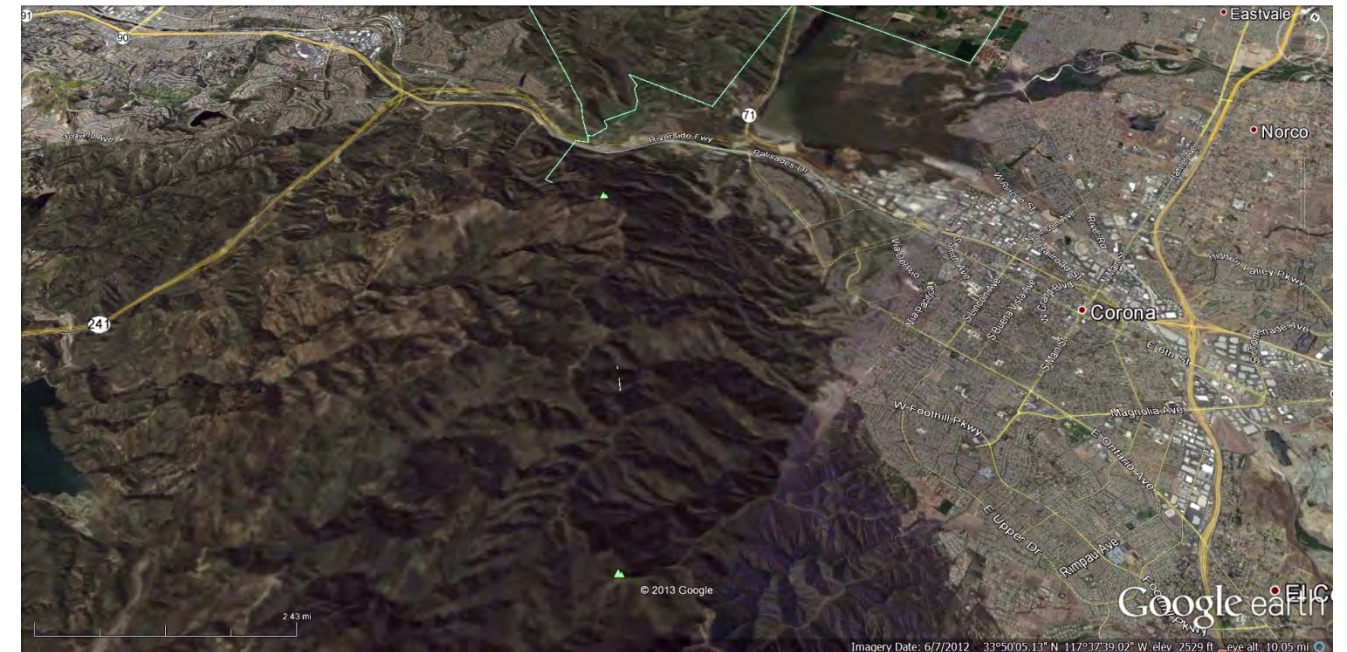


**PRELIMINARY GEOTECHNICAL INVESTIGATION AND
GRADING PLAN REVIEW FOR TTM# 26544,
SKYLINE HEIGHTS EIR, CITY OF CORONA, CALIFORNIA**



***Prepared for:* Richland Development Corporation**

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PROJECT # 134-02

May 20, 2013



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May 20, 2013
Project # 134-02

Attn: Mr. Mike Byer

SUBJECT: **Preliminary Geotechnical Investigation
And Grading Plan Review for
Tentative Tract Map No. 36544, Skyline Heights EIR
City of Corona, County of Riverside, California**

References: See Appendix I

Gentlemen:

In accordance with your request, Ginter & Associates, Inc. (G&A) presents the results of a preliminary geotechnical investigation and EIR level grading plan review for the subject site. G&A bases this report on its experience with similar projects in the area and information predominately contained in Pacific Soils Engineering, Inc. (PSE) referenced reports.

This firm has retained the services of Mr. Mike Mills, Consulting Engineering Geologist, who has authored all of PSE's referenced reports. Mr. Mills was instrumental in providing the synthesis of PSE's data and especially the Alquist-Priolo Earthquake Fault Zone Investigations.

This document will be suitable for inclusion into the EIR and for submittal to the reviewing agencies. G&A's review of the data and Tentative Tract Map No 36544 indicates that the proposed development is feasible from a geotechnical standpoint, provided the recommendations presented herein are adhered to during site development.

Our firm appreciates the opportunity to provide our services and look forward to working with you and the other team consultants. If you have any questions or require additional information, please contact the undersigned.

Thank you.

GINTER & ASSOCIATES, INC.

Dave Ginter

Dave Ginter
President



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1.0 Introduction:

1.1 PURPOSE:

This report has been prepared to provide a preliminary geologic and geotechnical engineering evaluation and grading plan review, suitable for inclusion as an appendix for the EIR, for the Skyline Heights development site, in the city of Corona, California.

This EIR-level document presents the data and analyses regarding the geology, soil properties geologic hazards and associated mitigation measure and general grading/foundation considerations and incorporates the data and findings from previous investigations by others.

The Tentative Tract Map (TTM) No. 36544 was prepared by KWC Engineers dated April 2, 2013, at a scale of 1"=60'. Data developed during previous subsurface investigations and referenced reports by others were transferred to the TT base (Figures 1 through 7, in pockets) which provides the basis for this review.

1.2 SCOPE OF WORK:

The scope of this evaluation consisted of the following:

- Review the referenced site and adjacent development reports
- Review available laboratory data to evaluate engineering properties of the on-site subsurface materials
- Review and synthesize Alquist-Priolo Earthquake Fault Zone investigation reports by Pacific Soils Engineering, Inc. (PSE) for the subject site
- Transfer geologic and geotechnical information generated from previous site investigations to the 1"=60' scale TTM plans prepared by KWC Engineers
- Delineate buttress keys, stabilization keys, fill keys, sub-drain locations and settlement monument locations
- Delineate fill areas below 50 ft. from finish grade requiring 93% relative compaction
- Delineate cut areas where potentially heavy ripping and/or blasting may be required

- Prepare this report, which characterizes the site's geological and geotechnical engineering aspects and provides geotechnical recommendations suitable for inclusion into the EIR

2.0 Project Description:

2.1 SITE LOCATION AND EXISTING CONDITIONS:

The Skyline Heights project consists of two spheres of development separated by Mabey Canyon. The northern portion is triangle-shaped, consisting of approximately 12 acres bounded on the west by Cleveland National Forest, on the northeast by the proposed extension of Foothill Parkway and on the south by Mabey Canyon (Plates I and II). The southern portion consists of 400 acres±, is irregular in shape and is situated near the mouths of Mabey, Tin Mine and Kroonen Canyons, in the city of Corona, California (Figure 1). Undeveloped land borders this portion of the site on the west, northwest and much of the south. An existing residential community abuts part of the east boundary and the northeast boundary.

North Portion of Development

An unimproved road emanating from Mabey Canyon provides access to this area. Vehicular access however, is limited, owing to the steep topography and non-maintenance of the road..

This area is in an almost natural state. Steep topography with a thin soil mantle has precluded agricultural development common elsewhere in the general area. Very dense chaparral punctuated by sporadic stands of scrub oak covers most slopes.

Clay mining occurred in the Temescal Canyon area from the early 1900's (Gray, 1961). Scattered bulldozer trenches and access roads were cleared for the previous subsurface investigations and remnants of clay exploration roads are evident. No large-scale mining appears to have been carried out in this area.

This portion of the development generally consists of steep-sided northeast trending "main" ridges with similar steep-sided "spur" ridges on the northeastern flank of the Santa Ana Mountains. Side-slope ratios vary from 2:1 to 1:1 with local gentler and steeper sections. Consequently, v-shaped canyons and narrow, spine-like ridges abound.

Four large canyons drain this area; three low-order canyons drain northwesterly into Wardlow Canyon and the fourth drains the center of this area into high-order Mabey Canyon to the south. These major drainages are v-shaped in their upper and middle reaches, where

incised into bedrock and flat in their lower reaches, where emanating onto plains of high-order Mabey and Wardlow Canyons.

The Mabey Canyon drainage flows into a large flood control basin near its mouth (Plate I), which intercepts flow and directs it northeast via lined channels into Temescal Canyon and eventually, the Santa Ana River system.

South Portion of Development

Unimproved roads emanating from Mabey Canyon Road and Mangular Avenue provide access to this area however, vehicular access is limited because of steep topography and environmental issues.

This portion consists of approximately 400 acres and was subjected to clay prospects and mines in the early 1900's. One successful clay prospect was the McKnight Clay Mine (Gray, 1961) southeast adjacent to the area (Plate V) that is now a tree nursery. Other poorly-preserved and undeveloped prospects and related access roads are scattered over the site. For example, an exploration trench (PSE 2005) encountered a small adit near proposed Lot 116.

Northwest trending "main" ridges and "spur" ridges with steep sides dominate the geomorphology. Low-order drainages feed three high-order drainages that extend over large areas of the northeastern Santa Ana Mountains. These high-order drainages are, from north to south, Mabey, Kroonen and Tin Mine Canyons (Plate I). Both following and breaching the local geologic structures and lithologies, these major drainage are commonly v-shaped in their upper and middle reaches, where incised into bedrock and flat in their lower reaches.

2.2 PROPOSED DEVELOPMENT

The proposed development involves two spheres of development – one north of Mabey Canyon, consisting of 44 lots and a larger one, south of Mabey Canyon, consisting of 246 lots for single-family homes. Access to the project will be from Foothill Parkway, which is proposed along the northeast side of the project. It is our understanding that the grading for the extension of Foothill Parkway will be completed prior to the subject site grading.

Cuts and fills to approximate maximum design depths of 130 ft. (vicinity of Lot 123), excluding utility excavation, and 138 ft. (vicinity of Lot 223), respectively are proposed. Cut slopes range from small 2 ft. high side yard split slopes to 140 ft. at 2:1 slope ratios. One 1.5:1 cut slope approximately 80 ft. high is proposed (Figure 6). Fill slopes range from small, less than 2 ft. high

side yard split slopes to 160 ft. at 2:1 slope ratios. One 1.5:1 slope approximately 130ft. high is proposed on the northern perimeter (see Sheet 2).

3.0 Site Geological/Geotechnical Investigations:

3.1 PREVIOUS REGIONAL INVESTIGATIONS:

The California Division of Mines and Geology (CDMG) published several geological reports pertaining to the geologic setting of the northern Santa Ana Mountains and Temescal Canyon, including the separating Whittier-Elsinore Fault Zone (WEFZ) (Gray, 1961; “Jenkins Edition” CDMG, 1965). These deal mainly with regional-scale mapping or local economic geology. Weber (1977), however, published detailed geologic fault maps and fault activity analyses of the WEFZ and unnamed local faults. He identified and named three NW-trending local segments of the WEFZ: The Tine Mine Fault (TMF), Main Street Fault (MSF) and Eagle Fault (EaF). Weber mapped the MSF near and almost parallel to the northeast site boundary (Plate 1); the TMF as bisecting Skyline Heights (Plate I); and the EaF about 200 ft. to the south. Gray et al. (2002) later incorporated the work of Weber and Gray into one map. The Gray (1961), Weber (1977) and Gray et al. (2002) maps are similar, but differ in the location and number of minor mapped faults.

Owing to increasing urbanization of the Corona and Temescal Canyon areas, the CDMG (1980) studied local faults and concluded that major strands of the WEFZ are likely active (Holocene) as exemplified by youthful geomorphic features such as right-laterally-displaced Holocene channels. Likewise, apparent fault scarps, displaced Holocene alluvium and a sag pond at “Glen Ivy Marsh” about 10 miles to the southeast. Accordingly, the CDMG (1980) placed the MSF in an Alquist-Priolo (A-P) Zone that requires site-specific fault rupture hazards investigation prior to development. Recently, the CGS (2003) expanded the local A-P Zones and included both the MSF and elements of the TMF (Plate I).

In 2003, the Riverside County Planning Department (2002), Temescal Canyon Area Plan) included the TMF, MSF and associated faults as seismic hazards.

3.2 SITE-SPECIFIC INVESTIGATIONS:

Between 2004 and 2007, Pacific Soils Engineering, Inc. (PSE) carried out a variety of field investigations within and adjacent to the Skyline Heights property limits. Most dealt with the

presence or absence of active faults within several adjacent parcels. For detailed descriptions and analyses, the reader is referred to the full reports as listed in Appendix A. For reference and contest, key documents are summarized as follows:

1. An A-P-level investigation of the “Pourhakimi” property along the east margin of the Skyline Heights project (PSE, 2004; Plate II) that concluded that active elements of the WEFZ are not present in the areas that were to be, and that are now, occupied by habitable structures. PSE did, for conservatism, impose a provisional setback along the northeast boundary of that site that is about parallel to the fault zone. Also completed, was a preliminary geotechnical investigation of the site. The site was then developed using another firm as geologists and geotechnical engineers of record.
2. An A-P-level investigation (PSE, 2005) focused on the Tin Mine (TMF) and related faults mapped on the Skyline Heights property, that encompasses the part south of Mabey Canyon. Based on aerial photograph and literature review, geologic mapping and logging of 13-excavator trenches up to 1170 ft. long and 17 excavator “pits”, PSE concluded and reported that, with the exception of a small “un-dateable” fault, the TMF and related faults are not active within the investigated areas using A-P Act criteria. Such will be summarized in detail later in this document.
3. An A-P-level investigation (PSE, 2007) of the “Lot 84” property that comprises the area of Skyline Heights north of Mabey Canyon (Plate I). PSE concluded, based on the presence of overlying un-displaced pre-Holocene sediments, that faults mapped onsite are not active according to A-P. Further, a fault interpreted as the extension of the aforementioned “un-dateable” fault encountered in the southern part of Skyline Heights is in fact, overlain by un-faulted pre-Holocene sediments, as reported by PSE.
4. Apparently, in a preliminary geotechnical investigation (unpublished) of the southern Skyline Heights property PSE undertook 16 seismic lines; drilling, logging and sampling of fifteen 30-inch diameter bucket auger borings using a truck-mounted drill rig; and excavating of and logging 35 trenches and one fault trench (FT-1) using a track-mounted backhoe. The borings ranged in depth from 4 to 75 feet below existing ground surface and were logged by an engineering geologist. Relatively “undisturbed” ring and bulk samples were obtained from the borings and transported to PSE’s laboratory for testing.

Unfortunately, owing a great deal to the closing of PSE, the field data and test results are not available for review and assessment. Only summaries within the text of the unpublished report are now available.

The unpublished geotechnical review by PSE further notes, A D-6 bulldozer created both access pathways and pads for the drill rig, where environmental constraints allowed. It should be noted that the boring locations were limited by site physical constraints including steep topography and environmentally sensitive areas. Detailed evaluation of geologic conditions and possible remedial measures should be based upon additional exploration (perhaps at the Grading Plan Review stage) when environmental clearances can be obtained in a timely manner and, finally, during grading.

3.3 ADJACENT GEOLOGICAL INVESTIGATIONS:

Since the imposition of the aforementioned A-P Zones, several nearby site-specific A-P investigations have taken place (Plate I), as discussed in the PSE (2005) report. Terra Geosciences (1994) investigated the property northeast of the southern Skyline Heights area by placing three trenches in essence perpendicular to the CDMG (1980) MSF. That company found no displacement of “older alluvium” and concluded that the fault probably passes south of its trenches. Neblett and Associates, Inc. (N&A) further delineated the MSF near the northeast border of the Skyline Heights parcel near the mouth of Kroonen Canyon based mainly on small offsets of probable Holocene alluvium. That firm concluded that the trace of the MSF is likely active.

IGC, Inc. (1990, 1991) explored a northeast-trending fault mapped by Weber (1977) northerly adjacent to the Skyline Heights parcel. IGC concluded that the fault is not active on the basis of limited dateable sediments. It is not clear, however, if the IGC feature is a likely fault or, rather, merely a typical clay bed within the Silverado Formation. According to PSE (2005), the descriptions of the lithologies and colors of the features in the IGC trench log suggest that the features are probably clay beds.

Further, according to PSE (2004) in a report of its fault investigation of the Pourhakimi property, geological investigations of the Centex property east of the “Pourhakimi” property (Plate II) identified the active MSF and a pre-Holocene splay of the fault northeasterly offsite to “Pourhakimi” (GeoSoils, Inc., 2001; Shlemon, 2003, personal communication). The active strand was mapped in part at the base of a northeast-facing faceted spur ridge southeast of the study site. Based on field observations, much if not all, of the MSF setback imposed along the northeastern “Pourhakimi” site likely falls within the Foothill Parkway right-of-way. In sum, the 2003 PSE investigation, as well others, demonstrated that the main MSF lies northeast of the southern part of the Skyline Heights parcel.

Of importance is a fault investigation of the TMF carried out by RMA (personal communication with representative of RMA and field observation by PSE personnel, 2005). RMA trenched the TMF as projected southeast from the Skyline Heights property and exposed un-faulted alluvium. Further, RMA mapped then-recent cut slopes and encountered no TMF, but rather, only un-faulted Pleistocene sediments and bedrock across the inferred trace of the fault. Thus, the TMF, if extant at that site, is not active according to A-P standards.

3.4 ADJACENT DEVELOPMENT:

Foothill Parkway has been completed to the northeast Skyline Heights border. As shown on Figures 1 through 7, when completed, the parkway will border north of the proposed Skyline Heights development. Typical cuts and fills are planned to accommodate the road. Thus, cut and fill slopes and fill prisms will border the proposed development. Coordination with the Foothill Parkway authorities will be required to afford adequate disposal of subdrains, adequate bearing qualities of parkway soils interfacing with the Skyline Heights development and adequate stability of cut slopes; or implementation of mitigating design and construction where unfavorable geologic/geotechnical features would affect construction of Skyline Heights.

4.0 Regional Geology:

The following discussion is in essence a synthesis of the PSE reports (2004, 2005, 2007 and unpublished 2008) pertaining to the geological setting of the site and its environs.. The PSE reports focus mainly on fault rupture issues rather than general geotechnical issues, but a great deal of geological information in the reports is quite useful for EIR-level assessment. Where applicable, this firm updates or expands on the aforementioned reports.

Pertinent references describing the regional geology are provided in Appendix I. In brief, the study site occupies a part of the northeastern slope of the Santa Ana Mountains near their boundary with Temescal Canyon (Plate I; Elsinore-Chino Trough). These mountains are one of many northwest-trending southern California ranges separated by broad valleys within the Peninsular Ranges Geomorphic (and structural) Province of southern California (Woodford et al., 1971; Gray, 1961). The province extends from the Transverse Ranges on the north to Baja California on the south.

The Santa Ana Mountains have a Jurassic and Cretaceous metamorphic and igneous core on which Cretaceous through Pliocene marine and non-marine rocks have been deposited (McColloch et al., 2000; 1961). Along the northeast flank of the mountains, the Cretaceous and Tertiary rocks were strongly tilted and folded during later regional deformation (Plate IV). Pleistocene to modern sediments derived

mainly from the upper reaches of the Santa Ana Mountains locally mantle those rocks. These sediments are mainly elevated Pleistocene river terrace and alluvial fan deposits and Holocene fans on valley floors (Temescal, Santa Ana Canyons, modern alluvium in incised drainages and toe-slope colluvium).

The northwest-trending Peninsular Ranges blocks parallel regional continental borderland faults that extend from the Mojave Desert in the east to the Channel Islands on the west. These fault zones are typically exhibit right-lateral strike-slip with associated reverse and normal dip-slip components. The approximately 150-mile long Whittier-Elsinore (WEFZ) that parallels the northeast Santa Ana Mountains, with elements mapped on the Skyline Heights site, is a classic example.

About 2000 feet east of Skyline Heights, the WEFZ splays into three major strands (Plates I and IV). The easternmost strand, the Chino Fault (CF) trends north-northwest across the Santa Ana River and along the east side of the Puente Hills, northeast of the study site. The center strand is mapped as the Main Street Fault (MSF) that, traditionally, is the northern continuation of the Elsinore Fault (EF). The MSF, based on mapping by Weber, (1977) and CGS, (2003), seemingly dies out southeast of Wardlow Canyon, near Mabey Canyon and the study site (Plates I and IV). The westernmost splay is the traditionally mapped Whittier Fault (WF). Near Fresno Canyon, the WF "bends" west-northwest across Santa Ana Canyon and then resumes a northwest trend along the west flank of the Puente Hills. The southernmost strand (s) of the WF is seemingly the Eagle Fault (EaF) of Weber (1977). According to most investigators, the study site thus lies within a tectonic wedge formed by the branching of the WEFZ into three identifiable stands.

Many WEFZ segments are active according to A-P standards. PSE (2002a, 2002b), Millman (1988) and Rockwell et al. (1986) documented surface displacements on the WEFZ in the last 11,000 years (Holocene) and the CGS (2003) has therefore, placed many of these segments in A-P Zones based on the foregoing and the presence of tectonic geomorphic features such as faceted spurs, displaced drainages and linear troughs. Among the faults so zoned, are the MSF and TMF mapped near and on-site, which gave rise to a many local A-P-level investigations as noted in this document and which bear heavily on local geological assessments.

5.0 Local Geology:

5.1 STRATIGRAPHY:

In general, the project highlands are underlain by resistant Cretaceous and Paleocene sedimentary rocks that are locally mantled by older Pleistocene alluvial "terrace" deposits. Whereas the lowland surfaces support mainly Holocene alluvial channel and fan deposits (Figures 1 through 7). The ages, distributions and geometries of the sedimentary bedrock are useful for assessing the presence/absence, style and locations of faults within and near the project. In turn,

the characteristics of the Pleistocene and Holocene alluvial deposits are useful indicators for constraining locations, slip geometries and ages (time of last movement) of local faults. Accordingly, the distinguishing and salient characteristics, distribution and ages of the various deposits are discussed in detail herein. Additionally, information relevant to the engineering characteristics and analyses thereof the onsite soils is included herein.

5.1.1 Cretaceous Trabuco Formation (Kt):

The Trabuco Formation is mainly brown, weakly to moderately lithified cobble to boulder conglomerate, marked by occasional lenses and beds of distinctive maroon silty conglomerate that crops out in the southwest corner of the site (See Plate I of the PSE 2005 report). Clasts are mainly metamorphic, igneous and volcanic fragments that are usually more weathered than clasts within the overlying Ladd Formation. Following the regional structural grain, this formation extends from northwest to southeast south of Tin Mine Canyon (RMA, 2005; personal communication; PSE, 2005).

Important to Skyline Heights development, RMA (personal communication and field observations by senior PSE personnel) trenches placed during an A-P investigation south of Skyline Heights (Plate II) exposed distinct Trabuco "maroon beds" on the south flank of Tin Mine Canyon, thereby documenting formational continuity across Tin Mine Canyon. This contradicts previous geologic maps (Gray, 1961; Weber, 1977; Gray et al., 2002) that depict the formation "fault-truncated" north of the canyon.

The Trabuco Formation grades upward into the Cretaceous Ladd Formation (Gray, 1961). The contact between the two formations is thus arbitrary and herein taken to be the top of the uppermost Trabuco "maroon bed".

5.1.2 Cretaceous Ladd Formation:

Two formal members of the Ladd Formation crop-out northwest and south of Kroonen Canyon (Figures 1 through 7). These are the Baker Canyon Conglomerate and Holz Shale members that usually were readily identifiable in PSE trenches and, locally, in outcrop. In contrast, north and east of the canyon, massive conglomerates and sandstones are deemed "undifferentiated Ladd Formation" (Gray, 1961; Gray et al., 2002), which may also encompass Williams Formation sandstone. Figures 2, 4 and 5) identify these units as "Ladd Formation Conglomerate" (Klc). Clearly identifiable and mappable Williams Formation does crop out in the northern part of the site, northwest of Mabey Canyon.

5.1.2.1 Baker Canyon Conglomerate (Klbe):

Thick-bedded to massive pebble and cobble conglomerate interspersed with buff to gray arkosic to lithic marine sandstone typify the Baker Canyon Conglomerate at and near Skyline Heights. As noted by Gray (1961) and as observable in many natural and anthropic exposures, the lower part of the member consists mainly of distinctly bedded, volcanic and metamorphic cobble conglomerate. Distinctly inter-bedded buff conglomerate and buff to gray thick-bedded sandstone characterize the upper part of the formation exposed onsite. This unit is generally well-indurated (cemented) below a weathered halo and therefore, supports steep anti-dip and dip slopes and is a major "ridge former".

The Baker Canyon Conglomerate is conformable with, and grades into, both the underlying Trabuco Formation and the overlying Holz Shale Member of the Ladd Formation (PSE, 2005)--Trench T-1 of the 2005 investigation clearly exposed a gradational contact between those two members. The transition zone, based on PSE investigations, varies from about 50 to about 100-feet thick. The contacts shown on the enclosed plates are thus arbitrary and could change as extensive in-grading exposures might dictate.

5.1.2.2 Holz Shale (Klh):

Locally fossiliferous, gray to bluish gray where fresh and gray to yellowish brown where weathered shale, claystone, siltstone and silty sandstone comprise the Holz Shale. Generally, the unit is thin-bedded and has well-developed bedding plane parting (PSE, 2005; Schoellhamer et al., 1981). Calcareous concretions up to three feet in diameter occur throughout the Holz Shale. Despite the presence of concretions, the unit is poorly resistant to weathering and forms subdued topography relative to the underlying Cretaceous conglomerate and the overlying Tertiary sandstone.

Where observed by PSE (2005), the Holz Shale is usually fractured (squeezed) and is replete with small-scale "flow" folds symptomatic of plastic deformation. This contrasts with the brittle behavior of the overlying and underlying Silverado Formation and Baker Conglomerate where faults, fractures and similar brittle deformation predominate.

Holz Shale forms a narrow V-shaped band typified by subdued, linear topography in the western part of the subject site (Figures 4 and 6). The outcrop pattern

describes a regional, closed, locally overturned, southeast-plunging syncline (Plate IV).

PSE mapping of the Holz Shale varies markedly from published regional geologic maps (Gray, 1961; Weber, 1977; Gray et al., 2002; Morton and Miller, 2006). This disparity likely stems from the new trench and road cut exposures that are now available, but were not to the earlier investigators.

Most importantly, PSE identified local ridges of Baker Canyon Conglomerate and topographic lows carved in previously unidentified Holz Shale, rather than the Silverado Formation depicted by the earlier investigators. The absence of Tertiary Silverado and the presence of "Cretaceous-against-Cretaceous" relationships suggest limited structural relief and stratigraphic separation. Accordingly, large-scale fault displacements are therefore, not necessary to explain local bedrock geometry and distribution.

Ladd Formation Conglomerate (Klc)

A large lithesome of well-indurated conglomerate interspersed with scattered sandstone beds underlies most of the area south of Mabey Canyon and north of Kroonen Canyon (Figures 2, 4 and 5) Based on its stratigraphic position, Gray (1961) concluded that it as part of the Ladd Formation. In many respects it is similar to the Baker Conglomerate, except that it is typically massive with few plane partings. Many measured attitudes represent textural changes rather than planar partings. Accordingly, the geological structural pattern is not particularly well-defined. Thus, PSE (unpublished) took a conservative approach in it slope stability analyses. Additional geological information could lead to different structural interpretations, as noted above.

Refractive seismic lines summarized in PSE (unpublished) suggest that at depth, this unit is well lithified. For reference, Section 8.1, herein, is an initial assessment of rippability of site bedrock south of Mabey Canyon.

5.1.2.3 Cretaceous Williams Formation (Kw):

Buff to brown, generally massive conglomerate and scattered claystone beds compose the local Williams Formation, as mapped by Gray (1961) and Morton and Miller (2006). Abundant, at least partly "grussified" granitoid cobbles and boulders and, at least local absence of stratification characterize this formation. According to Schoellhamer et al. (1981), the granitoid clasts differentiate this

formation from the Ladd Formation that supports mainly volcanic and metamorphic cobbles. This formation underlies the northern part of the study property and possibly crops out along the ridge northwest of Kroonen Canyon.

5.1.2.4 Paleocene Silverado Formation (Tsi):

Non-marine and locally marine light buff, to light gray, to yellow silty sandstone, intercalated red and gray/white claystone and scattered cobble conglomerate characterize the Silverado Formation. PSE (2005) identified well-cemented fossiliferous beds scattered throughout the on-site Silverado Formation.

Bedding varies from thin to massive, but is usually identifiable in natural and anthropic outcrops. The sandstone is moderately resistant, the limy fossiliferous beds are very resistant and the claystones are poorly resistant. Consequently, differential erosion of the red clay beds imparts steep, linear strike-parallel ridges and ravines. Hard limy beds form particularly evident spines along some ridgelines.

Red, white and gray mottled clay beds locally mark the base of the Silverado Formation. A few to a few tens of feet thick, these are likely sedimentary clays derived from nearby clay-rich saprolites that formed on an erosion surface prior to deposition of the remainder of the Silverado Formation (Gray, 1961; Schoellhamer et al., 1981; McCulloh et al., 2000).

These clays were probably therefore transported short distances into topographic lows on the Ladd Formation surface. These beds, which may be the lateral equivalent of the Claymont clay bed, were mined for clay at the McKnight Clay Mine and at the Kroonen Clay Prospect (Gray, 1961). These beds often are composed of commercial quality clay, but lack the volume to be commercially viable.

The contact between the basal red bed sequence and the underlying Ladd Formation is depositional, based on exposures in trenches excavated by PSE in 2005. Here, the contact is undulatory and rip-up clasts of the underlying Holz Shale are intercalated.

One particular limy, fossiliferous bed is depicted on the geologic map Figures 6 and 7, along with two smaller, discontinuous ones. Composed of *Ostrea, sp.* and other mollusks, this bed is traceable from the northeast property line westward to

near the south-center where it is tightly folded into a synform, as attested to by V-shaped outcrop pattern.

In sum, the red and limy beds are readily identifiable and often laterally continuous and are thus, excellent local and regional key beds to determine the presence or absence of bedrock faults.

5.2 QUATERNARY ALLUVIUM:

Based on studies by PSE, regional and local Quaternary alluvium consists of: 1) Older (Pleistocene) alluvial fan and "terrace" deposits elevated above the modern (Holocene) drainage; and 2) Holocene alluvium occupying the floors of active channels and swales, where detritus is seasonally transported, deposited and eroded. PSE (2004, 2005, 2007) provided detailed discussions of the relative ages of the various alluvial units. For this EIR document this firm provides the following summary/synthesis of the earlier PSE assessments.

5.2.1 Ages of Alluvial Deposits:

Lamar (1986), Millman (1986, 1988) set forth soil and morpho-stratigraphic chronosequences for northern Temescal Canyon alluvial deposits based on relative soil development, geomorphic position, radiocarbon dates and on comparisons with other numerically dated southern California soils (at Ventura and Norco). Vaughan and et al. (1999) developed a similar soil stratigraphy along the EF south of the study site, near Temecula and Pala, California. In brief, nine *geomorphic surfaces* were identified and the underlying deposits were inferred to be up to perhaps 200,000 to perhaps 300,000 years old. The age of each surface is deduced from relative soil development that provides a minimum age for the deposit on which it forms. The actual deposits are therefore, older. In general, the ages of the fan/stream/valley fill surfaces (and thus, usually the underlying deposits) increase with elevation; that is, the modern lowlands are the youngest and the highest elevated, the oldest.

As noted by PSE (2005), both soil-stratigraphic and morpho-stratigraphic hierarchies are thus useful to assess the age of the Quaternary deposits in the Corona area. PSE (2002a, 2002b) developed a modified version of the Millman and Rockwell (1986) and Millman (1988) chronology for an investigation similar to this.

Figures 1 through 7 reflect the adoption of the Millman (1988) nomenclature. Relative ages of the deposits are depicted where feasible and where reasonably inferred. Their boundaries are not "precise" and hence, for this study, were generalized.

5.2.1.1 Older Alluvium, Pleistocene Age (Qoa)

Two principal areas of older alluvium that strongly impact dating of last slip along the TMF and MSF occur in the northern and central parts of Skyline Heights. A greater than 18-ft. thick cap of older alluvium sits as an elevated "terrace" along the south side of Mabey Canyon. PSE informally named this deposit the "Mabey Fanglomerate" (PSE 2005). PSE (2005) trenches also exposed more than 20 feet of older alluvium (Kroonen Deposit) near the head of a major tributary to Kroonen Canyon, where PSE (unpublished) Boring BH-6 penetrated 35 feet of older alluvium before encountering bedrock.

5.2.1.1.1 Mabey Fanglomerate

The Mabey Fanglomerate is more than 18 feet thick where it underlies an extensive Pleistocene terrace above Mabey Canyon (Figure 2). These deposits are typically poorly sorted, arkosic to lithic, subangular to well-rounded sand and gravels, replete with ubiquitous cut-and-fill structures. Also common, are well-sorted, current-laminated sands and moderately sorted basal cobble and boulder beds.

Regionally, this deposit occurs at elevations similar to other high elevation fanglomerates along the east flank of the Santa Ana Mountains adjacent to or near the WEFZ; namely, at least several tens of feet above active Temescal Canyon drainages and alluvial fans. Uplift along the Eagle Fault and associated "beheading" of the proximal parts of the fans by erosion has now both cut off these fans from their original source areas and elevated them above Temescal Canyon.

5.2.1.1.2 Kroonen Canyon Older Alluvium:

Another deposit of older alluvium occurs near the head of Kroonen Canyon (Figure 4) near PSE's 2005 Trench T-11. This deposit overlies the Ladd Formation in depositional contact. The aforementioned BA-6 revealed depth to bedrock to be approximately 35 feet at this location.

The Kroonen deposits are mainly sand, silty sand and clayey sand. Abundant gravel typical of most regionally extensive alluvial fan deposits is absent, although occasional gravel stringers and "stone lines" are scattered throughout the section.

5.2.1.2 *Younger Holocene Alluvium (Qal):*

Unconsolidated sediments occupying active channels in the major canyons and low order swales are shown as Holocene alluvium (Q₁₋₂ on the various Figures). These deposits vary in thickness from a few to more than 20 feet. These deposits are generally of low density and compressible.

5.2.1.3 *Surficial Cover:*

Although unmapped, prisms of compressible soils likely lie at the base of onsite slopes, or occupy swales on those slopes. These accumulations stem from weathering, dislodgement and downslope transport (by either water or creep) of debris derived from local bedrock. Although colluvium was encountered in a few of the exploratory trenches (PSE, unpublished), the steep nature of most of the slopes and heavy vegetative growth prevented exploration of these soils; therefore, a detailed distribution and depths of colluvium must await further subsurface exploration or be determined at the time of grading. Colluvium is estimated to range from approximately 5 to 15 feet in depth, as estimated by PSE (unpublished).

5.2.1.4 *Landslides:*

During the current study, PSE mapped several relatively small, surficial onsite landslides or suspected landslides. Most are concentrated in the northern part of the site on steep slopes comprised of Ladd Formation.

A moderately large landslide is mapped easterly subjacent to Lots 58 through 64. Most of the mapped limits of the feature are off-site. Nonetheless, as shown on the Tentative Plan, proposed development will be affected by the slide. For instance, removal of slide debris in the process of providing adequate support of the planned fill in this area could prove difficult near a property line that is within the slide limits. As noted, the offsite part of the landslide might need to be investigated in light of the current development plans.

Per the unpublished PSE geotechnical review of the study site, that firm reported that their BA-15 encountered 12.5 feet of landslide debris and trenches TP-22 through TP25 and TP33 aided identification of the southwest landslide limit. As a caveat, these logs are not currently available for G&A review. The offsite limits are thus estimated and therefore, questioned on the geotechnical map. Based on the PSE field investigation, the entire slide, if as shown mostly offsite, is seemingly a composite of relatively shallow failures associated with fractured and clay-rich Silverado Formation. Indeed, the currently mapped landslides are seemingly related to the presence of steep natural slopes, colluviated swales and fractured or clay-rich rock. Seepage (i.e. springs) may also contribute to local zones of instability. The generally lithified and steeply dipping bedrock is not usually amenable to deep-seated landslides.

5.2.1.5 Artificial Fill (afu):

Non-engineered fill exists at the Kroonen Clay Prospect near the center of the project. It is anticipated that artificial fill in this region locally reaches a depth ranging from 5 to 20 feet. Also, PSE (2005) fault trenches that for the most part are in areas of planned cut were backfilled with non-engineered fill.

Of importance, engineered fill has been placed near along the eastern margin of Skyline Heights development must therefore, be designed and constructed in a manner that affords adequate and code-compliant subsurface and surface drainage from the study site.

6.0 Groundwater:

Perched groundwater was encountered in one of the PSE borings at 39 feet below ground surface. No groundwater was encountered during PSE's fault investigations. Groundwater could potentially be trapped seasonally along alluvium/bedrock contacts and bedrock fractures. Weber (1977) estimated the regional groundwater surface to be 100 ft. to 200 ft. below the existing ground surface.

6.1 GEOLOGIC STRUCTURE:

6.1.1 Regional:

The Whittier-Elsinore Fault Zone (WEFZ) has been mapped by many investigators (Gray, 1961; Gray et al., 2002; Weber, 1977). Recent geological studies show the fault zone as active (Rockwell et al., 1986; Millman, 1988; Lamar, 1992). Recently, PSE

(2002, 2003) has identified active traces of the fault zone along the west margin of Temescal Canyon southeast of Skyline Heights.

The Skyline Heights site is northwest of where the CF and WF *sensu stricto* branch from the classic EF (Plate I). The CF trends north-northwest into Temescal Canyon and then along the east flank of the Puente Hills. The WF trends northwest from the “split”. Near Fresno Canyon, it “turns” west-northwest across Santa Ana Canyon and then resumes a northwest trend along the west flank of the Puente Hills.

Near Skyline Heights, the WF has been presumed to have two branches, the EaF and TMF (Plates I and IV). Weber (1977) and Gray et al. (2003) mapped the TMF from about Santa Ana and Wardlow Canyons in the northwest across Skyline Heights to Hagador Canyon in the southeast where it either dies out or merges with the EaF. Conversely, the EaF, off-site to the south is typically mapped as the southernmost trace of the WF *sensu stricto* (Dibblee, 2003).

