Geotechnical Report
and
Stabilization Design
for
Hacienda Road
Shoulder and Pavement Distress

Within the
City of La Habra Heights, California
Prepared for
Onward Engineering
300 South Harbor Boulevard, Suite 814
Anaheim, California, 92805

September 2008

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# Geotechnical Report

and

Stabilization Design

Hacienda Road Shoulder and Pavement Distress

City of La Habra Heights, California

September 2008

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Geotechnical Report
and
Stabilization Design
Hacienda Road Shoulder and Pavement Distress
City of La Habra Heights, California
September 2008

PROJECT DESCRIPTION
Two sections of Hacienda Road within the City of La Habra Heights, California have recently experienced subsurface distress or shoulder erosion. The upper section, which is referred to herein as Hacienda Road (Upper), is located between West Skyline Drive and Canada Sombre Road. This sidehill section consists of two areas of pavement distress each of which is located in embankment fill near the outside shoulder of the road. The locations of the lower and upper sites within this section, designated as Sites 1 and 2, respectively, are shown on Plate 1.

The lower section, which is referred to herein as Hacienda Road (Lower), is located between East Road and West Road. This section consists of two areas where streambank erosion has, or threatens to; undermine the shoulder of the road. The locations of the lower and upper sites within this section, designated as Sites 1 and 2, respectively, are shown on Plate 1.

This document supplements the original geotechnical reports prepared by TGR Geotechnical, Inc. and provides alternative recommendations to those made therein.

WORK SCOPE
The scope of this project includes:

(1) Review the geotechnical reports prepared by TGR Geotechnical, Inc. and evaluate their conclusions based upon their reported data and additional site visits.
(2) Conduct laboratory testing to supplement previous work as necessary to classify geologic materials at each site and estimate shear strength properties.
(3) Assess the feasibility of utilizing soil nails and/or micropiles to stabilize the distressed slopes at each site.
(4) Prepare a geotechnical report documenting findings at each site including preliminary design analyses of proposed stabilization alternatives and estimated construction costs.
(5) In consultation with the City, prepare construction drawings, specifications, and an engineer’s estimate for a final stabilization / subsurface drainage design at each site.

PREVIOUS WORK
The test borings, as shown on Plate 2 which is included as an attachment to this report, and logs of borings as described below and which are included in Appendix A (upper) and Appendix B (lower), are believed to be representative of the conditions likely to be encountered at each site, and were used as the basis for soil nail / micropile and streambank stabilization recommendations and design in conjunction with the appropriate levels of engineering judgment and experience. The sources of data are as follows:

Hacienda Road (Upper)
Geotechnical Distress Investigation Report, Hacienda Road Distress Section between West Skyline Drive and Canada Sombre Road in the City of La Habra Heights, California
November 14, 2006
Project No. 06-1597
TGR Geotechnical, Inc.
3037 S. Harbor Boulevard
Santa Ana, California 92704
(714) 641 7189
Hacienda Road (Lower)

Limited Geotechnical Investigation, Improvement to Hacienda Road between East Road and West Road, in the City of La Habra Heights, California.

December 1, 2006

Project No. 06-1697
TGR Geotechnical, Inc.

These reports have been reviewed by the author, Steven C. Devin, P.E., G.E., and accepted as reliable sources of subsurface and soil/rock property data. The author assumes responsibility for their use and application for design of alternative stabilization measures to those proposed by TGR Geotechnical, Inc.

HACIENDA ROAD (UPPER)

Two areas of distress were investigated which are referred to as Site 1 and Site 2 in this report and on the construction drawings. Hacienda Road crosses a canyon at each site and passes through a cut section between the sites. Pavement distress at the Hacienda Road (Upper) site consists of a series of cracks in the pavement parallel to the outside edge of the road embankment at both Sites 1 and 2. Sets of 3 sub-parallel linear to crescent shaped cracks were noted at each site. Proceeding from the outermost crack towards the road centerline, crack geometry at Site 1 varies as follows: a 48 foot long crack near the edge of pavement followed by a 12 foot long crack 3 feet back from the first, and then finally an 80 foot long crack 6 feet beyond the intermediate crack. Similarly at Site 2, crack geometry varies as follows: a 12 foot long crack near the edge of pavement followed by a 44 foot long crack 4 feet back from the first, and then finally a 21 foot long crack another 4 feet back from the intermediate crack. Cracks at Site 2 exhibited a more pronounced crescent shape than those at Site 1. Pavement cracking at Site 2 is shown in Photo 1.

Photo 1, Pavement Cracking at Hacienda Road (Upper) Site 2
Cross sections through the areas of pavement distress at Sites 1 and 2 are shown in Plates 3 and 4, respectively.

**Local Geology**

The Hacienda Road (Upper) site is located within the Puente Hills of the Peninsular Ranges Assemblage physiographic province. The Puente Hills are bounded to the east by the Chino Basin and to the north and west by the Los Angeles Basin. The site is underlain by the Yorba Member of the Puente Formation (Tpy) which is an assemblage of marine sedimentary rocks deposited during early Pliocene to late Miocene times (3.6 to 11.2 million years BP) (see Plate 1). This unit consists of “white to gray, thin bedded, micaceous and siliceous siltstone and sandy siltstone which includes beds of fine-grained sandstone and white to pale-gray limy concretions and concretionary beds” (Morton and Miller, 2006). The Puente Formation is exposed in the cut slopes on the east side of Hacienda Road at the project site and was encountered during the subsurface exploration program.

**Landslides**

It is well documented that the Puente Formation is susceptible to landslides within the Puente Hills (Tan, 1988). Morton and Miller (2006) indicate that landslides are abundant and that most of the landslides are small to moderate size rotational failures. This suggests that failures are not likely to occur solely on bedding planes but rather in a weathered surficial rock mass.

**Seismicity**

The Hacienda Road (Upper) project is located in close proximity to the Whittier Fault Zone. The Whittier Fault Zone is classified as a Class A right-lateral – reverse oblique fault capable of a maximum $M_w$ 6.8 moment magnitude (Cao, et al, 2003). The project site is located within approximately 1600 feet of the surface expression of the 75° northeasterly dipping main fault. The site is in closer proximity to two splays of the main fault. One of these follows La Mirada Creek in the canyon to the southwest and approaches to within 300 to 400 feet of the site. Another splay north of the site approaches to within 500 feet. Other nearby faults located within 21± kilometers (13 miles) include: (a) the Puente Hills Blind Thrust Fault ($M_w$ 7.1); (b) the San Jose Fault ($M_w$ 6.4); (c) the Upper Elysian Park Blind Thrust Fault ($M_w$ 6.4); (d) the Sierra Madre ($M_w$ 7.2); (e) the Raymond Fault ($M_w$ 6.5); and (f) the Chino-Central Avenue (Elsinore) Fault ($M_w$ 6.7).

Probabilistic ground motions were estimated using 2002 data from the USGS Java calculator and from FRISKSP analyses. In both cases peak ground accelerations (PGA) were estimated for events with return periods of 475 and 1000 years. Peak ground accelerations which were estimated from the 2002 seismic hazard maps (i.e. the Java calculator) and from several attenuation relationships in FRISKSP are presented in the Table 1. The modal magnitude was estimated as $M_w$ 6.42 at a distance of 5.98 kilometers. The modal magnitude and distance are the combination of magnitude and distance which make the greatest hazard contribution to ground motions at the site.

**Table 1, Probabilistic Ground Motions**

<table>
<thead>
<tr>
<th>Source / Attenuation Relationship</th>
<th>Peak Ground Acceleration (PGA)</th>
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<tr>
<td></td>
<td>475 year</td>
</tr>
<tr>
<td>2002 Seismic Hazard Maps (USGS Java Calculator)</td>
<td>0.45g</td>
</tr>
<tr>
<td>Bozorgnia, et al (1999) Soft Rock - Corrected</td>
<td>0.52g</td>
</tr>
<tr>
<td>Bozorgnia, et al (1999) Holocene Soil - Corrected</td>
<td>0.43g</td>
</tr>
<tr>
<td>Sadigh, et al. (1997) Soft Rock</td>
<td>0.71g</td>
</tr>
<tr>
<td>Boore, et al. (1997) Rock</td>
<td>0.35g</td>
</tr>
</tbody>
</table>
Subsurface Exploration

Sampling and Penetration Testing

Five exploratory test borings, which were supervised and logged by a TGR Geotechnical, Inc. Engineer and a Certified Engineering Geologist, were drilled on October 4th and 5th, 2006. Borings were advanced with a 24 inch diameter bucket auger drill rig. Borings were advanced to depths of approximately 15 to 50 feet. A Modified California Ring Sampler (CRS) with an O.D. of 3 inches and an I.D. of 2.42 inches was driven at 5 foot intervals using a 2150 pound Kelly-bar, a 1350 pound Kelly-bar, and a 650 pound Kelly-bar for depths of up to 24, 44, and 50 feet, respectively. The Kelly-bar falls freely 12 inches and the number of blows required to drive the CRS sampler 12 inches is recorded as \(N_{\text{CRS-Kelly}}\). Driving is terminated if more than 50 blows are required to drive the sampler 6 inches. The field blow counts for each sampling interval are shown in the boring logs (\(N_{\text{CRS-Kelly}}\)). These are not equivalent standard penetration test (SPT) blow counts (ASTM D 1586). Since this method of driving the sampler utilizes falling weights which vary with sampling depth, it is necessary to convert these to an equivalent energy basis if the results are to be used for direct comparison with other tests made at various depths. This is accomplished by simply dividing the recorded blow counts by the theoretical energy delivered to the sampler (i.e. the potential energy = \(W\times h\)).

An overburden correction factor is also applied in order to reference all blow counts to an overburden pressure of 1 atmosphere (Liao and Whitman, 1986). An approximate estimate of the number of blows to drive the CRS 1 foot using the SPT 140 pound hammer with a 30 inch drop (\(N_{\text{SPT-140}}\)) can be made by simply taking a ratio of the delivered energy conservatively assuming 100 percent efficiency in both cases. The final estimate of equivalent SPT blow counts (\(N_{160}\)) is then made using a conversion factor of 0.56 applied to the CRS-140 blow counts. Estimated SPT (\(N_{160}\)) values for each boring are plotted with depth in Plate 5.

Groundwater

Groundwater was not encountered in any of the borings at the time of drilling.

Boring B-1

Boring B-1 was located within the Site 2 zone of pavement distress near the outside shoulder of the road. This boring encountered 19 inches of asphalt concrete pavement at the surface (with no base course) overlying loose to medium dense Sand and Silt FILL to a depth of 41.5 feet. Below this depth, a 2 foot thick layer of colluvium was encountered overlying bedrock. The colluvium was described as stiff, highly plastic Silt. Unfortunately, no samples of this layer were obtained by TGR Geotechnical field personnel. Below the colluvium and an irregular contact surface, Puente Formation claystone/siltstone interbedded with sandstone was encountered to the bottom of the boring at 50 feet. The bedding planes dip 5° to 15° to the west and northwest between depths of 44 to 46 feet. Dry unit weights in the fill varied from 72 pounds per cubic foot (pcf) at 6 feet to 95 pcf at 11 feet while a dry unit weight of 82 pcf was measured for a sample obtained from 46 feet in the bedrock. Moisture contents varied from 12 percent by weight at 11 feet to 24 percent at 21 feet. The degree of saturation increased linearly from 36.2 percent at 5 feet to 47.2 percent at 15 feet (@ a rate of 1.1 % / ft). It then increased at more than double the initial rate to 60.0 percent at 20 feet. Below 20 feet, the saturation decreased linearly at a rate of 1.3 % / ft to 43.3 percent at 35 feet. A collapse consolidation test (ASTM D 5333) was performed on a sample obtained at 5 feet. Direct shear tests (ASTM D 3080) were performed on a sample of fill from a depth of 15 feet and on a sample of claystone/siltstone from a depth of 45 feet. Test results are discussed below and presented in Appendix C.

