.18         992.00        1400.00
   2            354.90         989.74        5654.87
   3            364.59         987.24        5654.87
   4            375.16         984.24           0.00
Allowable Pullout Stress =     1500.0(psf)
Allowable Tendon Stress  =    36000.0(psi)
Allowable Nail Head Load =     7000.0(lbs)
Soil Nail No.  5       4 Load Points Apply to This Nail
Load Diagram Type =   1
POINT NO.     X-COORD.(ft)   Y-COORD.(ft)   FORCE(lbs)
   1            346.11         988.00        1400.00
   2            352.65         986.31        5654.87
   3            362.42         983.78        5654.87
   4            370.26         981.53           0.00
Allowable Pullout Stress =     2000.0(psf)
Allowable Tendon Stress  =    36000.0(psi)
Allowable Nail Head Load =     7000.0(lbs)
Soil Nail No.  6       4 Load Points Apply to This Nail
Load Diagram Type =   1
POINT NO.     X-COORD.(ft)   Y-COORD.(ft)   FORCE(lbs)
   1            346.04         984.00        1400.00
   2            352.58         982.31        5654.87
   3            362.34         979.78        5654.87
   4            370.18         977.53           0.00
Allowable Pullout Stress =     2000.0(psf)
Allowable Tendon Stress  =    36000.0(psi)
Allowable Nail Head Load =     7000.0(lbs)
NOTE - An Equivalent Line Load Is Calculated For Each Row Of Soil Nails
       Assuming A Uniform Distribution Of Load Horizontally Between
       Individual Nails.
A Critical Failure Surface Searching Method, Using A Random
Technique For Generating Circular Surfaces, Has Been Specified.
2000 Trial Surfaces Have Been Generated.
 100 Surface(s) Initiate(s) From Each Of    20 Points Equally Spaced
Along The Ground Surface Between  X = 335.00(ft)
                            and  X = 347.00(ft)
Each Surface Terminates Between   X = 370.00(ft)
                            and  X = 450.00(ft)
Unless Further Limitations Were Imposed, The Minimum Elevation
At Which A Surface Extends Is  Y =      0.00(ft)
 8.00(ft) Line Segments Define Each Trial Failure Surface.
Following Are Displayed The Ten Most Critical Of The Trial
      Failure Surfaces Evaluated. They Are
      Ordered - Most Critical First.
      * * Safety Factors Are Calculated By The Modified Bishop Method * *
      Total Number of Trial Surfaces Attempted =  2000
      Number of Trial Surfaces With Valid FS = 2000
      Statistical Data On All Valid FS Values:
         FS Max =   4.915   FS Min =   1.126   FS Ave =   1.665
         Standard Deviation =    0.465   Coefficient of Variation =   27.93 %
      Failure Surface Specified By 17 Coordinate Points
        Point      X-Surf       Y-Surf
         No.        (ft)         (ft)
          1        336.263      982.000
          2        344.209      981.073
          3        352.202      980.733
          4        360.198      980.984
          5        368.154      981.823
          6        376.026      983.246
          7        383.773      985.245
          8        391.350      987.810
          9        398.718      990.926
```

10	405.837	994.577
11	412.667	998.742
12	419.172	1003.399
13	425.316	1008.523
14	431.065	1014.086
15	436.389	1020.057
16	441.259	1026.404
17	444.592	1031.483

Circle Center At X = 352.804 ; Y = 1089.225 ; and Radius = 108.493

Factor of Safety

*** 1.126 ***

Individual data on the 28 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	7.9	460.6	0.0	0.0	0.	0.	69.1	0.0	0.0
2	1.8	216.1	0.0	0.0	0.	0.	32.4	0.0	0.0
3	0.2	162.7	0.0	0.0	0.	0.	24.4	0.0	0.0
4	0.3	694.4	0.0	0.0	0.	0.	104.2	0.0	0.0
5	5.7	18879.0	0.0	0.0	0.	0.	2831.9	0.0	0.0
6	2.8	9990.2	0.0	0.0	0.	0.	1498.5	0.0	0.0
7	5.2	19759.6	0.0	0.0	0.	0.	2963.9	0.0	0.0
8	8.0	33048.4	0.0	0.0	0.	0.	4957.3	0.0	0.0
9	1.8	8104.2	0.0	0.0	0.	0.	1215.6	0.0	0.0
10	6.0	26465.2	0.0	0.0	0.	0.	3969.8	0.0	0.0
11	4.0	17136.7	0.0	0.0	0.	0.	2570.5	0.0	0.0
12	3.0	12711.9	0.0	0.0	0.	0.	1906.8	0.0	0.0
13	0.8	3246.3	0.0	0.0	0.	0.	486.9	0.0	0.0
14	4.2	17595.2	0.0	0.0	0.	0.	2639.3	0.0	0.0
15	3.4	13729.8	0.0	0.0	0.	0.	2059.5	0.0	0.0
16	7.4	29270.0	0.0	0.0	0.	0.	4390.5	0.0	0.0
17	7.1	26615.4	0.0	0.0	0.	0.	3992.3	0.0	0.0
18	6.8	23445.8	0.0	0.0	0.	0.	3516.9	0.0	0.0
19	4.3	13537.3	0.0	0.0	0.	0.	2030.6	0.0	0.0
20	2.2	6338.2	0.0	0.0	0.	0.	950.7	0.0	0.0
21	6.1	16032.5	0.0	0.0	0.	0.	2404.9	0.0	0.0
22	2.2	4944.2	0.0	0.0	0.	0.	741.6	0.0	0.0
23	3.6	7112.9	0.0	0.0	0.	0.	1066.9	0.0	0.0
24	5.3	8097.4	0.0	0.0	0.	0.	1214.6	0.0	0.0
25	3.6	3503.5	0.0	0.0	0.	0.	525.5	0.0	0.0
26	0.5	370.7	0.0	0.0	0.	0.	55.6	0.0	0.0
27	0.7	424.7	0.0	0.0	0.	0.	63.7	0.0	0.0
28	3.3	906.2	0.0	0.0	0.	0.	135.9	0.0	0.0

Failure Surface Specified By 17 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	335.000	982.000
2	342.920	980.875
3	350.904	980.358
4	358.903	980.454
5	366.872	981.161
6	374.763	982.476
7	382.531	984.391
8	390.129	986.894
9	397.513	989.971
10	404.641	993.604
11	411.470	997.772
12	417.959	1002.450
13	424.073	1007.610
14	429.773	1013.223
15	435.027	1019.256
16	439.805	1025.672
17	443.396	1031.358

Circle Center At X = 353.652 ; Y = 1084.830 ; and Radius = 104.508

Factor of Safety

*** 1.126 ***

Failure Surface Specified By 18 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	336.263	982.000
2	344.209	981.069
3	352.200	980.687
4	360.198	980.858
5	368.165	981.581
6	376.064	982.851
7	383.856	984.663
8	391.504	987.009
9	398.973	989.876
10	406.226	993.252
11	413.228	997.120
12	419.948	1001.462
13	426.351	1006.257
14	432.409	1011.482
15	438.092	1017.113
16	443.372	1023.122
17	448.226	1029.482
18	449.915	1032.044

Circle Center At X = 353.749 ; Y = 1096.365 ; and Radius = 115.694

Factor of Safety

*** 1.126 ***

Failure Surface Specified By 16 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	335.632	982.000
2	343.568	980.993
3	351.562	980.676
4	359.553	981.053
5	367.482	982.119
6	375.288	983.867
7	382.914	986.284
8	390.303	989.352
9	397.398	993.047
10	404.147	997.343
11	410.499	1002.206
12	416.407	1007.600
13	421.825	1013.486
14	426.714	1019.818
15	431.037	1026.550
16	432.549	1029.424

Circle Center At X = 351.248 ; Y = 1072.830 ; and Radius = 92.163

Factor of Safety

*** 1.127 ***

Failure Surface Specified By 18 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	336.895	982.000
2	344.828	980.966
3	352.815	980.519
4	360.814	980.660
5	368.781	981.389
6	376.672	982.702
7	384.446	984.592
8	392.059	987.048
9	399.472	990.057
10	406.643	993.604
11	413.533	997.668
12	420.106	1002.228
13	426.326	1007.260
14	432.159	1012.735
15	437.574	1018.624
16	442.541	1024.895
17	447.033	1031.515
18	447.171	1031.755