The MSF named and mapped by Weber (1977) is the northernmost active segment of the EF. However, the fault apparently terminates near Mabey Canyon (Weber, 1977; CGS, 2003) as reflected by a lack of geomorphic expression and by the presence of unbroken pre-Holocene sediments. Here slip is inferentially transferred to the CF, WF or regional warping.

The subject region has been subject to transpression since at least Pliocene time (last 5,000,000 years) and perhaps earlier (Wright, 1991; Lamar, 1992; McCulloh et al., 2000). Geologic structure between the EaF and MSF (Figure 4) reflects that transpression. Specifically, the Cretaceous and Paleogene bedrock dips steeply and in many places is overturned (Tan, 1991; Dibblee, 2003). For example, this study identified a previously unmapped “Kroonen syncline” between the EaF and MSF (Figures 5 and 6) that apparently reflects northeast-vergent compression along the MSF, TMF or EaF. Further, a PSE investigation about two to three miles northwest, exposed a northeast-vergent thrust separating overlying Cretaceous rocks on the southwest from Pleistocene older alluvium on the northeast, a clear indicator of local compression where the WF deflects the west-northwest near Santa Ana Narrows.

Because Skyline Heights is between major faults, the local structural patterns are inherently complex. Accordingly, the simple, northeast dipping homocline shown by Dibblee (2003) is initially deceiving, particularly near the site where several folds and dip reversals occur (Tan, 1991; Weber, 1977; PSE, 2005). Also present are other faults both sub-parallel to and antithetic to the EaF, TMF, MSF and CF. In light of the regional and

local structural complexity, and because the MSF and TMF are in A-P Zones, PSE set forth the following interpretation of local geologic structure as it pertains to the location, character and activity levels of on-site faults.

6.1.2 Local Structure:

Rockwell et al. (1986) suggested that 5 to 7 mm/yr. of right slip takes place along the EF segment of the WEFZ. About 2 to 3 mm/yr. slip is transferred to the WF, or is manifested as regional up-warping in the eastern Los Angeles Basin, (Santa Ana Mountains and Puente Hills), with the remainder portioned to the CF (Rockwell et al., 1982). The EF to WF slip is left stepping. According to Weber, 1977; Dolan et al., undated; Gath, 1997; Rockwell et al., 1992; the WF slip component is reflected by displacements of postulated 140,000 year-old fluvial terraces in Santa Ana Canyon northwest and at the Brea Oil Field to the north.

The TMF and EaF segments of the WF are traditionally judged to transfer the slip from the EF to the WF. As, Dibblee, 2003; Tan, 1991; Morton, 1994; Weber, 1977; depict the EaF as the southern segment of the WF. The TMF is typically mapped as an “intermediate” strand, between the northern terminus of the MSF and the southern end of the EaF.

Previous regional geological mapping (Gray, 1961; Gray et al., 2003; Weber, 1977; Tan, 1991) shows different interpretations for fault locations and patterns.

6.1.2.1 Kroonen Syncline:

This previously unrecognized regional syncline trends northwest across the central part of Skyline Heights (Plate IV; Figure 6). The fold is locally overturned to the northeast and plunges to the southeast. It is readily identified by many measurements of attitude (strike and dip), by mapped outcrop patterns, by a particularly evident fossiliferous bed in the Silverado Formation and by the well-defined Holz Shale – Silverado contact (Figure 6). Additionally, PSE (2005) Trenches T3 and T-11 (Plates 2, 5 and 12 through 12b) exposed the hinge line or “core” of the syncline.

The fossiliferous bed(s) marks continuous north and south limbs both on and near the site. The continuous fossiliferous beds are demonstrably unbroken, thus

documenting the absence of northwest-trending faults east of PSE (2005) Trench T-2.

The geometry and stratigraphic position of the Kroonen syncline suggest that considerable compression took place between the EaF and MSF rather than the TMF and the MSF. Much of the south limb of the Kroonen syncline is south of the TMF and adjacent to the EaF, suggesting that, at most, the TMF played only a minor role in syncline formation. The Kroonen syncline is likely an old structure, later exhumed by uplift along the MSF or related faults, or by regional warping. Weber, 1997; Wright, 2001; Lamar, 1992; and McCulloh and other, 2000; demonstrate or posit post-Paleogene and pre-Quaternary deformation, including transpression, in the area now occupied by the Santa Ana Mountains and Puente Hills. Pleistocene-aged older alluvium near the hingeline of the Kroonen syncline (Kroonen deposit) is not folded (PSE T-5 and T-11), and the Mabey Fanglomerate does not exhibit evidence of compressional folding. Therefore, intense folding occurred prior to their deposition.

The aforementioned “anticline” north of Kroonen Canyon is not mapped, but postulated, for massive conglomerate in the Ladd Formation north of Kroonen Canyon yielded few bedding attitudes useful for delineating geological structures. Alternatively, the well-indurated conglomerate north of the canyon may be less amenable to folding than the Holz Shale and Baker Canyon.

Hence, the small faults exposed in Trench T-6 (2005), for example, may attest to brittle deformation rather than the more plastic deformation exhibited by the Holz Shale, Baker Canyon Conglomerate and the Silverado Formation.

6.1.2.2 Holz Shale Geometry

The PSE (2005) geologic map (Figures 1 through 7) differs in the distribution of the Holz Shale and Silverado Formation from earlier maps that depicted a fault bounded “slice” of Silverado Formation “wedged” between the conglomerates and sandstone of the Ladd Formation on the ridge where PSE (2005) Trenches T-3, T-4 and T-11 were placed (Figures 4 and 6)). Likewise, previous investigators interpreted the Holz Shale as locally missing, thereby requiring large horizontal and/or vertical displacement along faults such as the TMF and the “Whittier Fault” as portrayed by Gray et al. (2002). The recent PSE trenching, however based on abundant fossil (Photograph 1) and lithologic evidence, exposed Holz Shale and Baker Canyon Conglomerate underlying most of the ridge; as well as

the hingeline of the Kroonen syncline (T-11, Station 11+70). Thus presence of the Cretaceous sedimentary rock and Kroonen syncline, and the absence of the Silverado Formation reduce the need for significant slip along the TMF or related faults. Rather, the structural relationships are better explained as normal stratigraphic sequences, as observed in the field. Likewise, similar “basal” clays in the Silverado Formation at clearly depositional contact (McKnight Clay Mine, T-3, T-11) are also present where the TMF is mapped as separating the Silverado Formation from the Holz Shale (T-1, M-11). This relationship also strongly suggests minimal slip, for the basal Silverado is not “faulted out”.

A Kroonen syncline and a TMF with small vertical displacement are consistent with mapped geology (Figures 4 and 5). Likewise, an anticline at Kroonen Canyon is also plausible. In sum, the Silverado Formation does not daylight on the north slope of Kroonen Canyon; rather, it crops out north of the canyon, near the MSF.

Additionally, differential erosion of a poorly resistant band of Holz Shale between Cretaceous and Paleocene sand stone and conglomerate apparently gives rise to the strong lineament heretofore usually attributed to Holocene movement on the TMF near and northwest of Kroonen Canyon.

6.1.2.3 Local Faults

Main Street Fault (MSF), as named and mapped by Weber (1977) generally parallels the northeast site boundary (Plate I). The MSF characteristically separates Paleocene sedimentary rocks on the southwest from Pleistocene alluvium on the northeast. Owing to youthful slip along the fault, overlying Holocene deposits are likewise displaced (Weber, 1977; GeoSoils, 2001; Shlemon, 2003, personal communication; Treiman, 2004, personal communication; hence the CGS (2003) placed the MSF in an A-P Zone (Plate I).

The MSF is typified by right-lateral strike slip and “south-side-up” vertical separation (Weber, 1977; Lamar, 1992). PSE (2004) notes stratigraphic and geomorphologic evidence for MSF Holocene slip such as right-lateral deflected drainages, including Bedford and Main Street Canyons. Additionally, northeast-facing spurs (scarps) also mark the fault trace. Two reasonably distinct scarps are particularly evident east of Skyline Heights near Mangular Avenue (Plate III). Pleistocene alluvial fans that emanate from the high order canyons along the mountain front are beheaded by right lateral slip along the MSF, are now cut-off

from their source canyons Plate III) and are elevated above Temescal Canyon floor.

The MSF terminates at or near Mabey Canyon (Weber, 1977) in the northeast corner of Skyline Heights, where both stratigraphic and geomorphic evidence for the fault are “lost” (Plate I; Lineament F, Plate III).

The CGS (2003) maps the “main” branch of the MSF as trending into and terminating near the mesa in the northwest corner of Skyline Heights (similar to Plate I) with the corresponding A-P enclosing the entire mesa. Accordingly, PSE (2005) placed their Trench T-7 on the mesa where it exposed 18 ft. of upper to middle Pleistocene alluvium informally deemed the “Mabey Fanglomerate”. The fanglomerate, as logged by PSE (2005), is stratified and capped by a pre-Holocene, unbroken relict paleosol.

The southern contact between the Mabey Fanglomerate and the Ladd Formation is a buttress unconformity of the 2005 PSE report as indicated by the “stepped” contact, lack of shears and presence of Ladd Formation rip-up clasts in the fanglomerate. Therefore, based on the PSE investigation, the “A-P Zoned” MSF is not active at T-7 or passes off-site to the north.

Additionally, PSE’s (2005) Trench T-12 (Figure 2) intercepted a fault juxtaposing Ladd and Silverado Formations south of T-7 (PSE Plate 1; Plate 17; Station 4+90). According to PSE (2007). The T-12 fault is traceable across Mabey Canyon based on similar geological characteristics. North of Mabey Canyon, the fault is overlain by un-faulted pre-Holocene deposits and PSE (2007) judged the T-12 fault to be not active according to A-P standards.

Tin Mine Fault (TMF) - Weber, 1977; Gray, 1961; and Gray et al., 2002; mapped the TMF and related faults from about Santa Ana and Wardlow Canyons in the northwest, across Skyline Heights, to Hagador Canyon in the southeast where they die out or “merge” with the EaF (Plate I). The TMF is traditionally identified by the inferred juxtaposition of Mesozoic sedimentary rock on the southwest against Paleocene Silverado Formation on the northeast; and as a bold aerial photographic lineament (Plate III). High-order regional drainages, such as Wardlow, Mabey and Kroonen Canyons seemingly have strong right-lateral deflections usually attributed to slip along the fault. Therefore, the CGS (2003) placed elements of the TMF in an A-P Zone (Plate III).

In order to locate and date last displacement of the TMF, PSE (2005) placed Trenches T-1, T-2, T-3, T-4, T-5, much of T-6, T-8, T-9, T-10, T-11 and T-13, as well as other “checkpoint” backhoe pits, across the mapped or projected trace(s) of the fault. The exploration showed the aerial extent of the Silverado Formation at Kroonen Canyon is less than previously recognized; that a major (Kroonen) syncline sub-parallel the mapped trace of the TMF and that a bed of previously unrecognized Holz Shale extends across the southern part of Skyline Heights, forming the bold northwest-trending lineament usually attributed to the TMF. Importantly, the TMF proved to be less well-defined than previously thought and at Skyline Heights it does not displace upper Pleistocene deposits.

PSE’s 2005 Trench T-11 exposed a thick section of Baker Canyon Conglomerate and Holz Shale in its westernmost 1130 ft. (PSE Plates 12A and 12B of 2005)). The contact between the Holz Shale and the Silverado Formation (Station 11+30) is depositional rather than tectonic. Trench T-3 exposed a similar contact (Figure 6; PSE Plate 4 of 2005).

The TMF is identified as a relatively narrow fault dipping steeply to the southwest near Station 7+50 in PSE (2005) T-11 (Plate 12A & Photograph 6). Here, Holz Shale, siltstone and sandstone are probably juxtaposed with the sheared surfaces filled with carbonate. The fault zone, however, has no surface expression, attesting to the likely antiquity.

PSE’s Trench T-10 was placed to constrain the location of the TMF (Figure 6; Plate 2 of PSE 2005). Here too, a thick easily recognizable fault zone is absent. Near Station 2+70, however, a sheared zone may be the TMF. Additionally, an overturned local anticline and syncline are identifiable in T-2 and T-11.

PSE (2005) Trenches T-5, T-8, T-9 and T-13 overlap and intersect any reasonable projection of the TMF (Figure 2; Plate 1 of 2005) These trends expose stratified pre-Holocene and unbroken Kroonen deposits (2005 Appendix B), providing positive evidence of the lack of Holocene or latest Pleistocene surface rupture on the TMF.

Eagle Fault (EaF) – Weber (1977) identified the northeast-vergent Eagle Fault (EaF) that characteristically thrusts Jurassic and Cretaceous rocks over Cretaceous and Tertiary sediments along the northeast flank of the Santa Ana Mountains (Plate IV). Gray, 1961 and Gray, et al., 2002 mapped a similar fault south of C-4, south of Tin Mine Canyon. Weber (1977) suggested that most fault movement occurred in Pliocene time, the earliest displacements along the WEFZ. CGS

(2003) did not place the EaF in an A-P Zone. However, local and regional stratigraphic and structural relationships suggest that the EaF has contributed to slip along the WEFZ. For example, basement rock elevation in the Santa Ana Mountains (southwest block up) points to major structural relief. Also, compression in a block between the EaF (rather than the TMF) and the MSF probably gave rise to the Kroonen syncline. And finally, many investigators (Dibblee, 2003; Tan, 1991; Shoellhamer et al., 1981) “connected” the EaF to the south end of the clearly active WF.

Other Faults – Several investigators (Gray, 1961; Weber, 1977; Gray et al., 2002) have either inferred or mapped several other faults on, or near, Skyline Heights. Based on the present site-specific investigation, these faults either do not exist or are pre-Holocene in age. *Evidence points to bedding parallel differential erosion of sandstone and claystone beds in the Silverado Formation as the genesis of lineaments in the Silverado Formation (PSE, 2004, 2005, 2007).*

PSE (2005) demonstrated that the “Whittier Fault” of Gray et al. (2002) does not exist on or near the site. Specifically, the absence of Silverado Formation north of Kroonen Canyon precludes the presence of such a fault. Further, PSE Trenches T-6, T-8 and T-12 that would have intercepted the “Whittier Fault”, exposed no significant faults. The Ladd-Silverado contact and key fossiliferous and mudstone beds within the Silverado Formation are unbroken across the mapped trace of the “Whittier Fault”. Also, at the McKnight Clay Mine, as exposed in a road cut, the Silverado is in depositional contact with the Ladd Formation.

The 2005 and 2007 PSE trenches intersected many small faults and shears. These however, are not the “so-called” “Whittier Fault”, but rather occur near the axial (hinge) trace of the Kroonen syncline. As a class, these faults have displacements less than about one to two feet, are carbonate-filled and healed, and in many places capped by unbroken late Pleistocene soils or weathered surfaces.

Similar small faults and shears, as exposed in the trenches, occur in the hard, brittle conglomerate of the Ladd Formation north of Kroonen Canyon (Figure 5; Plate I of 2005). PSE (2005) T-12 exposes these faults that have small displacements, no obvious lateral continuity, are healed and do not displace late Pleistocene weathered surfaced. Further, they have no surface expression and many “die-out” upward and downward within the bedrock.

6.1.2.4 Age of the Main Street (MSF) and Tin Mine Faults (TMF)

The MSF and faults southeast of Skyline Heights that compose the classic EF segment of the WEFZ are well-documented Holocene (active) structures. For example, at Glen Ivy March, 10 to the southeast, Rockwell et al. (1986) documented five episodes of fault slip in the last 1,000 years, including a large (30-50 cm of vertical separation), post-1660 AD event. Similarly, Vaughan et al. (1999) reported at least four surface fault displacements in the last 3,500 to 4,500 years along the Temecula segment of the EF, about 45 miles southeast of Skyline Heights. Those investigators surmise that the last event was post 1660 AD, and perhaps was the same event as recorded at the Glen Ivy Marsh. PSE (2002, 2003) documented multiple latest Pleistocene and Holocene slip events along the EF at Indian and Horsethief Canyons southeast of Skyline Heights. At those sites, the active fault extended upwards from pre-Holocene sediments through Holocene sediments and finally through modern (Q₁₋₂) alluvium. Hence, one to two feet high scarps mark the active fault. Additionally, flowering upward zone, perhaps 25 to 37 feet wide in upper Pleistocene alluvium, narrowed to 1 to 5 feet wide “active zones”. These and similar studies (GeoSoils, 2001; Petra, 1999) southeast of Skyline Heights show that the active splays of the MSF are generally narrow zones that deform upper Holocene strata and even the ground surface. For example, linear scarps and faceted spurs south of Skyline Heights (Plate I) are aligned with and coincident with the mapped MSF. Local tectonic geomorphology thus well marks the MSF.

PSE notes that, in contrast, tectonic geomorphic features indicative of active faulting are absent near Mabey Canyon. This strongly suggests low to nil activity of the MSF in this area. The CGS (2003) thus terminated the MSF and PSE (2005) Trench T-7 clearly supports that conclusion, for it exposed un-faulted upper Pleistocene deposits that cross the mapped trace MSF of CGS (2003), Gray (1961) and Weber (1977). At Skyline Heights the “main” MSF is therefore pre-Holocene according to A-P definition.

The activity level of the TMF near Skyline Heights has been less well documented and thus traditionally more enigmatic than the MSF. PSE (2005) and RMA in a southeast adjacent parcel (Figure 2) demonstrated that the last local displacement of the TMF occurred in pre-Holocene time. These site-specific trench observations are supported by lack of geomorphic evidence for fault “recency”; for example, the ridge immediately east of Tin Mine Canyon is clearly not displaced (Plate III). The apparent linear ridges near the head of Kroonen Canyon result from differential erosion of the erodible Holz Shale that is bounded

by resistant sandstone and conglomerate. Also, the apparent right-lateral deflection of Kroonen Canyon near the mapped TMF appears to be more likely the confluence of three tributaries that follow shale/conglomerate contacts.

Further, the on-site trench exposures and absence of Kroonen and Tin Mine Canyon deflections strongly support the impression that the traditionally presumed large, right-lateral displacement of Mabey Canyon stems from erosion of the Holz Shale rather than recurrent, active faulting.

RMA (2005, personal communication) investigated the TMF in and along the south slope of Tin Mine Canyon (Plate II) and concluded that there is no evidence for a through-going active TMF in either Tin Mine Canyon alluvium or in the canyon wall. Although the RMA “alluvial: trenches did not expose a full Holocene section, <3,500 years bp radiocarbon dates and an underlying slightly developed, buried paleosol document no displacement in at least the past 5-6 ka; time sufficient to have recorded multiple recurrent events characteristic of the active WEFZ as depicted by Rockwell et al. (1986). RMA cuts along the south canyon wall exposed unbroken Ladd and Trabuco Formations and locally overlying Pleistocene terrace deposits, thus stratigraphically verifying either termination of the TMF or its lack of Holocene activity at and new Skyline Heights.

In sum, the Skyline Heights area, the on-site PSE (2004, 2005, 2007) and the near-site RMA investigations demonstrate that the TMF and much of the MSF are not active according to present State of California A-P definition.

6.1.2.5 Local Slip Partitioning

Decreasing slip along the north part of the MSF, including its cessation near Mabey Canyon and the lack of Holocene slip along the southern TMF, create an enigma regarding Holocene surface slip along the local segments of the WEFZ, for the zone is clearly active to the north and south. Skyline Heights is located at or near the zone where slip from the active EF that has included historical and latest Holocene surface rupture (Rockwell, 1995; PSE, 2002, 2003) is transferred to the northwest along the CF and WF. Numerous investigators, including the State of California (Peterson, et al., 1996) suggest that the long-term slip rate along the EF zone is about 5 mm/yr. south of partition to the WF and CF. Rockwell et al. (1992), The Working Group (1995) and Peterson et al. (1996) set forth a slip rate of about 2.5 mm/yr for the WF north of the Santa Ana River. The State of California (Peterson et al., 1996) suggests at least a 1.5 to 2 mm/yr slip

rate for the fault, with the remaining slip being transferred as regional-scale folding or warping in the east and north Los Angeles Basin. Transfer of slip to the WF from the EF traditionally has been judged to be via the TMF and the EaF. Based on the PSE investigations, however, it is not occurring along the TMF. An academic analysis of this issue is beyond the scope of the PSE investigations and this summary, but information gathered during by PSE and in referenced documents, suggests several explanations.

The presence of a currently unidentified and unstudied active step-over of the MSF east of Skyline Heights in Temescal Canyon cannot be discounted; or perhaps the fault exposed in PSE (2005) Trench T-12 transferred slip from the MSF to westerly faults such as the EaF. However, as deduced by PSE (2007) soil-stratigraphic evidence indicates that slip along that fault ceased before Holocene time. A series of west-northwest faults mapped by Weber (1977), Gray (1961) and Gray et al. (2002) between Mabey and Wardlow Canyons could act as the conduits of surface slip between the MSF and EaF.

Most investigators, including Tan (1991), Dibblee (2003) and McCulloh et al. (2000) identify a nearly west-trending "north branch" of the WF between the CF and the Santa Ana Narrows immediately south of the Santa Ana River. McCulloh et al. (2000) deduce that this branch is more recently active than the traditional "south branch" that ties into the EaF. Similarly, a series of so related west-northwest faults mapped by Weber (1977), Gray (1961), et al. (2002) north of Mabey and Wardlow Canyons could act as the conduits of youthful slip between the EF and WF. This would accommodate reversal of dip slip from northeast-side down south of the Narrows to northeast-side down northwest of the Narrows.

Alternatively, Gray, et al. (2002), Gath (1997, 2002) and Gath et al. (2002) suggest part of the strain deficit between the EF, CF and WF is consumed by up-warping of the Santa Ana Mountains, rather than by discrete fault slip. The phenomenon may be occurring at the EF-WF step-over area near Skyline Heights. It is also possible that northeast-vergent, possibly active thrust faults like that exposed near Fresno Canyon north of Skyline Heights are manifestations of that uplift.

The TMF in essence may therefore be a local fault related to compression between the EaF and MSF, as indicated by the Kroonen syncline, that ceased before the latest Pleistocene time. Possible stratigraphic mismatches along the mapped TMF, such as the differences in the Ladd Formation north and south of

Kroonen Canyon and truncation of the Holz Shale at Tin Mine Canyon, seemingly point to some slip along the fault. Conversely, stratigraphic facies changes and pinch-outs could account for same.

6.2 SUMMARY OF RESULTS OF A-P EFZ INVESTIGATIONS

- Skyline Heights is underlain by bedrock of the Cretaceous Ladd and Paleocene Silverado Formations. These are locally capped with Quaternary alluvium and landslides.
- The California Geological Survey (2003) placed A-P Zones encompassing both Tin Mine and Main Street Faults, both of which encroach onto Skyline Heights.
- Based on literature review, aerial photographic analyses, field mapping and logging of about 6000 lineal feet of fault trenches, PSE assessed that the on-site part of the Tin Mine Fault is *not active* according to A-P definitions.
- In their 2005 investigation, PSE at the time, concluded that the age of last movement along a fault exposed in their Trench T-12 (Figure 2) was unknown, owing to lack of dateable overlying sediments. Accordingly, PSE provisionally placed a habitable structures setback zone for the fault. Later, based on its 2007 investigation, PSE concluded that the fault was traceable across Mabey Canyon where it was overlain by un-displaced pre-Holocene deposits.
- PSE thus recommended removal of the provisional setback zone.
- The A-P trace of the MSF is overlain by un-faulted upper Pleistocene sediments and is thus *not active* at Skyline Heights, thereby precluding habitable structures setbacks.
- Bedding plane faulting associated with the Kroonen syncline is considered remote, because the near-parallel-limbed fold is not actively deforming, but rather is a relic of pre-Holocene deformation and because upper Pleistocene older alluvium is intact where it was observed overlying the bedrock in PSE trenches.
- Skyline Heights is in a region of active uplift and seismogenic faults capable of imparting strong ground motions at the site. Accordingly, to reduce the potential of resultant structural damage, PSE preliminarily recommends post-tensions slabs or some similar foundation reinforcements for design of proposed structures.
- All engineered structures must be designed in accordance with prevailing seismic codes. Likewise, this report is not intended to preclude site-specific geotechnical investigations.
- For future reference, the project geologist should observe and map the site during grading in order to document the absence of active faults; or if same are observed, make recommendations regarding appropriate mitigation.

7.0 Earthquake Hazards

The site is located in southern California, which is a tectonically active area. The type and magnitude of seismic hazards affecting a site are dependent on the distance to the causative fault, as well as the magnitude of the seismic event. The site is similar to most of southern California with respect to hazards associated with earthquakes. A detailed seismic hazard evaluation of the CBC Seismic Design Parameters and Peak Ground Acceleration Data determined utilizing USGS Earthquake Ground Motion Parameters Version 5.1.0 (Revised 2/10/2011) for the subject site is presented in Appendix III. The study reviewed the hazards associated with earthquakes that include primary hazards such as ground-shaking and surface rupture, as well as secondary seismic hazards such as liquefaction, seismic settlement and earthquake induced landsliding and are discussed in the following:

7.1 SEISMICITY

The potential for strong ground motion that affects future improvements is substantial due to the proximity of major active faults.

The horizontal ground motion representing 10% probability of exceedance in 50 years (475-year return period) as calculated from the USGS program (Appendix III) is 0.6436 Seismic Design Parameters are also included in Appendix III.

7.2 DYNAMIC SETTLEMENT

The following is a discussion of liquefaction and "dry sand" settlement, which can be associated with large seismic activity.

7.2.1 Liquefaction

Liquefaction is a phenomenon which tends to occur in saturated cohesionless soils during relatively severe earthquake ground motions. In general, during ground motion, saturated sands tend to compact and decrease in volume and if drainage is unable to occur, an increase in pore water pressure may result. If the pore water pressure becomes equivalent to the overburden pressure, the effective stress becomes zero and consequently, the soil loses its strength and is considered to be in a liquefied state. Post-liquefaction effects at a

site can manifest in several ways and may include ground deformations, loss of shear strength, lateral spread, dynamic settlement and flow failure.

Liquefaction potential is greatest in loose, poorly graded sands and silty sands with mean grain size in the range of 0.1 to 0.2 millimeters. Other factors that need to be considered are groundwater, confining stresses, relative density, intensity and duration of ground shaking. It is generally accepted that soils possessing clay content (particle size <0.005 mm.) greater than 15 to 20% may be considered non-liquefiable (Southern California Earthquake Center, 1999 and Blake, 1986).

The state of California has not studied the subject area relative to the potential for secondary seismic hazards such as liquefaction or earthquake-induced landsliding. Liquefaction requires relatively shallow groundwater and owing that groundwater is greater than several hundred feet in depth, the likelihood of liquefaction at the site is remote.

The Silverado and Ladd Formations, as well as un-weathered Pleistocene alluvium that underlie the site, are not considered susceptible to liquefaction. Holocene alluvium (Qal₁₋₂), landslide debris (Qls), topsoil and undocumented fills (Af) are potentially liquefiable, if saturated. Considering the recommended removal and replacement of those materials within the proposed development with drained, engineered fills, the susceptibility to liquefaction will be minimal upon completion of grading.

7.2.2 Dry Sand Settlement

Dry sand settlement is a result of earthquake-induced stresses creating strains within partially saturated soils. These strains cause a rearrangement of the soil structure, decreasing the void ratio of the soil, and ultimately creating settlement. The soils that typically are susceptible to dry sand settlement are loose, poorly graded sands and silty sands with mean grain size in the range of 0.1 to 0.2 mm. It is generally held that soils possessing clay content (particle size <0.005mm) greater than fifteen (15) to twenty (20) percent are typically not susceptible to dry sand settlement (Martin and Lew, 1999). Other factors that must be considered are confining stresses, relative density, intensity and duration of ground shaking, previous stress history and the age of the deposits. Based on a review of the data from PSE's subsurface investigation, the potential is considered

remote, provided the proposed design and remedial grading are accomplished as outlined herein.

7.2.3 Fault Rupture Potential

PSE conducted site-specific fault investigations of the Main Street, Tin Mine and seemingly associated faults (PSE 2004, 2005). Those investigations concluded that active faults as defined by Hart and Bryant (1997) do not exist in areas to be developed onsite. However, the age of last movement along a fault exposed in Trench T-12, in the northwest corner of the site is presently unknown owing to lack of dateable overlying sediments. Accordingly, PSE and this firm provisionally placed a habitable structures setback zone for the fault until such time when additional investigation may be undertaken. This is a conservative interpretation because the fault has no surface expression and its length is limited based on the un-faulted strata observed in the north-central part of the site south of Mabey Canyon. PSE (2005) also concluded that the A-P trace of the MSF is clearly overlain by un-faulted upper Pleistocene sediments near Mabey Canyon and is thus, not active at least in the northwest part of the site south of Mabey Canyon, thereby precluding the need for habitable structures setbacks there.

7.2.4 Earthquake Induced Landsliding

Owing to the steep natural slopes and, very locally, adverse bedding angles, the potential for earthquake-induced landsliding exists. Upon accomplishment of the remedial grading recommended in this report, seismically induced landsliding is not anticipated to negatively impact improvements on this project.

7.2.5 Seiches

A seiche is a free or standing wave oscillation on the surface of water in an enclosed or semi-enclosed basin. The wave can be initiated by an earthquake and can vary in height from several centimeters to a few meters. Because of the lack of elevated large basins in the project's vicinity, the potential for a seiche impacting the property is considered to be non-existent.

7.2.6 Other Potential Seismic Hazards

Other potential water-related hazards stemming from seismic activity include tsunamis and flooding resulting from dam failure. These are not considered likely, owing to the location of the site relative to potential sources.

7.3 MINERAL RESOURCES:

Previous prospecting of potential clay resources was carried out in the central portion of the site. The last date when prospecting or extraction took place on-site is not precisely known, but it was prior to 1950 (Gray, 1961), with most near the beginning of or before the 20th century. No active oil or gas wells exist and no mining of the gravels and/or sand has occurred to date. No other mineable resources are known to exist at the site. This firm understands that aggregate companies have explored on-site conglomerates, but found no evidence of mining development for sand or gravel.

7.4 OTHER POTENTIAL GEOLOGIC HAZARDS:

This portion of southern California is not susceptible to volcanic hazards such as lava flow. Land subsidence as a result of groundwater extraction would not affect this site, which is underlain by bedrock.

8.0 Material Properties:

8.1 RIPPABILITY:

The topsoil, undocumented artificial fill, landslide material, alluvium and older alluvium should be readily excavatable with conventional earth-moving.

The Silverado and Ladd formations underlying Skyline Heights are indurated, but have a seemingly upper weathered zone that yields rock amenable to dislodgement with a Caterpillar D-9 or equivalent bulldozer. However, the seismic refraction survey revealed that excavation difficulty is expected to increase with depth and very heavy ripping should be anticipated to attain design grades, especially in the northwesterly portion of the tract underlain by Ladd Foundation conglomerate (Klc). Sixteen (16) shallow seismic refraction lines were conducted for the PSE's investigation. Preliminarily, this data suggest that the rock is rippable to a depth of at

least 45 to 50 feet. Some of the borings encountered refusal or difficult coring. Cuts up to 130 feet, not including utility trenches, are proposed. Therefore, it is likely that excavating during grading will also encounter ledges and zones of extremely hard rock that may require blasting, particularly in the areas designated on Plate VI. Both the boring and seismic line information indicate that trenching with conventional trenching equipment will be extremely difficult in bedrock.

It should be anticipated that oversize rocks will be generated in the deeper cuts and locally in shallow cuts. Special disposal will likely be required. Typically, deep fills have been used to accommodate oversize rocks and boulders. Onsite rock crushing operations have been employed on similar developments producing large quantities of oversized rock.

8.2 HYDRO-CONSOLIDATION

The hydro-consolidation process is a response to the introduction of water into collapse prone alluvial/colluvial soils or landslide materials. Upon initial wetting, the soils structure and apparent strength are altered and a virtually immediate settlement response occurs. Given that all loose, dry, alluvial and landslide materials will be removed within the project limits and replaced as compacted fill as discussed in Section 9, it is our opinion that hydro-collapse will not significantly affect the subject site.

8.3 COMPRESSIBILITY

The onsite materials that are compressible include weathered surficial soils, existing non-engineered artificial fill, colluvium/alluvium, weathered landslide deposits and weathered bedrock. Compressible materials, will require removal from fill areas prior to placement of fill and where exposed at grade in cut areas. Recommended removal depths are presented in Section 9, and earthwork adjustment estimates are presented in Section 9, below.

8.4 EXPANSION POTENTIAL

Based on the laboratory tests performed during this investigation by PSE, it is anticipated that the majority of onsite soils would classify as "very low" to "medium" in expansion potential according to Table 18-I-B of the 1997 UBC. However, portions of the Silverado Formation could produce materials ranging from very high to critical in expansion potential.

8.5 SHEAR STRENGTH CHARACTERISTICS

The following table presents a summary of "averaged" shear strength parameters obtained from the data collected during the subsurface investigation by PSE.

TABLE 9.1		
<u><i>Material Description</i></u>	<u><i>Cohesion (PSF)</i></u>	<u><i>Friction Angle, ϕ (Degree)</i></u>
Compacted Fill	300	35
Bedrock-Silverado Formation (Tsi)	800	32
Bedrock-Ladd Formation (Kle, Klbe, Klh)	250	40
Along Bedding (Residual Strength)	100	20

8.6 EARTHWORK ADJUSTMENTS

The onsite materials are anticipated to shrink and bulk to varying degrees when removed and re-compacted as structural fill. The values presented in Table 9.2 are deemed appropriate for estimating purposes and may be used in an effort to balance earthwork quantities. These values may be used in conjunction with the enclosed geotechnical maps when evaluating the various geologic units. These shrinkage estimates should be applied to the cut volume, as well as the materials removed, as part of the remedial grading. In addition, subsidence of 0.1 feet should be used to estimate the consequences of processing removal bottoms. As is the case with every project, contingencies should be made to adjust the earthwork balance when grading is in progress and actual conditions are better defined.

TABLE 9.2		
<i>Geologic Unit</i>	<i>Approximate Range (90% fill zone)</i>	<i>Approximate Range (93% fill zone)</i>
Alluvium/Colluvium (Qal/Qcol)	12% shrinkage	
Silverado Formation (Tsi)	5 to 7% bulk	
Ladd Formation (Kle, Klbe, Klb) Rippable Blasting	12 to 15% bulk 18 to 20% bulk	10 to 12% bulk 15 to 18% bulk
Existing Fill (afu)	10% shrinkage	12% shrinkage
Landslide (Qls)	10% shrinkage	10% shrinkage

8.7 CHEMICAL ANALYSES

Soluble sulfate tests presented by PSE indicate that the soluble sulfate concentrations of the onsite materials are less than 0.1%. As such, soluble sulfate potential for the majority of onsite soils in this phase can be classified as "negligible" in accordance with Table 19-A-4 of the 1997 UBC. Resistivity and pH tests by PSE indicate that the onsite soils are "mildly corrosive" to "corrosive" to buried ferrous metals when classified by the criteria presented by the National Association of Corrosion Engineers (NACE, 1984).

9.0 Earthwork Conclusions and Recommendations

Based on the data compiled from recent exploratory studies and from the referenced reports with respect to the subject 100-scale Tentative Tract Map, it is our opinion that the subject site is suitable for the proposed development, provided the recommendations presented herein are incorporated in the design and construction of the proposed improvement.

9.1 SITE PREPARATION AND REMOVALS

All grading shall be accomplished under the observation and testing of the Project Geotechnical Engineer and Engineering Geologist in accordance with the recommendations contained herein,

the current codes practiced by the City of Corona and this firm's **Earthwork Specifications** (Appendix IV).

Loose, compressible residual soil, non-engineered onsite fill, colluvium, alluvium, landslide debris, older alluvium and weathered bedrock should be removed from fill areas prior to placement of fill and should be removed from shallow cut areas, where exposed at finish grades. Guidelines to determine the depth of removals are presented below; however, the exact extent of the removals must be determined in the field during grading, when observation and evaluation of the greater detail afforded by those exposures can be performed by the Geotechnical engineer and/or Engineering Geologist.

The minimum backcut ratio is 1:1 (horizontal to vertical). The bottoms of all removal areas should be observed, mapped and approved by the Engineering Geologist prior to fill placement. It is recommended the bottoms of removals be surveyed and documented by the Project Civil Engineer.

Groundwater, if encountered during grading, should be evaluated by the Geotechnical Engineer and/or Engineering Geologist. In general, groundwater is not anticipated to adversely affect grading although saturated soils and free water may be encountered along canyons and within alluvium removals. If groundwater is excessive, remedial measures such as horizontal drains or under drains may need to be installed. Further, settlement monuments and plates (see Plate VI) will need to be installed during and after completion of grading to monitor settlement monuments and plates will be used to determine when the majority of the primary settlement has been completed to allow for the release of affected lots for construction.

9.1.1 Stripping:

Vegetation, debris and other deleterious materials are unsuitable as structural fill material and should be disposed of offsite prior to commencing removals and placement of compacted fills. The thick, natural vegetative growth onsite is significant and will require consideration for removal. An onsite mulching operation may be considered. The mulched materials could then be used in future landscape areas.

9.1.2 Soil and Weathered Bedrock (unmapped):

Loose, compressible onsite soils and underlying weathered bedrock should be removed to expose the underlying competent bedrock materials prior to placement of compacted fill and when exposed in shallow cut area. An average removal depth of 5 feet is anticipated

for removal of topsoil and weathered bedrock. In general, onsite soils and weathered bedrock are suitable to be re-used as structural fill when properly moisture-conditioned.

9.1.3 Undocumented Artificial Fill (locally mapped as afu):

Fills associated with "recreational" grading, construction of unpaved access roads, mining operations and existing trench backfill should be removed prior to fill placement. Generally, these artificial fills range from a depth of 3 to 20 feet. Local areas of undocumented fill associated with previous geotechnical investigations are not plotted on the map. Other fills, including roadway fills and trench backfill, are not mapped due to their localized extent and minor thickness. Removals should extend below the undocumented fill until competent materials are encountered.