Boring B-2

Boring B-2 was located in the vicinity of Site 2 on the opposite (east) side of Hacienda Road from B-1 and outside of the pavement area. This boring was only approximately located. This boring was advanced to a depth of 17 feet where a corrugated metal pipe was encountered and the boring terminated. A loose, low plastic Sandy Silt FILL was encountered from the ground surface to the bottom of the boring. Dry unit weights in the fill varied from 80 to 93 pcf at depths of 6 and 16 feet, respectively. Moisture contents varied from 13 to 15 percent. The degree of saturation increased in an essentially linear fashion from 31.7 percent at 5 feet to 49.9 percent at 15 feet. A direct shear test (ASTM D 3080) was performed on a sample taken from a depth of 10 feet. Test results are discussed below and presented in Appendix C.
Boring B-3

Boring B-3 was made near the outside shoulder of the road in a turnout between Sites 1 and 2. This location is within a cut section of road between the two canyon fills. Seven inches of asphalt concrete pavement (with no base course) were encountered overlying approximately 2.5 feet of FILL. Below this depth, interbedded siltstone and sandstone of the Puente Formation were encountered to the bottom of the boring at 30 feet. The rock encountered between 3 and 6 feet was highly weathered. Typical bedding thickness varied between ¼ and 3 inches with occasional gypsum inter-layers. Bedding planes generally dip to the northwest at 8° to 28° with the exception of 15 to 22 feet where folding reverses dip direction to 24°- N40E at 20 feet. Dry unit weights in the bedrock varied from 91 pcf to 107 pcf at depths of 29 and 6 feet, respectively. The degree of saturation varied between 62.1 and 76.9 percent in the upper 20 feet of the boring and then increased to 82.4 percent at 29 feet. Direct shear tests (ASTM D 3080) were performed on samples obtained from depths of 10 and 29 feet. Test results are discussed below and presented in Appendix C.

Boring B-4

Boring B-4 was located in the Site 1 zone of pavement distress near the outside shoulder of the road. This boring encountered 24 inches of asphalt concrete (with no base course) near the surface. Below this depth a layer of loose Sand and Silt which TGR describes as colluvium extends to bedrock at a depth of 18 feet. Interbedded sandstone and siltstone of the Puente Formation was encountered from 18 feet to the bottom of the boring at 30 feet. Bedding planes rotate dip direction between 21 and 24 feet. The dip and dip direction are 35°- N60W at 21 feet rotating to 43°- S50W and 45°- S55W at 24 and 26 feet, respectively. Dry unit weights varied from 82 pcf to 88 pcf in the colluvium and 93 to 95 pcf in the bedrock. Moisture contents varied from 19 to 24 percent in the colluvium and 20 to 24 percent in the bedrock. The degree of saturation in the colluvium increased rapidly from 51.2 percent at 5 feet to 70.8 percent at 10 feet. Below 10 feet it decreased to 52.2 percent at 15 feet. The saturation in the Puente Formation siltstone / sandstone increased from 66.5 percent at 20 feet to 83.7 percent at 25 feet. The rate of increase from 15 feet to 25 feet was essentially constant irrespective of material type. A collapse consolidation test (ASTM D 5333) was performed on a sample obtained from a depth of 6 feet. A bulk sample was obtained from 2 to 5 feet in the colluvium and a modified Proctor test run (ASTM D 1557 Method B). Test results are discussed below and presented in Appendix C.

Boring B-5

Boring B-5 was located in the vicinity of Site 1 on the opposite side of the road from B-4 and outside the pavement area. This boring was only approximately located. Interbedded sandstone and siltstone of the Puente Formation was encountered for the entire depth of the boring. The boring was terminated at a depth of 15 feet. Bedding planes dip 10°- N30W at 2 feet and then average 32°- S70W to the bottom of the boring. Reported dry unit weights varied from 78 pcf at 6 feet to 104 pcf at 11 feet. Reported moisture contents varied from 46 percent at 6 feet to 16 percent at 11 feet. When the degree of saturation was computed using this data it was found to exceed 100 percent at 5 feet which means that there is an error in either the dry unit weight and/or moisture content reported for this sample. The degree of saturation computed at 10 feet was 69.6 percent. A direct shear test (ASTM D 3080) was run on a sample obtained from 10 feet. Unfortunately a direct shear test was not run on the sample from 6 feet where the reported dry unit weight was only 78 pcf and the reported natural moisture content was 46 percent. Test results are discussed below and presented in Appendix C.

The boring logs are included as TGR Plates 1 through 6 in Appendix A (upper). Surveyed ground surface elevations at each boring are summarized in Table 2.

<table>
<thead>
<tr>
<th>Boring</th>
<th>Elevation (ft)</th>
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<tbody>
<tr>
<td>B-1</td>
<td>818.9</td>
</tr>
<tr>
<td>B-2</td>
<td>820.7</td>
</tr>
<tr>
<td>B-3</td>
<td>806.5</td>
</tr>
<tr>
<td>B-4</td>
<td>795.2</td>
</tr>
<tr>
<td>B-5</td>
<td>792.9</td>
</tr>
</tbody>
</table>
Laboratory Testing

Laboratory tests were performed on select samples as shown on the Boring Logs and described above. The moisture content and dry unit weight were determined for each CRS interval and are shown on the logs. The degree of saturation of each sample was computed using reported data and an assumed specific gravity of 2.7 for the solid particles. Direct shear tests (ASTM D 3080) and collapse consolidation tests (ASTM D 5333) were performed on select samples as shown on the logs. A Modified Proctor Density (ASTM D 1557) was performed on a bulk sample of colluvium obtained from 2 to 5 feet in Boring B-4. Laboratory test results are presented in Appendix C.

Laboratory classification tests including grain size analyses and/or Atterberg limits tests were not performed on any samples obtained from this site.

Consolidation Tests

Collapse consolidation tests (ASTM D 5333) were performed on samples obtained from a depth of 5 feet in Borings B-1 and B-4. The collapse index \( I_c \) at an effective overburden pressure of 1600 psf was determined for each sample. The collapse index is the percentage of settlement that a partially saturated sample experiences when rapidly saturated by inundation under an applied load. Initial void ratios were computed for each sample assuming a specific gravity of 2.7 for the solids. Virgin compression indices were also computed for each sample following inundation. These results are summarized in Table 3.

Table 3, Collapse Consolidation Test Summary

<table>
<thead>
<tr>
<th>Boring</th>
<th>Depth (ft)</th>
<th>Dry Unit Weight, ( \gamma_d ) (pcf)</th>
<th>Moisture Content, ( w ) (%)</th>
<th>Initial Void Ratio, ( e_i )</th>
<th>Compression Index, ( C_c )</th>
<th>Collapse Index, ( I_c )</th>
<th>Degree of Collapse</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1</td>
<td>5.0</td>
<td>72</td>
<td>18</td>
<td>1.34</td>
<td>0.23</td>
<td>4.0</td>
<td>Moderate</td>
</tr>
<tr>
<td>B-4</td>
<td>5.0</td>
<td>82</td>
<td>20</td>
<td>1.05</td>
<td>0.23</td>
<td>0.1</td>
<td>Slight</td>
</tr>
</tbody>
</table>

Maximum Dry Density (Unit Weight)

The maximum dry density (unit weight) of a bulk sample of colluvium from Boring B-4 was determined in accordance with ASTM D 1557, Method B. The maximum dry unit weight thus determined was 96 pcf at an optimum compaction moisture content of 25 percent. The in situ moisture content of this sample was not measured but the CRS sample immediately below had a moisture content of 20 percent along with a dry unit weight of 82 pcf. The consequent relative compaction (RC) of the CRS sample was 85 percent. Elsewhere, assuming the same moisture-density relationship for fill/colluvium across the site, the RC varied from a low of 75 percent at 5 feet in Boring B-1 to 99 percent at 10 feet in the same boring. Typically, RC varied from 83 to 92 percent in the fill and colluvium in Borings B-1, B-2, and B-4. The RC can, in an approximate way, be correlated with the volume change properties summarized in Table 3.

Direct Shear Tests

Consolidated-Drained (CD) Direct Shear tests (ASTM D 3080) were performed on relatively undisturbed samples of fill and bedrock obtained using a CRS from Borings B-1, B-2, B-3, and B-5. Samples were soaked in water for 24 hours under the applied test normal stress. Following consolidation, samples were placed in the shear box and reloaded with the same surcharge load for a 1 hour period prior to shearing. Three specimens from each sample were sheared under applied normal stresses of 1000, 2000, and 4000 psf. The maximum shear displacement in each test was 0.25 inches which restricts the ability to obtain a constant volume friction angle for each test. The strain rate reported by TGR for each sample was 0.2 in/sec (1.2 in/min). This is an extremely rapid rate for a fine grained, relatively low permeability material (e.g. Silt) and may have had an effect on test results whereby excess pore water pressures generated during shear cannot dissipate rapidly enough relative to the rate of shearing. The sample is thus sheared in a partially-drained condition rather than the fully drained condition assumed in the test. Excess pore water pressures may result in an apparent cohesion \( c \) in an otherwise cohesionless material and may also reduce the measured friction angle relative to the
fully drained condition. These considerations must be accounted for when utilizing the reported shear strength data summarized in Table 4. The results summarized in Table 4 are based on a re-analysis of raw load-displacement data provided by TGR and differ from results presented in their report. Included in the results are the peak shear strength values of apparent cohesion and the peak friction angle. Post-peak values determined at a shear displacement of 0.25 inches are also included in some cases.

Table 4, Hacienda Road (Upper) - Direct Shear Test Summary

<table>
<thead>
<tr>
<th>Boring</th>
<th>Depth (ft)</th>
<th>Description</th>
<th>γd (pcf)</th>
<th>Sat. w (%)</th>
<th>w (%)</th>
<th>cpeak (psf)</th>
<th>c0.25&quot; (psf)</th>
<th>φpeak</th>
<th>φ0.25&quot;</th>
<th>Dilation (ψ°) or Contraction</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1</td>
<td>15</td>
<td>Sand and Silt (FILL)</td>
<td>83</td>
<td>38</td>
<td>18</td>
<td>12</td>
<td>-</td>
<td>33.4</td>
<td>-</td>
<td>3° contractive</td>
</tr>
<tr>
<td>B-1</td>
<td>45</td>
<td>Claystone/Siltstone</td>
<td>82</td>
<td>39</td>
<td>24</td>
<td>642</td>
<td>-</td>
<td>25.4</td>
<td>-</td>
<td>5° contractive</td>
</tr>
<tr>
<td>B-2</td>
<td>10</td>
<td>Silt (FILL)</td>
<td>87</td>
<td>35</td>
<td>15</td>
<td>69</td>
<td>-</td>
<td>32.4</td>
<td>-</td>
<td>contractive contractive</td>
</tr>
<tr>
<td>B-3</td>
<td>10</td>
<td>Sandstone/Siltstone - weathered</td>
<td>102</td>
<td>24</td>
<td>15</td>
<td>426</td>
<td>253</td>
<td>28.1</td>
<td>28.1</td>
<td>8° contractive contractive</td>
</tr>
<tr>
<td>B-3</td>
<td>30</td>
<td>Sandstone/Siltstone</td>
<td>91</td>
<td>32</td>
<td>26</td>
<td>564</td>
<td>450</td>
<td>30.4</td>
<td>26.8</td>
<td>12° 12° 2°</td>
</tr>
<tr>
<td>B-5</td>
<td>10</td>
<td>Sandstone/Siltstone</td>
<td>104</td>
<td>23</td>
<td>16</td>
<td>612</td>
<td>253</td>
<td>35.3</td>
<td>30.8</td>
<td>27° 20° 9°</td>
</tr>
</tbody>
</table>

**Sand / Silt (FILL).**

Direct shear tests were performed on samples of fill obtained from depths of 15 and 10 feet in Borings B-1 and B-2, respectively. A peak friction angle of 33.4° with an apparent cohesion of 12 psf was determined for the sample from B-1 while these same parameters were measured as 32.4° and 69 psf in B-2. The post-peak values were not determined due to insufficient shear displacement.

**Puente Formation.**

In the case of samples taken at a depth of 10 feet in borings B-3 and B-5, the sandstone / siltstone had well defined peaks in the load-displacement curves – particularly for boring B-5. These peaks occurred at average shear displacements of 0.1 and 0.08 inches, respectively. Both of these samples also had higher unit weights and lower saturation moisture contents (i.e. 102 and 104 pcf and; 24 and 23 percent, respectively) than other samples of the Puente Formation that were tested. These other samples include a sample taken from a depth of 45 feet in B-1 and a sample taken from a depth of 30 feet in B-3. The dry unit weight and the saturation moisture content for each of these samples were 82 and 91 pcf and 39 and 32 percent, respectively. Based upon reported bedding dip, all shear surfaces were oblique to the bedding planes although their absolute orientation is unknown as is the direction of shear relative to the bedding plane. Likewise, the samples were described as sandstone and siltstone but the type of material on the shear surface was not reported.

The peak friction angle varied in the sandstone / siltstone from 28.1° to 35.3° while the peak apparent cohesion varied from 426 psf to 612 psf. The finer grained sample taken from B-1 and described as claystone / siltstone had a peak friction angle of 25.4° and an apparent cohesion of 642 psf. The shear strength of this sample may be representative of the available strength on the bedding plane when the nearby bedding plane dips of 5° to 15° are considered. Due to the indurated (i.e. cemented) nature of many sedimentary rocks, a cohesion intercept can be expected at peak strength but should diminish rapidly during post-peak straining due to the breaking of interparticle bonds.