```
Circle Center At X =   354.902 ; Y =  1089.175 ; and Radius =   108.677
       Factor of Safety
      ***    1.127   ***
Failure Surface Specified By 18 Coordinate Points
  Point       X-Surf      Y-Surf
   No.         (ft)        (ft)
    1        338.790      982.000
    2        346.753      981.235
    3        354.750      981.021
    4        362.743      981.360
    5        370.693      982.250
    6        378.563      983.686
    7        386.315      985.663
    8        393.912      988.169
    9        401.318      991.195
   10        408.498      994.724
   11        415.417      998.740
   12        422.042     1003.224
   13        428.341     1008.155
   14        434.286     1013.509
   15        439.846     1019.261
   16        444.996     1025.383
   17        449.712     1031.845
   18        449.831     1032.035
Circle Center At X =   353.846 ; Y =  1096.818 ; and Radius =   115.801
       Factor of Safety
      ***    1.128   ***
Failure Surface Specified By 16 Coordinate Points
  Point       X-Surf      Y-Surf
   No.         (ft)        (ft)
    1        336.895      982.000
    2        344.834      981.016
    3        352.828      980.697
    4        360.820      981.044
    5        368.756      982.055
    6        376.580      983.723
    7        384.238      986.037
    8        391.677      988.980
    9        398.845      992.532
   10        405.693      996.669
   11        412.172     1001.361
   12        418.238     1006.577
   13        423.849     1012.279
   14        428.966     1018.429
   15        433.553     1024.983
   16        436.644     1030.290
Circle Center At X =   352.683 ; Y =  1076.555 ; and Radius =    95.864
       Factor of Safety
      ***    1.128   ***
Failure Surface Specified By 16 Coordinate Points
  Point       X-Surf      Y-Surf
   No.         (ft)        (ft)
    1        336.263      982.000
    2        344.185      980.882
    3        352.172      980.436
    4        360.169      980.664
    5        368.118      981.565
    6        375.963      983.133
    7        383.648      985.356
    8        391.118      988.219
    9        398.320      991.701
   10        405.203      995.778
   11        411.719     1000.421
   12        417.820     1005.596
   13        423.463     1011.266
   14        428.608     1017.392
   15        433.218     1023.930
```

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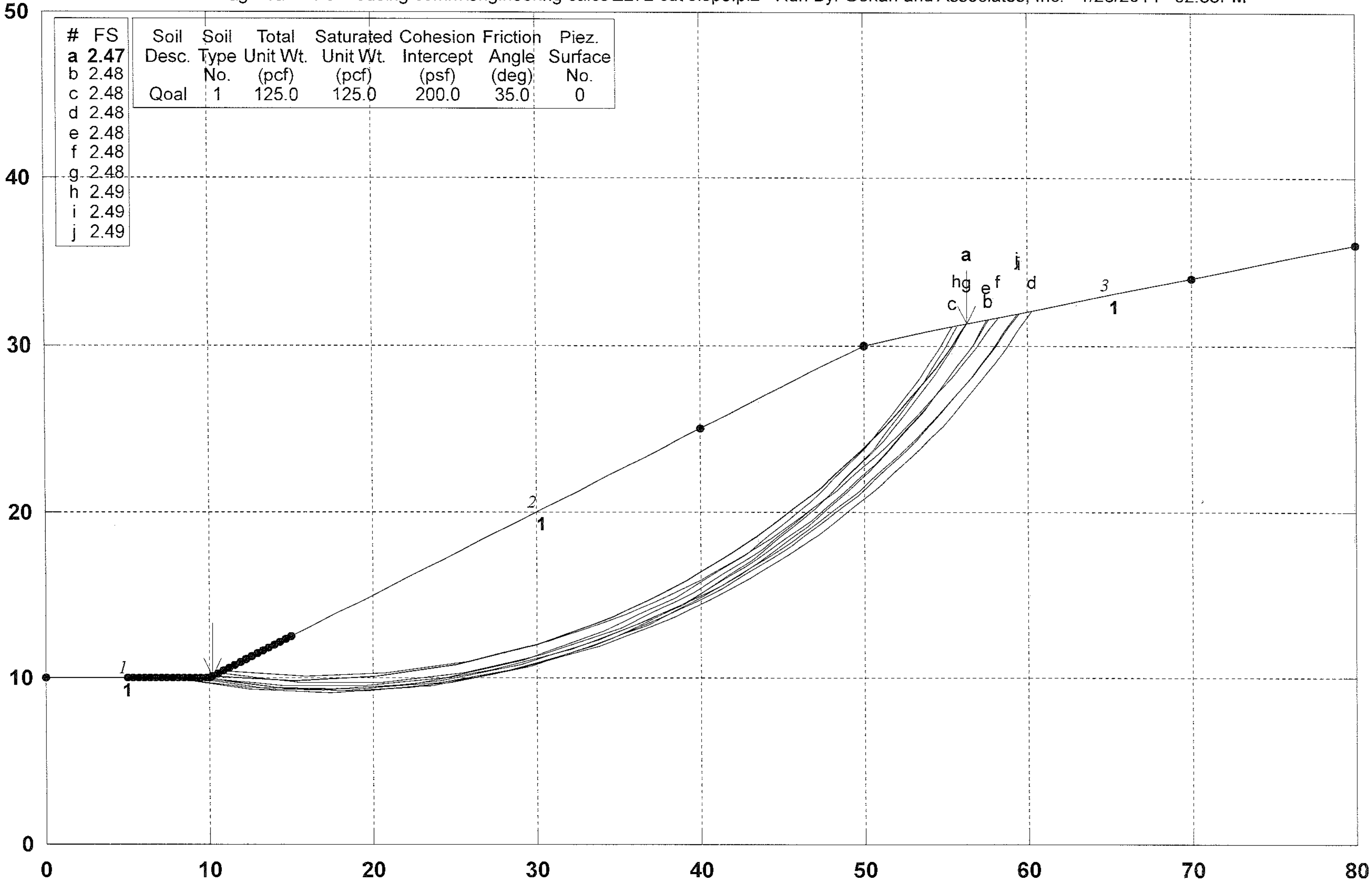
   16        436.986     1030.363
 Circle Center At X =   353.474 ; Y =  1075.224 ; and Radius =    94.799
       Factor of Safety
      ***    1.129   ***
 Failure Surface Specified By 15 Coordinate Points
   Point       X-Surf      Y-Surf
    No.         (ft)        (ft)
     1        337.526      982.000
     2        345.474      981.082
     3        353.471      980.880
     4        361.454      981.393
     5        369.360      982.620
     6        377.124      984.548
     7        384.684      987.164
     8        391.980      990.445
     9        398.953      994.366
    10        405.548      998.896
    11        411.710     1003.997
    12        417.391     1009.630
    13        422.546     1015.748
    14        427.132     1022.303
    15        431.034     1029.103
 Circle Center At X =   351.733 ; Y =  1070.176 ; and Radius =    89.313
       Factor of Safety
      ***    1.129   ***
 Failure Surface Specified By 15 Coordinate Points
   Point       X-Surf      Y-Surf
    No.         (ft)        (ft)
     1        336.263      982.000
     2        344.182      980.861
     3        352.174      980.520
     4        360.161      980.980
     5        368.062      982.236
     6        375.798      984.276
     7        383.290      987.079
     8        390.465      990.617
     9        397.250      994.855
    10        403.578      999.751
    11        409.384     1005.254
    12        414.610     1011.311
    13        419.205     1017.860
    14        423.122     1024.835
    15        424.369     1027.693
 Circle Center At X =   351.582 ; Y =  1060.418 ; and Radius =    79.901
       Factor of Safety
      ***    1.130   ***
          **** END OF GSTABL7 OUTPUT ****
```

WO 2272-1-0-101 20 foot high cut slope

x:\2272-1 agoura hills sr housing comm\engineering calcs\2272 cut slope.pl2 Run By: Gorian and Associates, Inc. 1/28/2014 02:33PM



GSTABL7 v.2 FSmin=2.47
Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7

***** GSTABL7 *****

** GSTABL7 by Garry H. Gregory, P.E. **
** Original Version 1.0, January 1996; Current Version 2.005, Sept. 2006 **

****

SLOPE STABILITY ANALYSIS SYSTEM
Modified Bishop, Simplified Janbu, or GLE Method of Slices.
(Includes Spencer & Morgenstern-Price Type Analysis)
Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
Nonlinear Undrained Shear Strength, Curved Phi Envelope,
Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.

****

Analysis Run Date: 1/28/2014
Time of Run: 02:33PM
Run By: Gorian and Associates, Inc.
Input Data Filename: X:\2272-1 Agoura Hills Sr Housing Comm\engineering calcs\2272 cut slope.dat
Output Filename: X:\2272-1 Agoura Hills Sr Housing Comm\engineering calcs\2272 cut slope.OUT
Unit System: English
Plotted Output Filename: X:\2272-1 Agoura Hills Sr Housing Comm\engineering calcs\2272 cut slope.PLT
PROBLEM DESCRIPTION: WO 2272-1-0-101
20 foot high cut slope

BOUNDARY COORDINATES
3 Top Boundaries
3 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	10.00	10.00	10.00	1
2	10.00	10.00	50.00	30.00	1
3	50.00	30.00	80.00	36.00	1

Default Y-Origin = 0.00(ft)
Default X-Plus Value = 0.00(ft)
Default Y-Plus Value = 0.00(ft)

ISOTROPIC SOIL PARAMETERS
1 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	125.0	125.0	200.0	35.0	0.00	0.0	0

A Critical Failure Surface Searching Method, Using A Random
Technique For Generating Circular Surfaces, Has Been Specified.
3000 Trial Surfaces Have Been Generated.
100 Surface(s) Initiate(s) From Each Of 30 Points Equally Spaced
Along The Ground Surface Between X = 5.00(ft)
and X = 15.00(ft)
Each Surface Terminates Between X = 40.00(ft)
and X = 70.00(ft)
Unless Further Limitations Were Imposed, The Minimum Elevation
At Which A Surface Extends Is Y = 0.00(ft)
5.00(ft) Line Segments Define Each Trial Failure Surface.
Following Are Displayed The Ten Most Critical Of The Trial
Failure Surfaces Evaluated. They Are
Ordered - Most Critical First.
* * Safety Factors Are Calculated By The Modified Bishop Method * *
Total Number of Trial Surfaces Attempted = 3000
Number of Trial Surfaces With Valid FS = 3000
Statistical Data On All Valid FS Values:
FS Max = 7.352 FS Min = 2.472 FS Ave = 3.085
Standard Deviation = 0.421 Coefficient of Variation = 13.63 %
Failure Surface Specified By 12 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	10.172	10.086
2	15.165	9.820
3	20.160	10.053

4	25.106	10.785
5	29.954	12.007
6	34.656	13.708
7	39.165	15.870
8	43.435	18.472
9	47.423	21.487
10	51.090	24.886
11	54.399	28.634
12	56.285	31.257

Circle Center At X = 15.359 ; Y = 59.595 ; and Radius = 49.779

Factor of Safety

*** 2.472 ***

Individual data on the 12 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	5.0	862.2	0.0	0.0	0.	0.	0.0	0.0	0.0
2	5.0	2431.5	0.0	0.0	0.	0.	0.0	0.0	0.0
3	4.9	3646.1	0.0	0.0	0.	0.	0.0	0.0	0.0
4	4.8	4465.8	0.0	0.0	0.	0.	0.0	0.0	0.0
5	4.7	4875.3	0.0	0.0	0.	0.	0.0	0.0	0.0
6	4.5	4883.9	0.0	0.0	0.	0.	0.0	0.0	0.0
7	4.3	4525.5	0.0	0.0	0.	0.	0.0	0.0	0.0
8	4.0	3856.5	0.0	0.0	0.	0.	0.0	0.0	0.0
9	2.6	2150.1	0.0	0.0	0.	0.	0.0	0.0	0.0
10	1.1	780.7	0.0	0.0	0.	0.	0.0	0.0	0.0
11	3.3	1567.5	0.0	0.0	0.	0.	0.0	0.0	0.0
12	1.9	264.7	0.0	0.0	0.	0.	0.0	0.0	0.0

Failure Surface Specified By 13 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	9.138	10.000
2	14.103	9.414
3	19.103	9.349
4	24.082	9.806
5	28.987	10.779
6	33.763	12.258
7	38.359	14.226
8	42.725	16.664
9	46.813	19.542
10	50.578	22.832
11	53.980	26.496
12	56.982	30.495
13	57.595	31.519

Circle Center At X = 17.226 ; Y = 57.212 ; and Radius = 47.900

Factor of Safety

*** 2.476 ***

Failure Surface Specified By 12 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	9.483	10.000
2	14.453	9.454
3	19.453	9.462
4	24.421	10.021
5	29.298	11.127
6	34.022	12.764
7	38.536	14.914
8	42.785	17.550
9	46.717	20.639
10	50.283	24.143
11	53.440	28.021
12	55.414	31.083

Circle Center At X = 16.906 ; Y = 54.402 ; and Radius = 45.018

Factor of Safety

*** 2.480 ***

Failure Surface Specified By 13 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	9.138	10.000
2	14.103	9.413
3	19.102	9.311
4	24.087	9.695
5	29.012	10.563
6	33.828	11.905
7	38.492	13.708
8	42.958	15.957
9	47.184	18.629
10	51.130	21.699
11	54.760	25.138
12	58.038	28.913
13	60.268	32.054

Circle Center At X = 17.648 ; Y = 60.669 ; and Radius = 51.378
Factor of Safety
*** 2.481 ***
Failure Surface Specified By 13 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	8.793	10.000
2	13.746	9.315
3	18.744	9.162
4	23.729	9.543
5	28.645	10.453
6	33.437	11.883
7	38.048	13.816
8	42.426	16.230
9	46.523	19.098
10	50.289	22.386
11	53.684	26.057
12	56.668	30.069
13	57.512	31.502

Circle Center At X = 17.675 ; Y = 55.948 ; and Radius = 46.799
Factor of Safety
*** 2.482 ***
Failure Surface Specified By 12 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	10.862	10.431
2	15.854	10.141
3	20.850	10.336
4	25.804	11.014
5	30.669	12.168
6	35.399	13.788
7	39.951	15.858
8	44.280	18.359
9	48.347	21.267
10	52.114	24.555
11	55.544	28.193
12	58.218	31.644