9.1.4 Alluvium/Colluvium (map symbol Qal)

All alluvium deposits exposed at existing or cut grades should be entirely removed prior to fill placement. These deposits range in thickness from approximately 5 feet to 35 feet. G & A recommends that all alluvial and colluvial soils below proposed fill areas and in cut areas, be removed to expose competent bedrock or terrace deposits. Anticipated average removal depths for alluvium/colluvium areas are expected to vary to an estimated maximum thickness of 35 feet.

Groundwater was not encountered during PSE's subsurface investigation within the alluvium onsite. However, access was limited and certain alluvial areas could not be investigated such as within Krooner Canyon. If "saturated" alluvium is encountered that possesses a degree of saturation of 85% or more and is consistent with depth to bedrock, consideration may be given to leaving the saturated alluvium in-place, provided that settlement time-delay consequences are acceptable to the owner. If saturated alluvium is encountered and is going to remain in-place, an additional liquefaction investigation may be required and dynamic settlement recommendations for those areas will likely change.

9.1.5 Older Alluvium (Qoa)

All older alluvium should be removed to expose the underlying competent bedrock. Generally, these deposits range in depth from 10 to about 20 feet.

9.1.6 Landslide Debris (Qls)

All landslide debris that could adversely affect the proposed development should be removed to expose the underlying competent bedrock. Generally, these deposits range in depth from 10 to about 29 feet, but may range up to around 35 feet in depth.

9.1.7 Removals Along Grading Limits

Removals of unsuitable soils will be required prior to fill placement along the grading limit. A 1:1 projection, from toe of slope or grading limit, outward to competent materials should be established, when possible. Where removals are not possible due to grading limits, property line or easement restrictions, removals should be initiated at the grading boundary (property line easement, grading limit or outside the improvement) at a 1:1 ratio inward to competent materials. Where this reduced removal criterion is implemented, special maintenance zones may be necessary. Affected areas will be established, minimally, as a 1:1 projection from suitable removal bottom to finish ground surface. This condition is mainly expected in the slope areas. Where the projection intersects the face of the slope, the potential for distortion of the slope in response to post-grading settlements exists and provisions for future maintenance of such slopes should be provided.

9.2 SLOPE STABILITY AND REMEDIATION:

9.2.1 Analytical Methods:

Slope stability analyses were performed for the proposed highest cut slope (Section 1-1'), highest fill slope (Section 2-2'), buttressed slope (Section 3-3'), highest ascending natural slope (Section 4-4'), highest descending natural slope (Section 5-5'). Temporary backcut conditions for buttressed slopes as shown on Section 3-3' was also analyzed. Shear strength parameters established and used in 2006 analyses by previous consultant Pacific Soils Engineering, Inc. were utilized in our analyses. The slopes were analyzed by calculating the factors of safety for circular-type failures using the Modified Bishop's Method and block-type failure surfaces using Modified Janbu Method. GSTABL7, a computer program developed by Mr. Garry H. Gregory was utilized to evaluate slope stability factors of safety under static and pseudo-static conditions. For pseudo-static analyses, a horizontal seismic coefficient of 0.15g was utilized for the critical failure surfaces on each cross-section analyzed.

The shear strength parameters used in the analyses and the results of the stability analyses are provided in Appendix II. The results of the stability analyses indicate that the proposed slopes will provide factors of safety greater than 1.5 and 1.1 under static and pseudo-static conditions, respectively, conforming to the City of Corona standards and are acceptable as planned. Similarly, the factors of safety for the temporary back-cut conditions also meet the required safety factor of 1.25 or greater.

Surficial stability analyses were also performed for the cut and fill slopes at 2H:1V gradient. The surficial stability analyses are based on the Infinite Slope Method considering submerged slope condition to a depth of 4 feet and the results are included in Appendix II. The factors of safety for surficial stability are greater than the required safety factor of 1.5 and the proposed slopes are considered surficially stable.

9.2.2 Cut Slopes

The majority of cut slopes have been designed at a slope ratio of 2:1. The following is an analysis of cut slopes within the various geologic units onsite. The Engineering Geologist should observe cut slopes and cut slope stabilization backcuts during grading. Modifications to the recommendations presented herein will likely be required based upon conditions exposed by grading.

9.2.2.1 Williams Formation (KW)

Cut slopes within the Williams Formation may expose adverse bedding that will require remediation. The cut slope superjacent to Lots 278 and 279 is anticipated to require replacement with a buttress fill. Calculations for this buttress are presented in Appendix II (cross section 3-3').

Probable buttresses are anticipated for the cut slope above Lots 261 through 267 and the cut slope below the WQMP Basin and Lots 247 through 255, as depicted on Figure 1 and Figure 2 (pocket).

9.2.2.2 Ladd Formation

Cut slopes within the Ladd Formation may expose adverse bedding that will require remediation. The majority of the cut slopes should expose bedding dipping into the slope or at an angle steeper than the proposed cut slopes (greater than 60 degrees). G & A has analyzed the highest 2:1 cut slope for a cross-bedding condition (approximately 160 feet) and the results are present in Appendix II. Cut slopes within the Ladd Formation that expose bedding dipping into the slope or at an angle steeper than the proposed cut slopes are expected to

be grossly stable; however, the final determination should be made based on exposed conditions during grading.

Cut slopes within the Ladd Formation may expose conglomerate resulting in poorly cemented sands, cobbles and boulders at finish grade, producing a rough uneven surface. These cut slopes should be evaluated in the field by the Project Engineering Geologist or Geotechnical Engineer. Replacement of those slopes with compacted stabilization fills may be required.

Future exploration and in-grading observation could reveal more favorable geology in these specific areas.

9.2.2.3 Alluvium/Colluvium:

Cut slopes exposing alluvial/colluvial soils (Qal) will require replacement with a drained stabilization fill. Typically, such corrective grading will be constructed with a minimum key width corresponding to one-half the full height of the superjacent slope, but not less than 15 feet. Minimum key depths at the toe will be 2 feet. All stabilization fills should be provided with a backdrain and outlet system as detailed in Appendix IV.

9.2.2.4 Silverado Formation:

Cut slopes within the Silverado Formation are expected to be grossly stable; however the final determination should be made based on exposed conditions during grading. Cut slopes within the Silverado Formation may expose poorly cemented sands, cobbles and boulders at finish grade, which would produce a rough uneven surface. These cut slopes should be evaluated in the field by the Project Engineering Geologist or Geotechnical Engineer. Replacement of those slopes with compacted stabilization fills may be required.

9.2.3 Fill Slopes:

The majority of fill slopes on the project are designed at a maximum slope ratio of 2:1. The highest fill slope is approximately 160 feet in height and located subjacent to Lot 68. G&A performed a slope stability analysis of this slope and the results are present in Appendix II. Based on the results of the calculations, the 2:1 fill slopes 160 feet and less in height, when properly constructed with onsite materials, are expected to be grossly stable, as designed.

Keys should be constructed at the toe of all fill slopes where unsuitable soil removals do not accomplish a minimum key width of one-half the slope height plus bench widths or twenty (20) feet, whichever is greater. Minimum key depths at the toe should be 2 feet. Due to grading limit restrictions, removals of unsuitable soils beyond the proposed toe may be limited; therefore, additional slope maintenance should be expected in the areas where a 1:1 projection into approved materials cannot be achieved.

9.2.4 Natural Slopes:

Ascending and descending natural slopes exist within the site that range in height up to approximately 200 feet with slope ratios from 2:1 to locally 1.2:1. Stability analyses of the highest ascending slope of 160 feet above Lot 4 indicates a factor-of-safety in excess of the code minimums (see Appendix II).

9.3 SUBSURFACE DRAINAGE:

Canyon subdrains are proposed as depicted on Figures 1 through 7 and will consist of 6" and 8" diameter pipes and will be constructed in accordance with the details shown in Appendix IV. Final determination of drain locations will be made in the field. Outletting of subdrain systems will require coordination with the Project Civil Engineer in determining suitable facilities to accept to drain water.

9.4 OVEREXCAVATION:

Where a transition of cut and fill occurs across a finished lot, it will be necessary to overexcavate the bedrock portions and replace it with compacted fill. The depth of bedrock overexcavation should be equal to 1/3 the maximum fill depth (after removals) on the fill side of the transition. Anticipated transition lots are shown on Figures 1-7.

We recommend the cut pads be overexcavated a minimum of 5 feet to provide a uniform foundation material. Some highly expansive soils of the Silverado Formation may be encountered at grade in some areas. Consideration should be given to increasing the overexcavation depth to mitigate for highly expansive soils, if warranted.

Difficult excavation with a backhoe can be expected in some street areas. We recommend street overexcavation and replacement with compacted fill to facilitate utility emplacement in these street areas and driveways. Depths of overexcavation should extend one (1) foot below the deepest utility line.

9.5 FILL MATERIALS:

a) The excavated on-site soils may be used as compacted fill. The fill materials to be used as compacted engineered fill should be free of organics, debris, deleterious materials, and rocks over 6 inches maximum dimension, and must be approved by the Project Geotechnical Engineer or Project Geologist prior to use.

b) Import soils, if required, should be evaluated by the Project Geotechnical Engineer for suitability prior to delivery. Import soils should be free of trash, debris or other objectionable materials. All import fill should have engineering characteristics similar to the on-site soils.

9.6 FILL PLACEMENT, MOISTURE CONDITIONING, AND COMPACTION:

After approval of the over-excavation and prior to placement of any compacted engineered fill materials, the exposed removal bottoms should be scarified to a minimum depth of six (6) inches. The area should then be moisture conditioned approximately 1 to 2 percentage points above the optimum moisture content, and compacted to a minimum of 90 (or 93) percent of the applicable maximum density.

The planned development will involve deep fills in the canyon areas and side-hill fill slopes. In order to reduce the amount of fill settlement, the fill should be compacted to the minimum relative compaction shown below:

Fill Depth (Feet)	Minimum Relative Compaction (ASTM: D1557) (%)
0 – 50	90
Below 50	93

Fill slopes should also be constructed at the minimum relative compactions shown above, with the exception of the outer 3 feet of the slope face which can be placed at 90% relative compaction. Deep fill areas requiring 93% relative compaction are delineated in Plate VI.

Generation of oversized rock (over 8 inches in maximum dimension) from excavations in the bedrocks should be anticipated. Oversize rock may be placed in single file windrows not exceeding three feet in height and six (6) feet in width with a minimum equipment width (15 feet) separating each windrow or a rock blanket fill not exceeding two (2) to three (3) feet in height. Large erratic rocks should be buried individually. Granular fill should be thoroughly flooded in the rock voids and covered with a minimum two feet compacted fill blanket. Successive windrows should be staggered and none should be placed within 10 feet of finished grade or closer than 15 feet to compacted fill slope surfaces. Typical rock placement construction detail is provided in Appendix IV. Placement of rock windrows must be performed under the observation and recommendations of the Project Geotechnical Engineer.

Fill materials should be spread in thin lifts, moisture conditioned to about 1 to 2 percentage points over optimum moisture content and compacted to a minimum of 90 (or 93) percent of the laboratory maximum dry density as determined in accordance with ASTM Test Method D-1557. Each lift should be treated in a like manner until the desired finished grades are achieved. The grading contractor should have suitable and sufficient compaction equipment in operation to achieve the required compaction. When necessary, earthmoving equipment may be utilized for compaction or temporarily halted in order to permit adequate compaction of fills.

9.7 SLOPE CONSTRUCTION:

1. Care should be taken during grading to prevent spillage of loose materials over slopes to remain natural. Should loose soil be spilled onto natural slopes by the grading contractor, removal of the loose fill will be required.

2. Fill should be properly benched into firm bedrock or compacted fill as directed by the Project Engineering Geologist and/or Geotechnical Engineer during grading. Typical benching should include 4-foot verticals exposing approved, competent material (see Appendix D – Typical Benching Detail).
3. In order to minimize surficial slumps on compacted fill slopes, the following grading procedures should be used:
 - a) Where possible, fill slopes should be constructed by overfilling a minimum of three (3) horizontal feet and then trimming back to expose the dense inner core of the slope surface. Compacted fill slopes should be back-rolled during construction at intervals not exceeding four (4) vertical feet. Care should be taken to construct the slope in a workmanlike manner so that it is positioned at its designed orientation and slope ratio.

Achieving a uniform slope surface by subsequent thin wedge filling must be avoided. Any add-on correction to a fill slope should be done by overfilling the affected area in horizontal, compacted lifts which must be benched into the existing fill prism. The overfilled slope may then be trimmed to the design gradient.

- b) Where fill slopes are planned above natural slopes and cannot be overfilled and trimmed back, the slopes should be rolled for the entire height of the slope with a sheepsfoot roller and then finished with a grid roller. If the desired compaction is not obtained in this manner, a vibratory sheepsfoot roller may be required. To be most effective, this equipment should be anchored and manipulated from a side-boom tractor. In lieu of a grid roller, the slope may be track rolled with a John Deere 450, Caterpillar D-8 dozer or equivalent. To obtain the required compaction and appearance of the slope face, the soil moisture should be maintained at above optimum from the time of mass filling to the completion of grid rolling.
- c) The grading contractor must take proper care to avoid spillage of loose material down the face of the slopes during grading and during drainage terrace and down-drain construction. Fine grading operations for benches and down-drains should not deposit loose trimmed soils on the finished slope surfaces. Loose soil materials will require removal.
- d) Seeding and planting of slopes should proceed as rapidly as possible to achieve a well-established and deep-rooted vegetal cover requiring minimal watering. The type of vegetation and watering schedule should be established by a landscape architect familiar with hillside maintenance.

9.8 EARTHWORK OBSERVATIONS AND TESTING:

- a) The site preparation, over-excavation, and earthwork should be performed under the observation and testing by a representative of the Project Geotechnical Engineer or Project Geologist.
- b) The fill should be tested at the time of placement to verify that the required compaction is achieved.
- c) The fill compaction should be determined in the field by the Sand Cone Method (ASTM: D 1556) or Nuclear Gauge Method (ASTM: D 2216), or other test method approved by the Project Geotechnical Engineer.
- d) During grading, an adequate number of field density tests should be performed using approved test procedures in order to determine compliance of earthwork to the project requirements. The frequency of field density testing should be in accordance with the recommendations of the Project Geotechnical Engineer.
- e) Quality control testing performed to determine the acceptability of the fill compaction should be based on the laboratory maximum dry density and optimum moisture content determined in accordance with ASTM: D 1557 test procedure.
- f) Any surface or subsurface obstructions, or questionable materials encountered during grading should be brought immediately to the attention of the Project Geotechnical Engineer. Deeper excavations may be required, should unsuitable soils be encountered locally, as determined by the Project Geotechnical Engineer and/or Project Geologist.

9.9 DEEP FILL SETTLEMENT MONITORING:

The planned development involves grade cuts in the higher elevations and fill placement in the lower portions of the site area. The maximum depth of proposed canyon fill is approximately 138 vertical feet. The areal loading from fill placement will cause settlement due to the compressibility of the fill materials. The magnitude of settlements would depend on the thickness of fill and the compressibility characteristics of fill materials. Considering the maximum fill depth and anticipated consolidation characteristics of the fill materials, the compacted fill may settle on the order of 6 to 8-inches. Due to the granular nature of the site soils, the majority of this settlement will occur during grading operations and long-term secondary settlement would be negligible.

In deep fill areas, settlement monuments should be installed and periodically surveyed by the Project Civil Engineer to evaluate the magnitude and progress of settlements. We propose positioning settlement monuments at strategic locations in the deeper fill areas as shown on Figure VI. These will be monitored on a regular basis until the primary settlement has stabilized before those areas released for construction

9.10 FOUNDATION DESIGN:

The compacted engineered fill at the project site is anticipated to be low to medium soil expansion potential. Presented below are preliminary geotechnical criteria for design of building foundations, based on medium soil expansion potential for the on-site soils:

Option 1 - Conventional Footing

Allowable Bearing Pressure (1)	=	1,500 psf
Minimum Footing Depth (2)	=	24 inches
Minimum Footing Width	=	Continuous: 12 inches Isolated: 24 inches
Passive Soil Pressure (3)	=	250 psf/ft.
Friction Coefficient	=	0.30 (ultimate)
Minimum Footing Reinforcement	=	Four No. 4 bars, 2 each at top and bottom
Garage Door Grade Beam	=	18 inches deep, Four No. 4 bars, Two each at top and bottom, tied into foundation elements.

- (1) The above value may be increased 250 lbs./sq.ft. for each additional foot exceeding the minimum embedment depth and/or width, subject to a maximum of 2,500 psf. Allowable bearing pressures may be increased by one-third for short term loading due to wind or seismic forces.
- (2) Footing depth is from lowest adjacent finished soil grade. Footings should be deepened, as necessary, to provide setback distance from adjacent slope in conformance with CBC criteria.
- (3) Value applies to level soil condition, and is subject to a maximum of 2000 psf.

Option 2 – Post-Tensioned Foundation

If post-tensioned slab/footing system is selected for the subject project, the design may be based on the geotechnical criteria shown below for medium soil expansion potential. These are based on the guidelines presented in the PTI, Third Edition design manual.

Thornthwaite Moisture Index = -20

Equilibrium Suction	=	3.6	
Edge Lift Moisture Variation Distance, e_m	=	5.1	Ft.
Edge Lift, y_m	=	1.1	In.
Center Lift Moisture Variation Distance, e_m	=	9.0	Ft.
Center Lift, y_m	=	0.47	In.
Minimum Footing Depth (Exterior)	=	18 Inches below adjacent grade	

The above foundation criteria are based on medium soil expansion consideration only. Foundation design should consider anticipated post-construction settlements, as appropriate.

Foundation design details such as slab thickness, concrete strength, reinforcements, etc. should be established by the Project Structural Engineer considering medium soil expansion and settlement potential provided below.

9.11 FOUNDATION SETTLEMENT:

Total static and differential post-construction settlements for footings designed and constructed in accordance with the criteria given below and supporting loads not exceeding the typical loadings for residential construction (column and wall loads on the order of 30 kips and 3 kips/lineal foot, respectively) are not anticipated to exceed the values shown below. The settlement estimates should be confirmed based on review of the foundation plans.

Structures should be designed for the following post-construction settlements:

- Total settlement = 1 inch
- Differential settlement = $\frac{3}{4}$ -inch (*)
- (*) Between similarly loaded column footings and for continuous footings and slabs over a distance of approximately 30 feet.

General Remarks

- (i) Footing depths should not be allowed to be affected adversely, such as through erosion softening, digging, landscaping, etc.
- (ii) Where foundations encroach closer than five (5) feet horizontally from the flow line of drainage swales, the footing should be deepened sufficiently to maintain the required embedment depth below the adjacent flow line.

- (iii) Foundation details such as concrete strength, reinforcements, etc. should be established by the Project Structural Engineer, considering the loading and medium soil expansion potential. The footing and slab reinforcements recommended in this report are minimum requirements. More restrictive criteria based on structural design considerations or Code requirements shall govern.
- (iv) Foundation excavations should be observed and approved by the Project Geotechnical Engineer prior to the placement of reinforcement or concrete. Forming of footing excavations may be required. Excavations should be free of slough and debris and thoroughly moisture conditioned prior to placing concrete.
- (v) Excavated material from footing and utility trenches should not be placed in slab-on-grade areas unless properly compacted and tested.

9.12 SLAB-ON-GRADE:

Recommended minimum criteria for slab-on-grade are shown below:

Concrete Floor Slabs

Concrete floor slabs should be 5 inches thick (minimum) and should be reinforced with No. 3 bars at 18 inches on center, each way at mid height. No. 3 bars at 18 inches on center should be provided connecting floor slabs to footings. In order to minimize migration of moisture up the concrete slab from soil subgrade and damage to floor coverings, a moisture barrier/water vapor retarder recommended beneath floor slabs as discussed hereinafter.

Moisture/Water Vapor Retarder for Concrete Slab-on-Grade

In order to reduce the potential for moisture/water vapor migration up through the slab and possibly affecting floor covering, a moisture/vapor retarder is recommended under concrete floor slab-on-grade. The moisture barrier should be properly installed, lapped and sealed in accordance with the manufacturer's specifications. Punctures and rips should be repaired prior to placement of sand.

As a minimum, this moisture/water vapor retarder should consist of 10-mil thick polyethylene ("Visqueen"), lapped and sealed, and placed mid-height within a 4-inch coarse sand layer. This moisture/water vapor retarder should be installed in accordance with manufacturer's specifications.

The following recommendations are based on the tentative guidelines by the American Concrete Institute (ACI, April 2001) to reduce the potential moisture/water vapor intrusion in concrete slab-on-grade. Based on our review of available literature, it appears that the ACI procedure would be more effective to help reduce potential moisture/water vapor migration up through concrete slab-on-grade.

Recommendations based on the ACI guidelines are presented below:

- The moisture/water vapor retarder should consist of high strength polyethylene membrane and should meet or exceed the ASTM: E-1745-97 Class C material requirements for water vapor permeance, tensile strength and puncture resistance. The vapor retarder should consist of “Moistop Plus” (Fortifiber Building Products Systems) or “Vapor Block” VB 15 (Americover, Inc.), or approved equal. The vapor retarder should be underlain by a capillary break comprised of minimum 4 inches thick pea gravel layer. The gravel layer should be placed and compacted on approved soil sub-grade.
- The membrane should be placed on approved gravel layer and properly lapped and sealed. Membranes intersecting utility pipes, sewer lines, ducts or drains must be properly wrapped around the penetrations and sealed. All punctures and rips in the membrane should be repaired prior to placement of concrete, following manufacturer’s recommendations. The vapor retarder should be installed in general accordance with the procedures outlined in ASTM: E-1643, and in conformance with the installation procedures recommended by the manufacturer.
- To minimize slab curling, a low slump concrete (low shrinkage mix design) should be used for the slab construction, as determined by the Project Structural Engineer.

The moisture/water vapor protection for concrete slab-on-grade should be selected based on cost and construction considerations, and considering potential future problems resulting from improper and uncontrolled landscape irrigation practices. Regardless of the moisture/water vapor retarder option selected, it should be emphasized that proper control of irrigation and landscape water adjacent to the structure is of paramount importance.

9.13 DRIVEWAY:

Driveway concrete slabs should be 5 inches thick (minimum) with No. 3 bars at 18 inches on center, each way at mid-height. The slab may be placed directly on properly prepared sub-grade. No moisture barrier is required under driveway slabs.

9.14 EXTERIOR FLATWORK:

Exterior concrete flatwork (e.g. sidewalks, walkways) should be 4 inches thick (minimum), with No. 3 bars at 24 inches on center, each way at mid-height and placed on properly prepared sub-

grade. Hardscape areas within two feet of the descending slopes should include a thickened edge deepened to provide a minimum five (5) feet horizontal setback between the bottom outside face of the thickened edge and slope face.

9.15 SUB-GRADE PRE-SATURATION:

Prior to concrete placement, the soil sub-grade should be thoroughly wetted to about 12 inches in depth at a moisture condition of about 2 to 3 percentage points above the optimum moisture content.

9.16 GENERAL:

Interior floor slabs and exterior concrete flatwork, including driveway, should be properly designed for the construction and service loading conditions, potential settlements and soil expansion. The structural details, such as slab thickness, concrete strength, reinforcing criteria, joint spacing, etc. should be established by the Project Structural Engineer. The recommended minimum reinforcements for concrete slabs provided above are intended for preliminary design only. More restrictive criteria as dictated by structural design or regulatory requirements shall govern.

9.17 RETAINING WALLS:

9.17.1 Foundations:

Retaining wall foundations may be supported on either compacted engineered fill or competent bedrock. If a bedrock/fill transition is encountered, the bedrock portion should be over-excavated minimum 2 feet and replaced with approved compacted fill. Alternatively, a cold joint should be constructed in both the footing and the wall.

Footings may be designed in accordance with the foundation design criteria presented in this report when embedded at least two feet below lowest adjacent grade. Footings located on or at top of slopes should be deepened, as necessary, to provide minimum lateral setback between footing and slope faces in accordance with CBC requirements.

9.17.2 Lateral Earth Pressures:

The earth pressures acting on retaining walls depend primarily on allowable wall movement, type of backfill materials, backfill slopes, wall inclination, surcharges, and any hydrostatic pressure. The following lateral earth pressures are recommended for vertical walls with no hydrostatic pressure and no surcharge loading:

Wall Condition	Backfill Type	Lateral Earth Pressure (Equivalent Fluid Pressure) (pcf)	
		Level	2:1 (H:V)
Active (Cantilever)	Sandy Soils	40	65

The surcharge effect of anticipated loads on the wall backfill (i.e. traffic, construction equipment, footings, etc.) should be included in the wall design. If the wall is free to deflect, 33 percent of the maximum surcharge load located within a distance equal to the height of wall should be used in the design.

9.17.3 Backfill

Retaining wall backfill should consist of predominantly granular non-expansive soils. The backfill should extend within a 45-degree plane from the wall footing.

Retaining wall backfill should be mechanically compacted to minimum 90 percent of the applicable laboratory maximum density and performed under the observation and testing of the Project Geotechnical Engineer. No jetting, ponding, or flooding should be permitted. No backfill should be placed against concrete until minimum design strengths are attained.

9.17.4 Backfill Drainage

The wall design should include waterproofing and weep holes or sub-drains or back-drains, as appropriate, for relieving possible hydrostatic pressures. If on-site soils are utilized for wall backfill, the design should include a prefabricated drainage blanket (such as Miradrain 2000, or equivalent).

As a minimum, sub-drains should consist of 4-inch diameter, perforated Schedule 40, PVC pipe or equivalent, embedded in approximately 3 cubic feet per lineal foot of ¾-inch (maximum) rock, or approved alternate. This permeable material should be enveloped in Geofabric consisting of Mirafi 140 or equivalent. The pipe and trench bottom should be sloped at a gradient of 2± percent to a suitable discharge outlet. Sub-drains placed behind retaining walls should be approved by the Project Geotechnical Engineer prior to backfill placement.

9.18 SOIL CORROSION:

The previous geotechnical consultants, Pacific Soils Engineering, Inc., performed preliminary testing and indicated that the site soils are corrosive to metals. We recommend that representative finished grade samples be tested for corrosion suites (Sulfate, Chloride, pH and resistivity) upon completion of grading. It may be necessary to retain a Corrosion Engineer for consultation.

9.19 UTILITY TRENCH BACKFILL:

Bedding material should consist of on-site sandy or imported materials exhibiting a San Equivalent (S.E.) value of 30 or greater.

The on-site soils are considered suitable for trench backfill, provided they are free of organic materials and oversize rocks.

Backfill of all exterior and interior trenches should be placed in thin lifts not exceeding 4 inches and mechanically compacted to achieve a relative compaction of not less than 90% based on ASTM: D1157. Care should be taken not to damage utility lines.

Utility trenches should not be located within the influence of footings. This is defined as a zone located below the footing and a line sloping at an inclination of 1:1 (horizontal to vertical) outward from the outside edge of footings. If utility lines are located within the zone of footings, the backfill should be compacted to a minimum 95 percent relative compaction or slurry backfilled (minimum 1-1/2 sack cemented sand mix).

To reduce potential water migration into building sub-grade through the granular bedding/shading layer and/trench backfill, utility trenches should be backfilled with the onsite finer grained materials or sand-cement slurry for minimum 3 feet length at their entry points.

Trenches in fill soils and alluvial deposits greater than 4 feet in depth should be shored or sloped back as required by the local regulatory agency, the state of California Division of Industrial Safety Construction Safety Orders, and Federal OSHA requirements.

9.20 ASPHALT CONCRETE PAVEMENT:

Preliminary asphalt concrete pavement sections for the planned streets in the development were developed utilizing the Cal Trans Method of Design and the Structural Section Design Guide for

California Cities and Counties. The traffic index for the proposed streets are not known at this time. Therefore, we are providing pavement sections for various traffic indices so that appropriate section can be chosen once the traffic index for the proposed streets become known.

The following recommended pavement sections were computed assuming an “R” value of 30 for site soils compacted as sub-grade material, and assumed traffic indices. The actual “R” value will depend on the soil conditions exposed at the planned street sub-grade elevations.

During rough grading, “R” value testing should be performed on the pavement sub-grade soils to confirm the pavement design basis. Further analysis and evaluation are necessary, if the design traffic index and the sub-grade “R” value are different from those used in our analyses.

Traffic Index	Recommended Minimum Pavement Section		
	Asphalt Concrete (inches)	Aggregate Base (inches)	Total (inches)
5.0	3.0	6.0	9.0
6.0	4.0	8.0	12.0
7.0	4.0	10.0	14.0
8.0	5.0	12.0	16.0

Note: AC = Asphalt Concrete, AB = Aggregate Base

Aggregate base should consist of crushed aggregate base (CAB) or crushed miscellaneous base (CMB) and should comply with the specifications outlined in the “Standard Specifications for Public Works Construction”, (“Green Book”). The base material should be compacted to minimum 95 percent of the maximum laboratory density, determined in accordance with ASTM: D-1557.

The soil sub-grade should be compacted to minimum 90 percent relative compaction. The subgrade soils should exhibit a firm and unyielding surface, in addition to the recommended minimum compaction. Final compaction and testing of pavement sub-grade should be performed just prior to placement of aggregate base.

9.21 SITE DRAINAGE:

All roof and surface drainage should be directed away from structures and their appurtenances and slopes to approved drainage facilities. Ponding of water should be avoided. For graded soil areas, a minimum gradient of 2 percent away from structures should be maintained.

The drainage patterns designed by Project Civil Engineer should be established at the time of fine grading and maintained throughout the life of the structure or, if altered, should be replaced with a properly designed area drain system.

Irrigation activities at the site should be monitored and controlled to prevent overwatering. Planter areas adjacent to structures should be avoided. If utilized, such planters should include measures to contain irrigation water and prevent moisture migration into walls and under foundations and slabs-on-grade.

9.22 SLOPE PLANTING, IRRIGATION AND MAINTENANCE:

General guidelines for slope planting, irrigation and maintenance are shown below:

- a) Slope planting should consist of appropriate drought resistant vegetation as recommended by the Landscape Architect. Landscaping of slopes should be completed as soon as possible and properly maintained.
- b) The property owner is responsible for proper irrigation and for maintenance and repair of installed irrigation systems. Leaks should be repaired immediately. Sprinklers should be adjusted to provide maximum coverage with a minimum of water usage and overlap. **Over-watering with consequent excessive runoff and ground saturation must be avoided.**
- c) If automatic sprinkler systems are installed, their use must be adjusted to account for natural rainfall conditions.
- d) All interceptor ditches, drainage terraces, down-drains, and any other drainage devices that have been installed must be maintained and cleaned.
- e) If rodent activity is present, the property owner must undertake a program for the elimination of burrowing animals. This must be an ongoing program in order to promote slope stability.

- f) Water must not be allowed to flow over the constructed or natural slopes. This may require the construction of berms or ditches along the top of slopes, if such devices are not in place.

9.23 ADDITIONAL INVESTIGATION:

Additional geologic and geotechnical investigation may be necessary during the 40-Scale Grading Plan Review phase to obtain supplemental information relative to the site geologic conditions and engineering properties of on-site materials and to confirm the preliminary recommendations provided herein.

10.0 Limitations:

This report has been prepared for the exclusive use of Richland Development Corporation relative to the design and construction of the proposed plan. This report is not intended for other parties, and it may not contain sufficient information for other purposes. The recommendations presented herein are of a general/conceptual nature and will be refined and detailed in forthcoming reports specific to various aspects of development. As such, the current recommendations may be subject to revision as additional detail of the project is made and additional data is developed, as well as in response to jurisdictional review requirements. This report is based on the plan as described and the information obtained as described herein.

The Owner or Owner Representative should make sure that the information and preliminary recommendations presented in this report are brought to the attention of the Project Architect and Project Engineer and incorporated into the project plans.

The findings contained in this report are based upon our evaluation and interpretation of the information obtained from the infiltration test pits and boring, as well as the geotechnical engineering analyses. The opinions and recommendations provided were based on the assumption that the geotechnical conditions, which exist across the site, are similar to those observed in the test excavations. The condition and characteristics of the sub-surface materials at locations and depths other than those excavated and observed may be different and no representations are made as to their quality and engineering properties. Should any conditions encountered during construction differ from those described herein, this office should be contacted immediately for evaluation of the actual conditions and for appropriate recommendations prior to continuation of work.

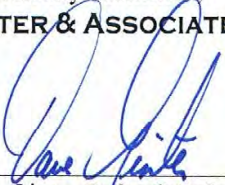
The finding and recommendations presented herein were obtained in accordance with currently accepted professional engineering principles and practice in the field of geologic and geotechnical engineering and reflect our best professional judgment. We make no other warranty, either expressed or implied.

This report is subject to review by the controlling authorities. If you have any questions or require additional information, please contact the undersigned.

We appreciate the opportunity to be of service and look forward to working with you and the other project consultants.

{Signatures on following page}

Respectfully submitted,
GINTER & ASSOCIATES, INC.