Post-peak friction angles measured at a shear displacement of 0.25 inches in the sandstone / siltstone varied from 26.8° to 30.8° while the apparent cohesion varied from 253 psf to 450 psf. The post-peak values for the claystone / siltstone sample were not determined since the load-displacement curve had just peaked at a shear displacement of 0.25 inches.
Colluvium

Unfortunately, direct shear tests were not performed on any of the samples of colluvium obtained from Boring B-4.

Evaluation

Asphalt concrete pavement thickness was measured as 24 inches in Boring B-4 at Site 1 and 19 inches in Boring B-1 at Site 2. No pavement base course was encountered in either of these borings. The pavement thickness diminished to 7 inches in Boring B-3 which was drilled in the stable cut area between Sites 1 and 2. This suggests that the ground surface has settled 17 inches at Site 1 and 12 inches at Site 2 since the time the road was constructed to its current configuration.

Below the pavement surface, loose to medium dense sand and silt extend to interbedded sandstone and siltstone/claystone of the Puente Formation. At Site 2, this loose sand and silt is interpreted as an “undocumented” fill with a moderate potential for collapse settlement (i.e. $I_c = 4.0$) near the surface. Considering the 39.9 foot thickness of fill at Site 2 and the estimated 12 inches of long term ground surface settlement, the fill has settled 2.5 percent since it was placed. This could be attributable to collapse settlements that may have occurred during a period of high groundwater. However, once these settlements had occurred they would have been unlikely to be repeated. Continuing road maintenance operations have resulted in the substantial pavement thicknesses noted above. Presumably, if the settlements had occurred all at once, the maintenance crews would have opted for an alternative solution to simply filling the disturbed areas with 19 to 24 inches of asphalt concrete. It is therefore reasonable to conclude that the settlement has occurred over a long period of time. The sand and silt fill would not experience the type of long term consolidation settlement experienced in saturated clays because it is associated with both the saturation and low permeability of these clays. Both of these factors are absent with the fill. Therefore, it is concluded that the most probable cause of the observed pavement distress and abnormal thickness is the long term creep of the sand and silt in the fill slope.

The probable cause of distress is further reinforced when considering Site 1 which has only a slight collapse potential near the surface and yet has experienced 17 inches of settlement, or 7.8 percent, since the time of construction.

Slope movements are often episodic and seasonal. Slope creep can occur within a zone of weakness within the fill or colluvium or it may occur along bedding planes or discontinuities in the rock. Considering the later, all measured bedding planes dip either into the slope away from the free face or essentially parallel to the face. Movement along these planes is therefore restricted by adjacent soil or rock and can be eliminated from further consideration as a potential source of the observed distress.

The shear strength of intact particulate geo-materials (i.e. soil and rock) is related to many factors including but not limited to:

1. whether the material is in a saturated or unsaturated condition;
2. whether the material is cohesive (i.e. clay) or cohesionless (i.e. sand) together with the distribution of particle sizes in cohesionless materials;
3. the degree of particle packing and structural arrangement as evident by the relative density or the saturated moisture content;
4. the permeability of the material and rate at which it is brought to failure; and
5. in the case of unsaturated materials, the degree of saturation and the magnitude of the consequent matric suction.

For the current discussion, the most important of these are the relative density and the unsaturated state of samples obtained from the borings.

A qualitative description of the relative density of driven samples is included in Plate 5. With the exception of the penetration test made at 15 feet ($N_{60}=15$), the estimated equivalent SPT ($N_{60}$) values varied from 8 to 11 blows per foot in the upper 20 feet of fill in Boring B-1 at Site 2. Below 20 feet, the equivalent ($N_{60}$) varied from 12 to 18 blows per foot. Similarly, the ($N_{60}$) values varied from 8 to 14 blows per foot in the colluvium (or fill?) in Boring B-4 at Site 1. The relative density of these materials is loose to
medium dense. Except at low confining pressures, loose cohesionless materials will not exhibit a substantial increase in peak shear strength relative to the strength available at greater shear displacements.

The shear strength in unsaturated materials is directly related to the surface tension that forms between water surfaces and particles in the unsaturated voids of the material. As the degree of saturation increases, the surface tension, or matric suction, decreases as does its contribution to shear strength. Consequently, as the degree of saturation increases there is an accompanying reduction in shear strength. The degree of saturation for each sample is plotted against depth in each boring in Plate 6. Assuming that the shear strength is lowest in the zones with the highest degree of saturation, the data in Plate 6 suggest that the weakest material lies at depths of 10 and 20 feet at Sites 1 and 2, respectively at the time these samples were obtained in early October. These zones occur in the colluvium and fill at Sites 1 and 2, respectively. Considering the time of year of sampling, the zone between the ground surface and these depths is likely to experience the greatest seasonal fluctuation in saturation and therefore the greatest seasonal variability in shear strength. It is therefore concluded that the shear zone lies between 0 and 10 feet in Boring B-4 at Site 1 and between 0 and 20 feet in Boring B-1 at Site 2.

**Stabilization Approach**

In the simplest terms, slope stabilization may be accomplished by any combination of reducing driving forces and/or increasing resisting forces. Driving forces may be reduced by simply removing material that under the force of gravity or the application of seismic ground motions places an excessive demand on the available resisting capacity within the slope. These resisting forces consist of some combination of favorable slope geometry and shear strength. A reduction in shear strength within the slope may be prevented through the installation of subsurface drains if the presence of groundwater is a factor contributing to instability. The shear strength within the soil or rock mass may be increased by the installation of reinforcement such as soil nails or micropiles. These methods of reinforcement essentially add a tensile resistance to the soil or rock mass unlike conventional stabilization schemes in which external structural elements impose external resisting forces through various means in order to provide stability. The approach taken herein is to use a combination of driving force reduction and resisting force augmentation through the utilization of *in situ* reinforcement (*i.e.* soil nails and micropiles).

**Design Parameters**

**Design Loads**

The following loads were used for design.

**Table 5, Design Loads**

<table>
<thead>
<tr>
<th>Load</th>
<th>Magnitude</th>
</tr>
</thead>
<tbody>
<tr>
<td>Static Surcharge</td>
<td>250 psf</td>
</tr>
<tr>
<td>Seismic Surcharge</td>
<td>0 psf</td>
</tr>
<tr>
<td>Peak Ground Acceleration (PGA)(^1)</td>
<td>0.52</td>
</tr>
<tr>
<td>Pseudo-Static Seismic Coefficient ((k_s))(^2)</td>
<td>0.19</td>
</tr>
</tbody>
</table>

\(^1\) 475 Year Return Period PGA determined from probabilistic FRISKSP analysis using an attenuation relationship for the Corrected PGA on Soft Rock (Bozorgnia, Campbell, and Niazi, 1999).

\(^2\) Seismic coefficient estimated using a maximum seismic displacement of 6 inches.

**Soil and Rock Properties**

Properties used for the analysis and design of the soil nail and micropile walls are presented in Tables 6 through 9. These were developed through the evaluation of test data, the back-analysis of existing slopes, and the application of experience and engineering judgment. Estimated statistical properties used in probabilistic design analyses are also presented.
The existing slopes were analyzed using both limit equilibrium (LE) and finite element (FEM) – shear strength reduction (SSR) analyses. Assumptions were made for the elastic and hydraulic properties of the soil and rocks used in these analyses. The slopes were evaluated under varying groundwater boundary conditions and using the friction angles reported in the TGR report. The cohesion necessary to achieve a factor of safety of unity (i.e. incipient failure) was evaluated as was, in the case of LE analyses, the location of the critical failure surface relative to the observed pavement distress. When groundwater seepage emerged from the slope face above the toe, critical failure surfaces were observed along the seepage face with computed factors of safety less than unity. Use of the FEM SSR method allowed displacements and the zones of maximum shear strain to be identified and compared with observed crack locations.

Table 6, Site 1 - Design Soil and Rock Properties

<table>
<thead>
<tr>
<th>Material</th>
<th>Maximum Depth Range (ft)</th>
<th>Total Unit Weight, γ&lt;sub&gt;total&lt;/sub&gt; (pcf)</th>
<th>Elastic Modulus, E (ksf)</th>
<th>Poisson's Ratio, ν</th>
<th>Shear Strength</th>
<th>Cohesion, c&lt;sup&gt;′&lt;/sup&gt; (psf)</th>
<th>Friction Angle, φ&lt;sup&gt;′&lt;/sup&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand and Silt – Loose – FILL/Colluvium (?)</td>
<td>0 – 18</td>
<td>100</td>
<td>420</td>
<td>0.35</td>
<td>32</td>
<td>32º</td>
<td></td>
</tr>
<tr>
<td>Sandstone – Siltstone – Claystone (Puente Formation)</td>
<td>18 +</td>
<td>117</td>
<td>1000</td>
<td>0.40</td>
<td>270</td>
<td>27º</td>
<td></td>
</tr>
</tbody>
</table>

Table 7, Site 2 - Design Soil and Rock Properties

<table>
<thead>
<tr>
<th>Material</th>
<th>Maximum Depth Range (ft)</th>
<th>Total Unit Weight, γ&lt;sub&gt;total&lt;/sub&gt; (pcf)</th>
<th>Elastic Modulus, E (ksf)</th>
<th>Poisson’s Ratio, ν</th>
<th>Shear Strength</th>
<th>Cohesion, c&lt;sup&gt;′&lt;/sup&gt; (psf)</th>
<th>Friction Angle, φ&lt;sup&gt;′&lt;/sup&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand and Silt – Loose - FILL</td>
<td>0 – 25</td>
<td>100</td>
<td>420</td>
<td>0.35</td>
<td>70</td>
<td>32º</td>
<td></td>
</tr>
<tr>
<td>Sand – Silt – Med. Dense - FILL</td>
<td>25 - 42</td>
<td>103</td>
<td>600</td>
<td>0.35</td>
<td>112</td>
<td>32º</td>
<td></td>
</tr>
<tr>
<td>Colluvium</td>
<td>42 - 45</td>
<td>110</td>
<td>1000</td>
<td>0.40</td>
<td>100</td>
<td>28º</td>
<td></td>
</tr>
<tr>
<td>Sandstone – Siltstone – Claystone (Puente Formation)</td>
<td>45 +</td>
<td>117</td>
<td>5000</td>
<td>0.40</td>
<td>270</td>
<td>27º</td>
<td></td>
</tr>
</tbody>
</table>

Table 8, Site 1 - Estimated Statistical Properties

<table>
<thead>
<tr>
<th>Material</th>
<th>Soil Property</th>
<th>Statistical Distribution</th>
<th>Mean</th>
<th>Standard Deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Colluvium - Silt and Sand</td>
<td>Cohesion</td>
<td>Normal</td>
<td>32 psf</td>
<td>10 psf</td>
</tr>
<tr>
<td>Colluvium - Silt and Sand</td>
<td>Friction Angle</td>
<td>Normal</td>
<td>32º</td>
<td>0.667º</td>
</tr>
<tr>
<td>Colluvium - Silt and Sand</td>
<td>Unit Weight</td>
<td>Normal</td>
<td>100 pcf</td>
<td>2 pcf</td>
</tr>
</tbody>
</table>

Table 9, Site 2 - Estimated Statistical Properties

<table>
<thead>
<tr>
<th>Material</th>
<th>Soil Property</th>
<th>Statistical Distribution</th>
<th>Mean</th>
<th>Standard Deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Colluvium - Silt and Sand</td>
<td>Cohesion</td>
<td>Normal</td>
<td>32 psf</td>
<td>10 psf</td>
</tr>
<tr>
<td>Colluvium - Silt and Sand</td>
<td>Friction Angle</td>
<td>Normal</td>
<td>32º</td>
<td>0.667º</td>
</tr>
<tr>
<td>Colluvium - Silt and Sand</td>
<td>Unit Weight</td>
<td>Normal</td>
<td>100 pcf</td>
<td>2 pcf</td>
</tr>
<tr>
<td>FILL - Sand and Silt - Medium Dense</td>
<td>Cohesion</td>
<td>Normal</td>
<td>112 psf</td>
<td>20 psf</td>
</tr>
<tr>
<td>FILL - Sand and Silt - Medium Dense</td>
<td>Friction Angle</td>
<td>Normal</td>
<td>32º</td>
<td>0.667º</td>
</tr>
<tr>
<td>FILL - Sand and Silt - Medium Dense</td>
<td>Unit Weight</td>
<td>Normal</td>
<td>103 pcf</td>
<td>2 pcf</td>
</tr>
</tbody>
</table>
Soil Nails and Micropiles

A constant allowable bond stress of 10 psi was used for both soil nails and micropiles. Effective drillhole diameters for determining soil-grout interface surface area were 4 and 6 inches for soil nails and micropiles, respectively. The resulting allowable tensile load was 1,508 pounds per lineal foot (plf) and 2,262 plf for soil nails and micropiles, respectively.