Circle Center At X = 16.355 ; Y = 61.634 ; and Radius = 51.497
Factor of Safety
*** 2.482 ***
Failure Surface Specified By 12 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	10.517	10.259
2	15.487	9.711
3	20.487	9.717
4	25.456	10.278
5	30.331	11.386
6	35.054	13.028
7	39.565	15.184
8	43.810	17.826
9	47.736	20.923

```
        10          51.294        24.436
        11          54.440        28.322
        12          56.325        31.265
     Circle Center At X =    17.947 ; Y =    54.505 ; and Radius =    44.865
            Factor of Safety
          ***    2.483   ***
     Failure Surface Specified By 12 Coordinate Points
       Point       X-Surf       Y-Surf
        No.         (ft)         (ft)
         1           9.828       10.000
         2          14.779        9.305
         3          19.778        9.190
         4          24.756        9.657
         5          29.646       10.699
         6          34.382       12.302
         7          38.900       14.444
         8          43.138       17.097
         9          47.040       20.224
        10          50.551       23.784
        11          53.625       27.727
        12          55.698       31.140
     Circle Center At X =    18.275 ; Y =    52.007 ; and Radius =    42.848
            Factor of Safety
          ***    2.486   ***
     Failure Surface Specified By 13 Coordinate Points
       Point       X-Surf       Y-Surf
        No.         (ft)         (ft)
         1           7.414       10.000
         2          12.379        9.410
         3          17.377        9.290
         4          22.365        9.642
         5          27.297       10.464
         6          32.130       11.746
         7          36.820       13.479
         8          41.326       15.646
         9          45.607       18.229
        10          49.626       21.203
        11          53.346       24.544
        12          56.735       28.220
        13          59.540       31.908
     Circle Center At X =    16.143 ; Y =    62.236 ; and Radius =    52.960
            Factor of Safety
          ***    2.489   ***
     Failure Surface Specified By 13 Coordinate Points
       Point       X-Surf       Y-Surf
        No.         (ft)         (ft)
         1           7.414       10.000
         2          12.369        9.329
         3          17.365        9.143
         4          22.356        9.443
         5          27.294       10.227
         6          32.133       11.487
         7          36.826       13.212
         8          41.329       15.384
         9          45.600       17.984
        10          49.598       20.987
        11          53.285       24.364
        12          56.626       28.084
        13          59.424       31.885
     Circle Center At X =    16.778 ; Y =    60.520 ; and Radius =    51.380
            Factor of Safety
          ***    2.490   ***
              **** END OF GSTABL7 OUTPUT ****
```

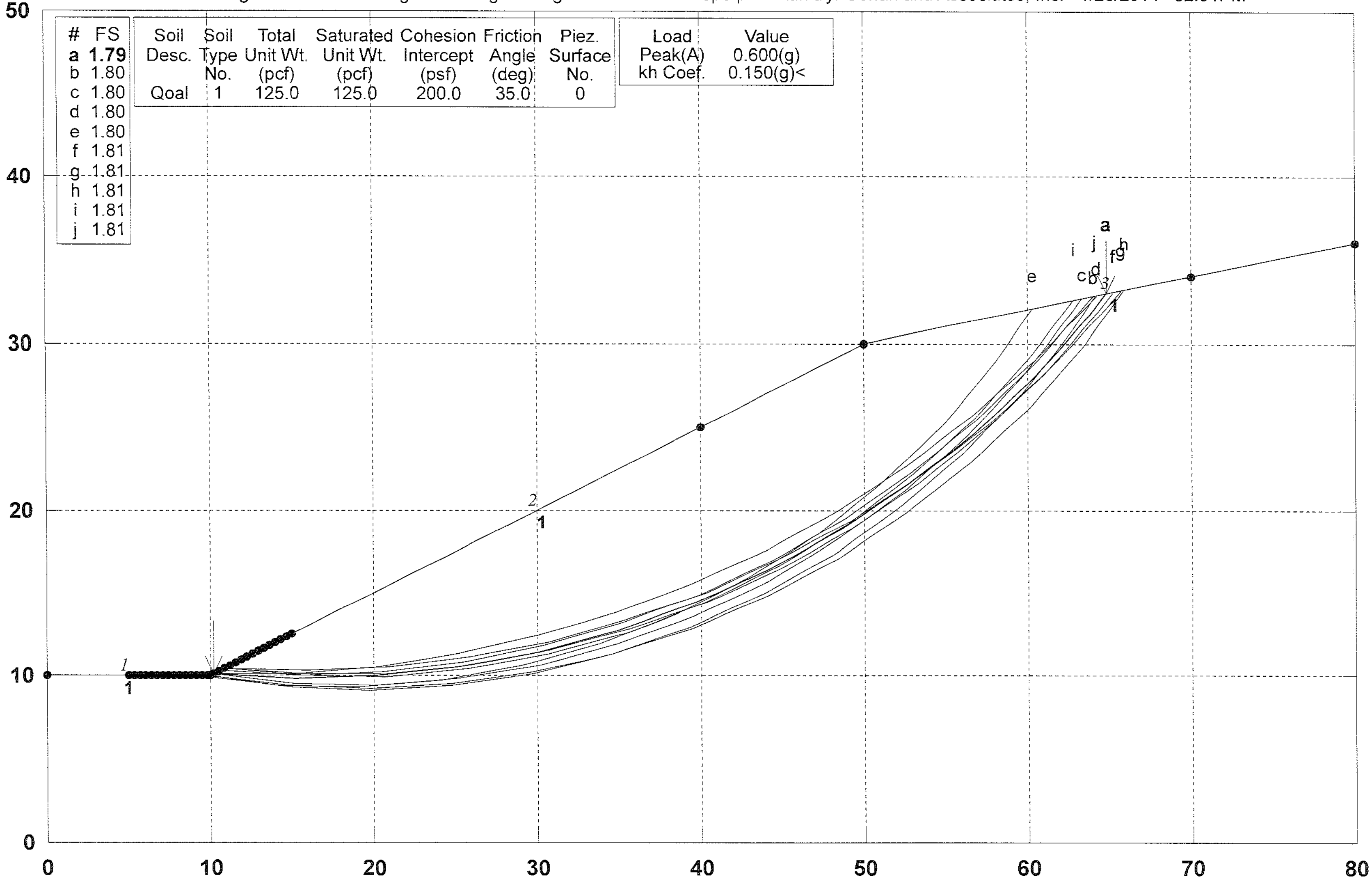
WO 2272-1-0-101 20 foot high cut slope pseudo-static

x:\2272-1 agoura hills sr housing comm\engineering calcs\2272 cut slope.pl2 Run By: Gorian and Associates, Inc. 1/28/2014 02:31PM

#	FS
a	**1.79**
b	1.80
c	1.80
d	1.80
e	1.80
f	1.81
g	1.81
h	1.81
i	1.81
j	1.81

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Qoal	1	125.0	125.0	200.0	35.0	0

Load	Value
Peak(A)	0.600(g)
kh Coef.	0.150(g)<



GSTABL7 v.2 FSmin=1.79

Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7

***** GSTABL7 *****

** GSTABL7 by Garry H. Gregory, P.E. **
** Original Version 1.0, January 1996; Current Version 2.005, Sept. 2006 **

****
SLOPE STABILITY ANALYSIS SYSTEM
Modified Bishop, Simplified Janbu, or GLE Method of Slices.
(Includes Spencer & Morgenstern-Price Type Analysis)
Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
Nonlinear Undrained Shear Strength, Curved Phi Envelope,
Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
****

Analysis Run Date: 1/28/2014
Time of Run: 02:31PM
Run By: Gorian and Associates, Inc.
Input Data Filename: X:\2272-1 Agoura Hills Sr Housing Comm\engineering calcs\2272 cut slope.dat
Output Filename: X:\2272-1 Agoura Hills Sr Housing Comm\engineering calcs\2272 cut slope.OUT
Unit System: English
Plotted Output Filename: X:\2272-1 Agoura Hills Sr Housing Comm\engineering calcs\2272 cut slope.PLT
PROBLEM DESCRIPTION: WO 2272-1-0-101
20 foot high cut slope pseudo-static

BOUNDARY COORDINATES
3 Top Boundaries
3 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	10.00	10.00	10.00	1
2	10.00	10.00	50.00	30.00	1
3	50.00	30.00	80.00	36.00	1

Default Y-Origin = 0.00(ft)
Default X-Plus Value = 0.00(ft)
Default Y-Plus Value = 0.00(ft)

ISOTROPIC SOIL PARAMETERS
1 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	125.0	125.0	200.0	35.0	0.00	0.0	0

Specified Peak Ground Acceleration Coefficient (A) = 0.600(g)
Specified Horizontal Earthquake Coefficient (kh) = 0.150(g)
Specified Vertical Earthquake Coefficient (kv) = 0.000(g)
Specified Seismic Pore-Pressure Factor = 0.000
A Critical Failure Surface Searching Method, Using A Random Technique For Generating Circular Surfaces, Has Been Specified.
3000 Trial Surfaces Have Been Generated.
100 Surface(s) Initiate(s) From Each Of 30 Points Equally Spaced Along The Ground Surface Between X = 5.00(ft)
and X = 15.00(ft)
Each Surface Terminates Between X = 40.00(ft)
and X = 70.00(ft)
Unless Further Limitations Were Imposed, The Minimum Elevation At Which A Surface Extends Is Y = 0.00(ft)
5.00(ft) Line Segments Define Each Trial Failure Surface.
Following Are Displayed The Ten Most Critical Of The Trial Failure Surfaces Evaluated. They Are Ordered - Most Critical First.
* * Safety Factors Are Calculated By The Modified Bishop Method * *
Total Number of Trial Surfaces Attempted = 3000
Number of Trial Surfaces With Valid FS = 3000
Statistical Data On All Valid FS Values:
FS Max = 5.574 FS Min = 1.794 FS Ave = 2.253
Standard Deviation = 0.317 Coefficient of Variation = 14.08 %
Failure Surface Specified By 14 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	10.172	10.086
2	15.166	9.826
3	20.164	9.961
4	25.136	10.490
5	30.051	11.410
6	34.877	12.714
7	39.586	14.396
8	44.148	16.443
9	48.534	18.844
10	52.716	21.583
11	56.670	24.644
12	60.370	28.008
13	63.793	31.652
14	64.847	32.969

Circle Center At X = 15.984 ; Y = 72.960 ; and Radius = 63.141

Factor of Safety

*** 1.794 ***

Individual data on the 14 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	5.0	860.2	0.0	0.0	0.	0.	129.0	0.0	0.0
2	5.0	2460.6	0.0	0.0	0.	0.	369.1	0.0	0.0
3	5.0	3790.5	0.0	0.0	0.	0.	568.6	0.0	0.0
4	4.9	4820.3	0.0	0.0	0.	0.	723.1	0.0	0.0
5	4.8	5532.6	0.0	0.0	0.	0.	829.9	0.0	0.0
6	4.7	5922.0	0.0	0.0	0.	0.	888.3	0.0	0.0
7	4.6	5995.2	0.0	0.0	0.	0.	899.3	0.0	0.0
8	4.4	5771.2	0.0	0.0	0.	0.	865.7	0.0	0.0
9	1.5	1889.6	0.0	0.0	0.	0.	283.4	0.0	0.0
10	2.7	3252.1	0.0	0.0	0.	0.	487.8	0.0	0.0
11	4.0	3867.0	0.0	0.0	0.	0.	580.0	0.0	0.0
12	3.7	2487.1	0.0	0.0	0.	0.	373.1	0.0	0.0
13	3.4	1106.5	0.0	0.0	0.	0.	166.0	0.0	0.0
14	1.1	72.9	0.0	0.0	0.	0.	10.9	0.0	0.0

Failure Surface Specified By 14 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	10.517	10.259
2	15.513	10.045
3	20.509	10.230
4	25.475	10.813
5	30.379	11.791
6	35.189	13.156
7	39.874	14.901
8	44.406	17.014
9	48.754	19.482
10	52.892	22.289
11	56.793	25.417
12	60.431	28.846
13	63.785	32.555
14	63.968	32.794