By: 
Dave Ginter R.G., C.E.G.
Principal Engineering Geologist/President

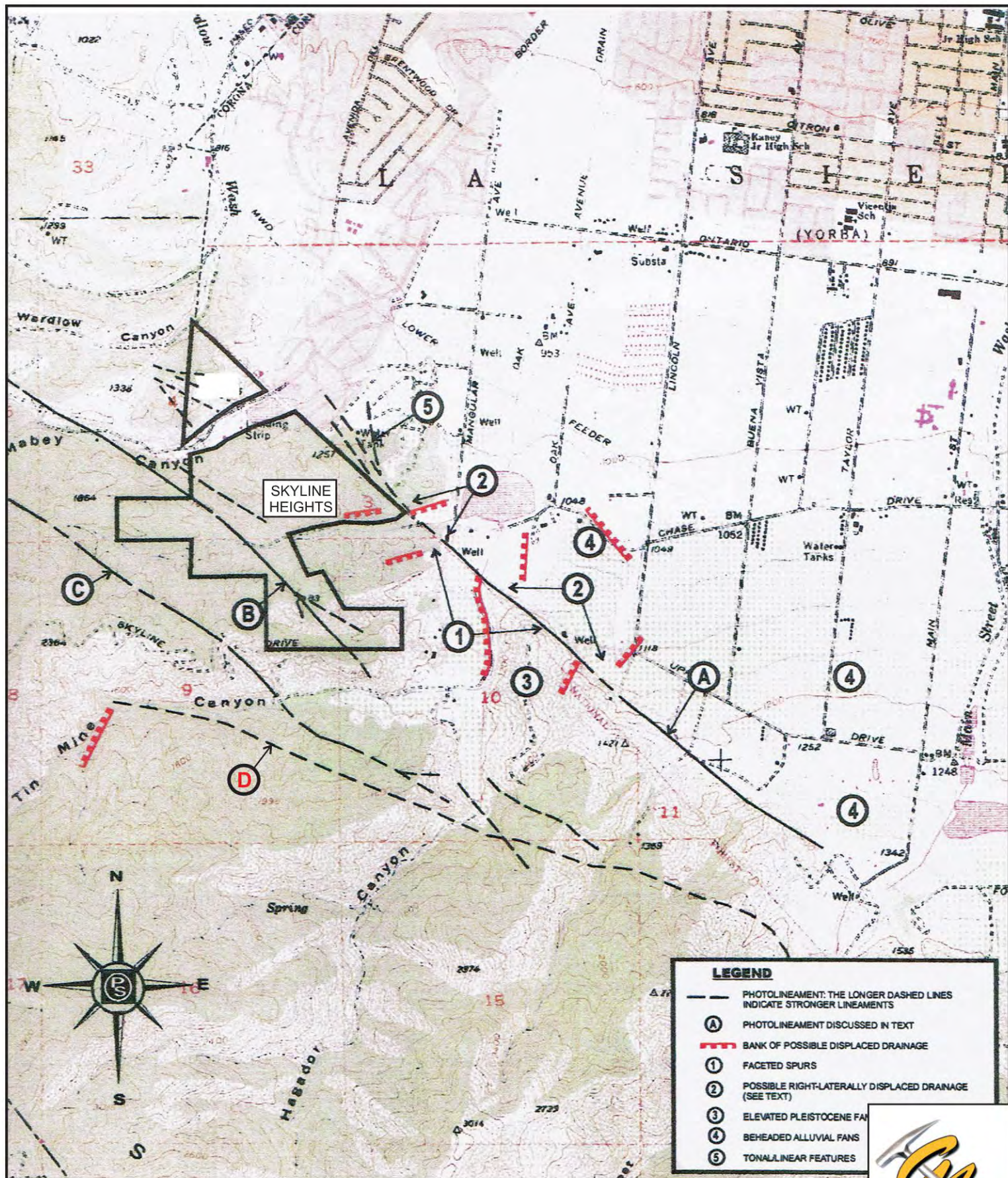


By: 
Mike Mills R.G., C.E.G.
Consulting Engineering Geologist



By: 
Vela "Ganesh" Ganeshwara, P.E., G.E.
Consulting Geotechnical Engineer





SOURCE MAP:
CORONA SOUTH
QUADRANGLE
SCALE: 1"=2200'
and PSE 11/30/07

Plate III REGIONAL PHOTOLINEMATIC/ TECTONIC GEOMORPHIC MAP



PROJECT NO. :
134-02
DATE :
May 20, 2013

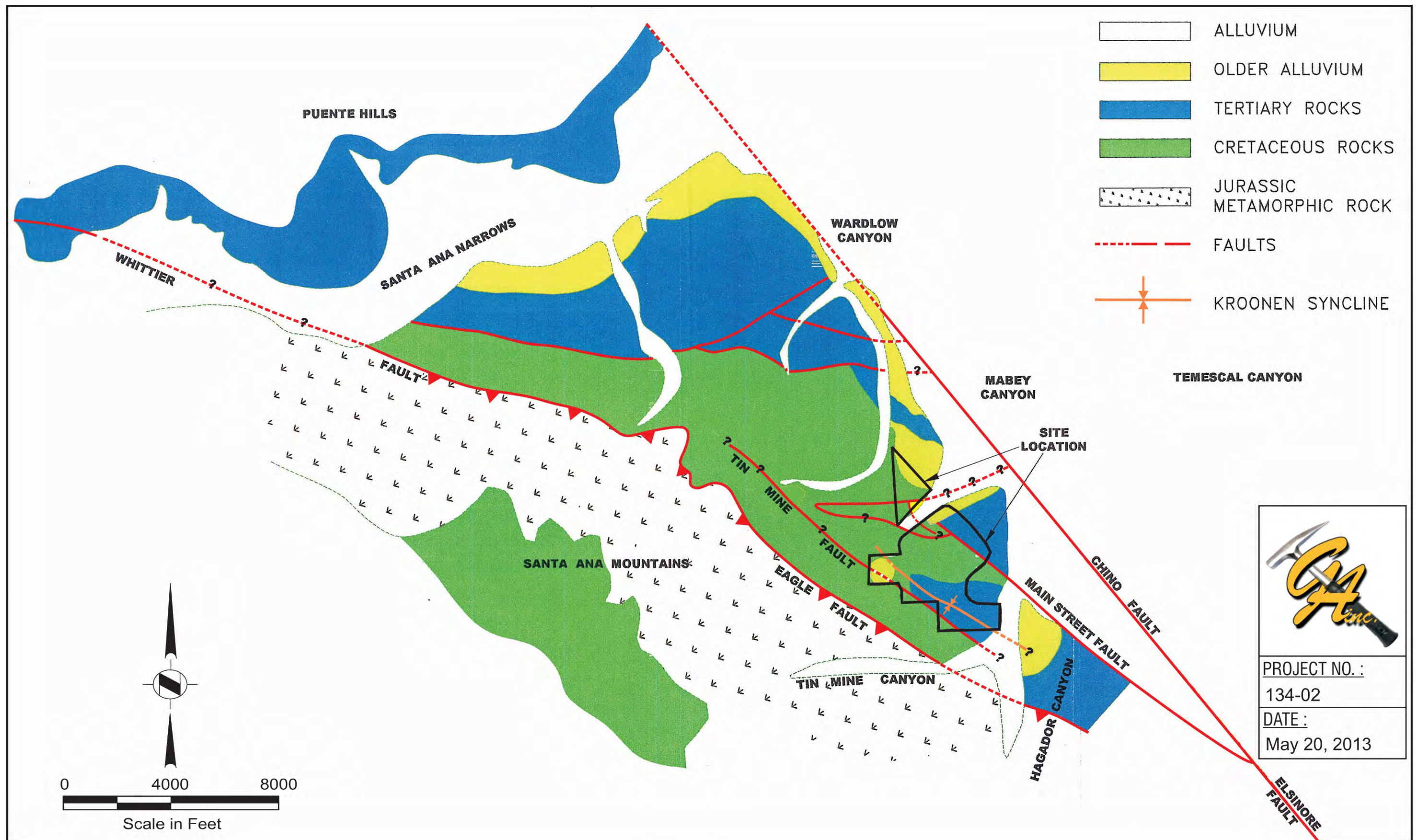
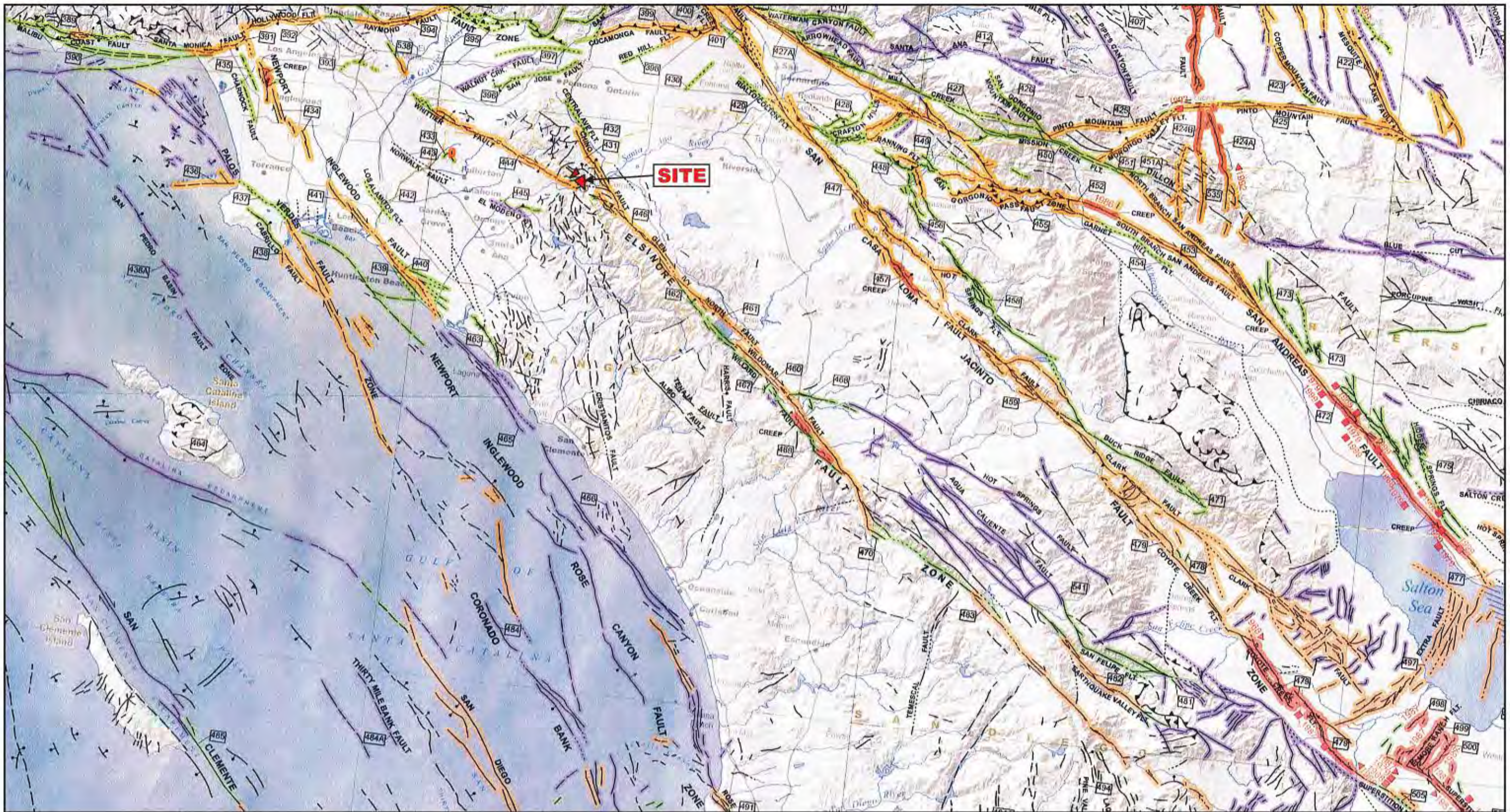


Plate IV
STYLIZED REGIONAL
STRUCTURAL MAP

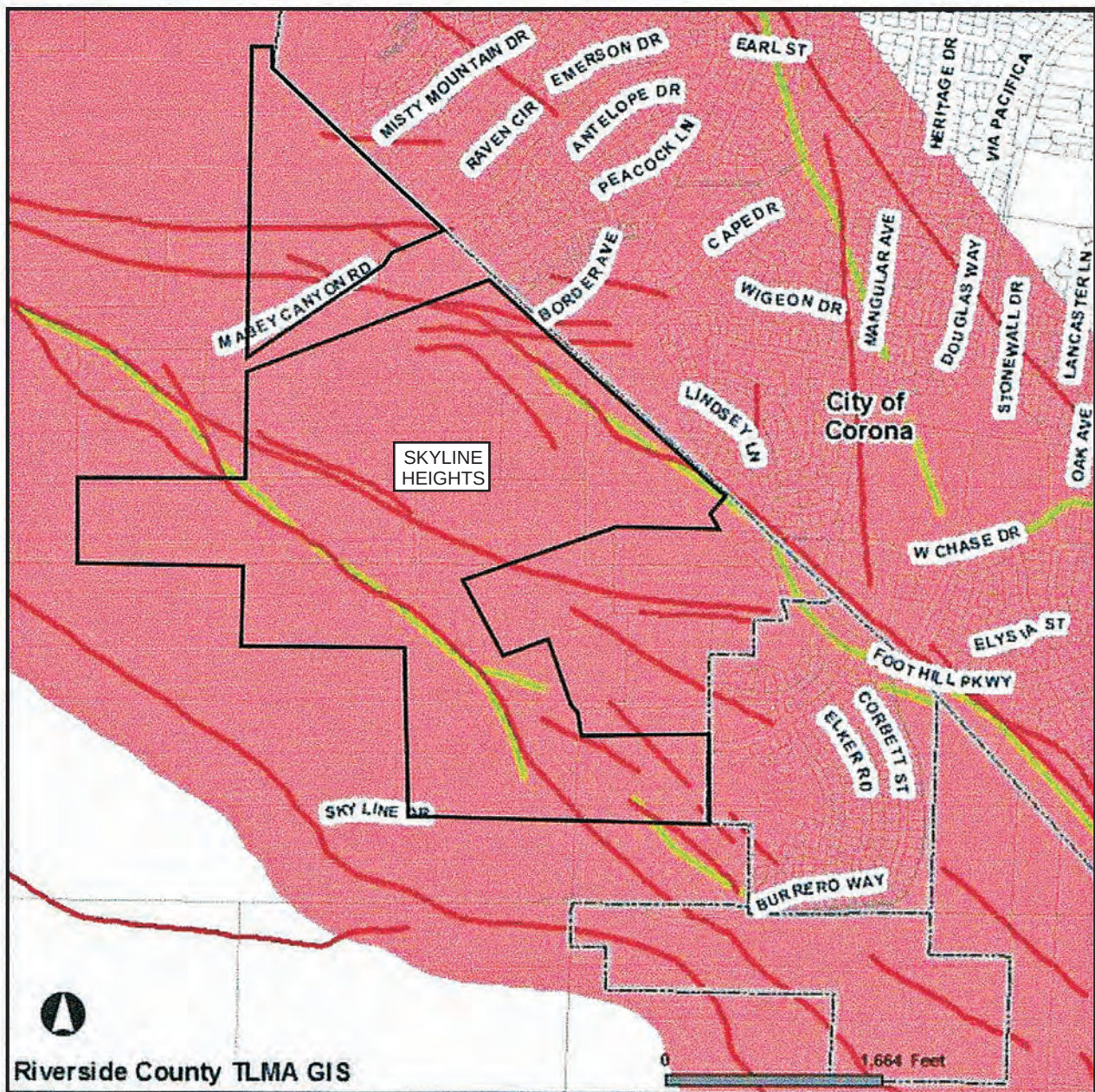


Source: Portion of C.G.S. Fault Activity Map
from C.W. Jennings and W.A. Bryant, 2010



Plate V REGIONAL FAULT MAP OF SOUTHERN CALIFORNIA

PROJECT NO.:	134-02
DATE:	May 20, 2013



FAULT ZONES

- | | | | |
|------------------|-------------------|---------------------|----------------|
| INTERSTATES | HIGHWAYS | PARCELS | ALQUIST-PRIOLO |
| RIVERSIDE COUNTY | COUNTY FAULT ZONE | ELSINORE FAULT ZONE | CITY |

IMPORTANT

Maps and data are to be used for reference purposes only. Map features are approximate, and are not necessarily accurate to surveying or engineering standards. The County of Riverside makes no warranty or guarantee as to the content (the source is often third party), accuracy, timeliness, or completeness of any of the data provided, and assumes no legal responsibility for the information contained on this map. Any use of this product with respect to accuracy and precision shall be the sole responsibility of the user.

REPORT PRINTED ON: Thu Jan 05 2012 13:07:33 GMT-0800 (Pacific Standard Time)

Version 111212

Plate VII RIVERSIDE COUNTY FAULT ZONES SKYLINE HEIGHTS

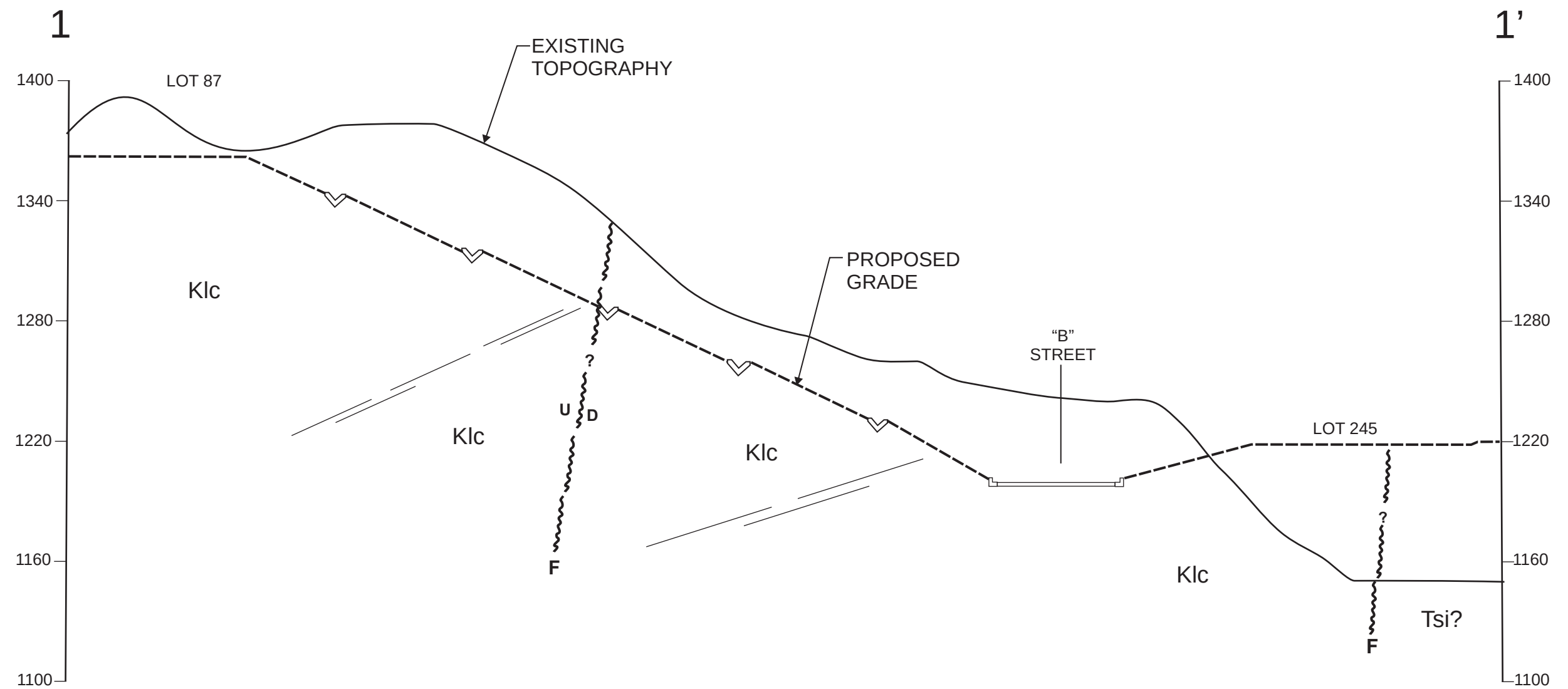


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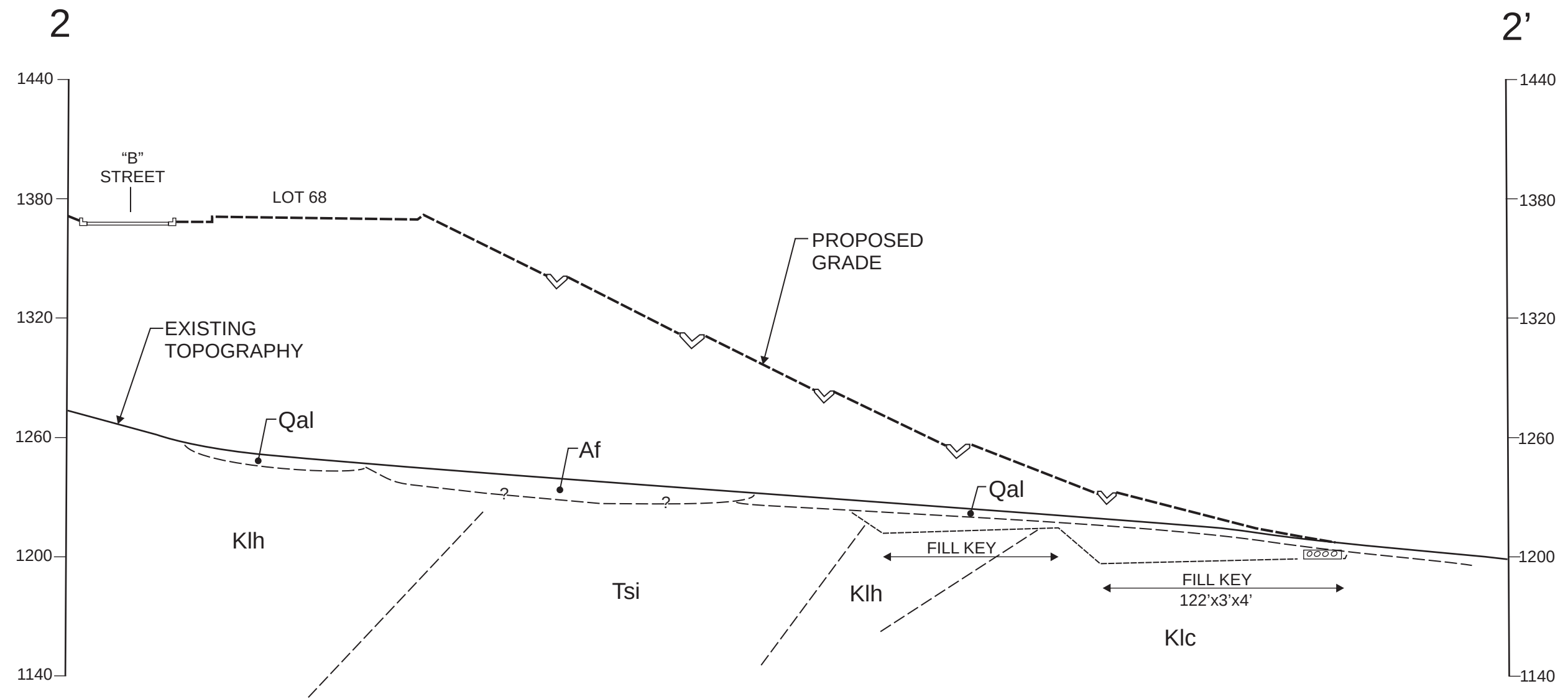
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May 20, 2013



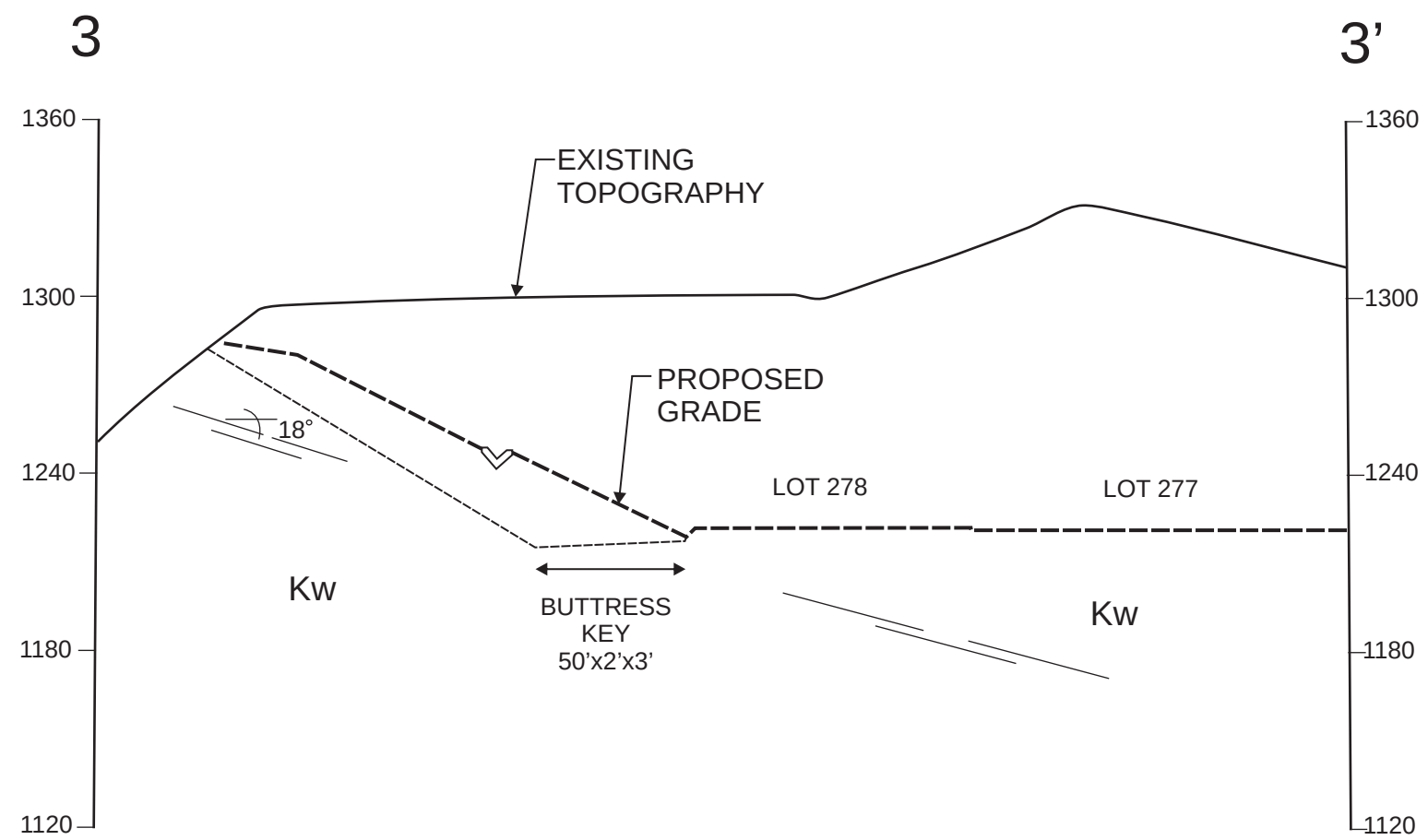
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CROSS-SECTION 2-2'
SCALE: 1"=60' H&V

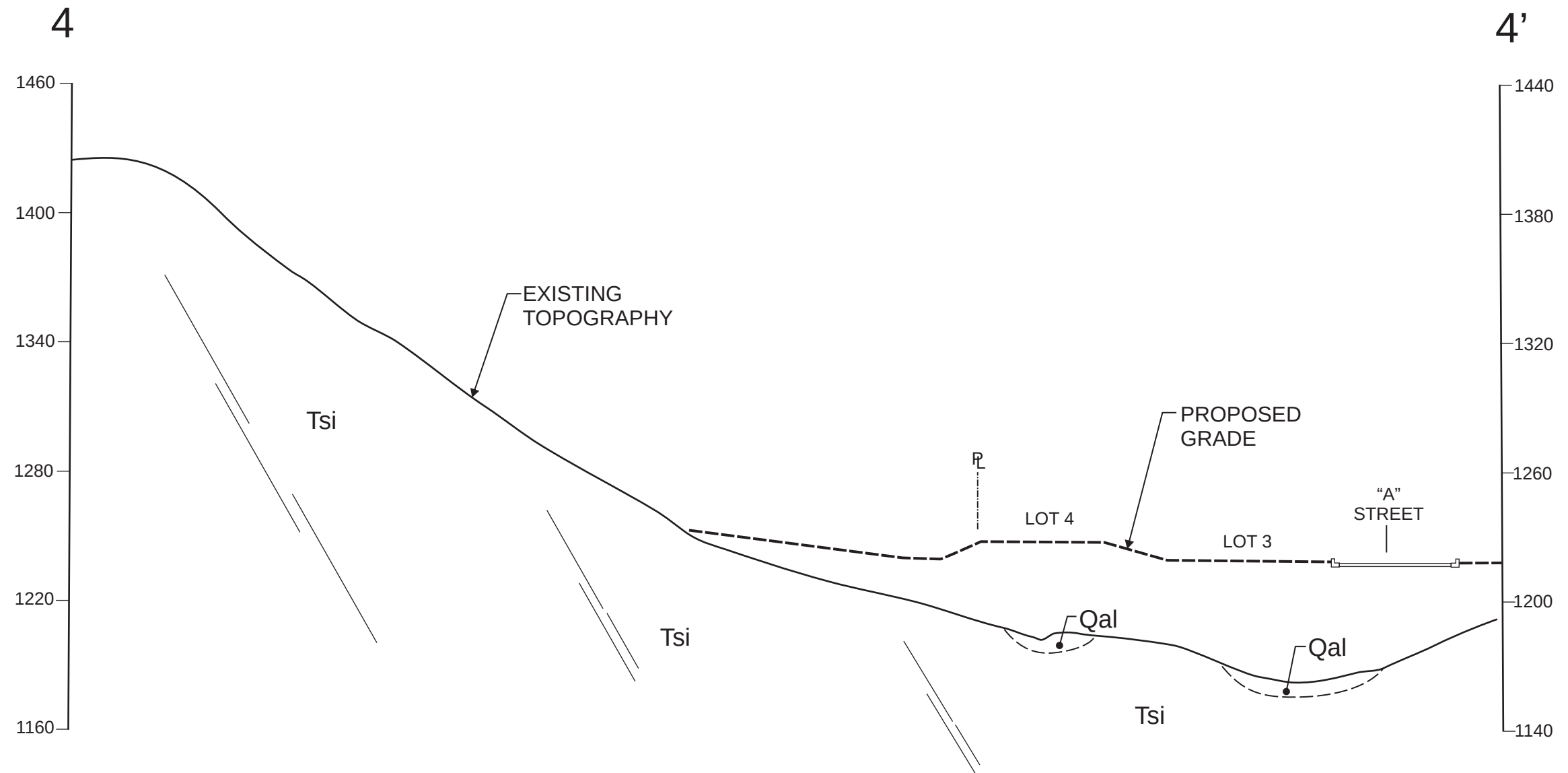


PROJECT NO. :
134-02
DATE :
May, 2013



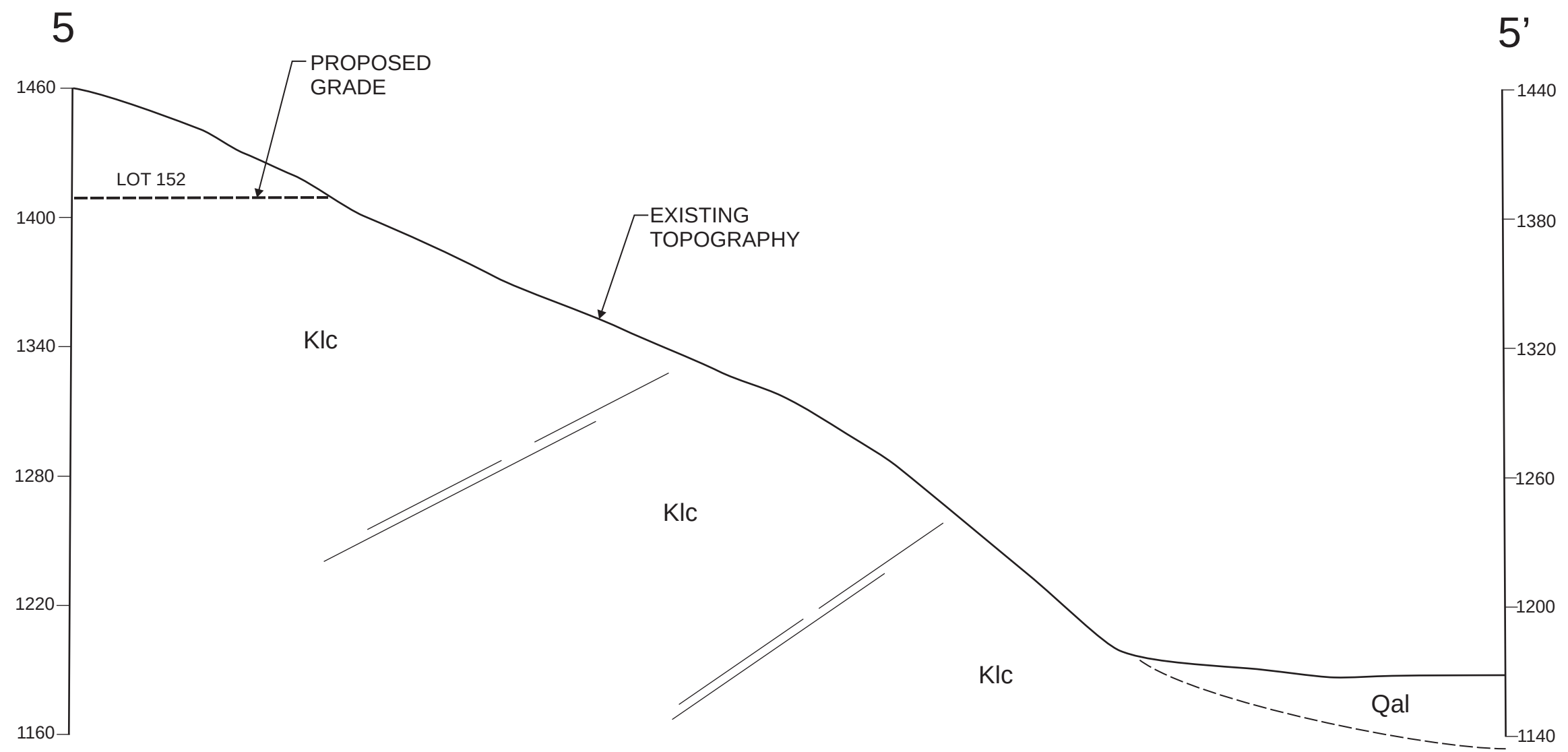
CROSS-SECTION 3-3'
 SCALE: 1"=60' H&V


PROJECT NO.: 134-02
DATE: May, 2013



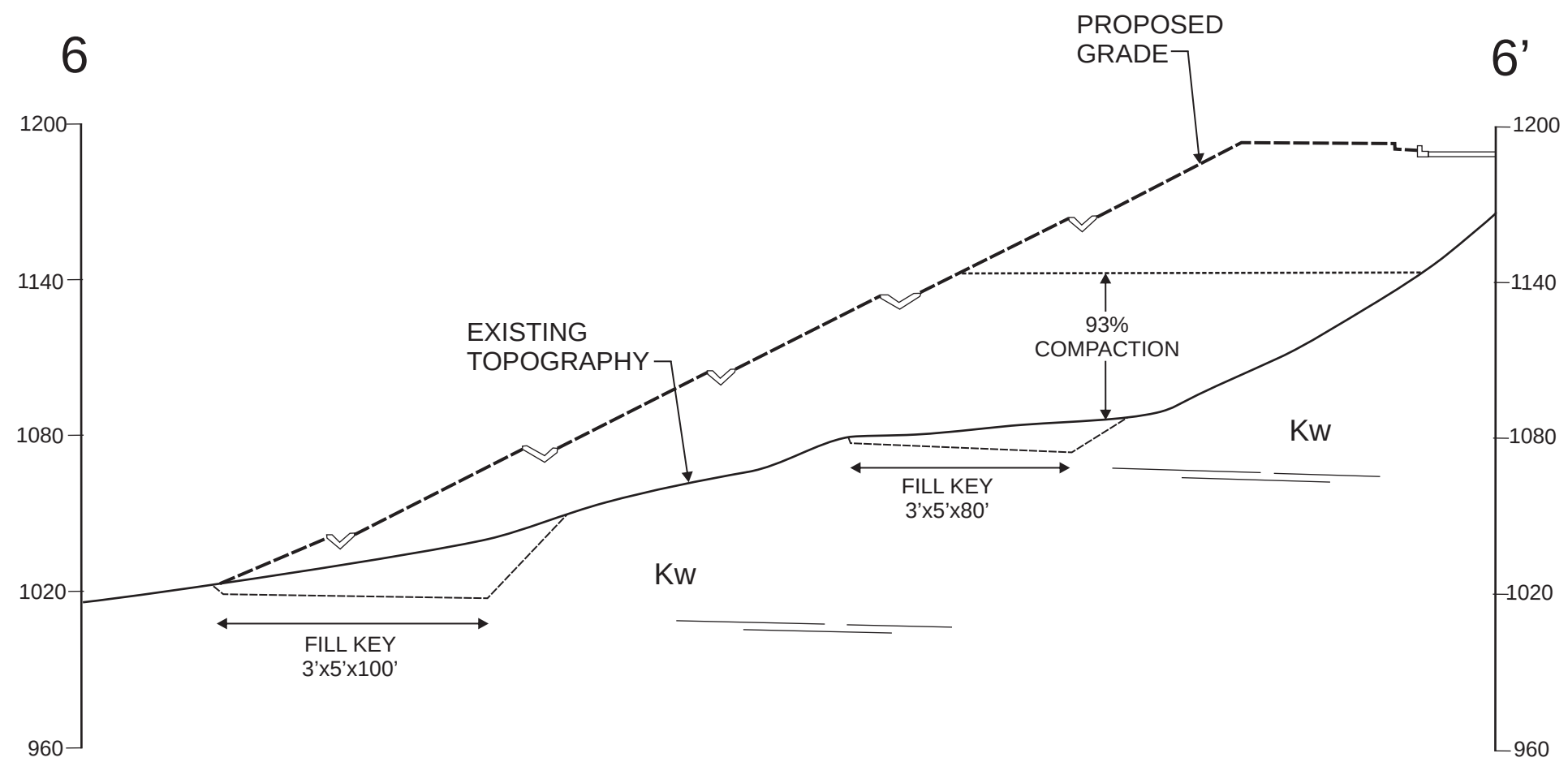
CROSS-SECTION 4-4'
SCALE: 1"=60' H&V


PROJECT NO. : 134-02
DATE : May, 2013



CROSS-SECTION 5-5'
SCALE: 1"=60' H&V


PROJECT NO. : 134-02
DATE : May, 2013



CROSS-SECTION 6-6'
SCALE: 1"=60' H&V

*** GSTABL7 ***

** GSTABL7 by Dr. Garry H. Gregory,
Ph.D., P.E., D.GE **

** Original Version 1.0, January 1996; Current Ver. 2.005.2,
Jan. 2011 **

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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: 5/10/2013
Time of Run: 10:39AM
Run By: Username
Input Data Filename: C:\SLOPE\Skyline\1-1\1-1.in
Output Filename: C:\SLOPE\Skyline\1-1\1-1.OUT
Unit System: English

Plotted Output Filename: C:\SLOPE\Skyline\1-1\1-1.PLT

PROBLEM DESCRIPTION: Skyline Height, PN:134-02, Section 1-1',
Highest Cut Slope, Static

BOUNDARY COORDINATES

14 Top Boundaries
14 Total Boundaries

Type Bnd	Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Below
	1	6.00	1218.00	78.00	1200.00	3
	2	78.00	1200.00	132.00	1200.00	3
	3	132.00	1200.00	189.00	1232.00	3
	4	189.00	1232.00	195.00	1232.00	3
	5	195.00	1232.00	258.00	1259.00	3
	6	258.00	1259.00	264.00	1259.00	3
	7	264.00	1259.00	324.00	1286.00	3
	8	324.00	1286.00	330.00	1286.00	3
	9	330.00	1286.00	392.00	1316.00	3
	10	392.00	1316.00	398.00	1316.00	3
	11	398.00	1316.00	462.00	1343.00	3
	12	462.00	1343.00	468.00	1343.00	3
	13	468.00	1343.00	510.00	1371.00	3
	14	510.00	1371.00	660.00	1371.00	3

User Specified Y-Origin = 1000.00(ft)

Default X-Plus Value = 0.00(ft)

Default Y-Plus Value = 0.00(ft)

1

ISOTROPIC SOIL PARAMETERS

3 Type(s) of Soil

Piez. Surface No.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param. (psf)	Pressure Constant (psf)
-------------------------	---------------------	----------------------------	--------------------------------	--------------------------------	----------------------------	-------------------------------------	-------------------------------

0	1	120.0	120.0	300.0	35.0	0.00	0.0
---	---	-------	-------	-------	------	------	-----

0	2	125.0	125.0	800.0	32.0	0.00	0.0
---	---	-------	-------	-------	------	------	-----

0	3	125.0	125.0	250.0	40.0	0.00	0.0
---	---	-------	-------	-------	------	------	-----

0							
---	--	--	--	--	--	--	--

0							
---	--	--	--	--	--	--	--

1							
---	--	--	--	--	--	--	--

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Circular Surfaces, Has Been Specified.