Several different types of soil nails and micropiles were analyzed. These are summarized in Table 10 which includes their design properties as well as wall facing capacities for different reinforcement patterns.

### Table 10, Alternative Soil Nails, Micropiles, and Wall Facings Analyzed

<table>
<thead>
<tr>
<th>Type - Description</th>
<th>Component / Mode</th>
<th>( F_y ) (ksi)</th>
<th>O.D. (in)</th>
<th>I.D. (in)</th>
<th>( A ) (in(^2))</th>
<th>( I ) (in(^4))</th>
<th>Allow. Pullout (plf)</th>
<th>Static ( T_{allow} ) (kips)</th>
<th>Seismic ( T_{allow} ) (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil Nail – Standard Nail, Grade 75 No. 9 Epoxy Coated All Thread Bar – Grouted in 4&quot; Drilled Hole</td>
<td>All Thread Bar</td>
<td>75</td>
<td>1.25</td>
<td>-</td>
<td>1.00</td>
<td>0.0491</td>
<td>1508</td>
<td>41.7</td>
<td>48.0</td>
</tr>
<tr>
<td>Soil Nail – SuperNail™, Hollow Steel Tube w/ internal No. 6 Epoxy Coated Bar grouted into tube - Composite Grouted in 4&quot; Drilled Hole</td>
<td>Steel Tube</td>
<td>42</td>
<td>1.66</td>
<td>1.38</td>
<td>0.62</td>
<td>0.184</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Inner Bar</td>
<td>75</td>
<td>0.75</td>
<td>-</td>
<td>0.44</td>
<td>0.0275</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Composite</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>1.06</td>
<td>0.2115</td>
<td>1508</td>
<td>24.7</td>
<td>40.4</td>
</tr>
<tr>
<td>Soil Nail – Launched Fiberglass Composite w/ Outer FG Tube and Inner No. 6 Steel Bar grouted into annulus - FG bar is perforated and launched into ground then inner bar is grouted w/ grout allowed to permeate surrounding ground under low pressure. (FG E=2800 ksi)</td>
<td>FG Tube</td>
<td>30</td>
<td>1.50</td>
<td>1.25</td>
<td>0.54</td>
<td>0.13</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Inner Bar</td>
<td>75</td>
<td>0.75</td>
<td>-</td>
<td>0.44</td>
<td>0.0275</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Composite</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>5.08</td>
<td>0.1308</td>
<td>566</td>
<td>24.7</td>
<td>40.4</td>
</tr>
<tr>
<td>Micropile – Williams T76N Hollow Injection Bar</td>
<td>Hollow Injection Bar</td>
<td>95</td>
<td>3.0</td>
<td>2.0</td>
<td>2.84</td>
<td>3.19</td>
<td>2262</td>
<td>149.9</td>
<td>245.3</td>
</tr>
<tr>
<td>Micropile – Titan 40/16 Hollow Injection Bar</td>
<td>Hollow Injection Bar</td>
<td>85</td>
<td>1.57</td>
<td>0.63</td>
<td>1.36</td>
<td>0.2942</td>
<td>2262</td>
<td>64.6</td>
<td>105.8</td>
</tr>
<tr>
<td>Wall Facing – 8 inch thick Shotcrete w/ 4x4-W4xW4 wire mesh</td>
<td>Flexure</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>13.5</td>
<td>18.4</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Punching Shear</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>26.1</td>
<td>35.6</td>
<td></td>
</tr>
<tr>
<td>Wall Facing – 8 inch thick Shotcrete w/ 4x4-W4xW4 wire mesh and No. 5 walers</td>
<td>Flexure</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>23.3</td>
<td>31.7</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Punching Shear</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>26.1</td>
<td>35.6</td>
<td></td>
</tr>
<tr>
<td>Wall Facing – 8 inch thick Shotcrete w/ 4x4-W4xW4 wire mesh and No. 6 walers</td>
<td>Flexure</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>27.4</td>
<td>37.4</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Punching Shear</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>26.1</td>
<td>35.6</td>
<td></td>
</tr>
</tbody>
</table>

SuperNail™ is a registered trademark of Soil Nail Launcher, Inc.

The interstitial and drillhole grout was considered to be cracked and not contribute to flexure rigidity of reinforcing elements. The FEM modeling results show that the tubular elements contribute bending resistance which has limited benefit to the performance of the overall structure. The primary resisting forces in the reinforcement are axial. Therefore, the additional installation time for the composite nail alternatives seems unwarranted unless additional corrosion protection is desired.

**Site 1**

The proposed stabilization design at Site 1 is shown in Plate 7 while its plan extent and finish grade contours are shown on Plate 2. The design consists of the removal of potentially unstable material downslope of the zone of pavement distress and the construction of a soil nail wall with a maximum height of approximately 12 feet along the exposed cut face. The alignment of the top of the wall was established in such a manner as to provide a 4 foot shoulder from the existing edge of traveled way on Hacienda Road along with adequate room for guardrail installation while removing material outward from the observed...
pavement cracking to the greatest extent possible. Soil nails consist of 30 foot long, Grade 75 - No. 9 epoxy coated threaded bars installed in 4 inch diameter drill holes. Nails are spaced at 4 feet horizontally and vertically along the wall face.

Both probabilistic limit equilibrium (LE) analyses and staged construction finite element (FEM) analyses were used in the stabilization design. Estimated statistical parameters used in the probabilistic LE analyses are presented in Table 8 above. SNAIL analyses were not performed at Site 1 because of the large reinforcement length to wall height ratio (i.e. 2.5).

A groundwater model was developed to estimate pore water pressures for use in the LE analyses. Isotropic hydraulic conductivities of $10^{-7}$ cm/sec for the colluvium and $3 \times 10^{-8}$ cm/sec for the Puente Formation were used together with an assumed total head of 790 feet at the upslope limit of the model to develop pore water pressures for use in the LE stability analyses. Under a worst case scenario with this groundwater level, a 0.19 pseudo-static seismic coefficient, and a full 250 psf surcharge, the area downslope of the wall is marginally stable with a mean factor of safety (FS) of 1.03 and a 39.8 percent probability of failure. These analyses are summarized in Appendix D which also includes a graphic with computed global FS for failure surfaces passing through the road. The factor of safety for all these surfaces exceeds 1.20 and therefore the design is acceptable.

FEM modeling of the staged construction process was performed in order to estimate the maximum tensile forces within the nails and the forces at the nail heads. Simulation of the construction process allowed the re-distribution of loads in the nails to be evaluated as the excavation proceeds and then finally under service loads. The FEM analyses for Site 1 are summarized in Plates 8 and 9. Plate 8 defines each stage and presents a summary of nail forces for each stage. The variation in tensile force along each nail for each stage is plotted in Plate 9. The maximum tensile forces in the nails were 11.97 kips and 10.92 kips for static and seismic loading, respectively. The maximum nail head tensile forces were 10.62 kips and 7.69 kips for static and seismic loading, respectively.

Site 2

The proposed stabilization design at Site 2 is shown in Plate 10 while its plan extent and finish grade contours are shown on Plate 2. The design consists of the removal of potentially unstable material downslope of the zone of pavement distress and the construction of a soil nail wall with a maximum height of approximately 16 feet along the exposed cut face. In addition, a micropile wall is proposed within the cut area to protect the upper wall in case of failure of the lower slope during an earthquake. The alignment of the top of the wall was established in such a manner as to provide a 4 foot shoulder from the existing edge of traveled way on Hacienda Road along with adequate room for guardrail installation while removing material outward from the observed pavement cracking to the greatest extent possible. Soil nails consist of 30 foot long, Grade 75 - No. 9 epoxy coated threaded bars installed in 4 inch diameter drill holes. Nails are spaced at 3 feet horizontally and vertically along the wall face. The micropile wall consists of pairs of micropiles tied together at the ground surface in a concrete pile cap and spaced at 2.5 feet along the wall. The front pile is vertical while the rear pile is battered 15°. Micropiles consist of Titan 40/16 hollow injection bars drilled a minimum of 5 feet into sound bedrock.

Both probabilistic limit equilibrium (LE) analyses and staged construction finite element (FEM) analyses were used in the stabilization design. Estimated statistical parameters used in the probabilistic LE analyses are presented in Table 9 above.

A review of laboratory shear strength data and back-analyses of the existing slope suggest that Site 2 is marginally stable under the conditions encountered at the time of drilling. When groundwater seepage emerging at the toe of the slope was considered, the slope rapidly became unstable as the groundwater level rose into the fill. It quickly became apparent that any stabilization scheme must be carried to bedrock and that it must ensure stability of the road if the foreslope fails. There are limitations however to 2 dimensional LE or FEM analyses in a narrow canyon fill where 3 dimensional end effects and confinement can have a substantial affect on the computed factor of safety.
Probabilistic LE analyses were performed assuming a failed foreslope and various assumptions as to the stabilizing contribution of the micropiles. The effect of micropile behavior on the LE factor of safety was checked for the following scenarios.

1. Micropiles functioning as shear resisting elements with a 100 pound capacity per pile when a potential failure surface is crossed.
2. Micropiles functioning as shear resisting elements with a 34,900 pound capacity per pile when a potential failure surface is crossed.
3. Micropiles functioning as passive tensile resisting elements grouted over their full length and having a tensile structural capacity of 10,000 pounds and an allowable pullout capacity of 1,440 pounds per lineal foot.

2D FEM modeling with the slender micropiles treated as strips having little flexure rigidity suggests that the primary effect is the development of axial forces within the micropiles. Considering these findings and recent research (Loehr and Brown, 2008) scenario 3 probably best represents field behavior.

The critical failure surface in these analyses occurred on the face of the slumped foreslope block. Additional trial surfaces were plotted in the vicinity of the micropile wall and soil nail wall. The minimum global FS for rotational failures passing through or beyond the micropile or soil nail wall was 1.16 for a pseudo-static seismic coefficient of 0.19 together with a 250 psf surcharge load and a slumped foreslope.

SNAIL analyses were also preformed as a check on the LE and FEM analyses. The results of these analyses are included in Appendix E.

FEM modeling of the staged construction process was performed in order to estimate the maximum tensile forces within the nails and the forces at the nail heads. Simulation of the construction process allowed the re-distribution of loads in the nails to be evaluated as the excavation proceeds and then finally under service loads. The FEM analyses for Site 2 are summarized in Plates 11 and 12. Plate 11 defines each stage and presents a summary of nail forces for each stage. The variation in tensile force along each nail for each stage is plotted in Plate 12. The maximum tensile forces in the nails were 15.18 kips and 13.13 kips for static and seismic loading, respectively. The maximum nail head tensile forces were 15.17 kips and 10.34 kips for static and seismic loading, respectively. Micropile axial forces were also estimated with a computed maximum 4.10 kip compressive force and a maximum 1.72 kip tensile force.

Construction Recommendations

Materials encountered in Borings B-1 and B-4 consist of loose to medium dense sands and silts at depths up to the proposed soil wall heights. While some cohesion was measured during direct shear tests of these materials, the Contractor is cautioned that stand up times may be very short and that temporary measures may be required prior to drilling nail holes and completing excavation of a given lift. Likewise, suitable means of stabilizing drillholes may be required during soil nail installation.

HACIENDA ROAD (LOWER)

Two areas of shoulder distress were investigated which are referred to as Site 1 and Site 2 in this report and on the construction drawings. La Mirada Creek flows along the toe of the road embankment on the west side of Hacienda Road at Site 2. The creek then crosses Hacienda Road in two 60 inch diameter corrugated steel pipes which discharge at a confluence with a tributary that flows in from the east. Downstream of the 60 inch pipes and the confluence with the tributary, the creek flows along east side of the toe of the road embankment at Site 1.