Circle Center At X = 15.701 ; Y = 72.621 ; and Radius = 62.577

Factor of Safety

*** 1.798 ***

Failure Surface Specified By 14 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	10.172	10.086
2	15.142	9.538
3	20.141	9.448
4	25.128	9.815
5	30.060	10.637
6	34.896	11.907

```
       7          39.595       13.615
       8          44.119       15.745
       9          48.428       18.281
      10          52.487       21.201
      11          56.262       24.479
      12          59.721       28.090
      13          62.835       32.002
      14          63.261       32.652
  Circle Center At X =    18.643 ; Y =     63.890 ; and Radius =     54.466
         Factor of Safety
        ***    1.801    ***
  Failure Surface Specified By 14 Coordinate Points
    Point       X-Surf       Y-Surf
     No.         (ft)         (ft)
       1          10.172       10.086
       2          15.172       10.132
       3          20.156       10.528
       4          25.101       11.273
       5          29.980       12.364
       6          34.771       13.794
       7          39.450       15.557
       8          43.994       17.645
       9          48.379       20.046
      10          52.586       22.749
      11          56.592       25.741
      12          60.378       29.006
      13          63.925       32.530
      14          64.196       32.839
  Circle Center At X =    12.041 ; Y =     81.161 ; and Radius =     71.099
         Factor of Safety
        ***    1.802    ***
  Failure Surface Specified By 13 Coordinate Points
    Point       X-Surf       Y-Surf
     No.         (ft)         (ft)
       1           9.138       10.000
       2          14.103        9.413
       3          19.102        9.311
       4          24.087        9.695
       5          29.012       10.563
       6          33.828       11.905
       7          38.492       13.708
       8          42.958       15.957
       9          47.184       18.629
      10          51.130       21.699
      11          54.760       25.138
      12          58.038       28.913
      13          60.268       32.054
  Circle Center At X =    17.648 ; Y =     60.669 ; and Radius =     51.378
         Factor of Safety
        ***    1.803    ***
  Failure Surface Specified By 14 Coordinate Points
    Point       X-Surf       Y-Surf
     No.         (ft)         (ft)
       1          10.862       10.431
       2          15.850       10.082
       3          20.850       10.144
       4          25.827       10.616
       5          30.749       11.495
       6          35.583       12.775
       7          40.295       14.447
       8          44.853       16.501
       9          49.228       18.922
      10          53.390       21.694
      11          57.309       24.798
      12          60.961       28.213
      13          64.320       31.917
      14          65.179       33.036
```

Circle Center At X = 17.621 ; Y = 70.750 ; and Radius = 60.697

Factor of Safety

*** 1.805 ***

Failure Surface Specified By 14 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	10.862	10.431
2	15.860	10.272
3	20.855	10.487
4	25.820	11.076
5	30.727	12.035
6	35.549	13.359
7	40.258	15.040
8	44.828	17.068
9	49.233	19.434
10	53.449	22.122
11	57.451	25.119
12	61.218	28.407
13	64.729	31.967
14	65.728	33.146

Circle Center At X = 15.514 ; Y = 76.815 ; and Radius = 66.547

Factor of Safety

*** 1.806 ***

Failure Surface Specified By 14 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	9.828	10.000
2	14.791	9.393
3	19.788	9.219
4	24.781	9.481
5	29.732	10.176
6	34.604	11.299
7	39.361	12.841
8	43.965	14.792
9	48.382	17.135
10	52.578	19.853
11	56.522	22.926
12	60.184	26.331
13	63.536	30.040
14	65.914	33.183

Circle Center At X = 19.291 ; Y = 66.541 ; and Radius = 57.327

Factor of Safety

*** 1.807 ***

Failure Surface Specified By 13 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	10.862	10.431
2	15.839	9.952
3	20.839	9.932
4	25.820	10.369
5	30.739	11.262
6	35.557	12.602
7	40.231	14.378
8	44.722	16.575
9	48.993	19.175
10	53.008	22.155
11	56.733	25.490
12	60.136	29.153
13	62.756	32.551

Circle Center At X = 18.563 ; Y = 64.364 ; and Radius = 54.480

Factor of Safety

*** 1.807 ***

Failure Surface Specified By 14 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	9.828	10.000
2	14.783	9.334

```
        3          19.779        9.130
        4          24.772        9.389
        5          29.720       10.109
        6          34.580       11.285
        7          39.310       12.905
        8          43.870       14.956
        9          48.220       17.421
       10          52.324       20.277
       11          56.145       23.502
       12          59.652       27.066
       13          62.813       30.940
       14          64.073       32.815
   Circle Center At X =    19.500 ; Y =    62.899 ; and Radius =    53.776
          Factor of Safety
         ***    1.807   ***
             **** END OF GSTABL7 OUTPUT ****
```

Surficial Slope Stability
(Seepage Parallel to Slope)

GORIAN & ASSOCIATES, INC.

Applied Earth Sciences
Geotechnical
Engineers
and Geologists

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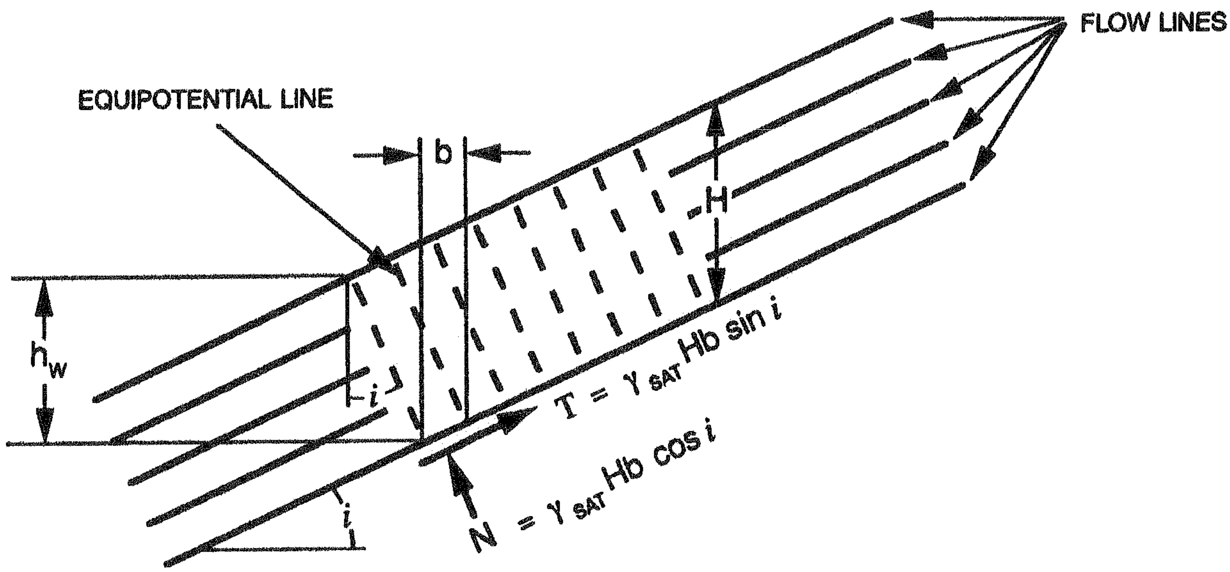

$$T = \frac{T}{b/\cos i} \quad \frac{\gamma_{SAT} H \sin i \cos i}{b/\cos i} = \gamma_{SAT} H \sin i \cos i \quad = \text{TANGENTIAL STRESS}$$

$$\sigma = \frac{N}{b/\cos i} - h_w \gamma_w = \gamma_{SAT} H \cos^2 i - h_w \gamma_w \quad : \gamma' H \cos^2 i = \text{NORMAL STRESS}$$

$$F.S. = \frac{C}{\gamma_{SAT} H \cos^2 i \tan i} + \frac{\gamma' \tan\phi}{\gamma_{SAT} \tan i}$$

$$F.S. = \frac{400}{200.18} + \frac{24.66}{62.60}$$

C	=	400	psf
ϕ	=	21.5	degrees
$\tan\phi$	=	0.39	
i	=	26.6	degrees
$\tan i$	=	0.50	
$\cos^2 i$	=	0.80	
H	=	4	feet
γ_{SAT}	=	125	pcf
γ'	=	62.6	pcf

FACTOR OF SAFETY = 2.39

2(h):1(v) Engineered Fill Slope

Reference: Soil Mech. and Found., Parcher, Means, 1967
Work Order: 2272-1-0-11
Log Number: 20524

GORIAN
& ASSOCIATES, INC.

Surficial Slope Stability
(Seepage Parallel to Slope)

Applied Earth Sciences
Geotechnical
Engineers
and Geologists

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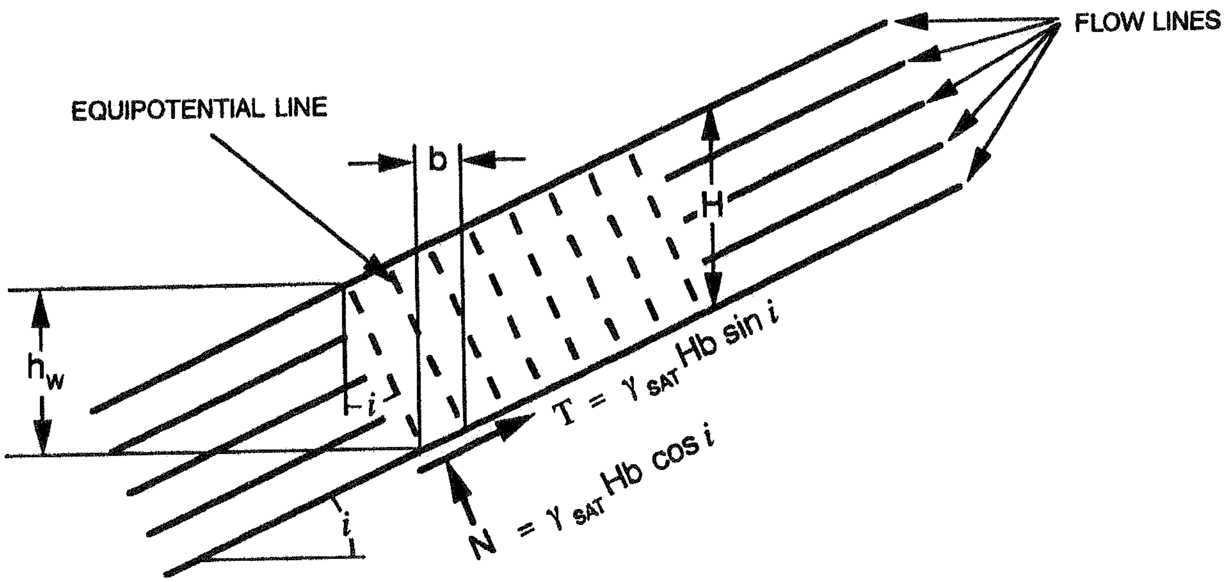
California 91361
805 497-9363
818 889-2137
FAX 805 373-6938



$$T = \frac{T}{b/\cos i} \quad \frac{\gamma_{SAT} H \sin i \cos i}{b/\cos i} = \gamma_{SAT} H \sin i \cos i \quad = \text{TANGENTIAL STRESS}$$

$$\sigma = \frac{N}{b/\cos i} - h_w \gamma_w = \gamma_{SAT} H \cos^2 i - h_w \gamma_w \quad : \gamma' H \cos^2 i = \text{NORMAL STRESS}$$

$$\text{F.S.} = \frac{C}{\gamma_{SAT} H \cos^2 i \tan i} + \frac{\gamma' \tan\phi}{\gamma_{SAT} \tan i}$$

$$\text{F.S.} = \frac{200}{200.18} + \frac{43.83}{62.60}$$

C	=	200	psf
ϕ	=	35	degrees
$\tan\phi$	=	0.70	
i	=	26.6	degrees
$\tan i$	=	0.50	
$\cos^2 i$	=	0.80	
H	=	4	feet
γ_{SAT}	=	125	pcf
γ'	=	62.6	pcf