Spaced

* *

Point No.	X-Surf (ft)	Y-Surf (ft)
1	135.333	1201.871
2	184.996	1207.672
3	234.145	1216.855
4	282.552	1229.378
5	329.989	1245.180

Circle Center At X = 75.421 ; Y = 1930.255 ; and Radius = 730.843

			Water Force	Water Force	Tie Force	Tie Force	Earthquake Force	
Surcharge Slice Load No. (lbs)	Width (ft)	Weight (lbs)	Top (lbs)	Bot (lbs)	Norm (lbs)	Tan (lbs)	Hor (lbs)	Ver (lbs)
1 0.0	49.7	68532.8	0.0	0.0	0.	0.	0.0	0.0
2 0.0	4.0	11427.1	0.0	0.0	0.	0.	0.0	0.0
3 0.0	6.0	17264.2	0.0	0.0	0.	0.	0.0	0.0
4 0.0	39.1	133043.1	0.0	0.0	0.	0.	0.0	0.0
5 0.0	23.9	101225.9	0.0	0.0	0.	0.	0.0	0.0
6 0.0	6.0	26398.1	0.0	0.0	0.	0.	0.0	0.0
7 0.0	18.6	83937.7	0.0	0.0	0.	0.	0.0	0.0
8 0.0	41.4	209276.6	0.0	0.0	0.	0.	0.0	0.0
9 0.0	6.0	31304.7	0.0	0.0	0.	0.	0.0	0.0
10 0.0	0.0	57.0	0.0	0.0	0.	0.	0.0	0.0
11 0.0	46.2	245613.4	0.0	0.0	0.	0.	0.0	0.0
12 0.0	15.8	86920.6	0.0	0.0	0.	0.	0.0	0.0
13 0.0	6.0	31912.7	0.0	0.0	0.	0.	0.0	0.0

14	23.1	116063.8	0.0	0.0	0.	0.	0.0	0.0
0.0								
15	40.9	184929.4	0.0	0.0	0.	0.	0.0	0.0
0.0								
16	2.3	9227.9	0.0	0.0	0.	0.	0.0	0.0
0.0								
17	3.7	14038.9	0.0	0.0	0.	0.	0.0	0.0
0.0								
18	37.7	135847.6	0.0	0.0	0.	0.	0.0	0.0
0.0								
19	4.3	15287.2	0.0	0.0	0.	0.	0.0	0.0
0.0								
20	35.1	63232.7	0.0	0.0	0.	0.	0.0	0.0
0.0								
21	0.8	34.6	0.0	0.0	0.	0.	0.0	0.0
0.0								

Failure Surface Specified By 10 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	156.833	1213.942
2	206.078	1222.599
3	254.701	1234.253
4	302.520	1248.858
5	349.357	1266.361
6	395.035	1286.695
7	439.384	1309.786
8	482.238	1335.545
9	523.436	1363.877
10	532.545	1371.000

Circle Center At X = 41.140 ; Y = 2018.615 ; and Radius = 812.948

Factor of Safety
*** 2.332 ***

1

Failure Surface Specified By 10 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	149.667	1209.918
2	198.902	1218.627
3	247.635	1229.814

4	295.739	1243.451
5	343.092	1259.504
6	389.573	1277.931
7	435.062	1298.684
8	479.444	1321.712
9	522.604	1346.955
10	559.320	1371.000

985.934 Circle Center At X = 2.805 ; Y = 2184.852 ; and Radius =

Factor of Safety
 *** 2.343 ***

Failure Surface Specified By 10 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	171.167	1221.988
2	220.629	1229.305
3	269.504	1239.850
4	317.582	1253.581
5	364.655	1270.436
6	410.521	1290.343
7	454.982	1313.217
8	497.846	1338.959
9	538.929	1367.458
10	543.380	1371.000

759.313 Circle Center At X = 85.078 ; Y = 1976.405 ; and Radius =

Factor of Safety
 *** 2.346 ***

1

Failure Surface Specified By 9 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	207.000	1237.143
2	256.874	1240.693

3	306.146	1249.195
4	354.325	1262.565
5	400.932	1280.670
6	445.503	1303.328
7	487.594	1330.316
8	526.787	1361.364
9	536.729	1371.000

501.130 Circle Center At X = 196.400 ; Y = 1738.161 ; and Radius =

Factor of Safety
*** 2.354 ***

Failure Surface Specified By 10 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	171.167	1221.988
2	220.680	1228.946
3	269.644	1239.070
4	317.857	1252.319
5	365.119	1268.638
6	411.235	1287.960
7	456.014	1310.204
8	499.272	1335.280
9	540.829	1363.082
10	551.160	1371.000

773.611 Circle Center At X = 88.741 ; Y = 1991.196 ; and Radius =

Factor of Safety
*** 2.354 ***

1

Failure Surface Specified By 9 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	185.500	1230.035
2	235.440	1232.485

3	284.903	1239.795
4	333.416	1251.896
5	380.518	1268.673
6	425.758	1289.965
7	468.704	1315.569
8	508.948	1345.241
9	537.556	1371.000

Circle Center At X = 185.421 ; Y = 1741.912 ; and Radius = 511.876

Factor of Safety
*** 2.355 ***

Failure Surface Specified By 10 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	156.833	1213.942
2	206.623	1218.518
3	255.793	1227.593
4	303.937	1241.090
5	350.658	1258.899
6	395.571	1280.872
7	438.305	1306.829
8	478.509	1336.555
9	515.851	1369.806
10	516.969	1371.000

Circle Center At X = 131.501 ; Y = 1763.456 ; and Radius = 550.098

Factor of Safety
*** 2.355 ***

1

Failure Surface Specified By 11 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	135.333	1201.871
2	185.317	1203.157

3	235.016	1208.632
4	284.081	1218.258
5	332.165	1231.967
6	378.929	1249.661
7	424.044	1271.216
8	467.192	1296.481
9	508.067	1325.277
10	546.382	1357.401
11	560.082	1371.000

Circle Center At X = 145.025 ; Y = 1797.204 ; and Radius = 595.412

Factor of Safety
 *** 2.356 ***

Failure Surface Specified By 10 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	149.667	1209.918
2	199.286	1216.080
3	248.225	1226.323
4	296.150	1240.578
5	342.732	1258.747
6	387.652	1280.706
7	430.603	1306.303
8	471.290	1335.365
9	509.435	1367.691
10	512.741	1371.000

Circle Center At X = 100.473 ; Y = 1810.521 ; and Radius = 602.614

Factor of Safety
 *** 2.358 ***

**** END OF GSTABL7 OUTPUT ****

*** GSTABL7 ***

Ph.D., P.E., D.GE ** ** GSTABL7 by Dr. Garry H. Gregory,

Jan. 2011 ** ** Original Version 1.0, January 1996; Current Ver. 2.005.2,

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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: 5/19/2013
Time of Run: 03:42PM
Run By: Username
Input Data Filename: C:\SLOPE\Skyline\6-6\6-1.in
Output Filename: C:\SLOPE\Skyline\6-6\6-1.OUT
Unit System: English

Plotted Output Filename: C:\SLOPE\Skyline\6-6\6-1.PLT

PROBLEM DESCRIPTION: Skyline Height, PN:134-02, Section 6-6',
Fill Slope with Geogrids, Static

BOUNDARY COORDINATES

4 Top Boundaries
10 Total Boundaries

Type	Boundary	X-Left	Y-Left	X-Right	Y-Right	Soil
Bnd	No.	(ft)	(ft)	(ft)	(ft)	Below
	1	0.00	1060.00	33.00	1060.00	3
	2	33.00	1060.00	39.00	1060.00	1
	3	39.00	1060.00	291.00	1191.00	1
	4	291.00	1191.00	420.00	1191.00	1
	5	33.00	1060.00	36.00	1056.00	3
	6	36.00	1056.00	120.00	1056.00	3
	7	120.00	1056.00	156.00	1085.00	3
	8	156.00	1085.00	228.00	1100.00	3
	9	228.00	1100.00	288.00	1110.00	3
	10	288.00	1110.00	420.00	1164.00	3

User Specified Y-Origin = 1000.00 (ft)

Default X-Plus Value = 0.00 (ft)

Default Y-Plus Value = 0.00 (ft)

1

ISOTROPIC SOIL PARAMETERS

3 Type(s) of Soil

Piez.	Soil	Total	Saturated	Cohesion	Friction	Pore	Pressure
Surface	Type	Unit Wt.	Unit Wt.	Intercept	Angle	Pressure	Constant
No.	No.	(pcf)	(pcf)	(psf)	(deg)	Param.	(psf)

0	1	120.0	120.0	300.0	35.0	0.00	0.0
0	2	125.0	125.0	800.0	32.0	0.00	0.0
0	3	125.0	125.0	250.0	40.0	0.00	0.0
1							

REINFORCING LAYER(S)

13 REINFORCING LAYER(S) SPECIFIED

REINFORCING LAYER NO. 1

10 POINTS DEFINE THIS LAYER

POINT NO.	X-COORD	Y-COORD	FORCE	INCLINATION FACTOR
1	39.00	1060.00	0.00	0.000
2	44.00	1060.00	1150.00	0.000
3	48.29	1060.00	1150.00	0.000
4	52.57	1060.00	1150.00	0.000
5	56.86	1060.00	1150.00	0.000
6	61.14	1060.00	1150.00	0.000
7	65.43	1060.00	1150.00	0.000
8	69.71	1060.00	1150.00	0.000
9	74.00	1060.00	1150.00	0.000
10	79.00	1060.00	0.00	0.000

REINFORCING LAYER NO. 2

10 POINTS DEFINE THIS LAYER

POINT NO.	X-COORD	Y-COORD	FORCE	INCLINATION FACTOR
1	58.24	1070.00	0.00	0.000
2	63.24	1070.00	1150.00	0.000
3	67.52	1070.00	1150.00	0.000
4	71.81	1070.00	1150.00	0.000
5	76.09	1070.00	1150.00	0.000
6	80.38	1070.00	1150.00	0.000
7	84.67	1070.00	1150.00	0.000
8	88.95	1070.00	1150.00	0.000
9	93.24	1070.00	1150.00	0.000
10	98.24	1070.00	0.00	0.000

REINFORCING LAYER NO. 3

10 POINTS DEFINE THIS LAYER

POINT NO.	X-COORD	Y-COORD	FORCE	INCLINATION FACTOR
1	77.47	1080.00	0.00	0.000
2	82.47	1080.00	1150.00	0.000
3	86.76	1080.00	1150.00	0.000
4	91.04	1080.00	1150.00	0.000
5	95.33	1080.00	1150.00	0.000
6	99.62	1080.00	1150.00	0.000
7	103.90	1080.00	1150.00	0.000
8	108.19	1080.00	1150.00	0.000
9	112.47	1080.00	1150.00	0.000

10	117.47	1080.00	0.00	0.000
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REINFORCING LAYER NO. 4

10 POINTS DEFINE THIS LAYER

POINT NO.	X-COORD	Y-COORD	FORCE	INCLINATION FACTOR
1	96.71	1090.00	0.00	0.000
2	101.71	1090.00	1150.00	0.000
3	106.00	1090.00	1150.00	0.000
4	110.28	1090.00	1150.00	0.000
5	114.57	1090.00	1150.00	0.000
6	118.85	1090.00	1150.00	0.000
7	123.14	1090.00	1150.00	0.000
8	127.42	1090.00	1150.00	0.000
9	131.71	1090.00	1150.00	0.000
10	136.71	1090.00	0.00	0.000

REINFORCING LAYER NO. 5

10 POINTS DEFINE THIS LAYER

POINT NO.	X-COORD	Y-COORD	FORCE	INCLINATION FACTOR
1	115.95	1100.00	0.00	0.000
2	120.95	1100.00	1150.00	0.000
3	125.23	1100.00	1150.00	0.000
4	129.52	1100.00	1150.00	0.000
5	133.80	1100.00	1150.00	0.000
6	138.09	1100.00	1150.00	0.000
7	142.38	1100.00	1150.00	0.000
8	146.66	1100.00	1150.00	0.000
9	150.95	1100.00	1150.00	0.000
10	155.95	1100.00	0.00	0.000

REINFORCING LAYER NO. 6

10 POINTS DEFINE THIS LAYER

POINT NO.	X-COORD	Y-COORD	FORCE	INCLINATION FACTOR
1	135.18	1110.00	0.00	0.000
2	140.18	1110.00	1150.00	0.000
3	144.47	1110.00	1150.00	0.000

4	148.75	1110.00	1150.00	0.000
5	153.04	1110.00	1150.00	0.000
6	157.33	1110.00	1150.00	0.000
7	161.61	1110.00	1150.00	0.000
8	165.90	1110.00	1150.00	0.000
9	170.18	1110.00	1150.00	0.000
10	175.18	1110.00	0.00	0.000

REINFORCING LAYER NO. 7

10 POINTS DEFINE THIS LAYER

POINT NO.	X-COORD	Y-COORD	FORCE	INCLINATION FACTOR
1	154.42	1120.00	0.00	0.000
2	159.42	1120.00	1150.00	0.000
3	163.71	1120.00	1150.00	0.000
4	167.99	1120.00	1150.00	0.000
5	172.28	1120.00	1150.00	0.000
6	176.56	1120.00	1150.00	0.000
7	180.85	1120.00	1150.00	0.000
8	185.13	1120.00	1150.00	0.000
9	189.42	1120.00	1150.00	0.000
10	194.42	1120.00	0.00	0.000

REINFORCING LAYER NO. 8

10 POINTS DEFINE THIS LAYER

POINT NO.	X-COORD	Y-COORD	FORCE	INCLINATION FACTOR
1	173.66	1130.00	0.00	0.000
2	178.66	1130.00	1150.00	0.000
3	182.94	1130.00	1150.00	0.000
4	187.23	1130.00	1150.00	0.000
5	191.51	1130.00	1150.00	0.000
6	195.80	1130.00	1150.00	0.000
7	200.08	1130.00	1150.00	0.000
8	204.37	1130.00	1150.00	0.000
9	208.66	1130.00	1150.00	0.000
10	213.66	1130.00	0.00	0.000

REINFORCING LAYER NO. 9

10 POINTS DEFINE THIS LAYER

POINT NO.	X-COORD	Y-COORD	FORCE	INCLINATION FACTOR
1	192.89	1140.00	0.00	0.000
2	197.89	1140.00	1150.00	0.000
3	202.18	1140.00	1150.00	0.000
4	206.46	1140.00	1150.00	0.000
5	210.75	1140.00	1150.00	0.000
6	215.04	1140.00	1150.00	0.000
7	219.32	1140.00	1150.00	0.000
8	223.61	1140.00	1150.00	0.000
9	227.89	1140.00	1150.00	0.000
10	232.89	1140.00	0.00	0.000

REINFORCING LAYER NO. 10

10 POINTS DEFINE THIS LAYER

POINT NO.	X-COORD	Y-COORD	FORCE	INCLINATION FACTOR
1	212.13	1150.00	0.00	0.000
2	217.13	1150.00	1150.00	0.000
3	221.42	1150.00	1150.00	0.000
4	225.70	1150.00	1150.00	0.000
5	229.99	1150.00	1150.00	0.000
6	234.27	1150.00	1150.00	0.000
7	238.56	1150.00	1150.00	0.000
8	242.84	1150.00	1150.00	0.000
9	247.13	1150.00	1150.00	0.000
10	252.13	1150.00	0.00	0.000

REINFORCING LAYER NO. 11

10 POINTS DEFINE THIS LAYER

POINT NO.	X-COORD	Y-COORD	FORCE	INCLINATION FACTOR
1	231.37	1160.00	0.00	0.000
2	236.37	1160.00	1150.00	0.000
3	240.65	1160.00	1150.00	0.000
4	244.94	1160.00	1150.00	0.000
5	249.22	1160.00	1150.00	0.000
6	253.51	1160.00	1150.00	0.000
7	257.79	1160.00	1150.00	0.000
8	262.08	1160.00	1150.00	0.000
9	266.37	1160.00	1150.00	0.000
10	271.37	1160.00	0.00	0.000

REINFORCING LAYER NO. 12

10 POINTS DEFINE THIS LAYER

POINT NO.	X-COORD	Y-COORD	FORCE	INCLINATION FACTOR
1	250.60	1170.00	0.00	0.000
2	255.60	1170.00	1150.00	0.000
3	259.89	1170.00	1150.00	0.000
4	264.17	1170.00	1150.00	0.000
5	268.46	1170.00	1150.00	0.000
6	272.75	1170.00	1150.00	0.000
7	277.03	1170.00	1150.00	0.000
8	281.32	1170.00	1150.00	0.000
9	285.60	1170.00	1150.00	0.000
10	290.60	1170.00	0.00	0.000

REINFORCING LAYER NO. 13

10 POINTS DEFINE THIS LAYER

POINT NO.	X-COORD	Y-COORD	FORCE	INCLINATION FACTOR
1	269.84	1180.00	0.00	0.000
2	274.84	1180.00	1150.00	0.000
3	279.13	1180.00	1150.00	0.000
4	283.41	1180.00	1150.00	0.000
5	287.70	1180.00	1150.00	0.000
6	291.98	1180.00	1150.00	0.000
7	296.27	1180.00	1150.00	0.000
8	300.55	1180.00	1150.00	0.000
9	304.84	1180.00	1150.00	0.000
10	309.84	1180.00	0.00	0.000

1

A Critical Failure Surface Searching Method, Using A Random
Technique For Generating Circular Surfaces, Has Been Specified.

2000 Trial Surfaces Have Been Generated.

Spaced 100 Surface(s) Initiate(s) From Each Of 20 Points Equally
Along The Ground Surface Between X = 0.00(ft)
and X = 100.00(ft)

Each Surface Terminates Between X = 200.00 (ft)
and X = 420.00 (ft)

Unless Further Limitations Were Imposed, The Minimum Elevation
At Which A Surface Extends Is Y = 0.00 (ft)

25.00 (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 = 5.030 FS Min = 1.770 FS Ave = 3.262
Standard Deviation = 0.864 Coefficient of Variation = 26.48 %

Failure Surface Specified By 13 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	52.632	1067.086
2	77.518	1069.470
3	102.191	1073.501
4	126.542	1079.160
5	150.464	1086.423
6	173.851	1095.257
7	196.600	1105.624
8	218.611	1117.477
9	239.787	1130.766
10	260.034	1145.430
11	279.264	1161.406
12	297.391	1178.623
13	308.800	1191.000

Circle Center At X = 29.189 ; Y = 1442.850 ; and Radius = 376.495

Factor of Safety
*** 1.770 ***

Individual data on the 13 slices

Surcharge Slice Load No. (lbs)	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor Ver (lbs) (lbs)	
1	24.9	15756.6	0.0	0.0	0.	0.	0.0	0.0
0.0								
2	24.7	44264.2	0.0	0.0	0.	0.	0.0	0.0
0.0								
3	24.4	66764.6	0.0	0.0	0.	0.	0.0	0.0
0.0								
4	23.9	83059.5	0.0	0.0	0.	0.	0.0	0.0
0.0								
5	23.4	93124.8	0.0	0.0	0.	0.	0.0	0.0
0.0								
6	22.7	97113.2	0.0	0.0	0.	0.	0.0	0.0
0.0								
7	22.0	95345.7	0.0	0.0	0.	0.	0.0	0.0
0.0								
8	21.2	88307.9	0.0	0.0	0.	0.	0.0	0.0
0.0								
9	20.2	76637.3	0.0	0.0	0.	0.	0.0	0.0
0.0								
10	19.2	61110.1	0.0	0.0	0.	0.	0.0	0.0
0.0								
11	11.7	29532.9	0.0	0.0	0.	0.	0.0	0.0
0.0								
12	6.4	11819.2	0.0	0.0	0.	0.	0.0	0.0
0.0								
13	11.4	8472.9	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	52.632	1067.086
2	77.532	1069.310
3	102.204	1073.349
4	126.514	1079.183
5	150.332	1086.779
6	173.530	1096.098
7	195.985	1107.089
8	217.575	1119.693
9	238.185	1133.843
10	257.705	1149.463
11	276.029	1166.469
12	293.061	1184.770
13	298.060	1191.000

Circle Center At X = 34.731 ; Y = 1408.099 ; and Radius = 341.482

Factor of Safety
*** 1.775 ***

1

Failure Surface Specified By 13 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	57.895	1069.822
2	82.746	1072.542
3	107.374	1076.842
4	131.676	1082.705
5	155.555	1090.108
6	178.913	1099.020
7	201.654	1109.404
8	223.686	1121.220
9	244.919	1134.417
10	265.266	1148.943
11	284.644	1164.738
12	302.975	1181.737
13	311.764	1191.000

Circle Center At X = 27.763 ; Y = 1460.112 ; and Radius = 391.451

Factor of Safety
*** 1.775 ***

Failure Surface Specified By 13 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	57.895	1069.822
2	82.685	1073.055
3	107.231	1077.797
4	131.441	1084.031
5	155.225	1091.734
6	178.493	1100.877
7	201.158	1111.426
8	223.136	1123.341
9	244.345	1136.578
10	264.704	1151.086
11	284.138	1166.813
12	302.573	1183.699
13	309.626	1191.000

Circle Center At X = 17.482 ; Y = 1476.456 ; and Radius = 408.637

Factor of Safety
*** 1.777 ***

1

Failure Surface Specified By 13 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	57.895	1069.822
2	82.797	1072.033
3	107.497	1075.892
4	131.887	1081.380
5	155.859	1088.475
6	179.308	1097.144
7	202.130	1107.350
8	224.225	1119.048
9	245.495	1132.185
10	265.846	1146.705
11	285.189	1162.543
12	303.439	1179.629
13	314.074	1191.000

376.624 Circle Center At X = 37.053 ; Y = 1445.869 ; and Radius =

Factor of Safety
*** 1.780 ***

Failure Surface Specified By 12 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	63.158	1072.558
2	87.969	1075.627
3	112.502	1080.434
4	136.637	1086.954
5	160.253	1095.156
6	183.234	1104.999
7	205.466	1116.434
8	226.838	1129.404
9	247.245	1143.845
10	266.586	1159.686
11	284.765	1176.848
12	297.785	1191.000

354.178 Circle Center At X = 32.256 ; Y = 1425.386 ; and Radius =

Factor of Safety
*** 1.794 ***

1

Failure Surface Specified By 13 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	52.632	1067.086
2	77.625	1067.661
3	102.473	1070.416
4	126.985	1075.332
5	150.973	1082.371
6	174.255	1091.478
7	196.653	1102.585
8	217.994	1115.606

9	238.116	1130.441
10	256.865	1146.978
11	274.098	1165.090
12	289.682	1184.638
13	293.901	1191.000

Circle Center At X = 58.568 ; Y = 1352.904 ; and Radius = 285.880

Factor of Safety
*** 1.798 ***

Failure Surface Specified By 12 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	47.368	1064.350
2	72.236	1066.920
3	96.840	1071.354
4	121.040	1077.627
5	144.699	1085.703
6	167.684	1095.537
7	189.864	1107.072
8	211.113	1120.243
9	231.310	1134.976
10	250.342	1151.188
11	268.100	1168.785
12	284.395	1187.566

Circle Center At X = 25.702 ; Y = 1395.617 ; and Radius = 331.975

Factor of Safety
*** 1.799 ***

1

Failure Surface Specified By 12 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	42.105	1061.614
2	67.033	1063.518

3	91.718	1067.474
4	115.992	1073.455
5	139.689	1081.420
6	162.647	1091.316
7	184.710	1103.073
8	205.726	1116.612
9	225.553	1131.841
10	244.054	1148.655
11	261.103	1166.940
12	272.488	1181.377

Circle Center At X = 31.794 ; Y = 1362.921 ; and Radius = 301.483

Factor of Safety
 *** 1.800 ***

Failure Surface Specified By 13 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	57.895	1069.822
2	82.529	1074.081
3	106.933	1079.509
4	131.050	1086.095
5	154.825	1093.824
6	178.205	1102.677
7	201.136	1112.636
8	223.565	1123.677
9	245.443	1135.775
10	266.719	1148.903
11	287.345	1163.031
12	307.273	1178.126
13	322.683	1191.000

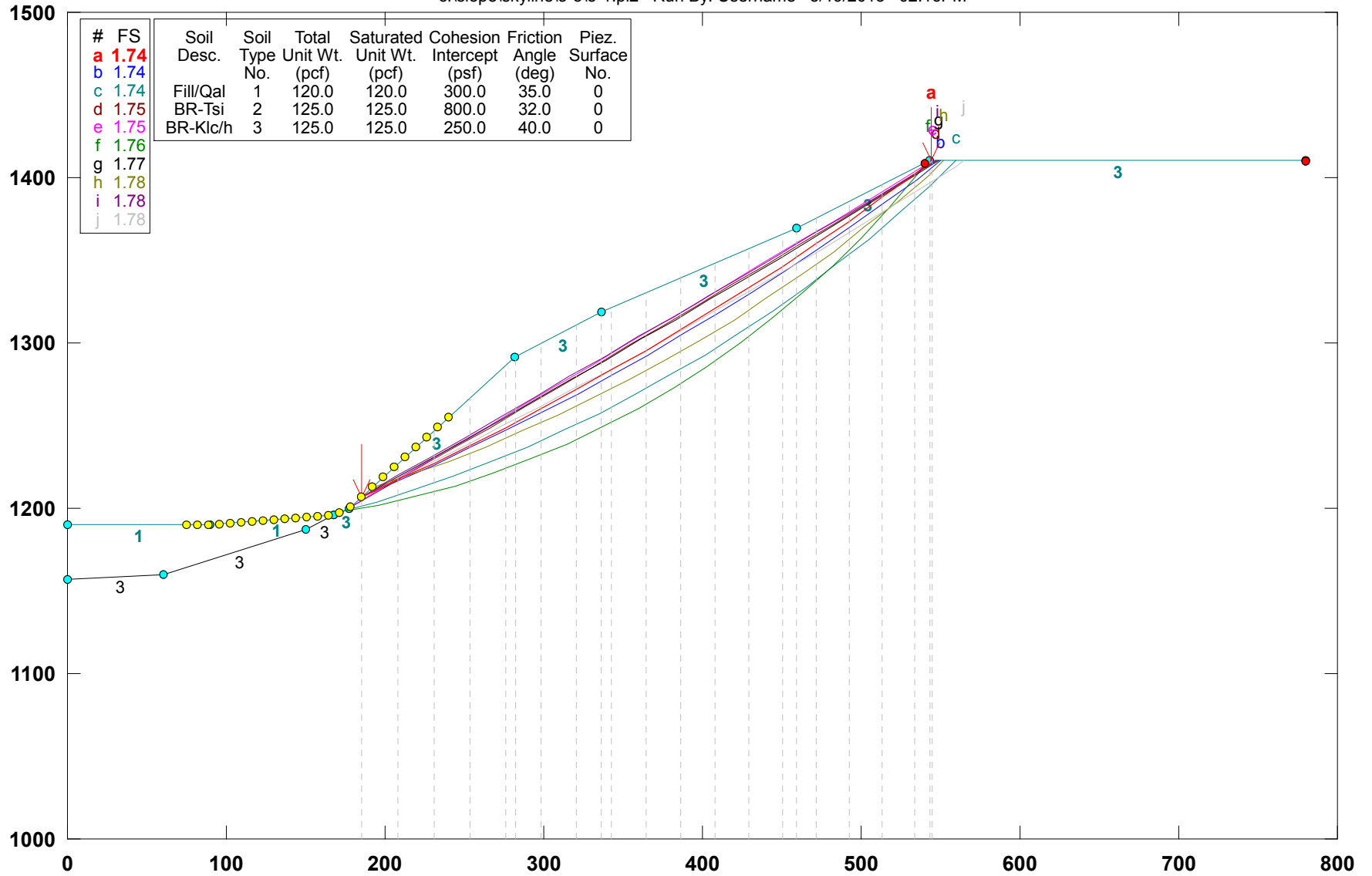
Circle Center At X = -19.026 ; Y = 1588.197 ; and Radius = 524.051

Factor of Safety
 *** 1.808 ***

**** END OF GSTABL7 OUTPUT ****

Skyline Height, PN:134-02, Section 5-5', Highest Descending Natural Slope, Static

c:\slope\skyline\5-5\5-1.pl2 Run By: Username 5/19/2013 02:16PM

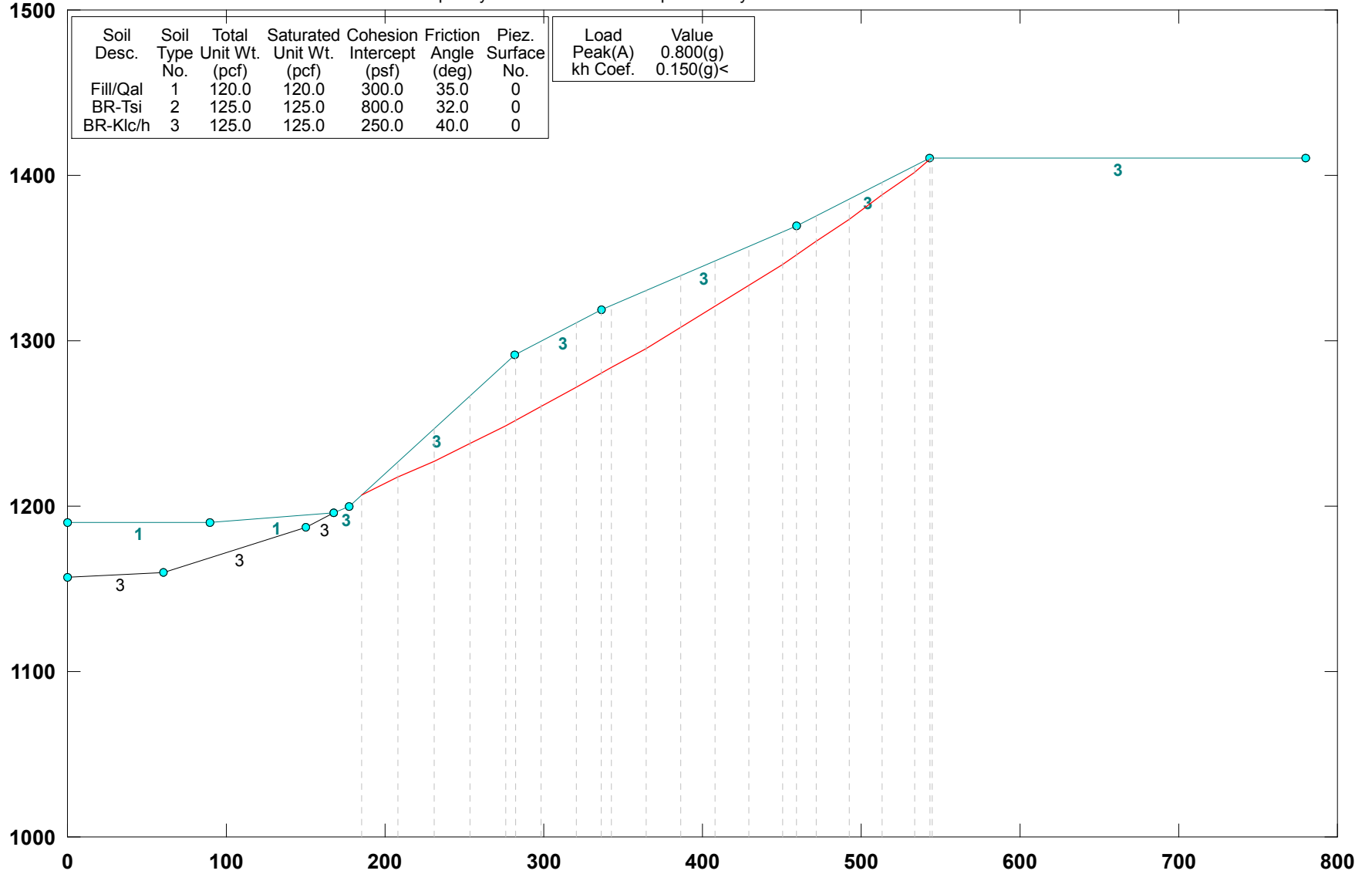


GSTABL7 v.2 FSmin=1.74

Safety Factors Are Calculated By The Modified Bishop Method

Skyline Height, PN:134-02, Section 5-5', Highest Descending Natural Slope, Pseudo

c:\slope\skyline\5-5\5-1 surface #1.plt Run By: Username 5/19/2013 02:18PM



GSTABL7 v.2 FSmin=1.27

Factor Of Safety Is Calculated By The Modified Bishop Method

*** GSTABL7 ***

** GSTABL7 by Dr. Garry H. Gregory,
Ph.D., P.E., D.GE **

** Original Version 1.0, January 1996; Current Ver. 2.005.2,
Jan. 2011 **

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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:	5/19/2013
	Time of Run:	02:18PM
	Run By:	Username
#1.in	Input Data Filename:	C:\SLOPE\Skyline\5-5\5-1 Surface
#1.OUT	Output Filename:	C:\SLOPE\Skyline\5-5\5-1 Surface
	Unit System:	English
#1.PLT	Plotted Output Filename:	C:\SLOPE\Skyline\5-5\5-1 Surface

PROBLEM DESCRIPTION: Skyline Height, PN:134-02, Section 5-5',
Highest Descending Natural Slope, Pseudo

BOUNDARY COORDINATES

8 Top Boundaries

11 Total Boundaries

Type	Boundary	X-Left	Y-Left	X-Right	Y-Right	Soil
Bnd	No.	(ft)	(ft)	(ft)	(ft)	Below
	1	0.00	1190.00	90.00	1190.00	1
	2	90.00	1190.00	168.00	1196.00	1
	3	168.00	1196.00	177.00	1200.00	3
	4	177.00	1200.00	282.00	1292.00	3
	5	282.00	1292.00	336.00	1319.00	3
	6	336.00	1319.00	459.00	1370.00	3
	7	459.00	1370.00	543.00	1410.00	3
	8	543.00	1410.00	780.00	1410.00	3
	9	0.00	1157.00	60.00	1160.00	3
	10	60.00	1160.00	150.00	1187.00	3
	11	150.00	1187.00	168.00	1196.00	3

User Specified Y-Origin = 1000.00(ft)

Default X-Plus Value = 0.00(ft)

Default Y-Plus Value = 0.00(ft)

1

ISOTROPIC SOIL PARAMETERS

3 Type(s) of Soil

Piez.	Soil	Total	Saturated	Cohesion	Friction	Pore	Pressure
Surface	Type	Unit Wt.	Unit Wt.	Intercept	Angle	Pressure	Constant
No.	No.	(pcf)	(pcf)	(psf)	(deg)	Param.	(psf)
	1	120.0	120.0	300.0	35.0	0.00	0.0
0							
	2	125.0	125.0	800.0	32.0	0.00	0.0
0							
	3	125.0	125.0	250.0	40.0	0.00	0.0
0							

Specified Peak Ground Acceleration Coefficient (A) = 0.800(g)

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

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

Specified Seismic Pore-Pressure Factor = 0.000

1

Trial Failure Surface Specified By 18 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	185.000	1207.010
2	207.900	1217.040
3	230.667	1227.368
4	253.297	1237.991
5	275.788	1248.908
6	298.134	1260.116
7	320.333	1271.615
8	342.381	1283.401
9	364.273	1295.473
10	386.006	1307.829
11	407.576	1320.467
12	428.981	1333.384
13	450.215	1346.579
14	471.276	1360.049
15	492.160	1373.792
16	512.863	1387.805
17	533.382	1402.087
18	544.441	1410.000

DEFLECTION ANGLE & SEGMENT DATA FOR SPECIFIED SURFACE(Excluding Last Segment)

Angle/Segment No.	Deflection(Deg)	Segment Length(ft)
1	0.75	25.00
2	0.74	25.00
3	0.75	25.00
4	0.74	25.00
5	0.75	25.00
6	0.74	25.00
7	0.75	25.00
8	0.75	25.00
9	0.75	25.00
10	0.74	25.00
11	0.75	25.00
12	0.74	25.00

			Water Force	Water Force	Tie Force	Tie Force	Earthquake Force	
Surcharge Slice Load No. (lbs)	Width (ft)	Weight (lbs)	Top (lbs)	Bot (lbs)	Norm (lbs)	Tan (lbs)	Hor (lbs)	Ver (lbs)
1 0.0	22.9	14361.4	0.0	0.0	0.0	0.0	2154.2	0.0
2 0.0	22.8	42245.2	0.0	0.0	0.0	0.0	6336.8	0.0
3 0.0	22.6	68617.2	0.0	0.0	0.0	0.0	10292.6	0.0
4 0.0	22.5	93490.6	0.0	0.0	0.0	0.0	14023.6	0.0
5 0.0	6.2	30138.0	0.0	0.0	0.0	0.0	4520.7	0.0
6 0.0	16.1	80596.6	0.0	0.0	0.0	0.0	12089.5	0.0
7 0.0	22.2	110304.5	0.0	0.0	0.0	0.0	16545.7	0.0
8 0.0	15.7	76926.6	0.0	0.0	0.0	0.0	11539.0	0.0
9 0.0	6.4	30810.1	0.0	0.0	0.0	0.0	4621.5	0.0
10 0.0	21.9	100559.2	0.0	0.0	0.0	0.0	15083.9	0.0
11 0.0	21.7	91217.5	0.0	0.0	0.0	0.0	13682.6	0.0

12	21.6	81043.9	0.0	0.0	0.0	0.0	12156.6	0.0
0.0								
13	21.4	70074.4	0.0	0.0	0.0	0.0	10511.2	0.0
0.0								
14	21.2	58323.9	0.0	0.0	0.0	0.0	8748.6	0.0
0.0								
15	8.8	20634.2	0.0	0.0	0.0	0.0	3095.1	0.0
0.0								
16	12.3	25779.0	0.0	0.0	0.0	0.0	3866.8	0.0
0.0								
17	20.9	36279.9	0.0	0.0	0.0	0.0	5442.0	0.0
0.0								
18	20.7	25675.0	0.0	0.0	0.0	0.0	3851.2	0.0
0.0								
19	20.5	14333.8	0.0	0.0	0.0	0.0	2150.1	0.0
0.0								
20	9.6	2623.3	0.0	0.0	0.0	0.0	393.5	0.0
0.0								
21	1.4	92.9	0.0	0.0	0.0	0.0	13.9	0.0
0.0								

Table 2 - Base Stress Data on the 21 Slices

Slice Mobilized No. Stress * (psf)	Alpha (deg)	X-Coord. Slice Cntr (ft)	Base Leng. (ft)	Available Shear Strength (psf)	Shear
1	23.65	196.45	25.00	601.87	
309.29					
2	24.40	219.28	25.00	1389.95	
927.94					
3	25.15	241.98	25.00	2132.27	
1536.42					
4	25.89	264.54	25.00	2829.56	
2132.72					
5	26.64	278.89	6.95	3244.62	
2519.14					
6	26.64	290.07	18.05	3335.27	
2593.64					
7	27.38	309.23	25.00	3291.67	
2610.16					
8	28.13	328.17	17.77	3228.58	
2607.58					
9	28.13	339.19	7.24	3177.95	
2564.32					
10	28.87	353.33	25.00	3007.46	
2464.91					
11	29.62	375.14	25.00	2741.12	
2274.35					
12	30.37	396.79	25.00	2452.38	
2054.56					

13	31.11	418.28	25.00	2142.05
1805.29				
14	31.86	439.60	25.00	1810.62
1526.58				
15	32.60	454.61	10.43	1560.69
1314.71				
16	32.60	465.14	14.57	1413.95
1175.56				
17	33.35	481.72	25.00	1189.84
978.78				
18	34.09	502.51	25.00	891.51
702.85				
19	34.84	523.12	25.00	572.58
398.00				
20	35.59	538.19	11.83	325.04
156.12				
21	35.59	543.72	1.77	206.39
36.89				

Sum of the Resisting Forces (including Pier/Pile, Tieback,
Reinforcing
Soil Nail, and Applied Forces if applicable) = 830985.44 (lbs)

Average Available Shear Strength (including Tieback, Pier/Pile,
Reinforcing,
Soil Nail, and Applied Forces if applicable) = 2009.16(psf)

Sum of the Driving Forces = 654998.00 (lbs)

Average Mobilized Shear Stress = 1583.66(psf)

Total length of the failure surface = 413.60(ft)

CAUTION - Factor Of Safety Is Calculated By The Modified
Bishop
Method. This Method Is Valid Only If The Failure
Surface
Approximates A Circular Arc.