The easterly half of Hacienda Road at Site 1 appears to be constructed on embankment fill adjacent to La Mirada Creek. The embankment rises about 5 to 6 feet above the poorly defined thalweg of the creek. The unpaved shoulder of the road embankment has eroded away over a length of approximately 145 feet as shown in Photo 2. The asphalt concrete pavement surface is starting to be undermined as well near the edge of travelled way. The most severe erosion extends for about 75 feet.
Photo 2, Shoulder Distress at Hacienda Road (Lower) Site 1

Photo 3, Shoulder Distress at Hacienda Road (Lower) Site 2
At Site 2, Hacienda Road appears to be constructed entirely on an embankment between La Mirada Creek and the easterly tributary. This embankment also rises about 5 to 6 feet above the adjacent lowlands. In general, the creek does not approach as closely to the road at Site 2 as it does at Site 1. However, there still has been considerable erosion of the road embankment as can be seen in Photo 3.

**Local Geology**

Both of the Hacienda Road (Lower) sites are located in the lowlands along La Mirada Creek. Morton and Miller (2006) have mapped these areas as very old alluvial fan deposits. The general topography, observed vegetation, and proximity to the creek would also suggest that more recent Holocene (i.e. within the past 11,000 years) alluvial deposits are likely present near the surface.

**Subsurface Exploration**

A total of two exploratory borings were drilled with a hand auger by TGR Geotechnical, Inc. personnel including one at each site as shown on Plate 13. The location of each boring is approximate.

Boring B-1 was drilled at Site 2 and advanced to a depth of 8 feet before being terminated without encountering bedrock. Groundwater was encountered at a depth of 2 feet. One CRS sample was obtained from 2 to 3.5 feet which had a moisture content of 25 percent and a dry unit weight of 98 pcf. A second sampling interval between 5 and 6.5 feet had no recovery. This boring encountered fine to medium grained Clayey SAND (SC) which TGR personnel interpreted as fill over its entire depth.

Boring B-2 was drilled at Site 1 and advanced to a depth of 8 feet before being terminated without encountering bedrock. Groundwater was encountered at a depth of 6 feet. Two CRS samples were obtained at depths of 2 to 3.5 and 6.5 to 8 feet, respectively. Corresponding moisture contents were 20 and 29 percent while corresponding dry unit weights were 90 and 93 pcf. A third sampling interval between 5 and 6.5 feet had no recovery. This boring encountered fine grained Clayey SAND (SC) which TGR personnel interpreted as fill over its entire depth.

**Laboratory Testing**

The moisture content and dry unit weight of CRS samples were determined and the results noted above.

A Consolidated Drained (CD) Direct Shear Test (ASTM D 3080) was performed on a relatively undisturbed sample obtained from a depth of 7 feet in Boring B-2 at Site 1. This sample was prepared and tested in the same manner as described for the samples from Hacienda Road (Upper). Shear displacement data were not obtained from TGR for this sample. The shear strength parameters obtained from this test as reported by TGR are presented in Table 11.

**Table 11, Hacienda Road (Lower) - Direct Shear Test Summary**

<table>
<thead>
<tr>
<th>Sample</th>
<th>Description</th>
<th>Friction Angle, $\phi$</th>
<th>Apparent Cohesion, $c$ (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-2 @ 7 feet</td>
<td>Brown Clayey SAND (SC)</td>
<td>33</td>
<td>50</td>
</tr>
</tbody>
</table>

**Evaluation and Recommendations**

Streambank erosion has occurred at two locations along the Hacienda Road (Lower) project site. The severity of this erosion varies but is most acute at Site 1 where the shoulder has entirely eroded away and the pavement surface is starting to be undermined (Photo 2). Left unchecked, erosion from future flood events along La Mirada Creek is likely to encroach into the traffic lanes on Hacienda Road.

Streambank stabilization measures are required both to protect the undamaged areas and to rehabilitate damaged areas. Suitable methods include the following.

1. The construction of a hybrid mass concrete wall to regain lost shoulder where the creek flows directly along the existing toe of the eroded and nearly vertical bank.
(2) Install High Performance Turf Reinforced Mat (HPTRM) where erosion is less severe to protect moderately eroded areas from future erosion and to re-build areas of lost embankment where the slopes have been over-steepened.

Proposed Stabilization Design

The proposed stabilization design consists of a combination of the two methods described above. At Site 1 it is proposed to stabilize the most severely damaged section using a mass concrete wall supported, and protected from scour, by ballistic soil nails / micropiles. The wall transition and approach areas at Site 1 should be protected with HPTRM (e.g. Propex Pyramat) anchored with 24 inch long pins spaced on an 18 inch offset grid. Boundaries and splices should be anchored at a spacing of 12 inch and splices must lap a minimum of 4 inches. Where stabilized slopes exceed a gradient of 1.5 horizontal to 1 vertical (1.5:1), percussion driven earth anchors (PDEA) such as Platipus S06E with a minimum tensile capacity of 3.3 kips and a design life greater than 30 years shall be provided at a maximum grid spacing of 3 feet. Due to the high groundwater conditions encountered in the borings, PDEA’s shall be driven 8 feet perpendicular to the slope.

At Site 2, the most severely eroded locations should be stabilized using TRM and engineered fill. These areas shall be reconstructed as to provide a minimum distance of 5 feet from the edge of pavement to the crest of the embankment slope. The toe of the engineered fill at these locations shall not encroach into the stream channel. The slope face may be constructed as steep as 0.5 to 1 provided TRM anchors with PDEA’s are provided on all embankment slopes as described above. Areas with slopes flatter than 1.5 to 1 which have not received new embankment material shall be protected by TRM anchored with pins as described above.

PROFESSIONAL STATEMENT

The recommendations and stabilization designs presented within this report are based upon the physical properties of soils and rock observed during a site reconnaissance and as described in the exploratory borings and laboratory test results presented by TGR Geotechnical, Inc. If during the course of construction, subsurface conditions are encountered which differ significantly from those detailed within this report, Steven C. Devin, P.E., G.E. should be contacted immediately in order to evaluate the applicability of this report and the stabilization designs presented herein to the changed conditions. Such an evaluation may result in changes to the recommendations made herein. The intent of this report was to evaluate site conditions and provide alternative stabilization methods. Although a site reconnaissance and subsurface exploration were made for the purposes described, the potential presence of soil or groundwater contamination was not investigated and no analytical laboratory testing was performed in this regard.

This report has been prepared for the exclusive use of Onward Engineering and the City of La Habra Heights and their retained design professionals in accordance with generally accepted geotechnical engineering practice common to the local area. No other warranty is made, express or implied.

REFERENCES


TGR Geotechnical, Inc. (2006), “Limited Geotechnical Investigation, Improvement to Hacienda Road between East Road and West Road, in the City of La Habra Heights, California,” Consulting Report.

ACKNOWLEDGEMENTS

The author would like to thank Mr. Majdi Ataya, P.E. and the staff of Onward Engineering for their patience and assistance in preparing this report. I would also like to thank Dr. Sanjay Govil, PhD, P.E., G.E. and his staff of TGR Geotechnical, Inc. for their assistance in providing clarifications when requested and for providing raw laboratory data from direct shear tests.
PLATES
Very old alluvial-fan deposits; Puente Formation; Soquel Member
Fernando Formation; Lower member
Fernando Formation; Upper member
Puente Formation; Yorba Member
Puente Formation; La Vida Member
Young landslide deposits?; Fernando Formation; Upper member, conglomerate
Fernando Formation; Lower member, conglomerate
Whittier Fault Zone
Hacienda Road (Upper)
Puente Formation; Soquel Member
Puente Formation; Boga Member
Young landslide deposits?
Puente Formation; La Vida Member
Puente Formation; Lower member
Puente Formation; Upper member
Puente Formation; Yorba Member
Puente Formation; La Vida Member
Puente Formation; Lower member
Puente Formation; Upper member
Puente Formation; Soquel Member
Puente Formation; Boga Member
Very old alluvial-fan deposits, Young landslide deposits?

Plate 1, Hacienda Road Project Locations and Surrounding Geology, La Habra Heights, California
Reference: Morton, D.M., Miller, F.K. Geologic Map of the San Bernardino and Santa Ana 30' x 60' Quadrangles, California
PLATE 5, Approximate Equivalent SPT Blow Counts
Corrected for Overburden Pressure
Hacienda Road (Upper) 10/04/2006

Approx. SPT ($N_1$) (blows/ft)

Depth (ft)

Boring B-1
Boring B-2
Boring B-3
Boring B-4
Boring B-5

Loose
Medium Dense
Dense
Very Loose
### Hacienda Road (Upper) Site 1 Finite Element Analyses Summary

No. 9 - 75 ksi Threaded Bar Soil Nails

<table>
<thead>
<tr>
<th></th>
<th>Excavation Stage 2</th>
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<th>Excavation Stage 3</th>
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<th>Excavation Stage 4</th>
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<td>Nail Head Force, $T_n$ (lb)</td>
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Axial Compression is positive

Notes:

- **Excavation Stage 1:** Preconstruction
- **Excavation Stage 2:** Excavate Lift 1 - Install **Row 1 Nails** - includes 250 psf Surcharge
- **Excavation Stage 3:** Excavate Lift 2 - Install **Row 2 Nails** - includes 250 psf Surcharge
- **Excavation Stage 4:** Excavate Lift 3 - Install **Row 3 Nails** - includes 250 psf Surcharge
- **Excavation Stage 5:** Pseudo-Static Seismic Stability Analysis with $k_h = 0.19$ - (No Surcharge) Shear Strength Reduction (SSR) Factor of Safety $= 1.39$

**Nail Design Forces:**

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Plate 9, Site 1 Nail Forces

Axial Force
Stage 2 - No. 9 - 75 ksi Threaded Bar Soil Nails with 250 psf Surcharge

Axial Force (lbs) (compression positive)
Distance (ft)
Row 1 Nails

Axial Force
Stage 3 - No. 9 - 75 ksi Threaded Bar Soil Nails with 250 psf Surcharge

Axial Force (lbs) (compression positive)
Distance (ft)
Row 1 Nails
Row 2 Nails

Axial Force
Stage 4 - No. 9 - 75 ksi Threaded Bar Soil Nails with 250 psf Surcharge

Axial Force (lbs) (compression positive)
Distance (ft)
Row 1 Nails
Row 2 Nails
Row 3 Nails

Axial Force
Stage 9 Seismic $K_s=0.19$
No. 9 - 75 ksi Threaded Bar Soil Nails (No Surcharge)

Axial Force (lbs) (compression positive)
Distance (ft)
Row 1 Nails
Row 2 Nails
Row 3 Nails
<table>
<thead>
<tr>
<th>Plate 11</th>
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<td>Hacienda Road (Upper) Site 2 Finite Element Analyses Summary</td>
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Axial Compression is positive

Notes:
- Excavation Stage 1: Preconstruction
- Excavation Stage 2: Excavate Lift 1 - Install Row 1 Nails - includes 250 psf Surcharge
- Excavation Stage 3: Excavate Lift 2 - Install Row 2 Nails - includes 250 psf Surcharge
- Excavation Stage 4: Excavate Lift 3 - Install Row 3 Nails - includes 250 psf Surcharge
- Excavation Stage 5: Excavate Lift 4 - Install Row 4 Nails - includes 250 psf Surcharge
- Excavation Stage 6: Excavate Lift 5 - Install Row 5 Nails - includes 250 psf Surcharge
- Excavation Stage 7: Install Micropiles
- Excavation Stage 8: Rotational failure of Foreslope - includes 250 psf Surcharge
- Excavation Stage 9: Pseudo-Static Seismic Stability Analysis with $k_s = 0.19$ - (No Surcharge) Shear Strength Reduction (SSR) Factor of Safety = 1.07

<table>
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<tr>
<th>Nail Design Forces:</th>
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<tr>
<th>Micropiles - Design Axial Forces</th>
<th>Vertical w/ 250 psf Surcharge</th>
<th>Vertical w/ No Surcharge ($k_s=0.19$)</th>
<th>Battered w/ 250 psf Surcharge</th>
<th>Battered w/ No Surcharge ($k_s=0.19$)</th>
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PLATE 12, Site 2, Nail and Micropile Force Summary

Axial Force
Stage 2 - No. 9 - 75 ksi Threaded Bar Soil Nails with 250 psf Surcharge

Axial Force
Stage 4 - No. 9 - 75 ksi Threaded Bar Soil Nails with 250 psf Surcharge

Axial Force
Stage 6 - No. 9 - 75 ksi Threaded Bar Soil Nails with 250 psf Surcharge

Axial Force
Stage 3 - No. 9 - 75 ksi Threaded Bar Soil Nails with 250 psf Surcharge

Axial Force
Stage 5 - No. 9 - 75 ksi Threaded Bar Soil Nails with 250 psf Surcharge

Axial Force
Stage 8 - No. 9 - 75 ksi Threaded Bar Soil Nails with 250 psf Surcharge

Row 1 Nails
Row 2 Nails
Row 3 Nails
Row 4 Nails
Row 5 Nails
PLATE 12, Site 2, Nail and Micropile Force Summary

**Axial Force**
Stage 9 Seismic \( k_h=0.19 \)
No. 9 - 75 ksi Threaded Bar Soil Nails (No Surcharge)

- Distance (ft)
- Axial Force (lbs) (compression positive)

**Row 1 Nails**
**Row 2 Nails**
**Row 3 Nails**
**Row 4 Nails**
**Row 5 Nails**
Appendix A
Hacienda Road (Upper)
TGR Geotechnical, Inc.
Boring Logs
THE FOLLOWING DESCRIBES THE TERMS AND SYMBOLS USED ON THE LOG OF BORINGS TO SUMMARIZE THE RESULTS OBTAINED IN THE FIELD INVESTIGATION AND SUBSEQUENT LABORATORY TESTING.