FACTOR OF SAFETY = 1.70

2(h):1(v) Cut Slope
in Alluvium

Reference: Soil Mech. and Found., Parcher, Means, 1967
Work Order: 2272-1-0-11
Log Number: 20524

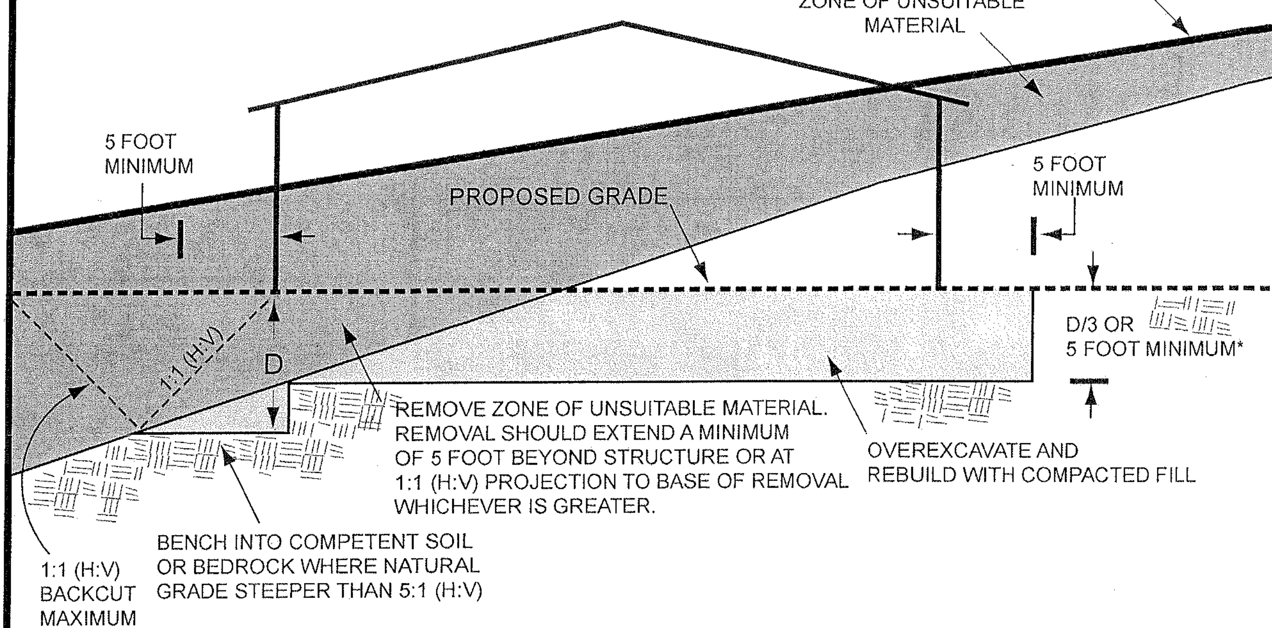
APPENDIX D

TYPICAL CONSTRUCTION DETAILS

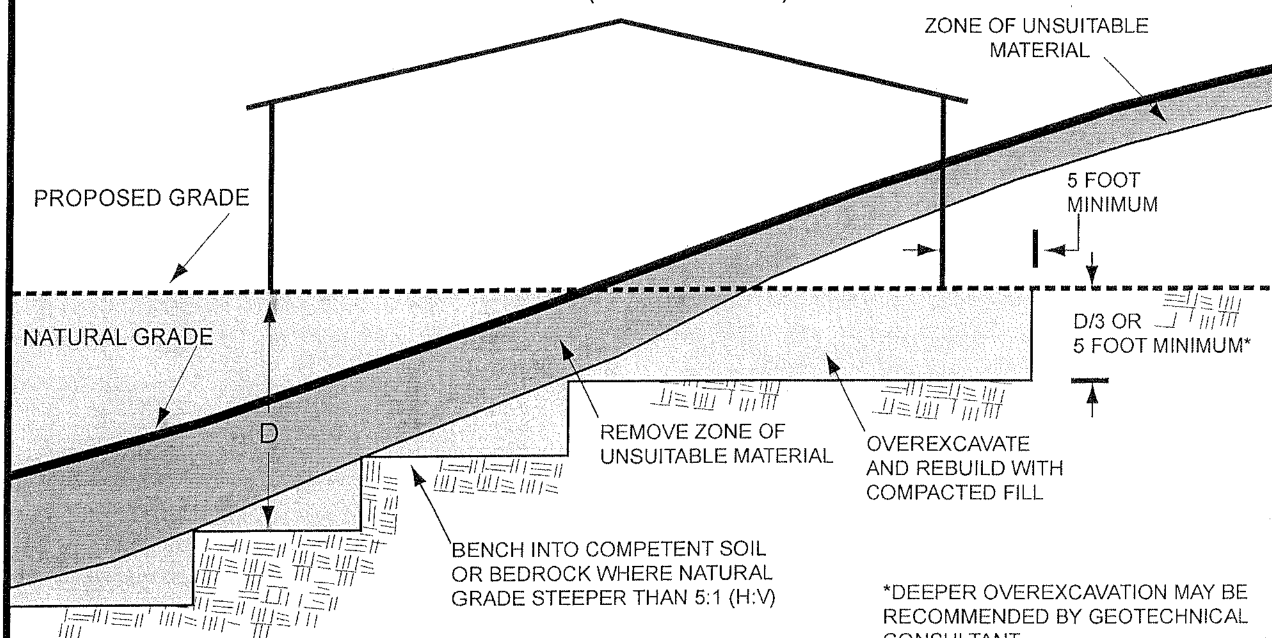
BUILDING PAD OVEREXCAVATION

TYPICAL DETAIL

CUT LOT



CUT-FILL (TRANSITION) LOT



DETAIL IS NOT TO SCALE

*DEEPER OVEREXCAVATION MAY BE RECOMMENDED BY GEOTECHNICAL CONSULTANT

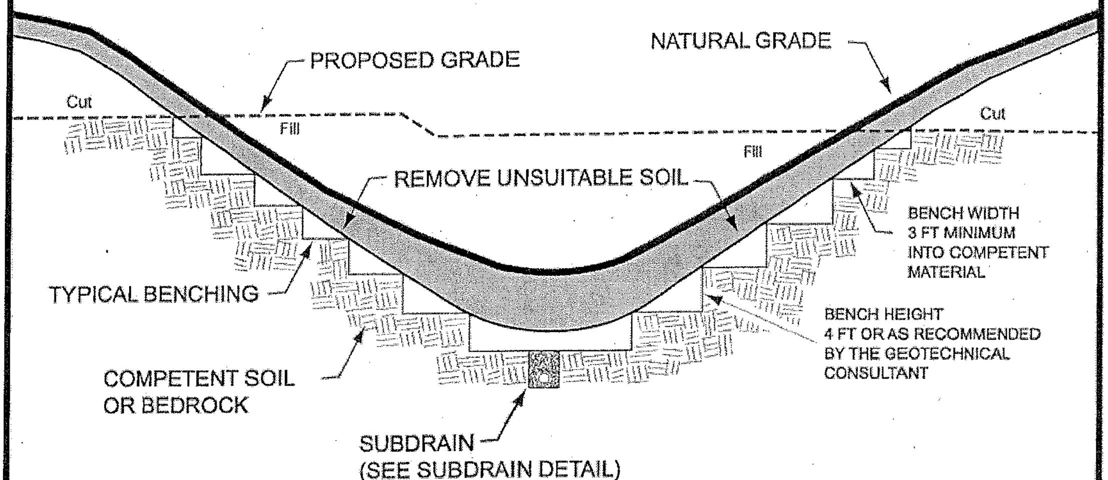
GORIAN & ASSOCIATES, INC.

overex1.ai

CANYON CLEANOUT AND BENCHING

TYPICAL DETAIL

(GENERALIZED TRANSVERSE SECTION)

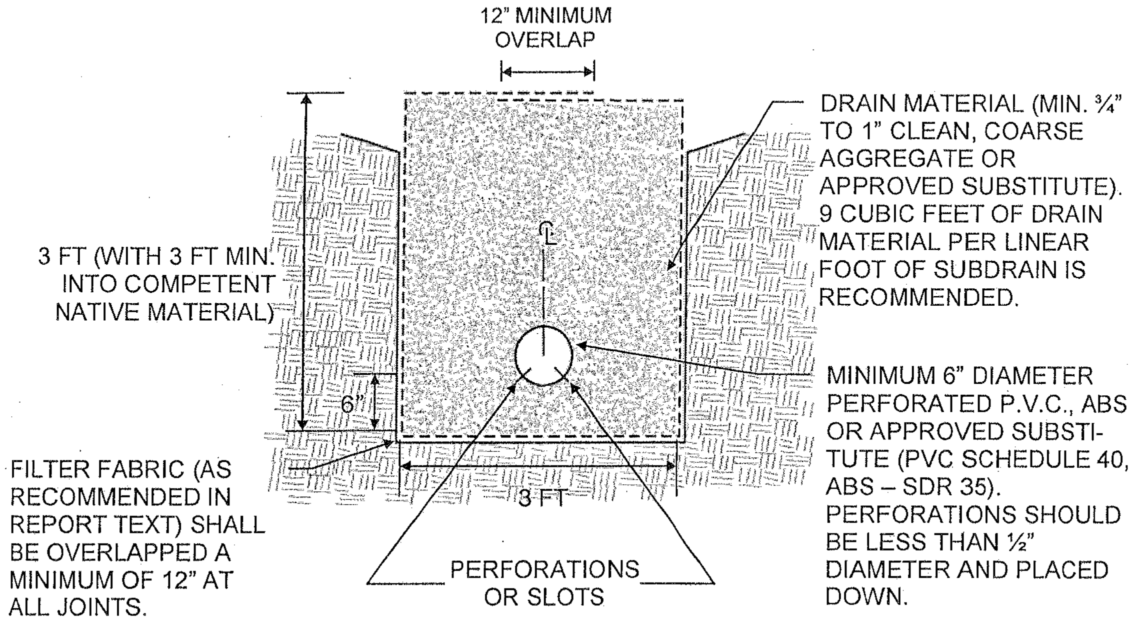


DETAIL IS NOT TO SCALE

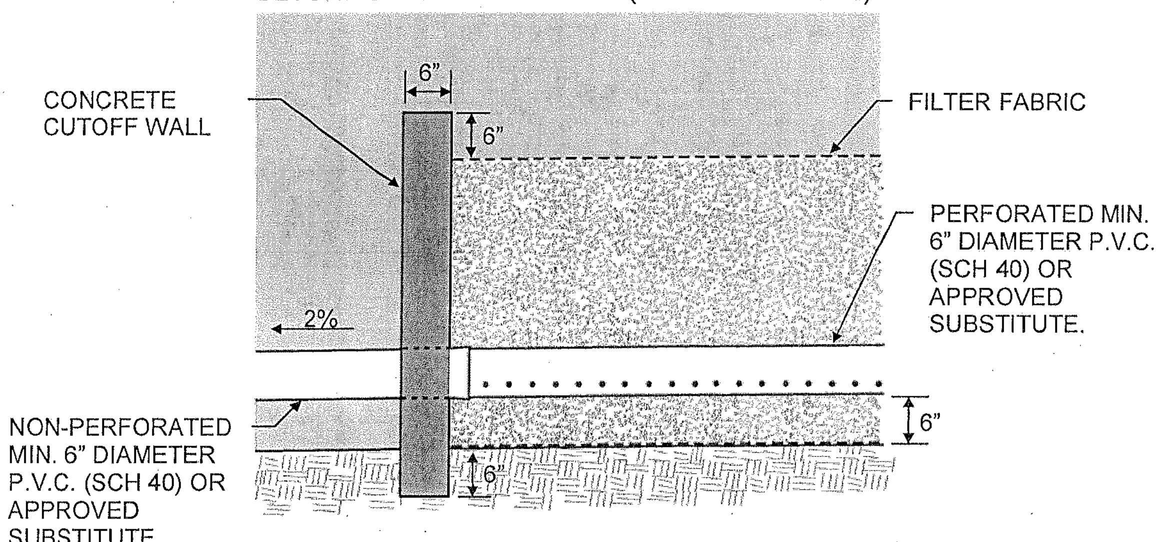
GORIAN & ASSOCIATES, INC.