*** SEISMIC SLOPE DISPLACEMENT DATA ***
(Note: kv is set = zero for displacement calculations)
Seismic Yield Coefficient (ky) = 0.2790(g)
Calculated Newmark Seismic Displacement = 0.638(ft)

Non-Symmetrical Sliding Resistance Has Been Specified
for Downhill Sliding.

**** END OF GSTABL7 OUTPUT ****

*** GSTABL7 ***

** GSTABL7 by Dr. Garry H. Gregory,
Ph.D., P.E., D.GE **

** Original Version 1.0, January 1996; Current Ver. 2.005.2,
Jan. 2011 **

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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: 5/19/2013
Time of Run: 02:16PM
Run By: Username
Input Data Filename: C:\SLOPE\Skyline\5-5\5-1.in
Output Filename: C:\SLOPE\Skyline\5-5\5-1.OUT
Unit System: English

Plotted Output Filename: C:\SLOPE\Skyline\5-5\5-1.PLT

PROBLEM DESCRIPTION: Skyline Height, PN:134-02, Section 5-5',
Highest Descending Natural Slope, Static

BOUNDARY COORDINATES

8 Top Boundaries
11 Total Boundaries

Type	Boundary	X-Left	Y-Left	X-Right	Y-Right	Soil
Bnd	No.	(ft)	(ft)	(ft)	(ft)	Below
	1	0.00	1190.00	90.00	1190.00	1
	2	90.00	1190.00	168.00	1196.00	1
	3	168.00	1196.00	177.00	1200.00	3
	4	177.00	1200.00	282.00	1292.00	3
	5	282.00	1292.00	336.00	1319.00	3
	6	336.00	1319.00	459.00	1370.00	3
	7	459.00	1370.00	543.00	1410.00	3
	8	543.00	1410.00	780.00	1410.00	3
	9	0.00	1157.00	60.00	1160.00	3
	10	60.00	1160.00	150.00	1187.00	3
	11	150.00	1187.00	168.00	1196.00	3

User Specified Y-Origin = 1000.00 (ft)

Default X-Plus Value = 0.00 (ft)

Default Y-Plus Value = 0.00 (ft)

1

ISOTROPIC SOIL PARAMETERS

3 Type(s) of Soil

Piez.	Soil	Total	Saturated	Cohesion	Friction	Pore	Pressure
Surface	Type	Unit Wt.	Unit Wt.	Intercept	Angle	Pressure	Constant
No.	No.	(pcf)	(pcf)	(psf)	(deg)	Param.	(psf)

0	1	120.0	120.0	300.0	35.0	0.00	0.0
0	2	125.0	125.0	800.0	32.0	0.00	0.0
0	3	125.0	125.0	250.0	40.0	0.00	0.0
1							

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Circular Surfaces, Has Been Specified.

2500 Trial Surfaces Have Been Generated.

100 Surface(s) Initiate(s) From Each Of 25 Points Equally Spaced
Along The Ground Surface Between X = 75.00 (ft)
and X = 240.00 (ft)

Each Surface Terminates Between X = 540.00 (ft)
and X = 780.00 (ft)

Unless Further Limitations Were Imposed, The Minimum Elevation
At Which A Surface Extends Is Y = 0.00 (ft)

25.00 (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
* *

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Total Number of Trial Surfaces Attempted = 2500

Number of Trial Surfaces With Valid FS = 2500

Statistical Data On All Valid FS Values:

FS Max = 5.719 FS Min = 1.737 FS Ave = 3.430
Standard Deviation = 0.908 Coefficient of Variation = 26.49 %

Failure Surface Specified By 18 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	185.000	1207.010
2	207.900	1217.040
3	230.667	1227.368
4	253.297	1237.991
5	275.788	1248.908
6	298.134	1260.116
7	320.333	1271.615
8	342.381	1283.401

9	364.273	1295.473
10	386.006	1307.829
11	407.576	1320.467
12	428.981	1333.384
13	450.215	1346.579
14	471.276	1360.049
15	492.160	1373.792
16	512.863	1387.805
17	533.382	1402.087
18	544.441	1410.000

Circle Center At X = -572.851 ; Y = 2968.688 ; and Radius = 1917.772

Factor of Safety
 *** 1.737 ***

Individual data on the 21 slices

			Water Force	Water Force	Tie Force	Tie Force	Earthquake Force	
Surcharge Slice Load	Width	Weight	Top	Bot	Norm	Tan	Hor	Ver
No. (lbs)	(ft)	(lbs)	(lbs)	(lbs)	(lbs)	(lbs)	(lbs)	(lbs)
1	22.9	14360.4	0.0	0.0	0.	0.	0.0	0.0
0.0								
2	22.8	42243.9	0.0	0.0	0.	0.	0.0	0.0
0.0								
3	22.6	68619.4	0.0	0.0	0.	0.	0.0	0.0
0.0								
4	22.5	93489.1	0.0	0.0	0.	0.	0.0	0.0
0.0								
5	6.2	30138.9	0.0	0.0	0.	0.	0.0	0.0
0.0								
6	16.1	80598.4	0.0	0.0	0.	0.	0.0	0.0
0.0								
7	22.2	110304.6	0.0	0.0	0.	0.	0.0	0.0
0.0								
8	15.7	76925.7	0.0	0.0	0.	0.	0.0	0.0
0.0								
9	6.4	30808.1	0.0	0.0	0.	0.	0.0	0.0
0.0								
10	21.9	100559.1	0.0	0.0	0.	0.	0.0	0.0
0.0								
11	21.7	91218.2	0.0	0.0	0.	0.	0.0	0.0
0.0								

12	21.6	81046.5	0.0	0.0	0.	0.	0.0	0.0
0.0								
13	21.4	70072.6	0.0	0.0	0.	0.	0.0	0.0
0.0								
14	21.2	58324.8	0.0	0.0	0.	0.	0.0	0.0
0.0								
15	8.8	20634.5	0.0	0.0	0.	0.	0.0	0.0
0.0								
16	12.3	25778.6	0.0	0.0	0.	0.	0.0	0.0
0.0								
17	20.9	36279.2	0.0	0.0	0.	0.	0.0	0.0
0.0								
18	20.7	25674.6	0.0	0.0	0.	0.	0.0	0.0
0.0								
19	20.5	14333.2	0.0	0.0	0.	0.	0.0	0.0
0.0								
20	9.6	2623.2	0.0	0.0	0.	0.	0.0	0.0
0.0								
21	1.4	92.8	0.0	0.0	0.	0.	0.0	0.0
0.0								

Failure Surface Specified By 18 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	185.000	1207.010
2	208.109	1216.547
3	231.075	1226.424
4	253.893	1236.639
5	276.558	1247.189
6	299.065	1258.072
7	321.409	1269.286
8	343.584	1280.829
9	365.588	1292.697
10	387.413	1304.889
11	409.057	1317.401
12	430.514	1330.231
13	451.779	1343.376
14	472.848	1356.834
15	493.716	1370.601
16	514.378	1384.674
17	534.831	1399.050
18	549.931	1410.000

Circle Center At X = -449.052 ; Y = 2776.418 ; and Radius = 1692.650

Factor of Safety
 *** 1.742 ***

Failure Surface Specified By 19 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	171.250	1197.444
2	195.344	1204.115
3	219.246	1211.440
4	242.940	1219.415
5	266.408	1228.034
6	289.631	1237.290
7	312.593	1247.176
8	335.277	1257.685
9	357.666	1268.810
10	379.742	1280.541
11	401.490	1292.871
12	422.894	1305.790
13	443.937	1319.288
14	464.603	1333.355
15	484.878	1347.981
16	504.747	1363.155
17	524.193	1378.866
18	543.204	1395.101
19	559.714	1410.000

Circle Center At X = -61.048 ; Y = 2083.389 ; and Radius = 915.893

Factor of Safety
*** 1.744 ***

Failure Surface Specified By 18 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	178.125	1200.986
2	200.086	1212.933
3	222.021	1224.927
4	243.930	1236.968
5	265.813	1249.056
6	287.671	1261.191
7	309.502	1273.372
8	331.307	1285.601
9	353.086	1297.876

10	374.838	1310.198
11	396.564	1322.567
12	418.264	1334.982
13	439.937	1347.443
14	461.583	1359.951
15	483.202	1372.506
16	504.794	1385.106
17	526.359	1397.753
18	547.139	1410.000

Circle Center At X = -5380.958 ; Y = 11445.780 ; and Radius = 11655.866

Factor of Safety
 *** 1.745 ***

1

Failure Surface Specified By 18 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	178.125	1200.986
2	199.941	1213.195
3	221.745	1225.426
4	243.536	1237.678
5	265.316	1249.953
6	287.083	1262.249
7	308.837	1274.567
8	330.580	1286.907
9	352.310	1299.268
10	374.027	1311.651
11	395.733	1324.056
12	417.425	1336.483
13	439.106	1348.931
14	460.774	1361.401
15	482.429	1373.893
16	504.072	1386.406
17	525.703	1398.941
18	544.742	1410.000

Circle Center At X = ***** ; Y = 23007.461 ; and Radius = 24982.035

Factor of Safety
 *** 1.751 ***

Failure Surface Specified By 19 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	171.250	1197.444
2	195.851	1201.892
3	220.248	1207.352
4	244.398	1213.816
5	268.260	1221.272
6	291.794	1229.707
7	314.959	1239.108
8	337.716	1249.458
9	360.026	1260.739
10	381.851	1272.932
11	403.153	1286.017
12	423.897	1299.970
13	444.046	1314.769
14	463.567	1330.388
15	482.426	1346.800
16	500.590	1363.977
17	518.029	1381.890
18	534.714	1400.508
19	542.239	1409.637

Circle Center At X = 75.952 ; Y = 1794.833 ; and Radius = 604.942

Factor of Safety
*** 1.761 ***

1

Failure Surface Specified By 18 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	185.000	1207.010
2	207.153	1218.596
3	229.265	1230.261
4	251.336	1242.002
5	273.366	1253.820
6	295.355	1265.716
7	317.302	1277.688
8	339.207	1289.736
9	361.070	1301.861

10	382.891	1314.062
11	404.668	1326.339
12	426.403	1338.693
13	448.094	1351.122
14	469.742	1363.626
15	491.346	1376.207
16	512.906	1388.862
17	534.422	1401.593
18	548.517	1410.000

Circle Center At X = -3116.615 ; Y = 7546.556 ; and Radius = 7147.763

Factor of Safety
*** 1.768 ***

Failure Surface Specified By 18 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	191.875	1213.033
2	215.727	1220.521
3	239.383	1228.608
4	262.828	1237.289
5	286.046	1246.559
6	309.022	1256.411
7	331.743	1266.840
8	354.194	1277.839
9	376.360	1289.400
10	398.227	1301.517
11	419.782	1314.182
12	441.010	1327.386
13	461.899	1341.122
14	482.434	1355.380
15	502.603	1370.151
16	522.394	1385.427
17	541.792	1401.197
18	552.079	1410.000

Circle Center At X = -92.316 ; Y = 2160.595 ; and Radius = 989.261

Factor of Safety
*** 1.779 ***

Failure Surface Specified By 18 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	185.000	1207.010
2	206.912	1219.045
3	228.812	1231.103
4	250.700	1243.183
5	272.575	1255.285
6	294.438	1267.410
7	316.289	1279.556
8	338.128	1291.725
9	359.954	1303.916
10	381.768	1316.128
11	403.570	1328.363
12	425.359	1340.620
13	447.136	1352.899
14	468.900	1365.200
15	490.652	1377.523
16	512.391	1389.868
17	534.118	1402.236
18	547.726	1410.000

Circle Center At X = ***** ; Y = 22818.020 ; and Radius =
24649.498

Factor of Safety
*** 1.782 ***

Failure Surface Specified By 19 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	185.000	1207.010
2	207.649	1217.594
3	230.228	1228.327
4	252.736	1239.208
5	275.171	1250.237
6	297.534	1261.414
7	319.822	1272.737
8	342.036	1284.207
9	364.173	1295.823
10	386.234	1307.584
11	408.217	1319.491

12	430.121	1331.541
13	451.945	1343.736
14	473.688	1356.074
15	495.350	1368.554
16	516.929	1381.177
17	538.425	1393.942
18	559.836	1406.848
19	564.989	1410.000

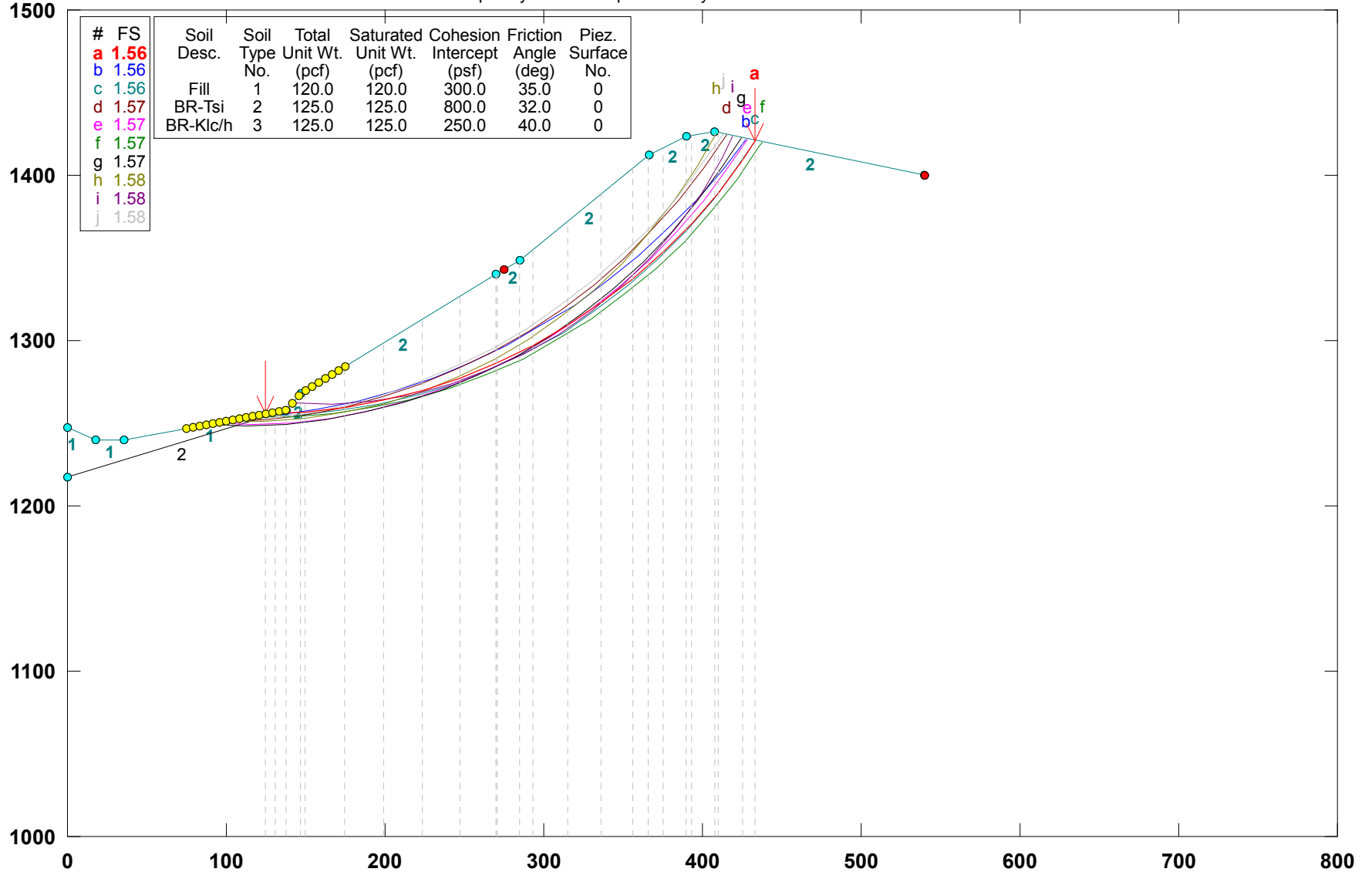
Circle Center At X = -1412.010 ; Y = 4653.974 ; and Radius =
3798.948

Factor of Safety
*** 1.782 ***

**** END OF GSTABL7 OUTPUT ****

Skyline Height, PN:134-02, Section 4-4', Highest Natural Slope, Static

c:\slope\skyline\4-4-1.pl2 Run By: Username 5/10/2013 11:37AM

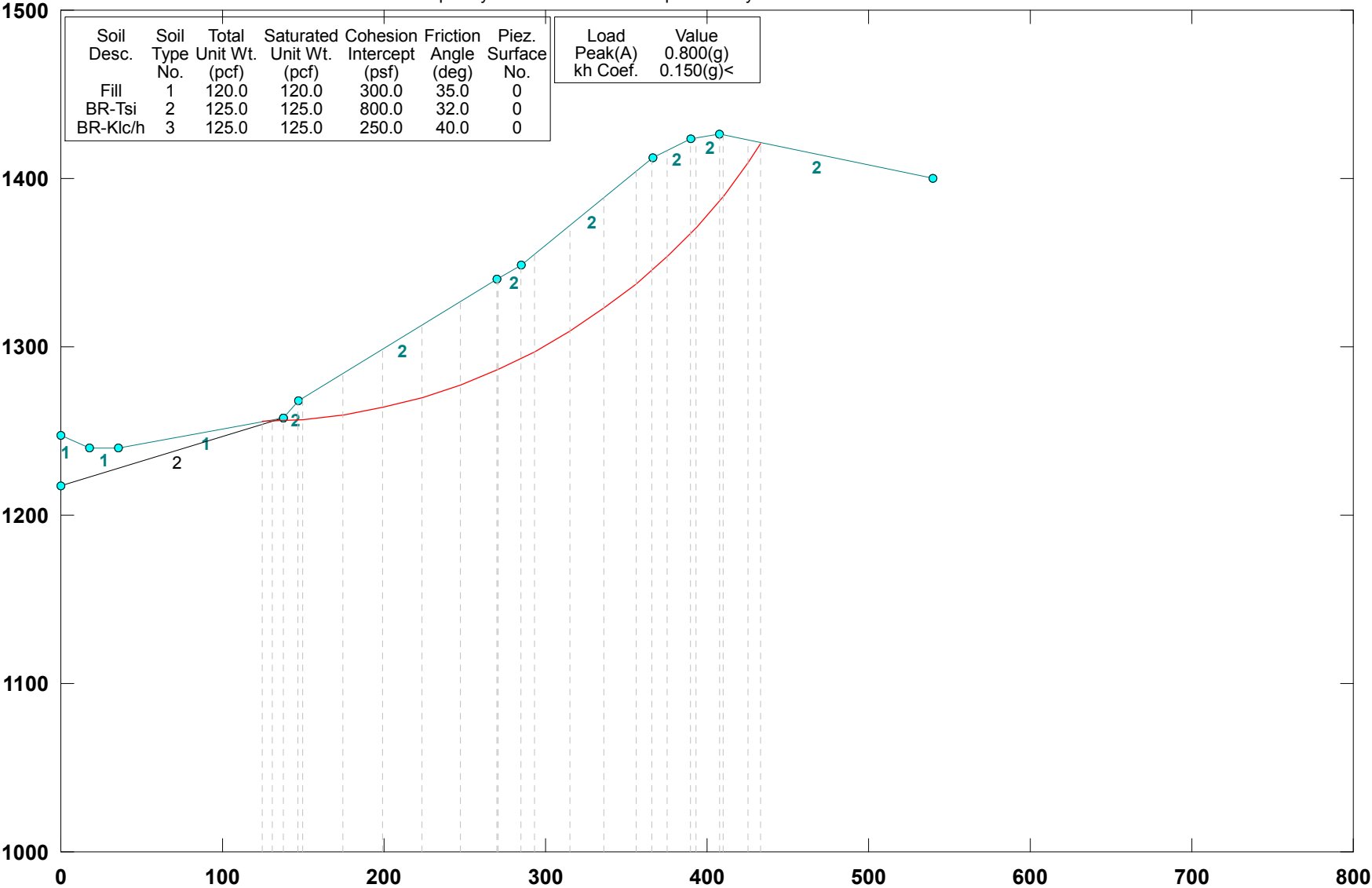


GSTABL7 v.2 FSmin=1.56

Safety Factors Are Calculated By The Modified Bishop Method

Skyline Height, PN:134-02, Section 4-4', Highest Natural Slope, Pseudostatic

c:\slope\skyline\4-4\4-1 surface #1.plt Run By: Username 5/10/2013 11:39AM



GSTABL7 v.2 FSmin=1.18
Factor Of Safety Is Calculated By The Modified Bishop Method

*** GSTABL7 ***

** GSTABL7 by Dr. Garry H. Gregory,
Ph.D., P.E., D.GE **

** Original Version 1.0, January 1996; Current Ver. 2.005.2,
Jan. 2011 **

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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:	5/10/2013
	Time of Run:	11:39AM
	Run By:	Username
#1.in	Input Data Filename:	C:\SLOPE\Skyline\4-4\4-1 Surface
#1.OUT	Output Filename:	C:\SLOPE\Skyline\4-4\4-1 Surface
	Unit System:	English
#1.PLT	Plotted Output Filename:	C:\SLOPE\Skyline\4-4\4-1 Surface

PROBLEM DESCRIPTION: Skyline Height, PN:134-02, Section 4-4',
Highest Natural Slope, Pseudostatic

BOUNDARY COORDINATES

10 Top Boundaries

11 Total Boundaries

Type	Boundary	X-Left	Y-Left	X-Right	Y-Right	Soil
Bnd	No.	(ft)	(ft)	(ft)	(ft)	Below
	1	0.00	1247.00	18.00	1240.00	1
	2	18.00	1240.00	36.00	1240.00	1
	3	36.00	1240.00	138.00	1258.00	1
	4	138.00	1258.00	147.00	1268.00	2
	5	147.00	1268.00	270.00	1340.00	2
	6	270.00	1340.00	285.00	1349.00	2
	7	285.00	1349.00	366.00	1412.00	2
	8	366.00	1412.00	390.00	1424.00	2
	9	390.00	1424.00	408.00	1426.00	2
	10	408.00	1426.00	540.00	1400.00	2
	11	0.00	1217.00	138.00	1258.00	2

User Specified Y-Origin = 1000.00(ft)

Default X-Plus Value = 0.00(ft)

Default Y-Plus Value = 0.00(ft)

1

ISOTROPIC SOIL PARAMETERS

3 Type(s) of Soil

Piez.	Soil	Total	Saturated	Cohesion	Friction	Pore	Pressure
Surface	Type	Unit Wt.	Unit Wt.	Intercept	Angle	Pressure	Constant
No.	No.	(pcf)	(pcf)	(psf)	(deg)	Param.	(psf)
	1	120.0	120.0	300.0	35.0	0.00	0.0
0							
	2	125.0	125.0	800.0	32.0	0.00	0.0
0							
	3	125.0	125.0	250.0	40.0	0.00	0.0
0							

Specified Peak Ground Acceleration Coefficient (A) = 0.800(g)

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

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

Specified Seismic Pore-Pressure Factor = 0.000

1

Trial Failure Surface Specified By 16 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	125.000	1255.706
2	149.977	1256.784
3	174.828	1259.510
4	199.444	1263.873
5	223.718	1269.854
6	247.544	1277.426
7	270.817	1286.557
8	293.435	1297.206
9	315.301	1309.326
10	336.317	1322.866
11	356.392	1337.765
12	375.439	1353.958
13	393.374	1371.375
14	410.118	1389.939
15	425.598	1409.570
16	433.438	1420.990

DEFLECTION ANGLE & SEGMENT DATA FOR SPECIFIED SURFACE(Excluding Last Segment)

Angle/Segment No.	Deflection(Deg)	Segment Length(ft)
1	3.79	25.00
2	3.79	25.00
3	3.79	25.00
4	3.79	25.00
5	3.79	25.00
6	3.79	25.00
7	3.79	25.00
8	3.79	25.00
9	3.79	25.00
10	3.79	25.00
11	3.79	25.00
12	3.79	25.00
13	3.79	25.00

Circle Center At X = 121.410(ft) ; Y = 1632.837(ft); and
Radius = 377.148(ft)

* * Factor Of Safety Is Calculated By The Modified Bishop
Method * *

Factor Of Safety For The Preceding Specified Surface = 1.182

Table 1 - Individual Data on the 23 Slices

Surcharge Slice Load No. (lbs)	Width (ft)	Weight (lbs)	Water Force	Water Force	Tie Force	Tie Force	Earthquake Force	
			Top (lbs)	Bot (lbs)	Norm (lbs)	Tan (lbs)	Hor (lbs)	Ver (lbs)
1	6.2	305.0	0.0	0.0	0.0	0.0	45.7	0.0
0.0								
2	6.8	1076.2	0.0	0.0	0.0	0.0	161.4	0.0
0.0								
3	9.0	7356.2	0.0	0.0	0.0	0.0	1103.4	0.0
0.0								
4	3.0	4521.9	0.0	0.0	0.0	0.0	678.3	0.0
0.0								
5	24.9	58614.6	0.0	0.0	0.0	0.0	8792.2	0.0
0.0								
6	24.6	91702.9	0.0	0.0	0.0	0.0	13755.4	0.0
0.0								
7	24.3	118153.2	0.0	0.0	0.0	0.0	17723.0	0.0
0.0								
8	23.8	137719.0	0.0	0.0	0.0	0.0	20657.9	0.0
0.0								
9	22.5	144830.6	0.0	0.0	0.0	0.0	21724.6	0.0
0.0								
10	0.8	5499.2	0.0	0.0	0.0	0.0	824.9	0.0
0.0								
11	14.2	97241.2	0.0	0.0	0.0	0.0	14586.2	0.0
0.0								
12	8.4	60162.7	0.0	0.0	0.0	0.0	9024.4	0.0
0.0								
13	21.9	166176.0	0.0	0.0	0.0	0.0	24926.4	0.0
0.0								
14	21.0	169820.5	0.0	0.0	0.0	0.0	25473.1	0.0
0.0								

15	20.1	166634.3	0.0	0.0	0.0	0.0	24995.1	0.0
0.0								
16	9.6	79763.6	0.0	0.0	0.0	0.0	11964.5	0.0
0.0								
17	9.4	76000.4	0.0	0.0	0.0	0.0	11400.1	0.0
0.0								
18	14.6	107990.6	0.0	0.0	0.0	0.0	16198.6	0.0
0.0								
19	3.4	22964.6	0.0	0.0	0.0	0.0	3444.7	0.0
0.0								
20	14.6	83559.5	0.0	0.0	0.0	0.0	12533.9	0.0
0.0								
21	2.1	9802.8	0.0	0.0	0.0	0.0	1470.4	0.0
0.0								
22	15.5	47027.6	0.0	0.0	0.0	0.0	7054.1	0.0
0.0								
23	7.8	6352.2	0.0	0.0	0.0	0.0	952.8	0.0
0.0								

Table 2 - Base Stress Data on the 23 Slices

Slice Mobilized No. Stress * (psf)	Alpha (deg)	X-Coord. Slice Cntr (ft)	Base Leng. (ft)	Available Shear Strength (psf)	Shear
1	2.47	128.09	6.18	326.23	
9.52					
2	2.47	134.59	6.83	878.50	
30.37					
3	2.47	142.50	9.01	1281.50	
156.52					
4	2.47	148.49	2.98	1710.12	
289.19					
5	6.26	162.40	25.00	2149.20	
596.45					
6	10.05	187.14	25.00	2859.86	
1160.22					
7	13.84	211.58	25.00	3398.78	
1782.43					
8	17.63	235.63	25.00	3777.23	
2405.28					
9	21.42	258.77	24.12	4000.33	
2969.64					
10	21.42	270.41	0.88	4146.02	
3096.46					
11	25.21	277.91	15.68	4070.90	
3416.45					
12	25.21	289.22	9.32	4209.16	
3551.31					
13	29.00	304.37	25.00	4291.29	
4014.09					

14	32.79	325.81	25.00	4363.11
4448.19				
15	36.58	346.35	25.00	4299.69
4687.22				
16	40.37	361.20	12.61	4130.82
4736.09				
17	40.37	370.72	12.39	4023.03
4595.88				
18	44.16	382.72	20.30	3590.74
4216.38				
19	44.16	391.69	4.70	3338.85
3874.28				
20	47.95	400.69	21.84	2755.03
3191.04				
21	47.95	409.06	3.16	2327.68
2590.49				
22	51.74	417.86	25.00	1615.32
1642.73				
23	55.53	429.52	13.85	737.98
416.40				

Sum of the Resisting Forces (including Pier/Pile, Tieback,
Reinforcing
Soil Nail, and Applied Forces if applicable) = 1165468.50 (lbs)

Average Available Shear Strength (including Tieback, Pier/Pile,
Reinforcing,
Soil Nail, and Applied Forces if applicable) = 3203.14(psf)

Sum of the Driving Forces = 986084.81 (lbs)

Average Mobilized Shear Stress = 2710.13(psf)

Total length of the failure surface = 363.85(ft)

CAUTION - Factor Of Safety Is Calculated By The Modified
Bishop
Method. This Method Is Valid Only If The Failure
Surface
Approximates A Circular Arc.

*** SEISMIC SLOPE DISPLACEMENT DATA ***
(Note: kv is set = zero for displacement calculations)
Seismic Yield Coefficient (ky) = 0.2538(g)
Calculated Newmark Seismic Displacement = 0.771(ft)

Non-Symmetrical Sliding Resistance Has Been Specified

for Downhill Sliding.

**** END OF GSTABL7 OUTPUT ****

*** GSTABL7 ***

** GSTABL7 by Dr. Garry H. Gregory,
Ph.D., P.E., D.GE **

** Original Version 1.0, January 1996; Current Ver. 2.005.2,
Jan. 2011 **

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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: 5/10/2013
Time of Run: 11:37AM
Run By: Username
Input Data Filename: C:\SLOPE\Skyline\4-4\4-1.in
Output Filename: C:\SLOPE\Skyline\4-4\4-1.OUT
Unit System: English

Plotted Output Filename: C:\SLOPE\Skyline\4-4\4-1.PLT

PROBLEM DESCRIPTION: Skyline Height, PN:134-02, Section 4-4',
Highest Natural Slope, Static

BOUNDARY COORDINATES

10 Top Boundaries
11 Total Boundaries

Type	Boundary	X-Left	Y-Left	X-Right	Y-Right	Soil
Bnd	No.	(ft)	(ft)	(ft)	(ft)	Below
	1	0.00	1247.00	18.00	1240.00	1
	2	18.00	1240.00	36.00	1240.00	1
	3	36.00	1240.00	138.00	1258.00	1
	4	138.00	1258.00	147.00	1268.00	2
	5	147.00	1268.00	270.00	1340.00	2
	6	270.00	1340.00	285.00	1349.00	2
	7	285.00	1349.00	366.00	1412.00	2
	8	366.00	1412.00	390.00	1424.00	2
	9	390.00	1424.00	408.00	1426.00	2
	10	408.00	1426.00	540.00	1400.00	2
	11	0.00	1217.00	138.00	1258.00	2

User Specified Y-Origin = 1000.00 (ft)

Default X-Plus Value = 0.00 (ft)

Default Y-Plus Value = 0.00 (ft)

1

ISOTROPIC SOIL PARAMETERS

3 Type(s) of Soil

Piez.	Soil	Total	Saturated	Cohesion	Friction	Pore	Pressure
Surface	Type	Unit Wt.	Unit Wt.	Intercept	Angle	Pressure	Constant
No.	No.	(pcf)	(pcf)	(psf)	(deg)	Param.	(psf)

0	1	120.0	120.0	300.0	35.0	0.00	0.0
0	2	125.0	125.0	800.0	32.0	0.00	0.0
0	3	125.0	125.0	250.0	40.0	0.00	0.0
1							

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Circular Surfaces, Has Been Specified.