**DENSITY AND CONSISTENCY**

The consistency of fine grained soils and the density of coarse grained soils are described on the basis of the Standard Penetration Test as follows:

<table>
<thead>
<tr>
<th>FINE GRAINED SOILS</th>
<th>Estimated Unconfined Compressive Strength (Tsf)</th>
<th>COARSE GRAINED SOILS</th>
</tr>
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<tbody>
<tr>
<td>Very Loose</td>
<td>&lt; 4</td>
<td>Very soft</td>
</tr>
<tr>
<td>Loose</td>
<td>4 – 10</td>
<td>Soft</td>
</tr>
<tr>
<td>Medium</td>
<td>10 – 30</td>
<td>Firm (medium)</td>
</tr>
<tr>
<td>Dense</td>
<td>30 – 50</td>
<td>Stiff</td>
</tr>
<tr>
<td>Very dense</td>
<td>&gt; 50</td>
<td>Very stiff</td>
</tr>
</tbody>
</table>

PARTICLE SIZE DEFINITION (As per ASTM D2487 and D422)

- Boulder => Larger than 12 inches
- Cobbles => 3 to 12 inches
- Coarse Gravel => 3/4 to 3 inches
- Fine Gravel => No. 4 to 3/4 inches
- Coarse Sands => No. 10 to No. 4 sieve
- Medium Sands => No. 40 to No. 10 sieve
- Fine Sands => No. 200 to 40 sieve
- Silt => 5μm to No. 200 sieve
- Clay => Smaller than 5μm

**SOIL CLASSIFICATION**

Soils and bedrock are classified and described based on their engineering properties and characteristics and using ASTM D2487 and D2488.

Percentage description of minor components

- Trace 1-10 % Some 20-35 %
- Little 10-20 % And or y 35-50 %

Stratified soils description

- Parting 0 to 1/16 inch thick Layer 1/2 to 12 inches thick
- Seam 1/16 to 1/2 inch thick Stratum > 12 inches thick
### LOG OF EXPLORATORY BORING B-1

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>FIELD RESULTS</th>
<th>LAB RESULTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td></td>
<td>18 72 C</td>
</tr>
<tr>
<td>5</td>
<td>Fill</td>
<td>12 95</td>
</tr>
<tr>
<td>10</td>
<td></td>
<td>18 83 S</td>
</tr>
<tr>
<td>15</td>
<td></td>
<td>24 81</td>
</tr>
<tr>
<td>20</td>
<td></td>
<td></td>
</tr>
<tr>
<td>25</td>
<td></td>
<td>23 80</td>
</tr>
</tbody>
</table>

**Summary of Subsurface Conditions**

- **Fill**: PAVEMENT, 19 inches Asphalt + No Base.
- **SAND/SILT (Fill)**, light brown, fine to medium grained, silty to sandy, fine to coarse sized gravel, gypsum, damp. ... loose, moist.
- ... medium dense.
- ... layer of light olive brown low plastic sandy clay.
**LOG OF EXPLORATORY BORING B-1**

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>FIELD RESULTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Graphic Log</td>
<td>Bulk Sample</td>
</tr>
<tr>
<td>10</td>
<td></td>
</tr>
<tr>
<td>38</td>
<td></td>
</tr>
<tr>
<td>40</td>
<td></td>
</tr>
</tbody>
</table>
| 45        |             |             |                         | Puente Formation (Tdp) | claystone/siltstone, tan to rust-brown, interbedded with sandstone, dense and hard.
| >50       |             |             |                         |                                  | well bedded sandstone and siltstone, thinly bedded. |
| 50        |             |             |                         |                                  | B: NS, 10W @ 46', thin sandstone bedding. |

**LAB RESULTS**

<table>
<thead>
<tr>
<th>Moisture Content (%)</th>
<th>Dry Density (lb/ft³)</th>
<th>Other Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>16</td>
<td>90</td>
<td></td>
</tr>
<tr>
<td>14</td>
<td>90</td>
<td></td>
</tr>
<tr>
<td>24</td>
<td>82</td>
<td>8</td>
</tr>
</tbody>
</table>

Boring terminated at approximately 50 feet below asphalt surface. No groundwater or seepage encountered. Hole backfilled with soil cuttings, compacted, and patched with AC.
LOG OF EXPLORATORY BORING B-2

Project Number: 06-1597
Project Name: Hacienda Road
Date Drilled: 10/4/06 - 10/4/06
Ground Elev: TBD

Logged By: Z.Y. A.S
Project Engineer: Zaher Yazeji
Drill Type: Bucket Auger
Drive Wt & Drop:

FIELD RESULTS

<table>
<thead>
<tr>
<th>Depth</th>
<th>Bulk Sample</th>
<th>Drive Sample</th>
<th>SPT Blowcount</th>
<th>Pocket Pen (ft)</th>
<th>USCS</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td></td>
<td></td>
<td>1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td></td>
<td></td>
<td>1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>15</td>
<td></td>
<td></td>
<td>4</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

SUMMARY OF SUBSURFACE CONDITIONS

Fill, light yellowish brown, low plastic, sandy fine grained, trace of gravel, gypsum, damp.

... soft.

Corrugated metal pipe encased in a thin coated concrete cover, at 17 feet below existing grade.

Boring terminated at approximately 17 feet below asphalt surface due to refusal.
No groundwater or seepage encountered.
Hole backfilled with soil cuttings, compacted, and patched with AC.
# LOG OF EXPLORATORY BORING B-3

**Project Number:** 06-1597  
**Logged By:** Z.Y. A.S  
**Project Name:** Hacienda Road  
**Project Engineer:** Zaher Yaziji  
**Date Drilled:** 10/5/06 - 10/5/06  
**Drive Type:** Bucket Auger  
**Ground Elev:** TBD  
**Drive Wt & Drop:**  

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>FIELD RESULTS</th>
<th>LAB RESULTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>FILL (FILL), light brown to grayish brown, mottled, with siltstone fragments, firm, moist.</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>PUEENTE FORMATION (Tp). Sandstone/Siltstone, interbedded, well-bedded, typically 1/4&quot; to 3&quot; thick, moderately jointed, highly weathered above 0'. B: N50E, 28NW @ 4', weathered bedrock. B: N35E, 19NW @ 8', 1/4&quot; thick sandstone bed.</td>
<td>16 107</td>
</tr>
<tr>
<td>10</td>
<td></td>
<td></td>
</tr>
<tr>
<td>15</td>
<td></td>
<td></td>
</tr>
<tr>
<td>17</td>
<td></td>
<td></td>
</tr>
<tr>
<td>20</td>
<td></td>
<td></td>
</tr>
<tr>
<td>25</td>
<td></td>
<td></td>
</tr>
<tr>
<td>28</td>
<td>Boring terminated at approximately 30 feet below existing asphalt surface. No groundwater or seepage encountered. Hole backfilled with soil cuttings, compacted, and patched with AC.</td>
<td>26 91 S</td>
</tr>
</tbody>
</table>

**SUMMARY OF SUBSURFACE CONDITIONS**

This Boring Log should be evaluated in conjunction with the complete geotechnical report. This Boring Log represents conditions observed at the specific location and date indicated. It is not warranted to be representative of subsurface conditions at other locations and times.
**LOG OF EXPLORATORY BORING B-4**

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Field Results</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 5</td>
<td>AC Fill</td>
</tr>
<tr>
<td>5 - 10</td>
<td>PAVEMENT, 24-inch Asphalt + No Base.</td>
</tr>
<tr>
<td>10 - 15</td>
<td>COLLUVIUM (F1), light brown sand and silt, fine to medium grained, mottled, loose, damp.</td>
</tr>
<tr>
<td>15 - 20</td>
<td>PUEENTE FORMATION (T1), interbedded sandstone and siltstone.</td>
</tr>
<tr>
<td>20 - 25</td>
<td>... weathered.</td>
</tr>
<tr>
<td>25 - 28</td>
<td>B: N30E, 35NW @ 21°</td>
</tr>
<tr>
<td>28 - 30</td>
<td>B: N40W, 43SW @ 24°</td>
</tr>
<tr>
<td>30 - 35</td>
<td>B: N35W, 45SW @ 26°</td>
</tr>
</tbody>
</table>

**SUMMARY OF SUBSURFACE CONDITIONS**

- MOISTURE CONTENT (%) | DRY BULK, Max. Corr. | OTHER TESTS |
- 20 | 82 | C
- 24 | 88 |
- 19 | 85 |

**LAB RESULTS**

- **Moisture Content (%)** 20, 24, 19
- **Dry Bulk, Max. Corr.** 82, 88, 85

This Boring Log should be evaluated in conjunction with the complete geotechnical report. This boring log represents conditions observed at the specific location and data indicated, it is not warranted to be representative of subsurface conditions at other locations and times.

**PLATE 5**

---

Steven C. Devin, P.E., G.E.  
Hacienda Road Slope Stabilization  
La Habra Heights, California
**LOG OF EXPLORATORY BORING B-5**

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Graphic Log</th>
<th>Bulk Sample</th>
<th>Bucket Sample</th>
<th>Pocket Pen</th>
<th>SPT Blow</th>
<th>Equivalent N</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6 - 10</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>11 - 15</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**FIELD RESULTS**

**SUMMARY OF SUBSURFACE CONDITIONS**

**PUENTE FORMATION (Tp), interbedded sandstone and siltstone.**

- B: N50E, 10NW @ 2'
- B: N15W, 30SW @ 4'
- B: N15W, 32SW @ 7'
- B: N25W, 32SW @ 10'

Boring terminated at approximately 15 feet below existing grade. No groundwater or seepage encountered. Hole backfilled with soil cuttings, compacted, and patched with AC.

**LAB RESULTS**

<table>
<thead>
<tr>
<th>Moisture Content (%)</th>
<th>Dry Density (pcf)</th>
<th>Other Tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>46</td>
<td>73</td>
<td></td>
</tr>
<tr>
<td>16</td>
<td>104</td>
<td>S</td>
</tr>
</tbody>
</table>

---

This Boring Log should be evaluated in conjunction with the complete geotechnical report. This Boring Log represents conditions observed at the specific location and date indicated. It is not warranted to be representative of subsurface conditions at other locations and times.
Appendix B
Hacienda Road (Lower)
TGR Geotechnical, Inc.
Boring Logs
# Hacienda Road Slope Stabilization
La Habra Heights, California

## LOG OF EXPLORATORY BORING B-2

### Sheet 1 of 1

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>FIELD RESULTS</th>
<th>LAB RESULTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 5</td>
<td>SAND (fill), brown, fine grained, clayey, moist.</td>
<td>20 60</td>
</tr>
<tr>
<td>5 - 20</td>
<td>No recovery, very wet.</td>
<td>29 93 5</td>
</tr>
<tr>
<td>20 - 29</td>
<td>Boring terminated at approximately 6 feet below existing grade. Water encountered at approximately 6 feet below existing grade. Hole backfilled with soil cuttings.</td>
<td></td>
</tr>
</tbody>
</table>

**Summary of Subsurface Conditions**

- **SF**: Shelby Tube
- **SS**: Standard Split Spoon
- **NC**: Modified California
- **W**: Water Table
- **A**: ATD

---

This Boring Log should be evaluated in conjunction with the complete geotechnical report. This Boring Log represents conditions observed at the specific location and date indicated. It is not warranted to be representative of subsurface conditions at other locations or times.

**PLATE 2**

---

Steven C. Devin, P.E., G.E.

---
LOG OF EXPLORATORY BORING B-1

Sheet 1 of 1

Project Number: 06-1697
Project Name: Hacienda Road 2
Date Drilled: 10/27/06 - 10/27/06
Ground Elev:  

Logged By: ZY
Logged By: Zaheer Yazoji

Logged By: Zaheer Yazoji

Sandy (Fill), brown, fine to medium grained, clayey, moist.