CANYON SUBDRAIN AND CUTOFF WALL

TYPICAL DETAIL



NOTE: CUTOFF WALL SHOULD EXTEND MINIMUM 6" BEYOND SUBDRAIN MATERIAL (IN ALL DIRECTIONS)



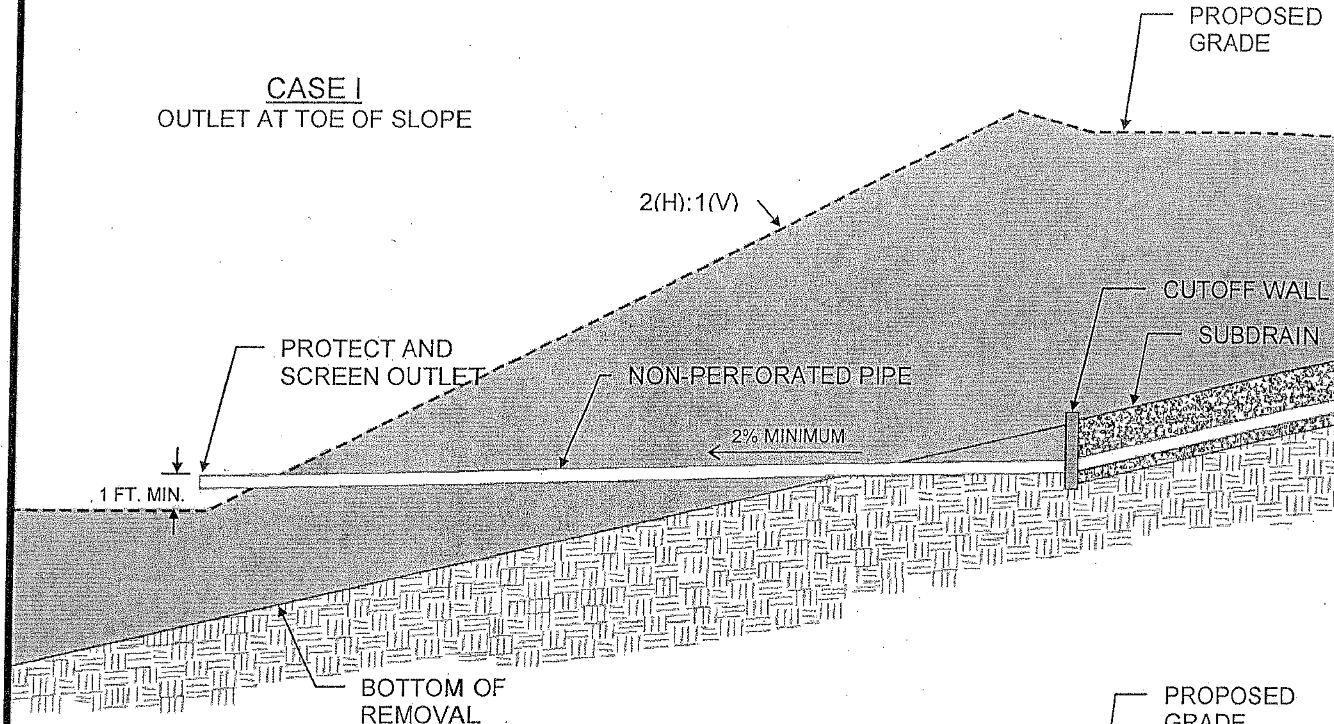
DETAIL IS NOT TO SCALE

NOTE: Subdrain pipe should be 6" diameter up to 500 ft long and 8" diameter over 500 ft long. Subdrain pipe should be 10" diameter where up to three 6" subdrain pipes or where two 8" diameter subdrain pipes are confluent unless specified otherwise by the geotechnical consultant. (See soils report)

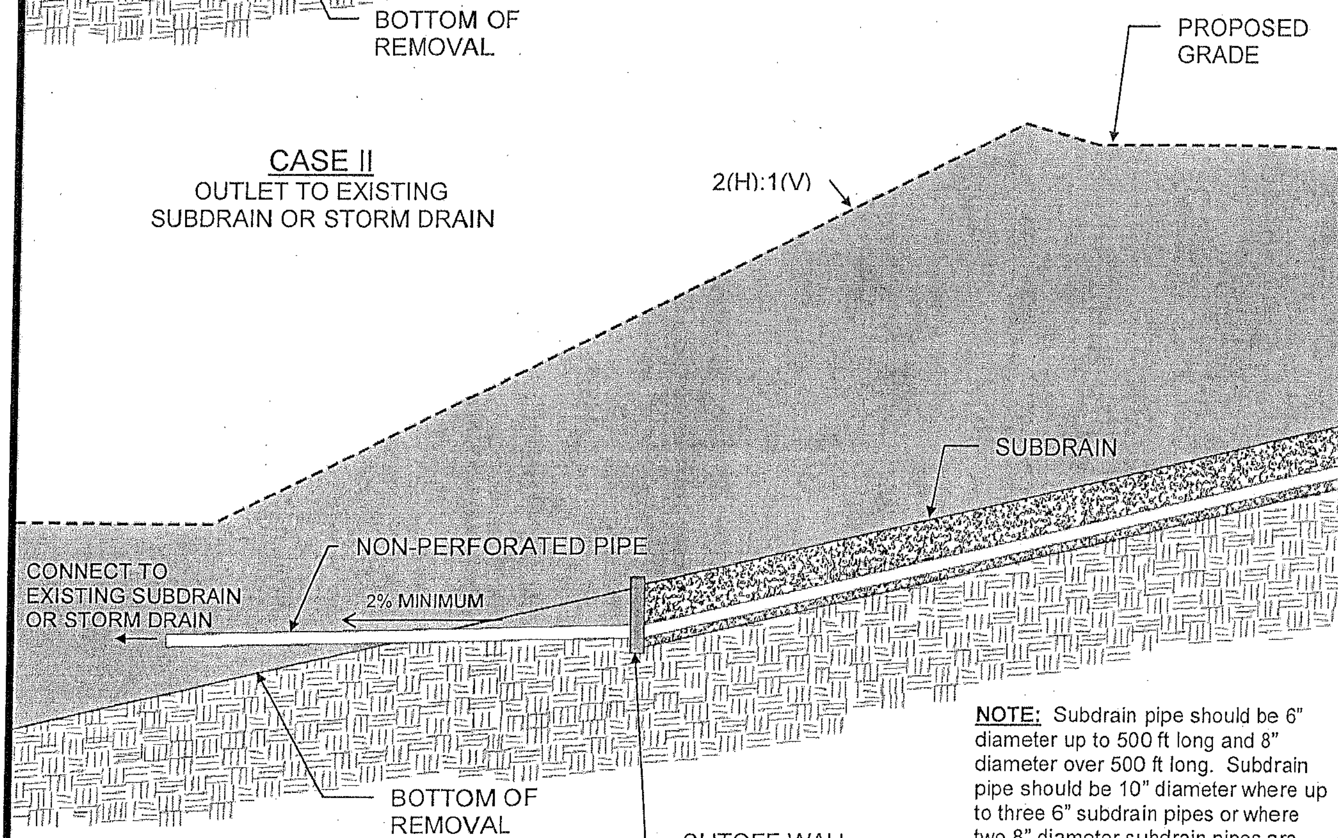
CANYON SUBDRAIN OUTLET TYPICAL DETAIL

(GENERALIZED LONGITUDINAL SECTION)

CASE I
OUTLET AT TOE OF SLOPE



CASE II
OUTLET TO EXISTING SUBDRAIN OR STORM DRAIN



NOTE: Subdrain pipe should be 6" diameter up to 500 ft long and 8" diameter over 500 ft long. Subdrain pipe should be 10" diameter where up to three 6" subdrain pipes or where two 8" diameter subdrain pipes are confluent unless specified otherwise by the geotechnical consultant.

DETAIL IS NOT TO SCALE

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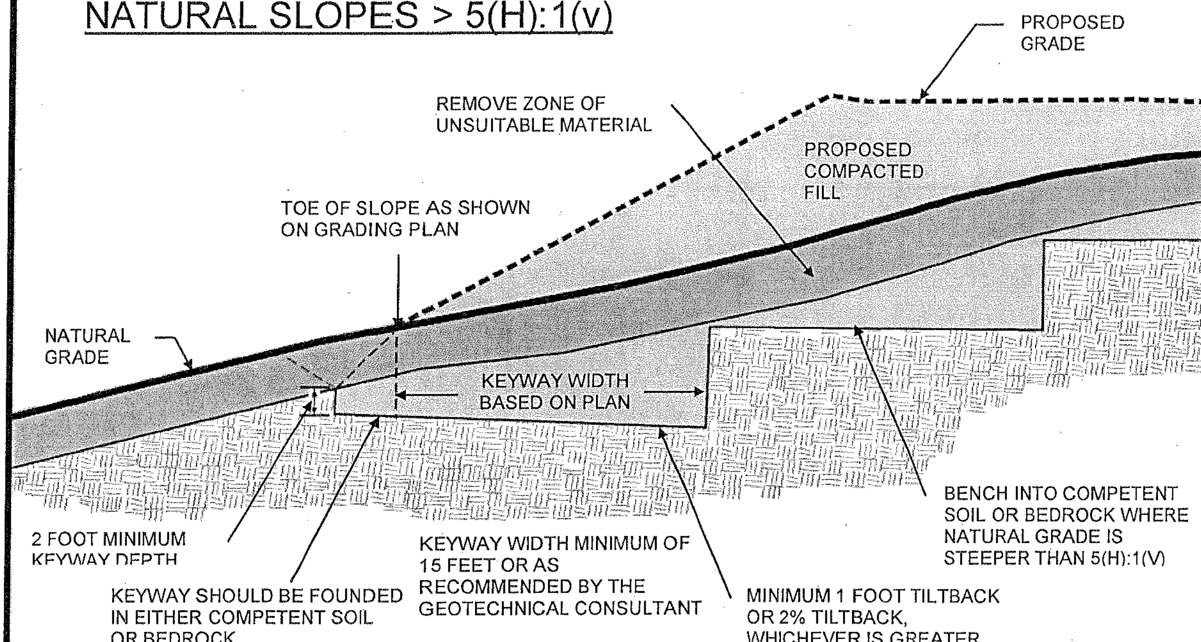
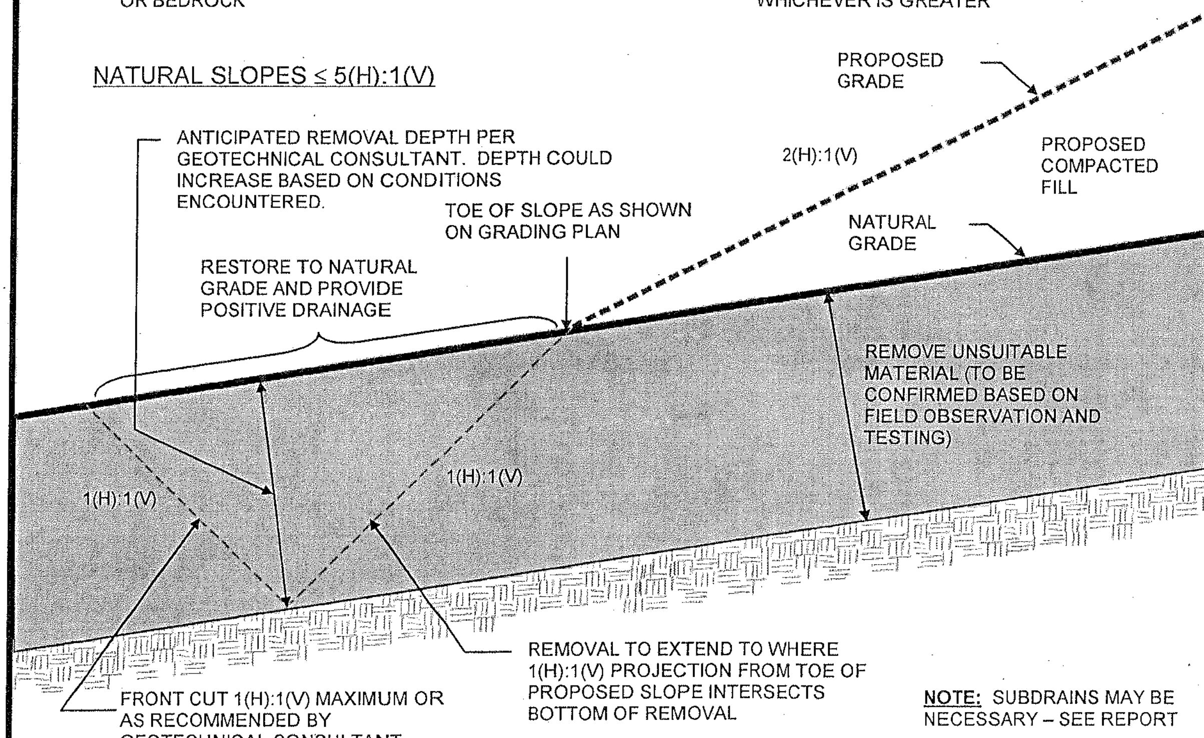
REMOVALS BEYOND TOE OF PROPOSED FILL SLOPES