2500 Trial Surfaces Have Been Generated.

9	315.301	1309.326
10	336.317	1322.866
11	356.392	1337.765
12	375.439	1353.958
13	393.374	1371.375
14	410.118	1389.939
15	425.598	1409.570
16	433.438	1420.990

Circle Center At X = 121.412 ; Y = 1632.833 ; and Radius = 377.144

Factor of Safety
*** 1.559 ***

Individual data on the 23 slices

Surcharge Slice Load No. (lbs)	Width (ft)	Weight (lbs)	Water Force		Tie Force		Earthquake Force	
			Top (lbs)	Bot (lbs)	Norm (lbs)	Tan (lbs)	Hor (lbs)	Ver (lbs)
1	6.2	304.9	0.0	0.0	0.	0.	0.0	0.0
0.0								
2	6.8	1076.2	0.0	0.0	0.	0.	0.0	0.0
0.0								
3	9.0	7356.2	0.0	0.0	0.	0.	0.0	0.0
0.0								
4	3.0	4521.5	0.0	0.0	0.	0.	0.0	0.0
0.0								
5	24.9	58613.6	0.0	0.0	0.	0.	0.0	0.0
0.0								
6	24.6	91703.1	0.0	0.0	0.	0.	0.0	0.0
0.0								
7	24.3	118153.2	0.0	0.0	0.	0.	0.0	0.0
0.0								
8	23.8	137716.4	0.0	0.0	0.	0.	0.0	0.0
0.0								
9	22.5	144832.1	0.0	0.0	0.	0.	0.0	0.0
0.0								
10	0.8	5497.5	0.0	0.0	0.	0.	0.0	0.0
0.0								
11	14.2	97243.1	0.0	0.0	0.	0.	0.0	0.0
0.0								
12	8.4	60166.0	0.0	0.0	0.	0.	0.0	0.0
0.0								

13	21.9	166171.4	0.0	0.0	0.	0.	0.0	0.0
0.0								
14	21.0	169822.9	0.0	0.0	0.	0.	0.0	0.0
0.0								
15	20.1	166639.1	0.0	0.0	0.	0.	0.0	0.0
0.0								
16	9.6	79761.2	0.0	0.0	0.	0.	0.0	0.0
0.0								
17	9.4	76001.7	0.0	0.0	0.	0.	0.0	0.0
0.0								
18	14.6	107990.2	0.0	0.0	0.	0.	0.0	0.0
0.0								
19	3.4	22962.9	0.0	0.0	0.	0.	0.0	0.0
0.0								
20	14.6	83560.6	0.0	0.0	0.	0.	0.0	0.0
0.0								
21	2.1	9802.4	0.0	0.0	0.	0.	0.0	0.0
0.0								
22	15.5	47029.3	0.0	0.0	0.	0.	0.0	0.0
0.0								
23	7.8	6352.1	0.0	0.0	0.	0.	0.0	0.0
0.0								

Failure Surface Specified By 16 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	108.333	1252.765
2	133.246	1254.849
3	157.996	1258.375
4	182.500	1263.332
5	206.675	1269.703
6	230.439	1277.465
7	253.713	1286.594
8	276.417	1297.058
9	298.477	1308.822
10	319.816	1321.846
11	340.364	1336.087
12	360.050	1351.496
13	378.809	1368.021
14	396.578	1385.608
15	413.296	1404.196
16	427.636	1422.132

Circle Center At X = 84.977 ; Y = 1682.466 ; and Radius = 430.336

Factor of Safety
 *** 1.562 ***

Failure Surface Specified By 16 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	120.833	1254.971
2	145.830	1255.349
3	170.742	1257.445
4	195.451	1261.250
5	219.840	1266.745
6	243.793	1273.905
7	267.197	1282.694
8	289.941	1293.073
9	311.917	1304.990
10	333.022	1318.392
11	353.155	1333.212
12	372.221	1349.383
13	390.130	1366.826
14	406.797	1385.459
15	422.144	1405.195
16	432.845	1421.106

Circle Center At X = 127.969 ; Y = 1617.626 ; and Radius = 362.725

Factor of Safety
*** 1.562 ***

Failure Surface Specified By 16 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	100.000	1251.294
2	124.970	1252.528
3	149.806	1255.379
4	174.406	1259.836
5	198.664	1265.880
6	222.479	1273.485
7	245.751	1282.620
8	268.380	1293.246
9	290.273	1305.318
10	311.335	1318.784
11	331.480	1333.590
12	350.621	1349.671

13	368.679	1366.960
14	385.577	1385.385
15	401.243	1404.868
16	415.103	1424.601

Circle Center At X = 93.521 ; Y = 1636.183 ; and Radius = 384.944

Factor of Safety
*** 1.569 ***

1

Failure Surface Specified By 17 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	91.667	1249.823
2	116.663	1249.384
3	141.633	1250.597
4	166.469	1253.458
5	191.062	1257.953
6	215.303	1264.064
7	239.088	1271.764
8	262.312	1281.018
9	284.874	1291.787
10	306.674	1304.023
11	327.619	1317.674
12	347.616	1332.678
13	366.577	1348.971
14	384.420	1366.482
15	401.067	1385.133
16	416.445	1404.844
17	428.118	1422.037

Circle Center At X = 110.810 ; Y = 1627.444 ; and Radius = 378.106

Factor of Safety
*** 1.571 ***

Failure Surface Specified By 17 Coordinate Points

Point	X-Surf	Y-Surf
-------	--------	--------

No.	(ft)	(ft)
1	116.667	1254.235
2	141.667	1254.197
3	166.612	1255.854
4	191.387	1259.199
5	215.878	1264.215
6	239.974	1270.881
7	263.561	1279.164
8	286.533	1289.028
9	308.784	1300.426
10	330.210	1313.307
11	350.714	1327.610
12	370.201	1343.271
13	388.581	1360.217
14	405.770	1378.370
15	421.689	1397.647
16	436.265	1417.958
17	437.631	1420.164

Circle Center At X = 129.724 ; Y = 1622.545 ; and Radius = 368.542

Factor of Safety
 *** 1.574 ***

1

Failure Surface Specified By 17 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	87.500	1249.088
2	112.490	1248.380
3	137.471	1249.362
4	162.328	1252.030
5	186.948	1256.372
6	211.218	1262.368
7	235.028	1269.991
8	258.268	1279.206
9	280.832	1289.970
10	302.617	1302.234
11	323.523	1315.943
12	343.455	1331.033
13	362.321	1347.436
14	380.036	1365.076
15	396.518	1383.874
16	411.692	1403.742
17	424.297	1422.790

369.694 Circle Center At X = 110.465 ; Y = 1618.069 ; and Radius =

Factor of Safety
*** 1.574 ***

Failure Surface Specified By 16 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	100.000	1251.294
2	124.999	1251.135
3	149.941	1252.840
4	174.687	1256.398
5	199.098	1261.791
6	223.040	1268.988
7	246.379	1277.949
8	268.985	1288.625
9	290.732	1300.955
10	311.501	1314.872
11	331.174	1330.298
12	349.643	1347.147
13	366.805	1365.326
14	382.565	1384.733
15	396.835	1405.260
16	408.956	1425.812

335.111 Circle Center At X = 114.656 ; Y = 1586.085 ; and Radius =

Factor of Safety
*** 1.575 ***

1

Failure Surface Specified By 15 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	141.667	1262.074
2	166.665	1261.758
3	191.598	1263.585

4	216.283	1267.541
5	240.538	1273.596
6	264.186	1281.707
7	287.052	1291.814
8	308.968	1303.842
9	329.774	1317.703
10	349.316	1333.296
11	367.450	1350.505
12	384.044	1369.203
13	398.975	1389.255
14	412.133	1410.512
15	418.887	1423.856

Circle Center At X = 157.844 ; Y = 1553.224 ; and Radius = 291.599

Factor of Safety
*** 1.575 ***

Failure Surface Specified By 16 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	100.000	1251.294
2	124.945	1252.956
3	149.734	1256.199
4	174.267	1261.008
5	198.445	1267.365
6	222.171	1275.244
7	245.349	1284.613
8	267.886	1295.434
9	289.690	1307.664
10	310.674	1321.253
11	330.753	1336.147
12	349.846	1352.286
13	367.877	1369.603
14	384.771	1388.031
15	400.462	1407.493
16	412.859	1425.043

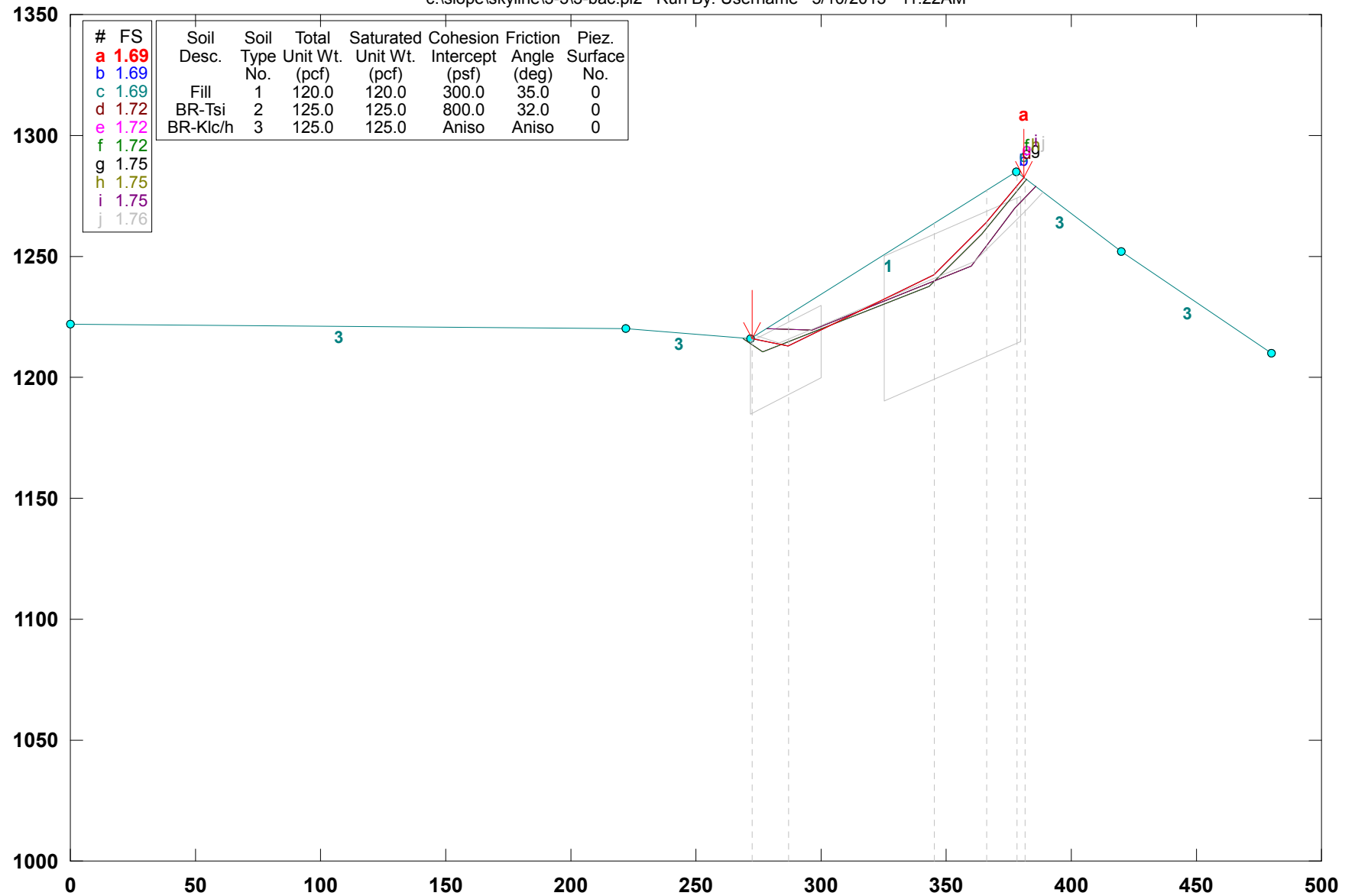
Circle Center At X = 86.419 ; Y = 1644.297 ; and Radius = 393.238

Factor of Safety
*** 1.577 ***

**** END OF GSTABL7 OUTPUT ****

Skyline Height, PN:134-02, Section 3-3', Temporary Backcut, Static

c:\slope\skyline\3-3\3-bac.pl2 Run By: Username 5/10/2013 11:22AM



GSTABL7 v.2 FSmin=1.69

Safety Factors Are Calculated By The Simplified Janbu Method for the case of c & phi both > 0

*** GSTABL7 ***

** GSTABL7 by Dr. Garry H. Gregory,
Ph.D., P.E., D.GE **

** Original Version 1.0, January 1996; Current Ver. 2.005.2,
Jan. 2011 **

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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: 5/10/2013
Time of Run: 11:22AM
Run By: Username
Input Data Filename: C:\SLOPE\Skyline\3-3\3-bac.in
Output Filename: C:\SLOPE\Skyline\3-3\3-bac.OUT
Unit System: English

Plotted Output Filename: C:\SLOPE\Skyline\3-3\3-bac.PLT

PROBLEM DESCRIPTION: Skyline Height, PN:134-02, Section 3-3',
Temporary Backcut, Static

BOUNDARY COORDINATES

5 Top Boundaries
5 Total Boundaries

Type	Boundary	X-Left	Y-Left	X-Right	Y-Right	Soil
Bnd	No.	(ft)	(ft)	(ft)	(ft)	Below
	1	0.00	1222.00	222.00	1220.00	3
	2	222.00	1220.00	272.00	1216.00	3
	3	272.00	1216.00	378.00	1285.00	1
	4	378.00	1285.00	420.00	1252.00	3
	5	420.00	1252.00	480.00	1210.00	3

User Specified Y-Origin = 1000.00 (ft)

Default X-Plus Value = 0.00 (ft)

Default Y-Plus Value = 0.00 (ft)

1

ISOTROPIC SOIL PARAMETERS

3 Type(s) of Soil

Piez.	Soil	Total	Saturated	Cohesion	Friction	Pore	Pressure
Surface	Type	Unit Wt.	Unit Wt.	Intercept	Angle	Pressure	Constant
No.	No.	(pcf)	(pcf)	(psf)	(deg)	Param.	(psf)
	1	120.0	120.0	300.0	35.0	0.00	0.0
0	2	125.0	125.0	800.0	32.0	0.00	0.0
0	3	125.0	125.0	250.0	40.0	0.00	0.0
0							

ANISOTROPIC STRENGTH PARAMETERS

1 soil type(s)

Soil Type 3 Is Anisotropic

Number Of Direction Ranges Specified = 3

Direction	Counterclockwise	Cohesion	Friction
Range	Direction Limit	Intercept	Angle
No.	(deg)	(psf)	(deg)

1	16.0	250.00	40.00
2	20.0	100.00	20.00
3	90.0	250.00	40.00

ANISOTROPIC SOIL NOTES:

- (1) An input value of 0.01 for C and/or Phi will cause Aniso C and/or Phi to be ignored in that range.
- (2) An input value of 0.02 for Phi will set both Phi and C equal to zero, with no water weight in the tension crack.
- (3) An input value of 0.03 for Phi will set both Phi and C equal to zero, with water weight in the tension crack.

Janbus Empirical Coef is being used for the case of c & phi both > 0
1

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Sliding Block Surfaces, Has Been Specified.

1500 Trial Surfaces Have Been Generated.

2 Boxes Specified For Generation Of Central Block Base

Length Of Line Segments For Active And Passive Portions Of Sliding Block Is 30.0

Box No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Height (ft)
1	272.00	1200.00	300.00	1215.00	30.00
2	325.00	1220.00	380.00	1245.00	60.00

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 Simplified Janbu Method * *

Total Number of Trial Surfaces Attempted = 1500

3	20.9	42310.7	0.0	0.0	0.	0.	0.0	0.0
0.0								
4	12.1	13938.1	0.0	0.0	0.	0.	0.0	0.0
0.0								
5	3.2	1237.4	0.0	0.0	0.	0.	0.0	0.0
0.0								

Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	272.152	1216.099
2	286.737	1212.939
3	345.004	1242.668
4	365.889	1264.205
5	381.158	1282.519

Factor of Safety
*** 1.690 ***

1

Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	272.152	1216.099
2	286.737	1212.939
3	345.004	1242.668
4	365.889	1264.205
5	381.158	1282.519

Factor of Safety
*** 1.690 ***

Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	268.827	1216.254
2	276.886	1210.848

3	343.164	1237.774
4	364.295	1259.070
5	382.043	1281.823

Factor of Safety
 *** 1.721 ***

1

Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	268.827	1216.254
2	276.886	1210.848
3	343.164	1237.774
4	364.295	1259.070
5	382.043	1281.823

Factor of Safety
 *** 1.721 ***

Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	268.827	1216.254
2	276.886	1210.848
3	343.164	1237.774
4	364.295	1259.070
5	382.043	1281.823

Factor of Safety
 *** 1.721 ***

1

Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	278.680	1220.348
2	296.601	1219.447
3	359.976	1245.709
4	377.424	1270.113
5	385.950	1278.754

Factor of Safety
*** 1.751 ***

Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	278.680	1220.348
2	296.601	1219.447
3	359.976	1245.709
4	377.424	1270.113
5	385.950	1278.754

Factor of Safety
*** 1.751 ***

1

Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	278.680	1220.348
2	296.601	1219.447
3	359.976	1245.709
4	377.424	1270.113
5	385.950	1278.754

Factor of Safety
*** 1.751 ***

Failure Surface Specified By 5 Coordinate Points

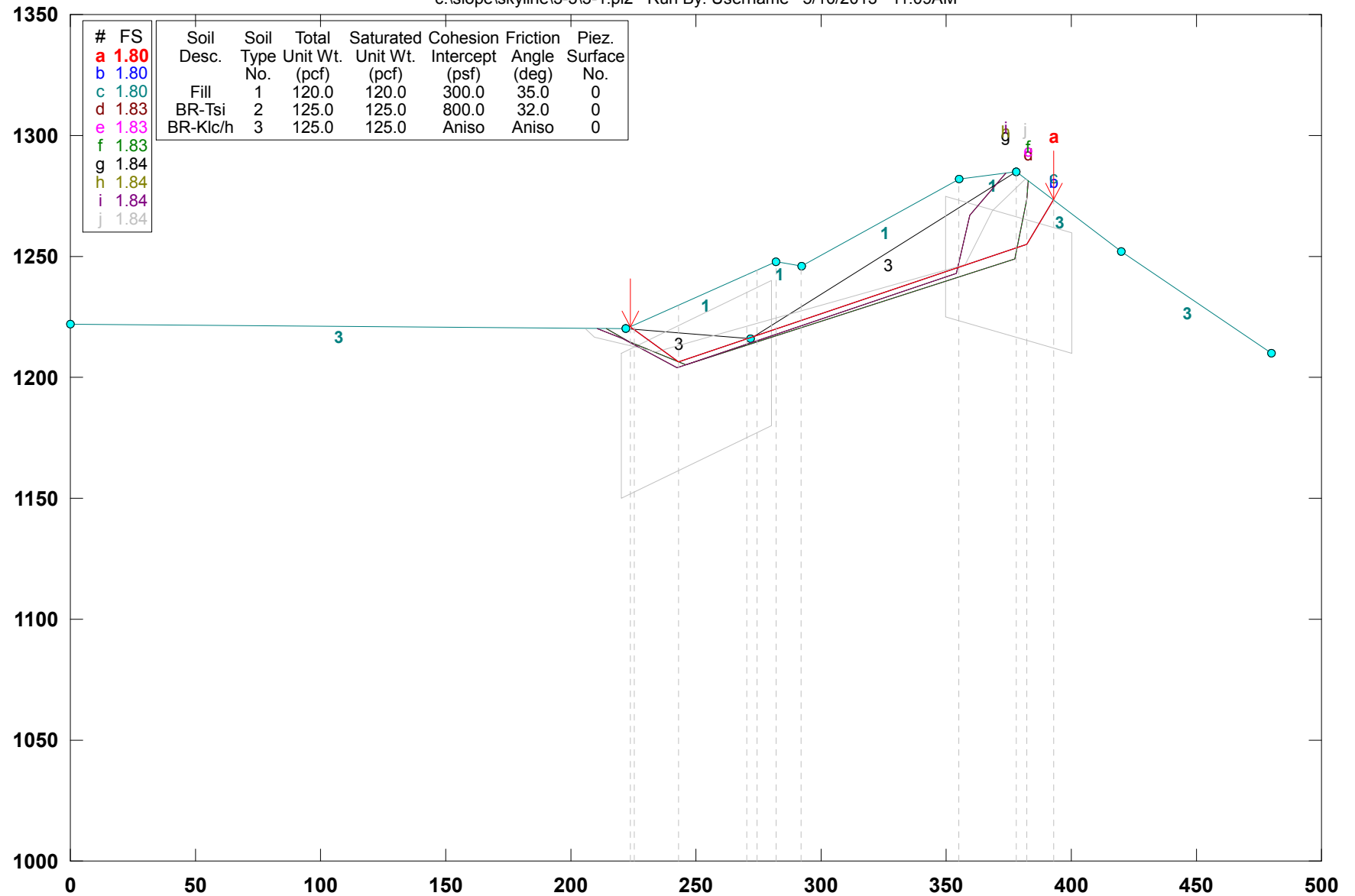
Point No.	X-Surf (ft)	Y-Surf (ft)
1	274.024	1217.318
2	283.490	1214.020
3	361.077	1247.762
4	382.077	1269.186
5	388.933	1276.410

Factor of Safety
*** 1.756 ***

**** END OF GSTABL7 OUTPUT ****

Skyline Height, PN:134-02, Section 3-3',Buttressed Slope, Static

c:\slope\skyline\3-3\3-1.pl2 Run By: Username 5/10/2013 11:09AM

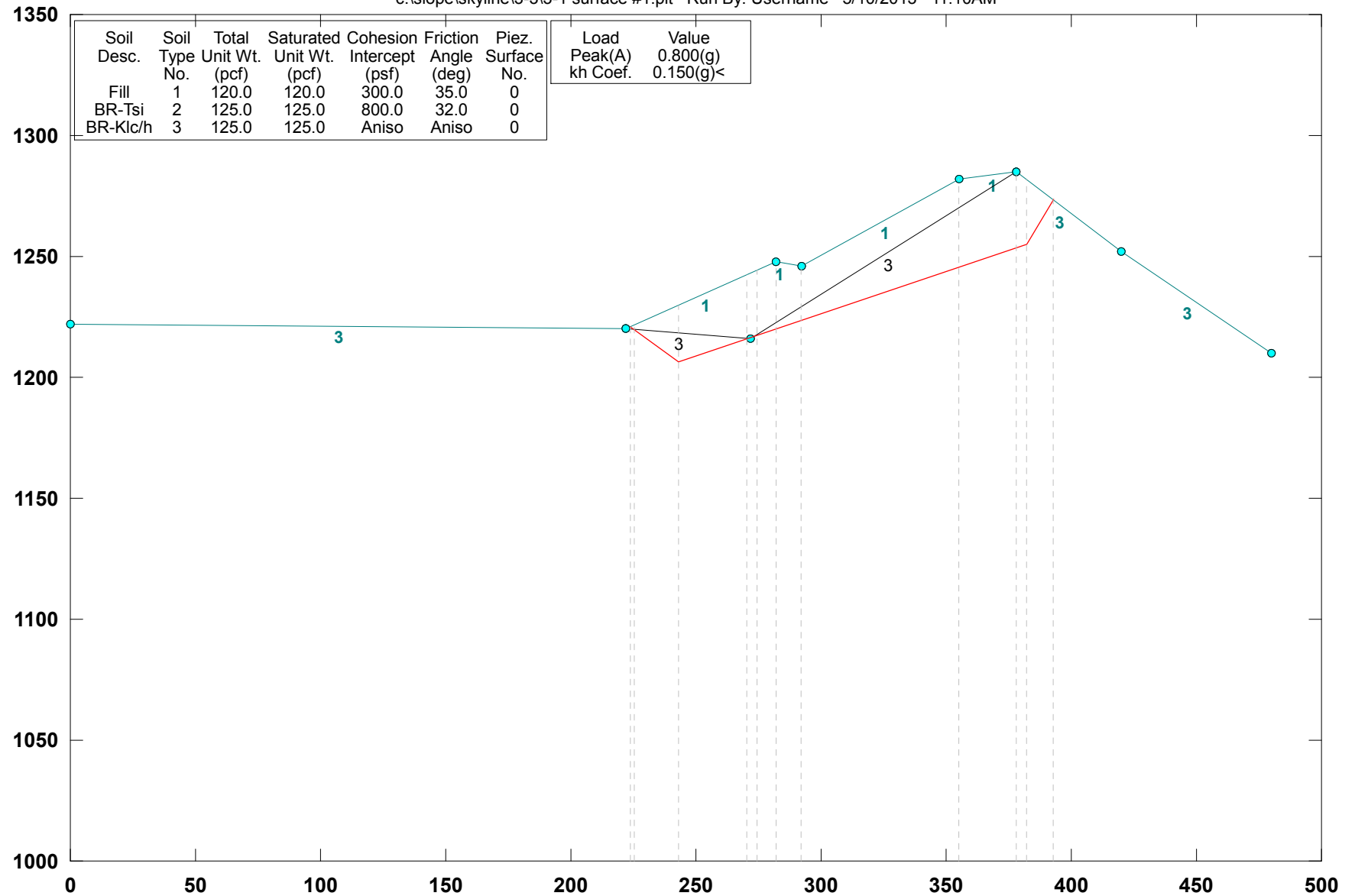


GSTABL7 v.2 FSmin=1.80

Safety Factors Are Calculated By The Simplified Janbu Method for the case of c & phi both > 0

Skyline Height, PN:134-02, Section 3-3',Buttressed Slope, Pseudostatic

c:\slope\skyline\3-3\3-1 surface #1.plt Run By: Username 5/10/2013 11:10AM



GSTABL7 v.2 FSmin=1.28

Factor Of Safety Is Calculated By The Simplified Janbu Method

*** GSTABL7 ***

** GSTABL7 by Dr. Garry H. Gregory,
Ph.D., P.E., D.GE **

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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:	5/10/2013
	Time of Run:	11:10AM
	Run By:	Username
#1.in	Input Data Filename:	C:\SLOPE\Skyline\3-3\3-1 Surface
#1.OUT	Output Filename:	C:\SLOPE\Skyline\3-3\3-1 Surface
	Unit System:	English
#1.PLT	Plotted Output Filename:	C:\SLOPE\Skyline\3-3\3-1 Surface

PROBLEM DESCRIPTION: Skyline Height, PN:134-02, Section 3-3',
Buttressed Slope, Pseudostatic

BOUNDARY COORDINATES

7 Top Boundaries

9 Total Boundaries

Type	Boundary	X-Left	Y-Left	X-Right	Y-Right	Soil
Bnd	No.	(ft)	(ft)	(ft)	(ft)	Below
	1	0.00	1222.00	222.00	1220.00	3
	2	222.00	1220.00	282.00	1248.00	1
	3	282.00	1248.00	292.00	1246.00	1
	4	292.00	1246.00	355.00	1282.00	1
	5	355.00	1282.00	378.00	1285.00	1
	6	378.00	1285.00	420.00	1252.00	3
	7	420.00	1252.00	480.00	1210.00	3
	8	222.00	1220.00	272.00	1216.00	3
	9	272.00	1216.00	378.00	1285.00	3

User Specified Y-Origin = 1000.00 (ft)

Default X-Plus Value = 0.00 (ft)

Default Y-Plus Value = 0.00 (ft)

1

ISOTROPIC SOIL PARAMETERS

3 Type(s) of Soil

Piez.	Soil	Total	Saturated	Cohesion	Friction	Pore	Pressure
Surface	Type	Unit Wt.	Unit Wt.	Intercept	Angle	Pressure	Constant
No.	No.	(pcf)	(pcf)	(psf)	(deg)	Param.	(psf)
	1	120.0	120.0	300.0	35.0	0.00	0.0
0	2	125.0	125.0	800.0	32.0	0.00	0.0
0	3	125.0	125.0	250.0	40.0	0.00	0.0
0							

ANISOTROPIC STRENGTH PARAMETERS

1 soil type(s)

Soil Type 3 Is Anisotropic

Number Of Direction Ranges Specified = 3

Direction Range No.	Counterclockwise Direction Limit (deg)	Cohesion Intercept (psf)	Friction Angle (deg)
1	16.0	250.00	40.00
2	20.0	100.00	20.00
3	90.0	250.00	40.00

ANISOTROPIC SOIL NOTES:

- (1) An input value of 0.01 for C and/or Phi will cause Aniso C and/or Phi to be ignored in that range.
- (2) An input value of 0.02 for Phi will set both Phi and C equal to zero, with no water weight in the tension crack.
- (3) An input value of 0.03 for Phi will set both Phi and C equal to zero, with water weight in the tension crack.

Specified Peak Ground Acceleration Coefficient (A) = 0.800(g)
Specified Horizontal Earthquake Coefficient (kh) = 0.150(g)
Specified Vertical Earthquake Coefficient (kv) = 0.000(g)

Specified Seismic Pore-Pressure Factor = 0.000

Janbu's Empirical Coef. is being used for the case of c & phi both > 0
1

Trial Failure Surface Specified By 4 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	223.904	1220.889
2	243.024	1206.640
3	382.150	1255.086
4	392.884	1273.305

Janbu's Empirical Coefficient (fo) = 1.046

* * Factor Of Safety Is Calculated By The Simplified Janbu Method * *

Factor Of Safety For The Preceding Specified Surface = 1.280

Table 1 - Individual Data on the 10 Slices

Surcharge Slice Load No. (lbs)	Width (ft)	Weight (lbs)	Water Force	Water Force	Tie Force	Tie Force	Earthquake Force	
			Top (lbs)	Bot (lbs)	Norm (lbs)	Tan (lbs)	Hor (lbs)	Ver (lbs)
1	1.6	178.0	0.0	0.0	0.0	0.0	26.7	0.0
0.0								
2	17.6	26916.4	0.0	0.0	0.0	0.0	4037.5	0.0
0.0								
3	27.3	81911.4	0.0	0.0	0.0	0.0	12286.7	0.0
0.0								
4	4.1	13159.7	0.0	0.0	0.0	0.0	1974.0	0.0
0.0								
5	7.6	24939.7	0.0	0.0	0.0	0.0	3741.0	0.0
0.0								
6	10.0	30246.8	0.0	0.0	0.0	0.0	4537.0	0.0
0.0								
7	63.0	226468.1	0.0	0.0	0.0	0.0	33970.2	0.0
0.0								
8	23.0	96669.6	0.0	0.0	0.0	0.0	14500.4	0.0
0.0								
9	4.1	15047.0	0.0	0.0	0.0	0.0	2257.0	0.0
0.0								
10	10.7	17881.0	0.0	0.0	0.0	0.0	2682.2	0.0
0.0								

Table 2 - Base Stress Data on the 10 Slices

Slice Mobilized Stress * (psf)	Alpha (deg)	X-Coord.	Base	Available	
		Slice Cntr (ft)	Leng. (ft)	Shear Strength (psf)	Shear
1	-36.69	224.69	1.95	799.36	-
54.30					
2	-36.69	234.25	21.89	3746.54	-
731.76					
3	19.20	256.66	28.88	1149.65	
1413.19					
4	19.20	272.35	4.36	2258.23	
1504.41					

5	19.20	278.21	8.04	1248.82
1546.26				
6	19.20	287.00	10.59	1157.05
1423.13				
7	19.20	323.50	66.71	1356.96
1691.35				
8	19.20	366.50	24.35	1570.28
1977.55				
9	19.20	380.08	4.39	1367.85
1705.95				
10	59.50	387.52	21.15	1536.53
1562.10				

Sum of the Resisting Forces (including Pier/Pile, Tieback,
Reinforcing
Soil Nail, and Applied Forces if applicable) = 318429.09 (lbs)

Average Available Shear Strength (including Tieback, Pier/Pile,
Reinforcing,
Soil Nail, and Applied Forces if applicable) = 1655.80(psf)

Sum of the Driving Forces = 260255.58 (lbs)

Average Mobilized Shear Stress = 1353.30(psf)

Total length of the failure surface = 192.31(ft)

*** SEISMIC SLOPE DISPLACEMENT DATA ***
(Note: kv is set = zero for displacement calculations)
Seismic Yield Coefficient (ky) = 0.3376(g)
Calculated Newmark Seismic Displacement = 0.436(ft)

Non-Symmetrical Sliding Resistance Has Been Specified
for Downhill Sliding.

**** END OF GSTABL7 OUTPUT ****

*** GSTABL7 ***

** GSTABL7 by Dr. Garry H. Gregory,
Ph.D., P.E., D.GE **

** Original Version 1.0, January 1996; Current Ver. 2.005.2,
Jan. 2011 **

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Prohibited)

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: 5/10/2013
Time of Run: 11:09AM
Run By: Username
Input Data Filename: C:\SLOPE\Skyline\3-3\3-1.in
Output Filename: C:\SLOPE\Skyline\3-3\3-1.OUT
Unit System: English

Plotted Output Filename: C:\SLOPE\Skyline\3-3\3-1.PLT

PROBLEM DESCRIPTION: Skyline Height, PN:134-02, Section 3-3',
Buttressed Slope, Static

BOUNDARY COORDINATES

7 Top Boundaries
9 Total Boundaries

Type	Boundary	X-Left	Y-Left	X-Right	Y-Right	Soil
Bnd	No.	(ft)	(ft)	(ft)	(ft)	Below
	1	0.00	1222.00	222.00	1220.00	3
	2	222.00	1220.00	282.00	1248.00	1
	3	282.00	1248.00	292.00	1246.00	1
	4	292.00	1246.00	355.00	1282.00	1
	5	355.00	1282.00	378.00	1285.00	1
	6	378.00	1285.00	420.00	1252.00	3
	7	420.00	1252.00	480.00	1210.00	3
	8	222.00	1220.00	272.00	1216.00	3
	9	272.00	1216.00	378.00	1285.00	3

User Specified Y-Origin = 1000.00 (ft)

Default X-Plus Value = 0.00 (ft)

Default Y-Plus Value = 0.00 (ft)

1

ISOTROPIC SOIL PARAMETERS

3 Type(s) of Soil

Piez.	Soil	Total	Saturated	Cohesion	Friction	Pore	Pressure
Surface	Type	Unit Wt.	Unit Wt.	Intercept	Angle	Pressure	Constant
No.	No.	(pcf)	(pcf)	(psf)	(deg)	Param.	(psf)
	1	120.0	120.0	300.0	35.0	0.00	0.0
0							
	2	125.0	125.0	800.0	32.0	0.00	0.0
0							
	3	125.0	125.0	250.0	40.0	0.00	0.0
0							

ANISOTROPIC STRENGTH PARAMETERS

1 soil type(s)

Soil Type 3 Is Anisotropic

Number Of Direction Ranges Specified = 3

Direction Range No.	Counterclockwise Direction Limit (deg)	Cohesion Intercept (psf)	Friction Angle (deg)
1	16.0	250.00	40.00
2	20.0	100.00	20.00
3	90.0	250.00	40.00

ANISOTROPIC SOIL NOTES:

- (1) An input value of 0.01 for C and/or Phi will cause Aniso C and/or Phi to be ignored in that range.
- (2) An input value of 0.02 for Phi will set both Phi and C equal to zero, with no water weight in the tension crack.
- (3) An input value of 0.03 for Phi will set both Phi and C equal to zero, with water weight in the tension crack.

Janbus Empirical Coef is being used for the case of c & phi both > 0
1

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Sliding Block Surfaces, Has Been Specified.

1500 Trial Surfaces Have Been Generated.

2 Boxes Specified For Generation Of Central Block Base

Length Of Line Segments For Active And Passive Portions Of Sliding Block Is 25.0

Box No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Height (ft)
1	220.00	1180.00	280.00	1210.00	60.00
2	350.00	1250.00	400.00	1235.00	50.00

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 Simplified Janbu Method * *

			Water Force	Water Force	Tie Force	Tie Force	Earthquake Force	
Surcharge Slice Load	Width	Weight	Top	Bot	Norm	Tan	Hor	Ver
No. (lbs)	(ft)	(lbs)	(lbs)	(lbs)	(lbs)	(lbs)	(lbs)	(lbs)
1 0.0	1.6	178.0	0.0	0.0	0.	0.	0.0	0.0
2 0.0	17.6	26916.2	0.0	0.0	0.	0.	0.0	0.0
3 0.0	27.3	81911.8	0.0	0.0	0.	0.	0.0	0.0
4 0.0	4.1	13160.0	0.0	0.0	0.	0.	0.0	0.0
5 0.0	7.6	24939.5	0.0	0.0	0.	0.	0.0	0.0
6 0.0	10.0	30246.8	0.0	0.0	0.	0.	0.0	0.0

7	63.0	226469.1	0.0	0.0	0.	0.	0.0	0.0
0.0								
8	23.0	96670.3	0.0	0.0	0.	0.	0.0	0.0
0.0								
9	4.2	15048.0	0.0	0.0	0.	0.	0.0	0.0
0.0								
10	10.7	17881.1	0.0	0.0	0.	0.	0.0	0.0
0.0								

Failure Surface Specified By 4 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	223.904	1220.888
2	243.024	1206.640
3	382.150	1255.086
4	392.884	1273.305

Factor of Safety
*** 1.798 ***

1

Failure Surface Specified By 4 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	223.904	1220.888
2	243.024	1206.640
3	382.150	1255.086
4	392.884	1273.305

Factor of Safety
*** 1.798 ***

Failure Surface Specified By 6 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	214.495	1220.068
2	223.125	1214.636

3	246.202	1205.021
4	377.475	1249.225
5	382.127	1273.788
6	382.754	1281.265

Factor of Safety
 *** 1.834 ***

1

Failure Surface Specified By 6 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	214.495	1220.068
2	223.125	1214.636
3	246.202	1205.021
4	377.475	1249.225
5	382.127	1273.788
6	382.754	1281.265

Factor of Safety
 *** 1.834 ***

Failure Surface Specified By 6 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	214.495	1220.068
2	223.125	1214.636
3	246.202	1205.021
4	377.475	1249.225
5	382.127	1273.788
6	382.754	1281.265

Factor of Safety
 *** 1.834 ***

1

Failure Surface Specified By 6 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	210.455	1220.104
2	220.579	1216.252
3	242.218	1203.732
4	353.842	1242.838
5	359.649	1267.155
6	373.674	1284.436

Factor of Safety
*** 1.839 ***

Failure Surface Specified By 6 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	210.455	1220.104
2	220.579	1216.252
3	242.218	1203.732
4	353.842	1242.838
5	359.649	1267.155
6	373.674	1284.436

Factor of Safety
*** 1.839 ***

Failure Surface Specified By 6 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	210.455	1220.104
2	220.579	1216.252
3	242.218	1203.732
4	353.842	1242.838
5	359.649	1267.155
6	373.674	1284.436

Factor of Safety
*** 1.839 ***

Failure Surface Specified By 6 Coordinate Points

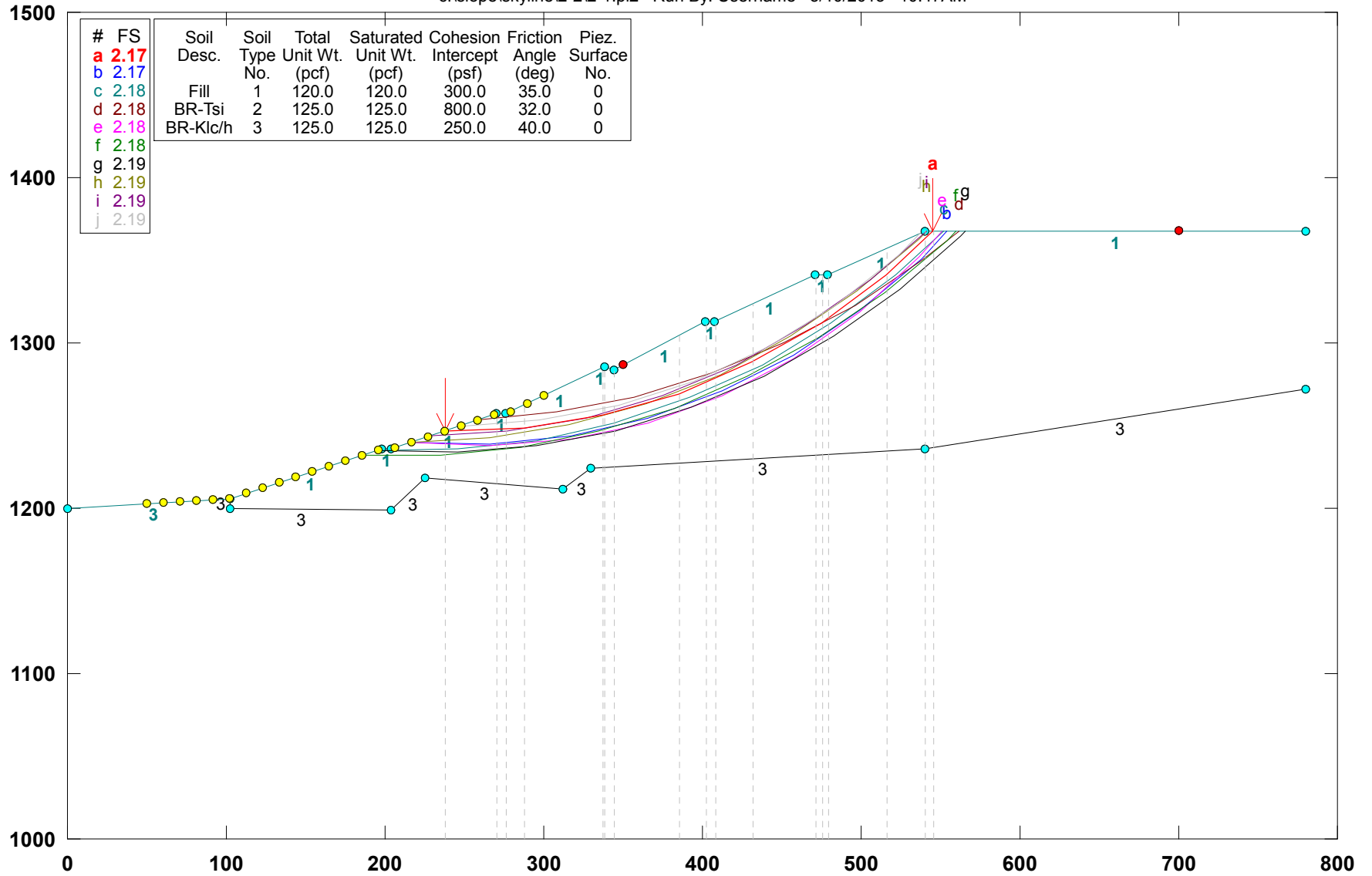
Point No.	X-Surf (ft)	Y-Surf (ft)
1	205.879	1220.145
2	209.550	1216.478
3	233.883	1210.742
4	357.346	1246.316
5	368.214	1268.830
6	381.565	1282.199