... wet.

SC

... No recovery, very wet.

Boring terminated at approximately 9 feet below existing grade.
Water encountered at approximately 2 feet below existing grade.
Hole backfilled with soil cuttings.

SUMMARY OF SUBSURFACE CONDITIONS

LAB RESULTS

Field Tests

Sandy (Fill), brown, fine to medium grained, clayey, moist.

... wet.

SC

... No recovery, very wet.

Boring terminated at approximately 9 feet below existing grade.
Water encountered at approximately 2 feet below existing grade.
Hole backfilled with soil cuttings.
Appendix C
Laboratory Test Results
APPENDIX C

Laboratory Testing Procedures and Test Results

Direct Shear Tests: Direct shear tests were performed on selected remolded and/or undisturbed samples which were soaked for a minimum of 24 hours under a surcharge equal to the applied normal force during testing. After transfer of the sample to the shear box, and reloading the sample, pore pressures set up in the sample due to the transfer were allowed to dissipate for a period of approximately 1-hour prior to application of shearing force. The samples were tested under various normal loads, a motor-driven, strain-controlled, direct-shear testing apparatus at a strain rate of less than 0.001 to 0.5 inches per minute (depending upon the soil type). The test results are presented in the test data.

<table>
<thead>
<tr>
<th>Sample Location</th>
<th>Sample Description</th>
<th>Friction Angle (degrees)</th>
<th>Apparent Cohesion (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1 @ 15 ft</td>
<td>SAND and SILT, light brown, fine to medium grained, silty/sandy, fine to coarse gravel, gypsum.</td>
<td>32</td>
<td>42</td>
</tr>
<tr>
<td>B-1 @ 45 ft</td>
<td>PUENTE FORMATION (Tp), claystone/siltstone, tan to rust brown, interbedded with sandstone.</td>
<td>27</td>
<td>492</td>
</tr>
<tr>
<td>B-2 @ 10 ft</td>
<td>SILT, light yellowish brown, low plastic, sandy fine grained, trace of gravel, gypsum.</td>
<td>31</td>
<td>98</td>
</tr>
<tr>
<td>B-3 @ 10 ft</td>
<td>PUENTE FORMATION (Tp), sandstone/siltstone interbedded, weathered.</td>
<td>28</td>
<td>270</td>
</tr>
<tr>
<td>B-3 @ 30 ft</td>
<td>PUENTE FORMATION (Tp), sandstone/siltstone hard.</td>
<td>27</td>
<td>402</td>
</tr>
<tr>
<td>B-5 @ 10 ft</td>
<td>PUENTE FORMATION (Tp), sandstone/siltstone hard.</td>
<td>33</td>
<td>234</td>
</tr>
</tbody>
</table>

Consolidation Tests: A Consolidation test was performed on a selected, relatively undisturbed ring sample. Sample was placed in a consolidometer and loads were applied in geometric progression. The percent consolidation for each load cycle was recorded as the ratio of the amount of vertical compression to the original 1-inch height. The consolidation pressure curves are presented in the test data.

C-1

TGR GEOTECHNICAL, INC.
Project No. 06-1597

Maximum Density Tests: The maximum dry density and optimum moisture content of typical materials were determined in accordance with ASTM Test Method D1557. The results of these tests are presented in the table below:

<table>
<thead>
<tr>
<th>Sample Location</th>
<th>Sample Description</th>
<th>Maximum Dry Density (pcf)</th>
<th>Optimum Moisture Content (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-4 @ 2-5 ft</td>
<td>Light Brown Sand and Silt, fine to medium grained.</td>
<td>96</td>
<td>25</td>
</tr>
</tbody>
</table>
Direct Shear Test Summary - Hacienda (Upper)

- Boring B-3 - 10ft Peak
- Boring B-3 - 10ft CV
- Boring B-5 - 10ft Peak
- Boring B-5 - 10ft CV
- Boring B-1 - 15ft Peak
- Boring B-1 - 15ft @ 0.25"
- Boring B-1 - 45ft - CV
- Boring B-1 - 45ft - Peak
- Boring B-2 10ft - Peak
- Boring B-2 - 10ft - @0.25"
- Boring B-3 - 30ft - Peak
- Boring B-3 - 30ft - @ 0.25"

Equations:

- $y = 0.5863x + 564$  
  $R^2 = 0.9998$
- $y = 0.4757x + 642$  
  $R^2 = 0.9978$
- $y = 0.708x + 612$  
  $R^2 = 0.9991$
- $y = 0.5049x + 450$  
  $R^2 = 0.9975$
- $y = 0.5961x + 279.5$  
  $R^2 = 0.9964$
- $y = 0.5338x + 252.5$  
  $R^2 = 0.9997$
- $y = 0.6351x + 68.5$  
  $R^2 = 1$
- $y = 0.66x + 12$  
  $R^2 = 0.9844$
- $y = 0.5338x + 252.5$  
  $R^2 = 0.9997$
- $y = 0.5863x + 564$  
  $R^2 = 0.9998$
Shear Stress vs Shear Disp.

Axial Disp. vs Shear Disp.

Parameters

Client: Hacienda Road

Location:

Job #: 06-1597
Sample: 3
Boring: 1
Depth: 15 ft.
Files: 06-1597-01-15-01.dat
Stress at Max Def: 768 0.145

Soil Type: Clay
Technician: TB
Axial Load: 1000 psf
Shear Rate: .02 in./sec.
Distance: 0.25 in.
Stress at Max Disp: 0.24 744

Maximum Load

768 psf
Shear Displacement at maximum Load
0.1450 in.

Date
10/16/2006

TGR Geotechnical
Shear Stress vs Shear Disp.

Axial Disp. vs Shear Disp.

Parameters
Client: Hacienda Road

Location:
Job #: 06-1597
Sample: 3
Boring: 1
Depth: 15 ft.
File: 06-1597-01-15-02.dat

Shear Stress at Max Def
1188 0.201

Maximum Load
1188 psf

Shear Displacement at maximum Load
0.2005 in.

Technician: TB
Axial Load: 2000 psf
Shear Rate: .02 in./sec.
Distance: 0.25 in.

Stress at Max Disp
0.246 1176

Date
10/16/2006

TGR Geotechnical
Parameters
Client: Hacienda Road
Location:
Job #: 06-1597
Sample: 3
Boring: 1
Depth: 15 ft.
File: 06-1597-01-15-04.dat
Stress at Max Def 2700 0.196

Soil Type: Clay
Technician: TB
Axial Load: 4000 psf
Shear Rate: .02 in./sec.
Distance: 0.25 in.
Stress at Max Disp 0.246 2592

Maximum Load
2700 psf
Shear Displacement at maximum Load
6.1955 in.

Date
10/16/2006

TGR Geotechnical
### Parameters

<table>
<thead>
<tr>
<th>Client: Hacienda Road</th>
</tr>
</thead>
<tbody>
<tr>
<td>Location:</td>
</tr>
<tr>
<td>Job #: 06-1597</td>
</tr>
<tr>
<td>Sample: 3</td>
</tr>
<tr>
<td>Boring: 1</td>
</tr>
<tr>
<td>Depth: 45 ft.</td>
</tr>
<tr>
<td>File: 06-1597-01-45-01.dat</td>
</tr>
<tr>
<td>Stress at Max Def: 1092</td>
</tr>
</tbody>
</table>

| Soil Type: Clay          |
| Technician: TB           |
| Axial Load: 1000 psf     |
| Shear Rate: .02 in./sec. |
| Distance: 0.25 in.       |

### Maximum Load

- 1092 psf
- Shear Displacement at maximum Load: 0.0405 in.
- Date: 10/16/2006

**Stress at Max Disp:**

- 1092  0.041
- 0.24  960
Parameters
Client: Hacienda Road

Location:
Job #: 06-1597
Sample: 3
Boring: 1
Depth: 45 ft.
File: 06-1597-01-45-02.dat
Stress at Max Def 1632 0.196

Soil Type: Clay
Technician: TB
Axial Load: 2000 psf
Shear Rate: .02 in./sec.
Distance: 0.25 in.
Stress at Max Disp 0.24 1572

Maximum Load
1632 psf
Shear Displacement at maximum Load
0.1955 in.

Date 10/16/2006

TGR Geotechnical
Parameters
Client: Hacienda Road

Location:
Job #: 06-1597
Sample: 3
Boring: 1
Depth: 45 ft.
File: 06-1597-01-45-04.dat
Stress at Max Def
2532

Shear Stress (kip)

Shear Displacement (in.)

Axial Disp. vs Shear Disp.

0.032

0.005

0.022

0.025

Shear Displacement (in.)

Shear Stress (kip)

1.195

0.532

0.05

0.1

0.15

0.2

0.25

0.1

0.2

0.25

Axial Displacement (in.)

0

0.05

0.1

0.15

0.2

0.25

Maximum Load
2532 psf
Shear Displacement at maximum Load
0.2256 in.

Date
10/16/2006

TGR Geotechnical
Parameters

Client: Hacienda Road

Location:
Job # 06-1597
Sample: 2
Boring: 2
Depth: 10 ft.
File: 06-1597-02-10-01.dat

Stress at Max Def 708 0.225

Maximum Load

Soil Type: Clay
Technician: TB
Axial Load: 1000 psf
Shear Rate: .02 in./sec.
Distance: 0.25 in.

708 psf
Shear Displacement at maximum Load 0.2250 in.

Date 10/16/2006

TGR Geotechnical
### Parameters

**Client:** Hacienda Road  
**Location:**  
**Job # 06-1597**  
**Sample:** 2  
**Boring:** 2  
**Depth:** 10 ft.  
**File:** 06-1597-02-10-02.dat  
**Stress at Max Def**  

<table>
<thead>
<tr>
<th>Stress at Max Def</th>
<th>1332</th>
<th>0.24</th>
</tr>
</thead>
</table>

**Soil Type:** Clay  
**Technician:** TB  
**Axial Load:** 2000 psf  
**Shear Rate:** .02 in./sec.  
**Distance:** 0.25 in.  
**Maximum Load**  

| Maximum Load | 1332 psf  
|--------------|----------|
| **Shear Displacement at maximum Load** | 0.2400 in.  
| **Date** | 10/16/2006 |

---

**TGR Geotechnical**
**Parameters**

Client: Hacienda Road

**Location:**
- Job #: 06-1597
- Sample #: 2
- Boring #: 2
- Depth: 10 ft.
- File: 06-1597-02-10-04.dat

**Soil Type:** Clay

**Technician:** TB

**Axial Load:** 4000 psf

**Shear Rate:** .02 in./sec.

**Distance:** 0.25 in.

**Stress at Max Def:**
- 2611
- 0.23

**Stress at Max Disp:**
- 0.246
- 2563

**Maximum Load:**
- 2611 psf

**Shear Displacement at Maximum Load:**
- 0.230 in.

**Date:**
- 10/16/2006

TGR Geotechnical
Parameters

Client: Hacienda Road

Location:
Job #: 06-1597
Sample: 2
Boring: 3
Depth: 10 ft.
File: 06-1597-03-10-01.dat
Stress at Max Def: 876 0.071

Soil Type: Clay
Technician: TB
Axial Load: 1000 psf
Shear Rate: .02 in./sec.
Distance: 0.25 in.
Stress at Max Disp: 0.246 804

Maximum Load
876 psf
Shear Displacement at maximum Load
0.0705 in.

Date
10/16/2006
Parameters

Client: Hacienda Road

Location:
Job # 06-1597
Sample: 2
Boring: 3
Depth: 10 ft.
File: 06-1597-03-10-02.dat

Stress at Max Def
1620 0.121

Soil Type: Clay
Technician: TB
Axial Load: 2000 psf
Shear Rate: .02 in./sec.
Distance: 0.25 in.

Maximum Load
1620 psf
Shear Displacement at maximum Load
0.1205 in.

Date
10/16/2006
Parameters
Client: Hacienda Road

Location:
Job #: 06-1597
Sample: 2
Boring: 3
Depth: 10 ft.
File: 06-1597-03-10-04.dat

Soil Type: Clay
Technician: TB
Axial Load: 4000 psf
Shear Rate: .02 in./sec.
Distance: 0.25 in.

Stress at Max Def
2520  0.106

Stress at Max Disp
0.245  2436

Maximum Load
2520 psf
Shear Displacement at maximum Load
0.1055 in.