TYPICAL DETAIL

<u>NATURAL SLOPES > 5(H):1(v)</u>



<u>NATURAL SLOPES ≤ 5(H):1(V)</u>

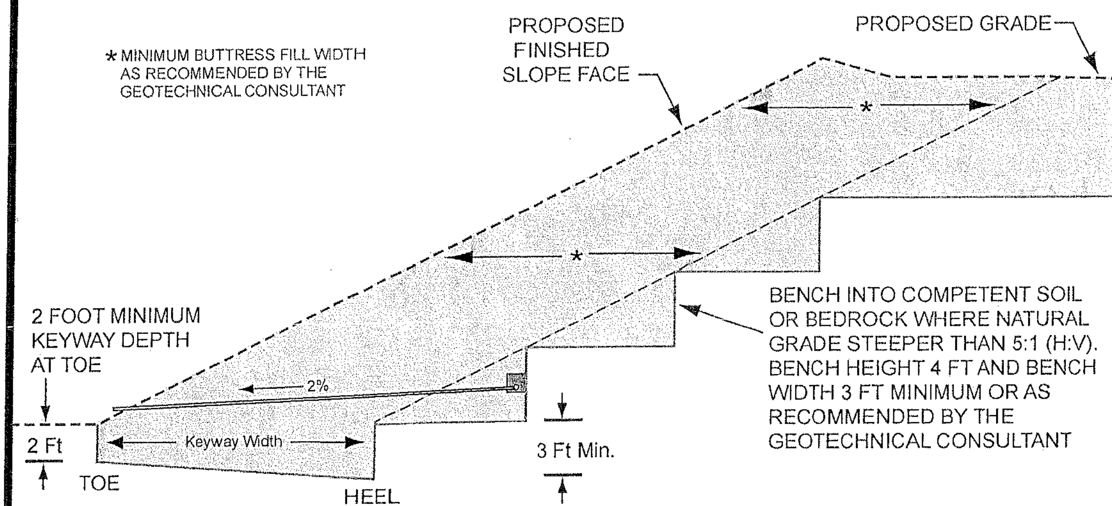


NOTE: SUBDRAINS MAY BE NECESSARY – SEE REPORT

DETAIL IS NOT TO SCALE

BUTTRESS FILL

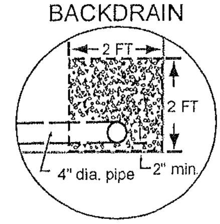
TYPICAL DETAIL



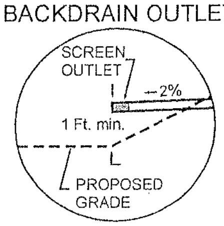
NOTES:

1. KEYWAY MINIMUM WIDTH AS RECOMMENDED BY THE GEOTECHNICAL CONSULTANT. KEYWAY SHOULD BE FOUNDED IN BEDROCK.
2. TILT KEYWAY INTO SLOPE; HEEL MINIMUM 1 FOOT DEEPER THAN TOE OR 2% GRADE TOWARD HEEL (WHICHEVER IS GREATER).
3. TYPICAL BACKDRAIN 4 CUBIC FEET OF CLEAN 3/4 INCH GRAVEL PER LINEAL FOOT OF BACKDRAIN. GRAVEL SHOULD BE WRAPPED IN FILTER FABRIC AND PLACED AT BACK OF KEYWAY AND EVERY 30 FEET (VERTICAL).
4. PIPE SHOULD BE 4 INCH PVC, ABS, OR APPROVED SUBSTITUTE (PVC SCHEDULE 40, ABS - SDR 35).
5. BACKDRAIN PIPE SHOULD BE PERFORATED AND PERFORATIONS SHOULD BE PLACED FACING DOWN.
6. OUTLET PIPE SHOULD BE NON-PERFORATED AND CONNECTED TO THE PERFORATED PIPE BY "L" OR "T" JOINTS.
7. BACKDRAIN SHOULD EXTEND LENGTH OF BUTTRESS FILL, SHOULD BE SLOPED TOWARD OUTLETS, AND OUTLETS PROVIDED EVERY 100 FEET OR AS SPECIFIED BY THE GEOTECHNICAL CONSULTANT. (MIN. 2 OUTLETS)

BACKDRAIN



BACKDRAIN OUTLET



DETAIL IS NOT TO SCALE

GORIAN & ASSOCIATES, INC.

BENCHMARK

RDBM TAG IN CONC CB 42 FEET WEST OF CENTERLINE OF KANAN ROAD AND 64 FEET NORTH OF CENTERLINE OF AGOURA ROAD.

BM NO.: CY10004A ELEVATION: 961.97 ADJ.: 1988

FLOOD PLAIN NOTE

COUNTY:	LOS ANGELES
ASSESSOR PARCEL #:	2061-001-025
FLOOD ZONE:	X
COMMUNITY NAME:	AGOURA HILLS
COMMUNITY #:	065037
COMMENT:	MINIMAL FLOOD HAZARD
MAP NUMBER:	065037C1243F
MAP DATE:	SEPTEMBER 26, 2008

LEGAL DESCRIPTION

PARCEL 2 OF PARCEL MAP 15762, IN THE CITY OF AGOURA HILLS, AS PER MAP FILED IN BOOK 175 PAGES 6 AND 7 OF PARCEL MAPS, IN THE OFFICE OF THE COUNTY RECORDER OF SAID COUNTY.

EXCEPT THEREFROM AN UNDIVIDED ONE-HALF INTEREST IN ALL OIL, GAS, MINERALS AND OTHER HYDROCARBON SUBSTANCES LYING BELOW THE SURFACE OF SAID LAND; BUT WITH NO RIGHT OF SURFACE ENTRY THERETO, AS PROVIDED IN THE DEED RECORDED DECEMBER 26, 1960 AS INSTRUMENT NO. 1450 IN BOOK D1076 PAGE 565, OFFICIAL RECORDS.

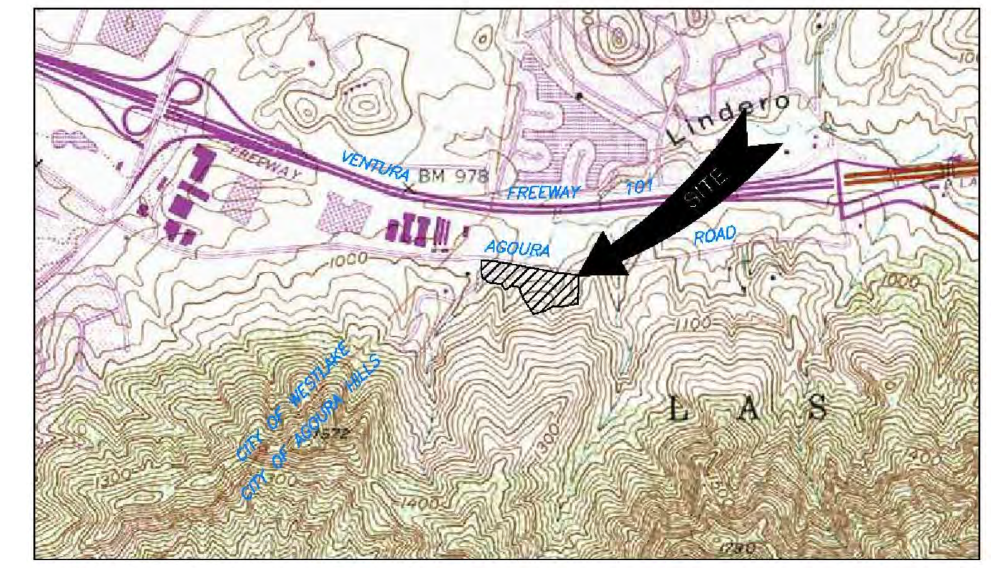
ALSO EXCEPT THEREFROM AN UNDIVIDED ONE-HALF INTEREST IN ALL OIL, GAS, MINERALS AND OTHER HYDROCARBON SUBSTANCES LYING BELOW THE SURFACE OF SAID LAND; BUT WITH NO RIGHT OF SURFACE ENTRY THERETO, AS PROVIDED IN THE DEED RECORDED DECEMBER 29, 1960 AS INSTRUMENT NO. 1452 IN BOOK D1076 PAGE 568, OFFICIAL RECORDS.

ALSO EXCEPT THEREFROM AN UNDIVIDED ONE-HALF INTEREST IN ALL OIL, GAS, MINERALS AND OTHER HYDROCARBON SUBSTANCES LYING BELOW THE SURFACE OF SAID LAND, BUT WITH NO RIGHT OF SURFACE ENTRY THERETO, AS PROVIDED IN THE DEED RECORDED MARCH 4, 1983 AS INSTRUMENT NO. 83-248390.