Factor of Safety
*** 1.840 ***

**** END OF GSTABL7 OUTPUT ****

Skyline Height, PN:134-02, Section 2-2', Highest Fill Slope, Static

c:\slope\skyline\2-2\2-1.pl2 Run By: Username 5/10/2013 10:47AM

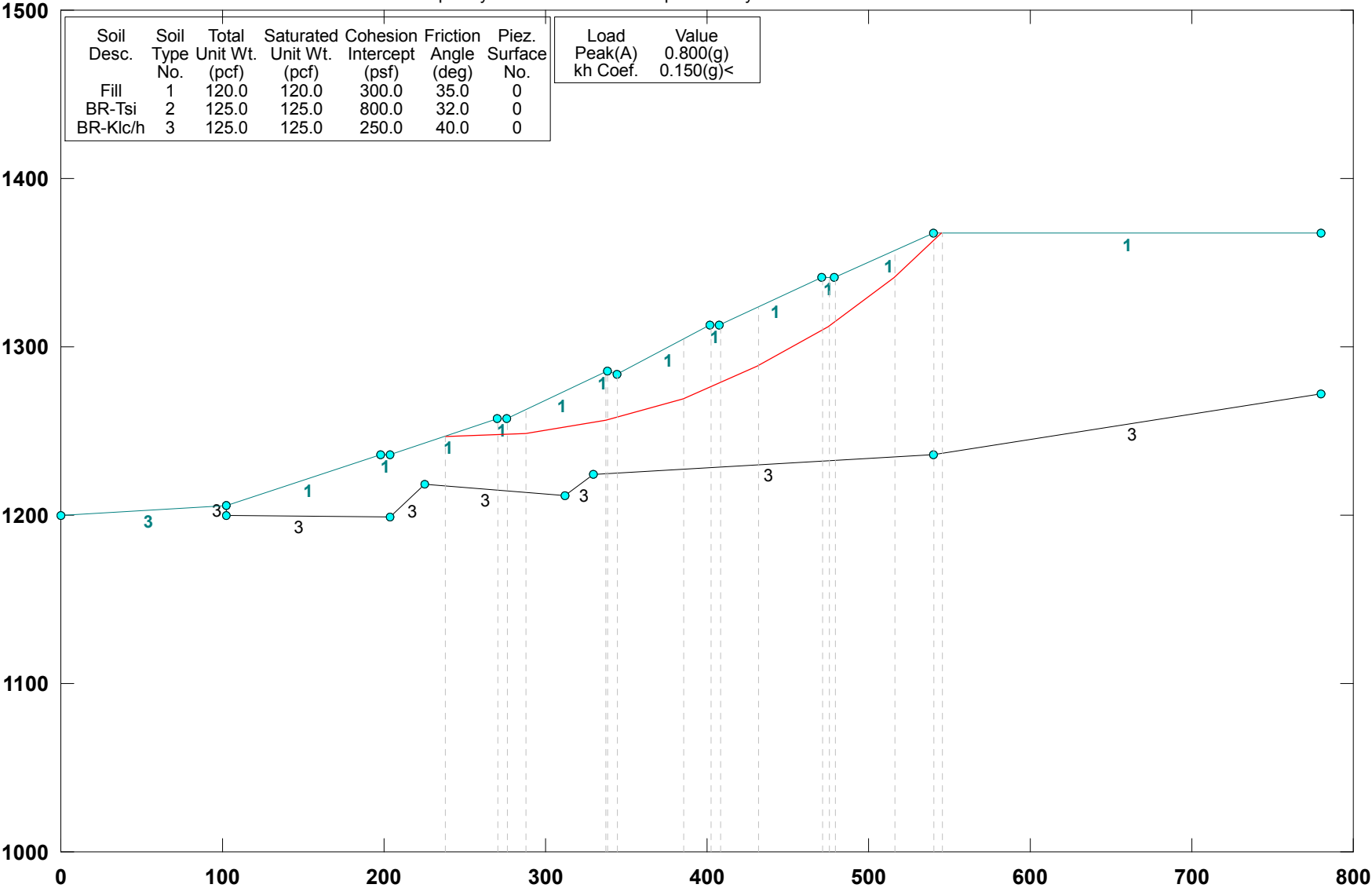


GSTABL7 v.2 FSmin=2.17

Safety Factors Are Calculated By The Modified Bishop Method

Skyline Height, PN:134-02, Section 2-2', Highest Fill Slope, Pseudostatic

c:\slope\skyline\2-2\2-1 surface #1.plt Run By: Username 5/10/2013 10:48AM



GSTABL7 v.2 FSmin=1.52
Factor Of Safety Is Calculated By The Modified Bishop Method

*** GSTABL7 ***

** GSTABL7 by Dr. Garry H. Gregory,
Ph.D., P.E., D.GE **

** Original Version 1.0, January 1996; Current Ver. 2.005.2,
Jan. 2011 **

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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:	5/10/2013
	Time of Run:	10:48AM
	Run By:	Username
#1.in	Input Data Filename:	C:\SLOPE\Skyline\2-2\2-1 Surface
#1.OUT	Output Filename:	C:\SLOPE\Skyline\2-2\2-1 Surface
	Unit System:	English
#1.PLT	Plotted Output Filename:	C:\SLOPE\Skyline\2-2\2-1 Surface

PROBLEM DESCRIPTION: Skyline Height, PN:134-02, Section 2-2',
Highest Fill Slope, Pseudostatic

BOUNDARY COORDINATES

13 Top Boundaries

20 Total Boundaries

Type	Boundary	X-Left	Y-Left	X-Right	Y-Right	Soil
Bnd	No.	(ft)	(ft)	(ft)	(ft)	Below
	1	0.00	1200.00	102.00	1206.00	3
	2	102.00	1206.00	198.00	1236.00	1
	3	198.00	1236.00	204.00	1236.00	1
	4	204.00	1236.00	270.00	1257.00	1
	5	270.00	1257.00	276.00	1257.00	1
	6	276.00	1257.00	338.00	1286.00	1
	7	338.00	1286.00	344.00	1284.00	1
	8	344.00	1284.00	402.00	1313.00	1
	9	402.00	1313.00	408.00	1313.00	1
	10	408.00	1313.00	471.00	1341.00	1
	11	471.00	1341.00	479.00	1341.00	1
	12	479.00	1341.00	540.00	1368.00	1
	13	540.00	1368.00	780.00	1368.00	1
	14	102.00	1206.00	102.10	1200.00	3
	15	102.10	1200.00	204.00	1199.00	3
	16	204.00	1199.00	225.00	1218.00	3
	17	225.00	1218.00	312.00	1212.00	3
	18	312.00	1212.00	330.00	1224.00	3
	19	330.00	1224.00	540.00	1236.00	3
	20	540.00	1236.00	780.00	1272.00	3

User Specified Y-Origin = 1000.00(ft)

Default X-Plus Value = 0.00(ft)

Default Y-Plus Value = 0.00(ft)

1

ISOTROPIC SOIL PARAMETERS

3 Type(s) of Soil

Piez.	Soil	Total	Saturated	Cohesion	Friction	Pore	Pressure
Surface	Type	Unit Wt.	Unit Wt.	Intercept	Angle	Pressure	Constant
No.	No.	(pcf)	(pcf)	(psf)	(deg)	Param.	(psf)
	1	120.0	120.0	300.0	35.0	0.00	0.0
0							
	2	125.0	125.0	800.0	32.0	0.00	0.0
0							

0 3 125.0 125.0 250.0 40.0 0.00 0.0

Specified Peak Ground Acceleration Coefficient (A) = 0.800(g)
Specified Horizontal Earthquake Coefficient (kh) = 0.150(g)
Specified Vertical Earthquake Coefficient (kv) = 0.000(g)

1 Specified Seismic Pore-Pressure Factor = 0.000

Trial Failure Surface Specified By 8 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	237.500	1246.659
2	287.465	1248.524
3	336.876	1256.177
4	385.064	1269.517
5	431.377	1288.361
6	475.188	1312.455
7	515.906	1341.474
8	545.216	1368.000

DEFLECTION ANGLE & SEGMENT DATA FOR SPECIFIED SURFACE(Excluding Last Segment)

Angle/Segment No.	Deflection(Deg)	Segment Length(ft)
1	6.67	50.00
2	6.67	50.00
3	6.67	50.00
4	6.67	50.00
5	6.67	50.00

Circle Center At X = 246.913(ft) ; Y = 1673.637(ft); and
Radius = 427.082(ft)

* * Factor Of Safety Is Calculated By The Modified Bishop
Method * *

Factor Of Safety For The Preceding Specified Surface = 1.515

Table 1 - Individual Data on the 16 Slices

			Water Force	Water Force	Tie Force	Tie Force	Earthquake Force	
Surcharge Slice Load No. (lbs)	Width (ft)	Weight (lbs)	Top (lbs)	Bot (lbs)	Norm (lbs)	Tan (lbs)	Hor (lbs)	Ver (lbs)
1 0.0	32.5	17799.5	0.0	0.0	0.0	0.0	2669.9	0.0
2 0.0	6.0	6491.4	0.0	0.0	0.0	0.0	973.7	0.0
3 0.0	11.5	15644.5	0.0	0.0	0.0	0.0	2346.7	0.0
4 0.0	49.4	127883.6	0.0	0.0	0.0	0.0	19182.5	0.0
5 0.0	1.1	3966.1	0.0	0.0	0.0	0.0	594.9	0.0
6 0.0	6.0	19930.6	0.0	0.0	0.0	0.0	2989.6	0.0
7 0.0	41.1	149964.0	0.0	0.0	0.0	0.0	22494.6	0.0
8 0.0	16.9	72764.1	0.0	0.0	0.0	0.0	10914.6	0.0
9 0.0	6.0	25467.4	0.0	0.0	0.0	0.0	3820.1	0.0
10 0.0	23.4	97032.1	0.0	0.0	0.0	0.0	14554.8	0.0
11 0.0	39.6	156614.0	0.0	0.0	0.0	0.0	23492.1	0.0
12 0.0	4.2	14924.2	0.0	0.0	0.0	0.0	2238.6	0.0
13 0.0	3.8	12436.2	0.0	0.0	0.0	0.0	1865.4	0.0
14 0.0	36.9	92316.0	0.0	0.0	0.0	0.0	13847.4	0.0
15 0.0	24.1	29754.2	0.0	0.0	0.0	0.0	4463.1	0.0
16 0.0	5.2	1477.3	0.0	0.0	0.0	0.0	221.6	0.0

Table 2 - Base Stress Data on the 16 Slices

Slice Mobilized	Alpha	X-Coord.	Base	Available	
No.	(deg)	Slice Cntr	Leng.	Shear Strength	Shear
Stress					
*		(ft)	(ft)	(psf)	
(psf)					

1	2.14	253.75	32.52	671.89
102.01				
2	2.14	273.00	6.00	1039.62
200.68				
3	2.14	281.73	11.47	1234.17
252.54				
4	8.80	312.17	50.00	1971.14
760.91				
5	15.47	337.44	1.17	2456.42
1381.35				
6	15.47	341.00	6.23	2328.04
1301.36				
7	15.47	364.53	42.61	2533.01
1429.01				
8	22.14	393.53	18.28	2784.68
2027.74				
9	22.14	405.00	6.48	2754.11
2003.57				
10	22.14	419.69	25.24	2698.83
1959.82				
11	28.81	451.19	45.22	2445.91
2104.15				
12	28.81	473.09	4.78	2228.72
1898.83				
13	35.48	477.09	4.68	1943.98
1853.73				
14	35.48	497.45	45.32	1543.16
1423.59				
15	42.15	527.95	32.50	821.19
714.58				
16	42.15	542.61	7.03	351.35
164.18				

Sum of the Resisting Forces (including Pier/Pile, Tieback,
Reinforcing
Soil Nail, and Applied Forces if applicable) = 632288.25 (lbs)

Average Available Shear Strength (including Tieback, Pier/Pile,
Reinforcing,
Soil Nail, and Applied Forces if applicable) = 1862.24(psf)

Sum of the Driving Forces = 417376.72 (lbs)

Average Mobilized Shear Stress = 1229.27(psf)

Total length of the failure surface = 339.53(ft)

Bishop
Surface

CAUTION - Factor Of Safety Is Calculated By The Modified
Method. This Method Is Valid Only If The Failure
Approximates A Circular Arc.

*** SEISMIC SLOPE DISPLACEMENT DATA ***
(Note: kv is set = zero for displacement calculations)
Seismic Yield Coefficient (ky) = 0.3786(g)
Calculated Newmark Seismic Displacement = 0.347(ft)

Non-Symmetrical Sliding Resistance Has Been Specified
for Downhill Sliding.

**** END OF GSTABL7 OUTPUT ****

*** GSTABL7 ***

Ph.D., P.E., D.GE ** ** GSTABL7 by Dr. Garry H. Gregory,

Jan. 2011 ** ** Original Version 1.0, January 1996; Current Ver. 2.005.2,

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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: 5/10/2013
Time of Run: 10:47AM
Run By: Username
Input Data Filename: C:\SLOPE\Skyline\2-2\2-1.in
Output Filename: C:\SLOPE\Skyline\2-2\2-1.OUT
Unit System: English

Plotted Output Filename: C:\SLOPE\Skyline\2-2\2-1.PLT

PROBLEM DESCRIPTION: Skyline Height, PN:134-02, Section 2-2',
Highest Fill Slope, Static

BOUNDARY COORDINATES

13 Top Boundaries
20 Total Boundaries

Type Bnd	Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Below
	1	0.00	1200.00	102.00	1206.00	3
	2	102.00	1206.00	198.00	1236.00	1
	3	198.00	1236.00	204.00	1236.00	1
	4	204.00	1236.00	270.00	1257.00	1
	5	270.00	1257.00	276.00	1257.00	1
	6	276.00	1257.00	338.00	1286.00	1
	7	338.00	1286.00	344.00	1284.00	1
	8	344.00	1284.00	402.00	1313.00	1
	9	402.00	1313.00	408.00	1313.00	1
	10	408.00	1313.00	471.00	1341.00	1
	11	471.00	1341.00	479.00	1341.00	1
	12	479.00	1341.00	540.00	1368.00	1
	13	540.00	1368.00	780.00	1368.00	1
	14	102.00	1206.00	102.10	1200.00	3
	15	102.10	1200.00	204.00	1199.00	3
	16	204.00	1199.00	225.00	1218.00	3
	17	225.00	1218.00	312.00	1212.00	3
	18	312.00	1212.00	330.00	1224.00	3
	19	330.00	1224.00	540.00	1236.00	3
	20	540.00	1236.00	780.00	1272.00	3

User Specified Y-Origin = 1000.00(ft)

Default X-Plus Value = 0.00(ft)

Default Y-Plus Value = 0.00(ft)

1

ISOTROPIC SOIL PARAMETERS

3 Type(s) of Soil

Piez. Surface No.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param. (psf)	Pressure Constant (psf)
	1	120.0	120.0	300.0	35.0	0.00	0.0
0	2	125.0	125.0	800.0	32.0	0.00	0.0
0	3	125.0	125.0	250.0	40.0	0.00	0.0
0							
1							

Spaced

100 Surface(s) Initiate(s) From Each Of 25 Points Equally
Along The Ground Surface Between X = 50.00(ft)
and X = 300.00(ft)

Unless Further Limitations Were Imposed, The Minimum Elevation
At Which A Surface Extends Is Y = 0.00 (ft)

Following Are Displayed The Ten Most Critical Of The Trial Failure Surfaces Evaluated. They Are Ordered - Most Critical First.

* *

Number of Trial Surfaces With Valid FS = 2500

FS Max = 6.963 FS Min = 2.171 FS Ave = 3.734
Standard Deviation = 0.909 Coefficient of Variation =

24.35 %

Point No.	X-Surf (ft)	Y-Surf (ft)
--------------	----------------	----------------

1	237.500	1246.659
2	287.465	1248.524
3	336.876	1256.177
4	385.064	1269.517
5	431.377	1288.361
6	475.188	1312.455
7	515.906	1341.474
8	545.216	1368.000

Circle Center At X = 246.914 ; Y = 1673.635 ; and Radius = 427.080

Factor of Safety
*** 2.171 ***

Individual data on the 16 slices

Surcharge Slice Load No. (lbs)	Width (ft)	Weight (lbs)	Water Force	Water Force	Tie Force	Tie Force	Earthquake Force	
			Top (lbs)	Bot (lbs)	Norm (lbs)	Tan (lbs)	Hor (lbs)	Ver (lbs)
1	32.5	17799.9	0.0	0.0	0.	0.	0.0	0.0
0.0								
2	6.0	6491.6	0.0	0.0	0.	0.	0.0	0.0
0.0								
3	11.5	15645.6	0.0	0.0	0.	0.	0.0	0.0
0.0								
4	49.4	127882.9	0.0	0.0	0.	0.	0.0	0.0
0.0								
5	1.1	3966.0	0.0	0.0	0.	0.	0.0	0.0
0.0								
6	6.0	19930.3	0.0	0.0	0.	0.	0.0	0.0
0.0								
7	41.1	149963.3	0.0	0.0	0.	0.	0.0	0.0
0.0								
8	16.9	72765.7	0.0	0.0	0.	0.	0.0	0.0
0.0								
9	6.0	25467.5	0.0	0.0	0.	0.	0.0	0.0
0.0								
10	23.4	97031.5	0.0	0.0	0.	0.	0.0	0.0
0.0								
11	39.6	156614.9	0.0	0.0	0.	0.	0.0	0.0
0.0								

12	4.2	14925.3	0.0	0.0	0.	0.	0.0	0.0
0.0								
13	3.8	12435.0	0.0	0.0	0.	0.	0.0	0.0
0.0								
14	36.9	92315.0	0.0	0.0	0.	0.	0.0	0.0
0.0								
15	24.1	29754.8	0.0	0.0	0.	0.	0.0	0.0
0.0								
16	5.2	1477.6	0.0	0.0	0.	0.	0.0	0.0
0.0								

Failure Surface Specified By 9 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	216.667	1240.030
2	266.655	1238.930
3	316.422	1243.742
4	365.273	1254.400
5	412.523	1270.754
6	457.510	1292.575
7	499.605	1319.558
8	538.217	1351.324
9	554.192	1368.000

Circle Center At X = 250.942 ; Y = 1661.158 ; and Radius = 422.521

Factor of Safety
*** 2.171 ***

1

Failure Surface Specified By 9 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	195.833	1235.323
2	245.833	1235.559
3	295.543	1240.931
4	344.439	1251.382
5	392.002	1266.801
6	437.729	1287.026
7	481.136	1311.842
8	521.764	1340.986
9	552.242	1368.000

486.000 Circle Center At X = 218.542 ; Y = 1720.792 ; and Radius =

Factor of Safety
*** 2.177 ***

Failure Surface Specified By 8 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	258.333	1253.288
2	308.101	1258.107
3	357.196	1267.576
4	405.186	1281.610
5	451.647	1300.087
6	496.168	1322.843
7	538.358	1349.677
8	561.943	1368.000

526.760 Circle Center At X = 233.015 ; Y = 1779.439 ; and Radius =

Factor of Safety
*** 2.180 ***

1

Failure Surface Specified By 9 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	216.667	1240.030
2	266.602	1237.490
3	316.450	1241.385
4	365.385	1251.653
5	412.595	1268.121
6	457.298	1290.518
7	498.754	1318.472
8	536.274	1351.520
9	550.725	1368.000

388.402 Circle Center At X = 261.328 ; Y = 1625.856 ; and Radius =

Factor of Safety
*** 2.183 ***

Failure Surface Specified By 10 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	185.417	1232.068
2	235.417	1231.951
3	285.190	1236.703
4	334.265	1246.277
5	382.175	1260.583
6	428.464	1279.485
7	472.694	1302.803
8	514.443	1330.316
9	553.316	1361.763
10	559.650	1368.000

Circle Center At X = 211.936 ; Y = 1742.173 ; and Radius =
510.794

Factor of Safety
*** 2.184 ***

1

Failure Surface Specified By 10 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	195.833	1235.323
2	245.815	1233.971
3	295.665	1237.836
4	344.842	1246.875
5	392.807	1260.992
6	439.041	1280.030
7	483.038	1303.784
8	524.319	1331.995
9	562.436	1364.355
10	565.918	1368.000

477.727 Circle Center At X = 233.936 ; Y = 1711.528 ; and Radius =

Factor of Safety
*** 2.185 ***

Failure Surface Specified By 9 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	216.667	1240.030
2	266.605	1242.519
3	316.001	1250.265
4	364.304	1263.182
5	410.973	1281.126
6	455.487	1303.896
7	497.349	1331.238
8	536.090	1362.847
9	541.195	1368.000

472.989 Circle Center At X = 218.129 ; Y = 1713.017 ; and Radius =

Factor of Safety
*** 2.189 ***

1

Failure Surface Specified By 8 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	227.083	1243.345
2	276.988	1246.439
3	326.277	1254.836
4	374.391	1268.440
5	420.780	1287.097
6	464.915	1310.594
7	506.293	1338.662
8	540.923	1368.000

466.809 Circle Center At X = 223.351 ; Y = 1710.139 ; and Radius =

Factor of Safety
*** 2.192 ***

Failure Surface Specified By 8 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	247.917	1249.974
2	297.802	1253.360
3	346.954	1262.529
4	394.705	1277.356
5	440.406	1297.640
6	483.436	1323.104
7	523.210	1353.402
8	536.926	1366.640

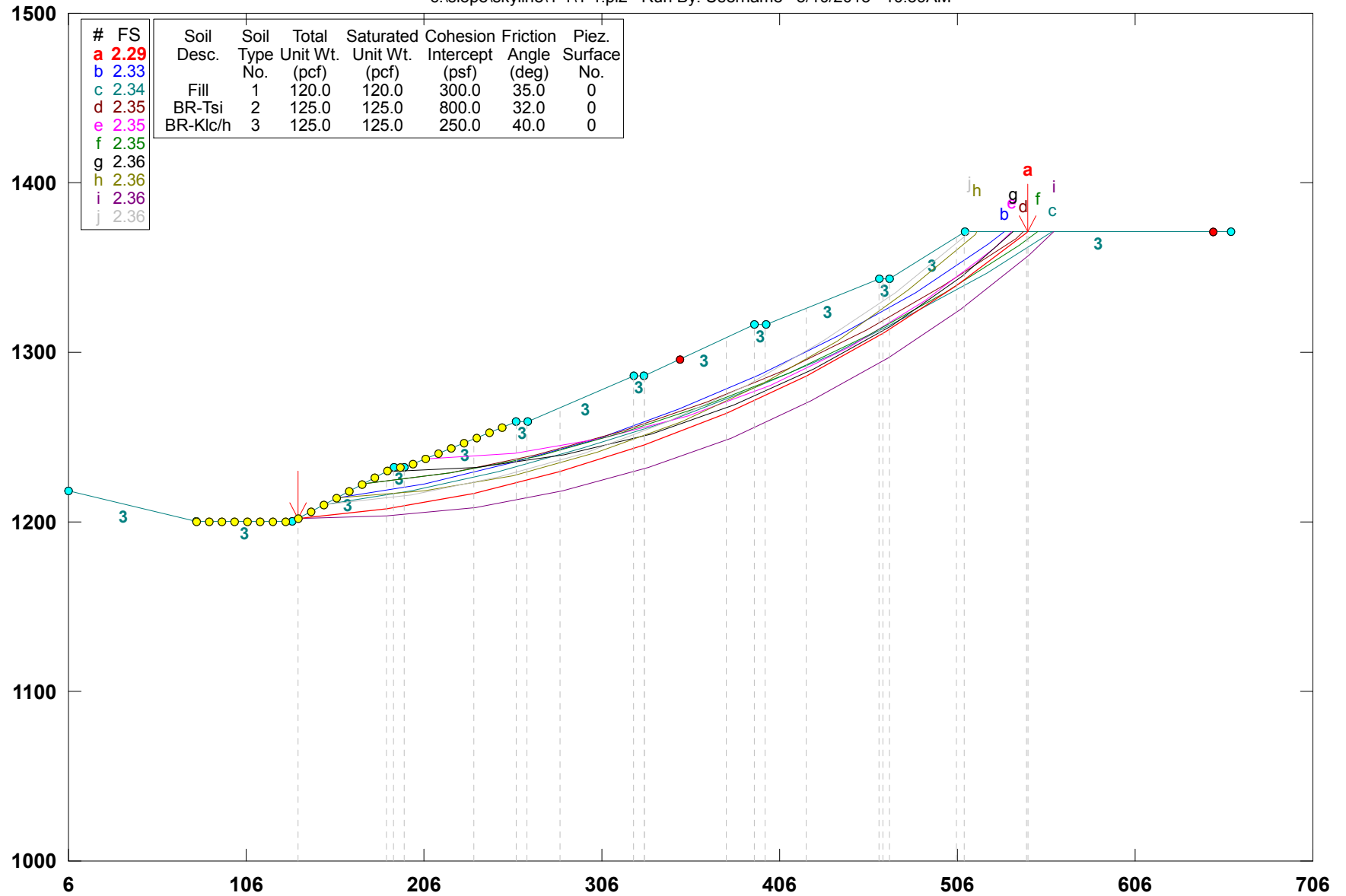
Circle Center At X = 244.733 ; Y = 1674.167 ; and Radius =
424.205

Factor of Safety
*** 2.193 ***

**** END OF GSTABL7 OUTPUT ****

Skyline Height, PN:134-02, Section 1-1', Highest Cut Slope, Static

c:\slope\skyline\1-1\1-1.pl2 Run By: Username 5/10/2013 10:39AM

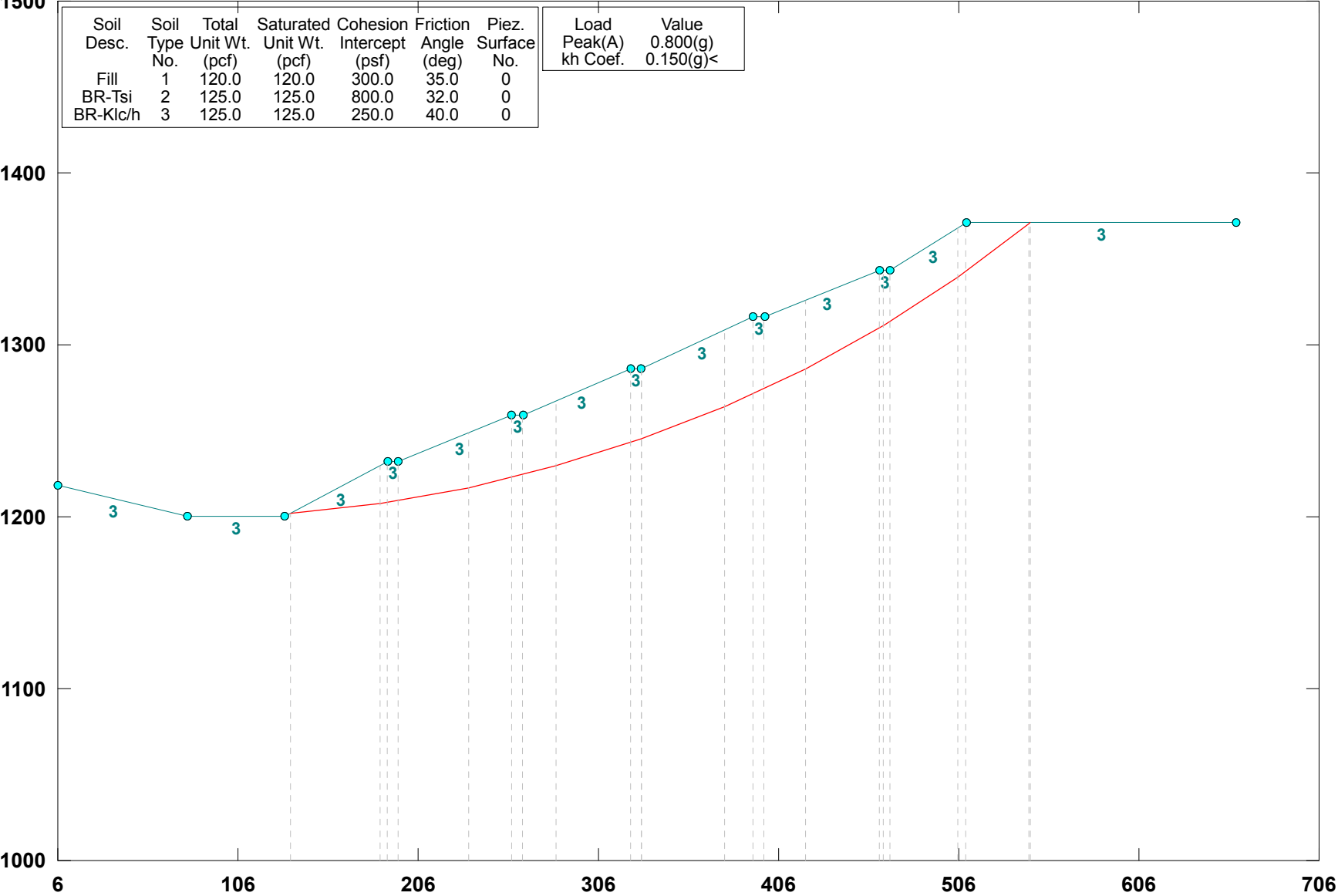


GSTABL7 v.2 FSmin=2.29

Safety Factors Are Calculated By The Modified Bishop Method

Skyline Height, PN:134-02, Section 1-1', Highest Cut Slope, Pseudostatic

c:\slope\skyline\1-1\1-1 surface #1.plt Run By: Username 5/10/2013 10:40AM



GSTABL7 v.2 FSmin=1.60
Factor Of Safety Is Calculated By The Modified Bishop Method

*** GSTABL7 ***

** GSTABL7 by Dr. Garry H. Gregory,
Ph.D., P.E., D.GE **

** Original Version 1.0, January 1996; Current Ver. 2.005.2,
Jan. 2011 **

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Prohibited)

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:	5/10/2013
	Time of Run:	10:40AM
	Run By:	Username
#1.in	Input Data Filename:	C:\SLOPE\Skyline\1-1\1-1 Surface
#1.OUT	Output Filename:	C:\SLOPE\Skyline\1-1\1-1 Surface
	Unit System:	English
#1.PLT	Plotted Output Filename:	C:\SLOPE\Skyline\1-1\1-1 Surface

PROBLEM DESCRIPTION: Skyline Height, PN:134-02, Section 1-1',
Highest Cut Slope, Pseudostatic

BOUNDARY COORDINATES

14 Top Boundaries

14 Total Boundaries

Type	Boundary	X-Left	Y-Left	X-Right	Y-Right	Soil
Bnd	No.	(ft)	(ft)	(ft)	(ft)	Below
	1	6.00	1218.00	78.00	1200.00	3
	2	78.00	1200.00	132.00	1200.00	3
	3	132.00	1200.00	189.00	1232.00	3
	4	189.00	1232.00	195.00	1232.00	3
	5	195.00	1232.00	258.00	1259.00	3
	6	258.00	1259.00	264.00	1259.00	3
	7	264.00	1259.00	324.00	1286.00	3
	8	324.00	1286.00	330.00	1286.00	3
	9	330.00	1286.00	392.00	1316.00	3
	10	392.00	1316.00	398.00	1316.00	3
	11	398.00	1316.00	462.00	1343.00	3
	12	462.00	1343.00	468.00	1343.00	3
	13	468.00	1343.00	510.00	1371.00	3
	14	510.00	1371.00	660.00	1371.00	3

User Specified Y-Origin = 1000.00(ft)

Default X-Plus Value = 0.00(ft)

Default Y-Plus Value = 0.00(ft)

1

ISOTROPIC SOIL PARAMETERS

3 Type(s) of Soil

Piez.	Soil	Total	Saturated	Cohesion	Friction	Pore	Pressure
Surface	Type	Unit Wt.	Unit Wt.	Intercept	Angle	Pressure	Constant
No.	No.	(pcf)	(pcf)	(psf)	(deg)	Param.	(psf)
	1	120.0	120.0	300.0	35.0	0.00	0.0
0	2	125.0	125.0	800.0	32.0	0.00	0.0
0	3	125.0	125.0	250.0	40.0	0.00	0.0
0							

Specified Peak Ground Acceleration Coefficient (A) = 0.800(g)

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

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

1

Specified Seismic Pore-Pressure Factor = 0.000

Trial Failure Surface Specified By 11 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	135.333	1201.871
2	184.996	1207.672
3	234.145	1216.855
4	282.552	1229.378
5	329.989	1245.180
6	376.234	1264.189
7	421.072	1286.316
8	464.292	1311.456
9	505.692	1339.493
10	545.077	1370.295
11	545.862	1371.000

DEFLECTION ANGLE & SEGMENT DATA FOR SPECIFIED SURFACE(Excluding Last Segment)

Angle/Segment No.	Deflection(Deg)	Segment Length(ft)
1	3.92	50.00
2	3.92	50.00
3	3.92	50.00
4	3.92	50.00
5	3.92	50.00
6	3.92	50.00
7	3.92	50.00
8	3.92	50.00

Circle Center At X = 75.423(ft) ; Y = 1930.249(ft); and
Radius = 730.837(ft)

* * Factor Of Safety Is Calculated By The Modified Bishop
Method * *

Factor Of Safety For The Preceding Specified Surface = 1.599

Table 1 - Individual Data on the 21 Slices

Surcharge Slice Load No. (lbs)	Width (ft)	Weight (lbs)	Water Force	Water Force	Tie Force	Tie Force	Earthquake Force	
			Top (lbs)	Bot (lbs)	Norm (lbs)	Tan (lbs)	Hor (lbs)	Ver (lbs)
1	49.7	68535.9	0.0	0.0	0.0	0.0	10280.4	0.0
0.0								
2	4.0	11426.6	0.0	0.0	0.0	0.0	1714.0	0.0
0.0								
3	6.0	17264.6	0.0	0.0	0.0	0.0	2589.7	0.0
0.0								
4	39.1	133044.4	0.0	0.0	0.0	0.0	19956.7	0.0
0.0								
5	23.9	101226.9	0.0	0.0	0.0	0.0	15184.0	0.0
0.0								
6	6.0	26398.1	0.0	0.0	0.0	0.0	3959.7	0.0
0.0								
7	18.6	83938.1	0.0	0.0	0.0	0.0	12590.7	0.0
0.0								
8	41.4	209274.6	0.0	0.0	0.0	0.0	31391.2	0.0
0.0								
9	6.0	31305.6	0.0	0.0	0.0	0.0	4695.8	0.0
0.0								
10	0.0	56.2	0.0	0.0	0.0	0.0	8.4	0.0
0.0								
11	46.2	245611.8	0.0	0.0	0.0	0.0	36841.8	0.0
0.0								
12	15.8	86923.0	0.0	0.0	0.0	0.0	13038.4	0.0
0.0								
13	6.0	31912.6	0.0	0.0	0.0	0.0	4786.9	0.0
0.0								
14	23.1	116062.5	0.0	0.0	0.0	0.0	17409.4	0.0
0.0								
15	40.9	184929.1	0.0	0.0	0.0	0.0	27739.4	0.0
0.0								
16	2.3	9228.4	0.0	0.0	0.0	0.0	1384.3	0.0
0.0								
17	3.7	14038.6	0.0	0.0	0.0	0.0	2105.8	0.0
0.0								
18	37.7	135850.8	0.0	0.0	0.0	0.0	20377.6	0.0
0.0								
19	4.3	15286.0	0.0	0.0	0.0	0.0	2292.9	0.0
0.0								
20	35.1	63232.4	0.0	0.0	0.0	0.0	9484.9	0.0
0.0								

21	0.8	34.6	0.0	0.0	0.0	0.0	5.2	0.0
0.0								

Table 2 - Base Stress Data on the 21 Slices

Slice Mobilized No. Stress * (psf)	Alpha (deg)	X-Coord. Slice Cntr (ft)	Base Leng. (ft)	Available Shear Strength (psf)	Shear
1	6.66	160.16	50.00	1326.66	
361.68					
2	10.58	187.00	4.07	2408.44	
922.28					
3	10.58	192.00	6.10	2426.51	
929.87					
4	10.58	214.57	39.82	2824.90	
1096.91					
5	14.50	246.07	24.64	3355.14	
1611.19					
6	14.50	261.00	6.20	3470.59	
1669.98					
7	14.50	273.28	19.16	3562.78	
1716.89					
8	18.42	303.28	43.69	3819.05	
2175.79					
9	18.42	326.99	6.31	3946.27	
2251.82					
10	22.35	329.99	0.01	3727.24	
2429.07					
11	22.35	353.12	49.99	3872.28	
2528.27					
12	26.27	384.12	17.58	3873.15	
2830.59					
13	26.27	395.00	6.69	3743.49	
2731.47					
14	26.27	409.54	25.73	3551.32	
2584.47					
15	30.19	441.54	47.35	3096.23	
2455.73					
16	30.19	463.15	2.65	2779.89	
2189.69					
17	34.11	466.15	4.48	2528.31	
2137.41					
18	34.11	486.85	45.52	2415.75	
2035.21					
19	38.03	507.85	5.47	2288.22	
2043.97					
20	38.03	527.54	44.53	1249.71	
1040.45					
21	41.93	545.47	1.06	195.04	
25.56					

Sum of the Resisting Forces (including Pier/Pile, Tieback,
Reinforcing
Soil Nail, and Applied Forces if applicable) = 1289039.00 (lbs)

Average Available Shear Strength (including Tieback, Pier/Pile,
Reinforcing,
Soil Nail, and Applied Forces if applicable) = 2857.83(psf)

Sum of the Driving Forces = 806200.94 (lbs)

Average Mobilized Shear Stress = 1787.37(psf)

Total length of the failure surface = 451.06(ft)

Bishop
Surface
CAUTION - Factor Of Safety Is Calculated By The Modified
Method. This Method Is Valid Only If The Failure
Approximates A Circular Arc.

*** SEISMIC SLOPE DISPLACEMENT DATA ***
(Note: kv is set = zero for displacement calculations)
Seismic Yield Coefficient (ky) = 0.4075(g)
Calculated Newmark Seismic Displacement = 0.299(ft)

Non-Symmetrical Sliding Resistance Has Been Specified
for Downhill Sliding.

**** END OF GSTABL7 OUTPUT ****

APPENDIX I

References

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