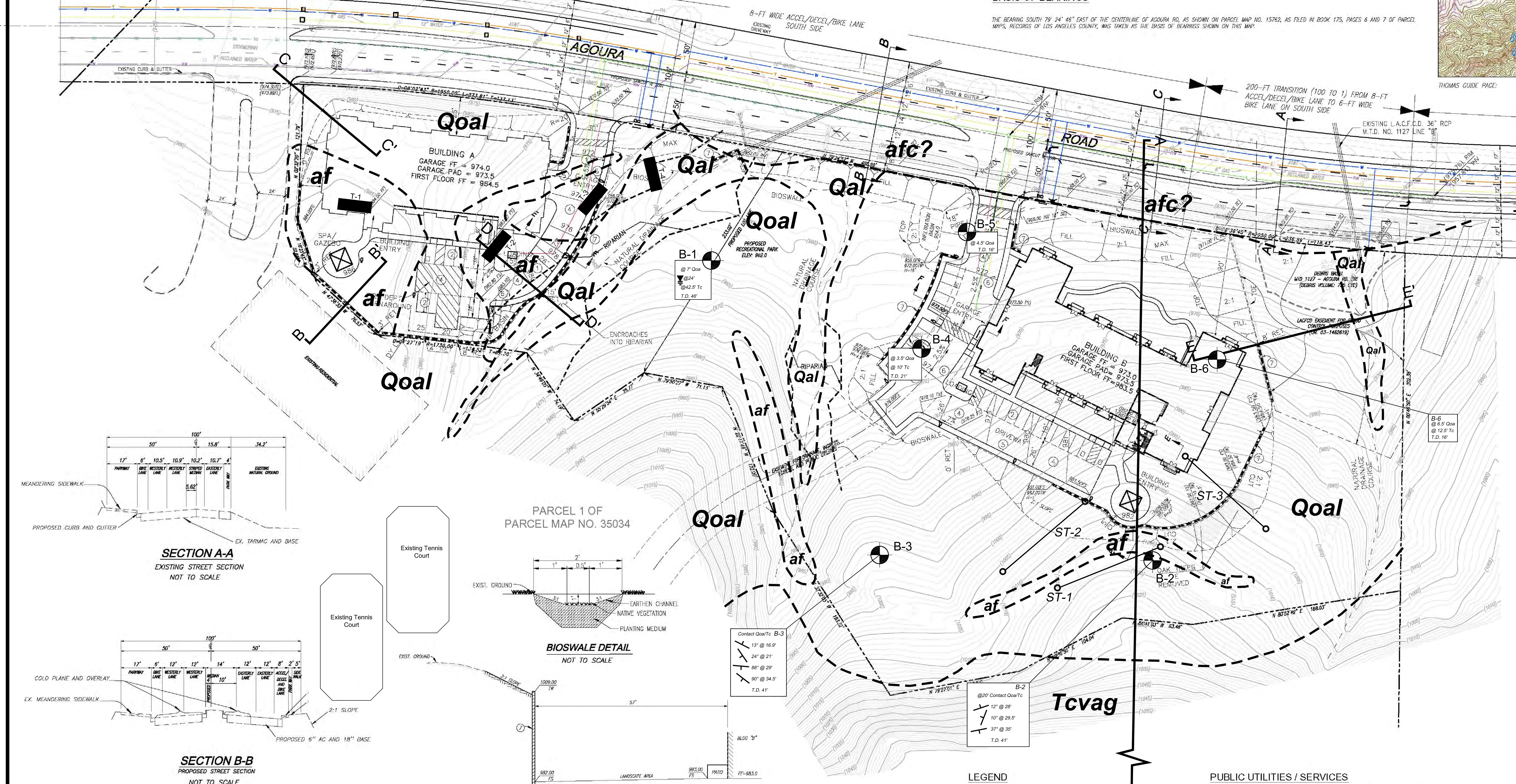
BASIS OF BEARINGS

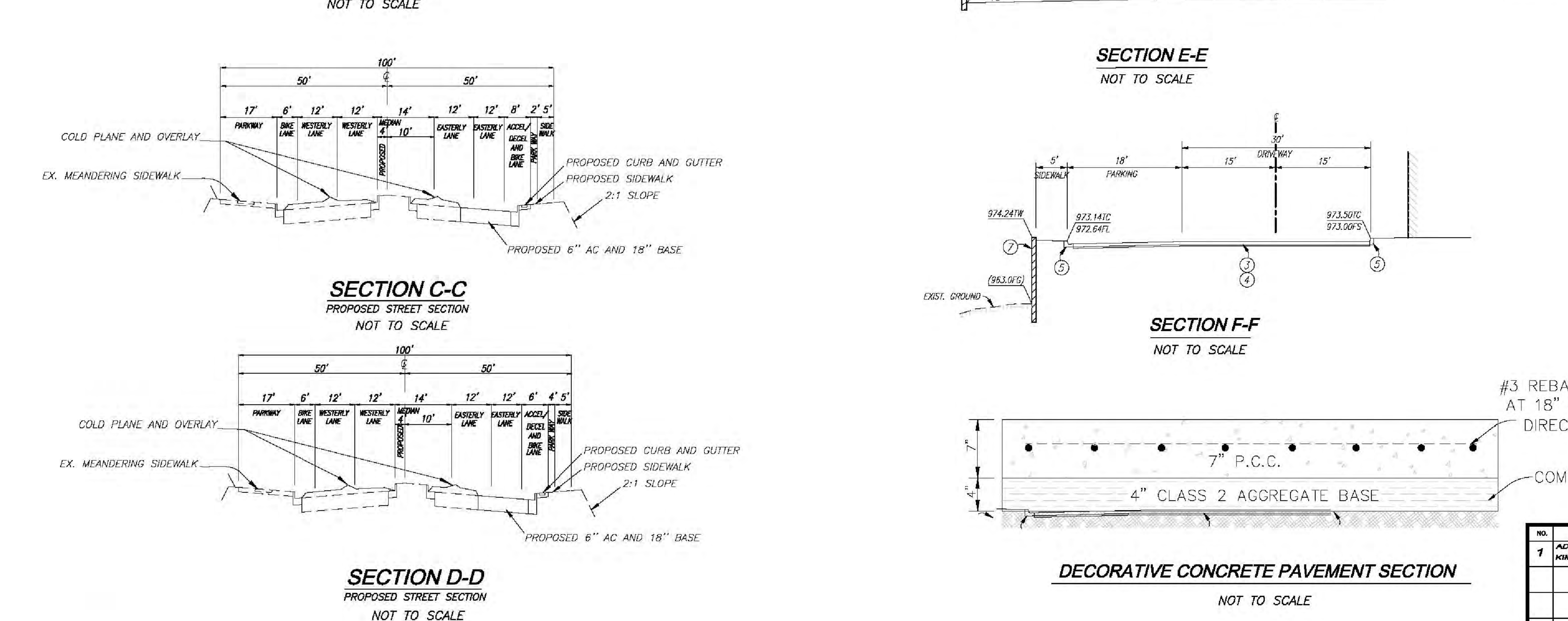
THE BEARING SOUTH 79' 24' 46" EAST OF THE CENTERLINE OF AGOURA RD, AS SHOWN ON PARCEL MAP NO. 15762, AS FILED IN BOOK 175, PAGES 6 AND 7 OF PARCEL MAPS, RECORDS OF LOS ANGELES COUNTY, WAS TAKEN AS THE BASIS OF BEARINGS SHOWN ON THIS MAP.

VICINITY MAP



THOMAS GUIDE PAGE: 557 F-6





CONSTRUCTION NOTES:

1. CONSTRUCT 3 INCHES OF A.C. OVER 10 INCHES OF AGGREGATE BASE
2. CONSTRUCT 3 INCHES OF A.C. OVER 7 INCHES OF AGGREGATE BASE.
3. DECORATIVE CONCRETE, 7 INCHES WITH NO. 3 BARS AT 18 INCHES ON CENTER IN BOTH DIRECTIONS OVER 4 INCHES OF CLASS 2 AGGREGATE BASE. SEE SECTION HEREON.
4. FIRELANE PER L.A.C.F.D. REQUIREMENTS.
5. 6" CONC. CURB PER APWA STD. PLAN 120-1
6. 6" CONC. CURB AND 18" GUTTER PER APWA STD. PLAN 120-1
7. RETAINING WALLS BY SEPARATE PERMIT
8. HANDICAP RAMP PER APWA STD. 111-2
9. 6" ROLLED CURB PER APWA STD. PLAN 121-1

PROPOSED LAND USAGE & AREAS

PROPOSED LAND USES: SENIOR HOUSING COMMUNITY

TOTAL NUMBER OF LOTS: 2

GORIAN & ASSOCIATES, INC. Applied Earth Sciences		
Job No.: 2272-0-0-101		Date: Jan. 2014
Scale: 1" = 40'	Drawn By: Approved By:	PLATE 1

EXPLANATION

afc	Certified Engineered Fill
Qal	Alluvium
Qoal	Older Alluvium
Tc	Calabasas Formation
Tcvag	Conejo Volcanics (Agglomerate)
	Approximate Contact Between Geologic Units
T-4	Approximate Location of Exploratory Backhoe Pit
B-6	Approximate Location of 24" Diameter Exploratory Boring
	Strike and Dip of Bedding
	Groundwater
ST-3	Approximate Location of Seismic Traverse
A A'	Geotechnical Cross Section

PUBLIC UTILITIES / SERVICES

WATER:	LAS VIRGENES MUNICIPAL WATER DIST. 4232 LAS VIRGENES ROAD CALABASAS, CA 91302 (818) 880-4110	SEWER:	LA COUNTY, DEPT. OF PUBLIC WORKS SEWER MAINTENANCE DIVISION 1000 S. FREMONT AVENUE, BLDG A9 EAST ALHAMBRA, CA 91803 (626) 300-3308
ELECTRICAL:	SOUTHERN CALIFORNIA EDISON 3589 FOOTHILL DRIVE THOUSAND OAKS, CA 91361 (805) 494-7016	CABLE:	TIME WARNER CABLE 2323 TELLER ROAD NEWBURY PARK, CA 91320 (805) 375-5213
TELEPHONE:	SBC (PAC BELL) 16201 RAYMER STREET, #115 VAN NUYS, CA 91406 (818) 373-6889	CABLE:	CHARTER COMMUNICATIONS 3806 CROSSCREEK ROAD MALIBU, CA 90265 (310) 456-9010
GAS:	SOUTHERN CALIFORNIA GAS 9400 OAKDALE AVENUE CHATSWORTH, CA 91313 (818) 701-3324	CALTRANS:	CALTRANS 5660 RESEDA BOULEVARD TARZANA, CA 91356 (805) 388-1426

UTILITY NOTE

UTILITY INFORMATION SHOWN HEREON WAS COMPILED FROM PUBLIC RECORDS. NO LIABILITY IS ASSUMED OR INFERRED BY HMK ENGINEERING AS TO THE ACCURACY OF SAID INFORMATION.

BOUNDARY DATUM

ESTABLISHED FROM THE LOCAL CITY OR COUNTY ENGINEERING BUREAU'S BOUNDING MONUMENTED CENTERLINES OR FROM POINTS OF RECORD AS SHOWN. IF ELEVATIONS ARE SHOWN, THEY ARE FOR INFORMATION ONLY AND ARE NOT FOR CERTIFICATION OF GRADE, DRAINAGE OR ANY OTHER ITEM WHICH MAY BE CONNECTED TO VERTICAL DATUM.

SCALE: 1"=40'

NO.	REVISION	REVISED BY	APPROVED BY	DATE
1	ADDED CITY'S WIDENING PER KIMLEY HORN'S PLANS	08/13/2013	MARK D HARDY	08/13/2013

HMK ENGINEERING INC. — CIVIL ENGINEERS · LAND SURVEYORS · LAND PLANNERS — 1552 EIGHTEENTH STREET, SANTA MONICA, CA 90404 PHONE (310) 449-5511

No. 36536 EXP. JUNE 30, 2014

(OWNER& DEVELOPER) AGOURA HILLS CENTER PROPERTIES, LLC 2985 E. HILLCREST DRIVE #107 THOUSAND OAKS, CA. 91362 (818) 865-0200 CONTACT: STEVE RICE

VESTING TENTATIVE TRACT MAP NUMBER 71742

LOCATED IN A CORPORATED TERRITORY OF THE COUNTY OF LOS ANGELES, STATE OF CALIFORNIA

SEPTEMBER 2012

SHEET 1 OF 1

GEOTECHNICAL CROSS SECTIONS

GORIAN & ASSOCIATES, INC.
Applied Earth Sciences

Job No.: 2272-0-0-101	Date: Jan. 2014
Scale: 1" = 40'	Drawn By: Approved By:
	PLATE 2