Date
10/17/2006

TGR Geotechnical
Parameters
Client: Hacienda Road

Location:
Job #: 06-1597
Sample: 6
Boring: 3
Depth: 30 ft.
File: 06-1597-03-30-01.dat

Soil Type: Clay
Technician: TB
Axial Load: 1000 psf
Shear Rate: .02 in./sec.
Distance: 0.25 in.
Stress at Max Def
1140
Stress at Max Disp
0.24

Maximum Load
1140 psf
Shear Displacement at maximum Load
0.1650 in.

Date
10/17/2006

TGR Geotechnical
Shear Stress vs Shear Disp.

Shear Displacement (in.)

Shear Stress (kpf)

0.0 0.05 0.1 0.15 0.2 0.25

1.84

1.472

1.104

0.736

0.369

0.0

Axial Disp. vs Shear Disp.

Axial Displacement (in.)

Shear Displacement (in.)

0.033

0.006

-0.022

0.0

0.05 0.1 0.15 0.2 0.25

Parameters

Client: Hacienda Road

Location:
Job # 06-1597
Sample: 6
Boring: 3
Depth: 30 ft.
File: 06-1597-03-30-02.dat
Stress at Max Def 1752 0.14

Soil Type: Clay
Technician: TB
Axial Load: 2000 psf
Shear Rate: .02 in./sec.
Distance: 0.25 in.
Stress at Max Disp 0.246 .1416

Maximum Load
1752 psf
Shear Displacement at maximum Load 0.1400 in.

Date 10/17/2006

TGR Geotechnical
Parameters
Client: Hacienda Road

Location:
Job #: 06-1597
Sample: 6
Boring: 3
Depth: 30 ft.
File: 06-1597-03-30-04.dat

Soil Type: Clay
Technician: TB
Axial Load: 4000 psf
Shear Rate: .02 in./sec.
Distance: 0.25 in.

Stress at Max Def
2904  0.166
Stress at Max Disp
0.246  2464

Maximum Load
2904 psf
Shear Displacement at maximum Load
0.1655 in.

Date
10/17/2006

TGR Geotechnical
Shear Stress vs Shear Disp.

Axial Disp. vs Shear Disp.

Parameters
Client: Hacienda Road

Location:
Job # 06-1597
Sample: 2
Boring: 5
Depth: 10 ft.
File: 06-1597-05-10-01.dat
Stress at Max Def 1344 0.071

Soil Type: Clay
Technician: TB
Axial Load: 1000 psf
Shear Rate: .02 in./sec.
Distance: 0.25 in.
Stress at Max Disp 0.246 828

Maximum Load
1344 psf
Shear Displacement at maximum Load
0.0705 in.

Date 10/17/2006

TGR Geotechnical
Parameters
Client: Hacienda Road

Location:
Job #: 06-1597
Sample: 2
Boring: 5
Depth: 10 ft.
File: 06-1597-05-10-02.dat
Stress at Max Def 1992

Soil Type: Clay
Technician: TB
Axial Load: 2000 psf
Shear Rate: .02 in./sec.
Distance: 0.25 in.
Stress at Max Disp 1992

Maximum Load
1992 psf
Shear Displacement at maximum Load
0.0550 in.
Date
10/17/2006

TGR Geotechnical
Parameters
Client: Hacienda Road

Location:
Job #: 06-1597
Sample: 2
Boring: 5
Depth: 10 ft.

File: 06-1597-05-10-04.dat

<table>
<thead>
<tr>
<th>Stress at Max Def</th>
<th>Stress at Max Disp</th>
</tr>
</thead>
<tbody>
<tr>
<td>3455</td>
<td>0.096</td>
</tr>
<tr>
<td>0.241</td>
<td>2784</td>
</tr>
</tbody>
</table>

Maximum Load
3456 psf
Shear Displacement at maximum Load
0.0960 in.

Date
10/17/2006

TGR Geotechnical
Hacienda Road Slope Stabilization
La Habra Heights, California

DIRECT SHEAR TEST

Project Number: 08-1597
Project Name: Hacienda Road

Telephone: 
Fax:

Specimen Identification | Classification | $\gamma$ | MC% | c | $\phi$
--- | --- | --- | --- | --- | ---
B-1 | SAND/SILT (Fill), light brown, fine to medium grained, silty to sandy, fine to coarse sized gravel, with gypsum, loose. | 83 | 18 | 42 | 32
<table>
<thead>
<tr>
<th>Specimen Identification</th>
<th>Classification</th>
<th>( \gamma )</th>
<th>MC%</th>
<th>c</th>
<th>( \phi )</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1</td>
<td>PUENTE FORMATION (Tn), claystone/gilstone, tan to rust-brown, interbedded with sandstone, hard.</td>
<td>82</td>
<td>24</td>
<td>492</td>
<td>27</td>
</tr>
</tbody>
</table>

**DIRECT SHEAR TEST**

<table>
<thead>
<tr>
<th>Project Number: 08-1597</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project Name: Hacienda Road</td>
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</tbody>
</table>

Telephone:
Fax:
### Hacienda Road Slope Stabilization

La Habra Heights, California

---

#### Specimen Identification and Classification

<table>
<thead>
<tr>
<th>Specimen Identification</th>
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<th>$\gamma_s$</th>
<th>MC%</th>
<th>c</th>
<th>$\phi$</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-2</td>
<td>SILT (FIII), light yellowish brown, low plastic, sandy fine grained, trace of gravel, gypsum.</td>
<td>87</td>
<td>15</td>
<td>98</td>
<td>31</td>
</tr>
</tbody>
</table>

---

#### DIRECT SHEAR TEST

- **Project Number:** 06-1597
- **Project Name:** Hacienda Road
**Hacienda Road Slope Stabilization**
La Habra Heights, California

---

**DIRECT SHEAR TEST**

<table>
<thead>
<tr>
<th>Specimen Identification</th>
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<th>$\gamma_d$</th>
<th>MC%</th>
<th>$c$</th>
<th>$\phi$</th>
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</thead>
<tbody>
<tr>
<td>B-3</td>
<td>PUENTE FORMATION (Tp), sandstone/siltstone</td>
<td>102</td>
<td>15</td>
<td>270</td>
<td>28</td>
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</tbody>
</table>

Interbedded, weathered.

---

**Project Number:** 06-1597
**Project Name:** Hacienda Road
Specimen Identification | Classification | $\gamma_d$ | MC% | c  | $\phi$
---|---|---|---|---|---
| B-3 | 10.0 | PUENTE FORMATION (Tp), sandstone/siltstone | 102 | 15 | 270 | 28
| interbedded, weathered. | |

DIRECT SHEAR TEST

Project Number: 08-1597
Project Name: Hacienda Road
Specimen Identification | Classification | $\gamma$ | MC% | c | $\phi$
--- | --- | --- | --- | --- | ---
B-5 | PUENTE FORMATION (Tp), sandstone/siltstone, hard. | 104 | 16 | 234 | 33

**DIRECT SHEAR TEST**

Project Number: 06-1597

Project Name: Hacienda Road
## Direct Shear Test

<table>
<thead>
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<th>Specimen Identification</th>
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<th>c</th>
<th>$\phi$</th>
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<tbody>
<tr>
<td>B-2</td>
<td>SAND(Fill), brown, fine grained, clayey.</td>
<td>123</td>
<td>92</td>
<td>32</td>
<td>33</td>
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</table>

**DIRECT SHEAR TEST**

---

Project Number: 06-1697

Project Name: Hacienda Road 2
<table>
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<th>Specimen Identification</th>
<th>Classification</th>
<th>$\gamma$</th>
<th>MC%</th>
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<tbody>
<tr>
<td>B-1</td>
<td>SAND/SILT (Fill), light brown, fine to medium grained, silty to sandy, fine to coarse size gravel, gypsum.</td>
<td>5.0</td>
<td>72</td>
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</tbody>
</table>
STRAIN, %

STRESS, Ksf

<table>
<thead>
<tr>
<th>Specimen Identification</th>
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<th>MC%</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-4</td>
<td>COLLUVIUM (Fill), light brown sand and silt, fine</td>
<td>5.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>to medium grained, mottled.</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

CONSOLIDATION TEST

Project Number: 09-1597
Project Name: Hacienda Road
MOISTURE-DENSITY RELATIONSHIP

- **Source of Material:** B-425
- **Description of Material:** COLLUVIUM (Fill), light brown sand with silt, fine to medium grained.
- **Test Method:** ASTM D1557 Method B

**TEST RESULTS**
- **Maximum Dry Density:** 96.0 PCF
- **Optimum Water Content:** 25.0 %

**ATTERBERG LIMITS**
- **LL:**
- **PL:**
- **PI:**

Curves of 100% Saturation for Specific Gravity Equal to:
- 2.80
- 2.70
- 2.60

**MOISTURE-DENSITY RELATIONSHIP**

- **Project Number:** 06-1597
- **Project Name:** Hacienda Road
Appendix D
Hacienda Road (Upper) Site 1
Design Calculations
Please Standby While File Is Scanned...

Minimum/Maximum Magnitudes Are : 5.0500  8.9500
Minimum/Maximum Distances Are  : 10.6725  928.6400

Number of Integration M-D Pairs   =  823257

Value of Ground Motion Measure (x) = 0.4996E+00

Total Computed Activity Rate      = 0.8653E+00
Total Computed Hazard, P[X>x]    = 0.2108E-02
Ratio of (Hazard/Activity Rate)   = 0.2436E-02

E[Magnitude|EQ] = 5.6578
E[Distance |EQ] = 261.1111
E[Epsilon-x|EQ] = 8.5726

E[Magnitude|(P[X>x]=0.2108E-02)] = 8.1332
E[Distance |(P[X>x]=0.2108E-02)] = 20.4397
E[Epsilon-x|(P[X>x]=0.2108E-02)] = 0.1106

Enter: 0 for Raw File; 1 for M-D Bins; 2 for Both
Punching Shear (Upper Wall)

\[ T_{0} = 10.5 \, k \]  
- For Seismic \( k = 0.17 \) (FEM)

\[ V_{F} = 0.58 \sqrt{4000} \left( \frac{\pi}{2} \right) (0.24) = 39.2 \, k \]

Nail Head: \( D_{e} = 8 + 4 = 12''\) (8''x8'' B - 4'' Shotcrete)

\[ T_{\text{allow}} = 39.2 / 1.5 = 26.1 \, k \text{ Static} \]

\[ T_{\text{allow}} = \frac{39.2}{1.1} = 35.6 \, k \text{ Seismic} \]

No. 9 Epoxy Coated TS ties:

\[ k = 0.06 \, \text{in}^2 \]

\[ R_{\text{Tallow}} = \frac{(1)(75)}{1.8} = 41.7 \, k \text{ Static} \]

\[ R_{\text{Tallow}} = \frac{(1)(75)}{1.35} = 55.6 \, k \text{ Seismic} \]

Facing Flexural Capacity

Upper

\[ \#6 \text{ bars} \]

\[ \sigma_{w} = 0.12 + \frac{2(0.44)}{3} = 0.41 \]

\[ \sigma_{\text{allow}} = 0.12 \]

\[ R_{FF} = 3.8(1)(0.41 + 0.12) \left( \frac{1}{2} \right) (65) = 41.1 \, k \text{ Static} \]

\[ R_{FF} = 3.8(1)(0.12 + 0.12) \left( \frac{1}{2} \right) (65) = 20.2 \, k \text{ Seismic} \]

\[ R_{FF} = 3.8(1)(0.32 + 0.12) \left( \frac{1}{2} \right) (65) = 34.9 \, k \]

\[ \text{Static FS} = 1.5 \]

\[ \text{Seismic FS} = 1.1 \]

\[ \#6 \text{ wires} + 4x4 - W4 x W4 \]

\[ R_{\text{allow}} = \frac{27.4 \, k}{27.4 \, k} \]

\[ \#5 \text{ wires} + 4x4 - W4 x W4 \]

\[ R_{\text{allow}} = \frac{23.3 \, k}{31.7 \, k} \]

\[ 4x4 - W4 x W4 \]

\[ R_{\text{allow}} = \frac{13.5 \, k}{18.4 \, k} \]
**Micropile**

**Williams T76N**

- **G.D.** = 3'
- **I.D.** = 2''
- **A** = \( \frac{2.84 \text{ in}^2}{2.84} = 0.9722 \text{ ft}^2\)
- **F_y = 270}{2.84} = 95 \text{ ksi}\)
- **I = \frac{\pi}{64} (3^4 - 2^4) = 3.19 \text{ in}^4 = 1.5387 \times 10^{-4} \text{ ft}^4\)
- **2.5' spacing**

**Supernail**

- **4'' Drill Hole**
- **A = 150.8 \text{ in}^2/\text{LF} 2.66''
- **F_y = \sqrt{\frac{377}{2.66}} = 42 \text{ kpsi}\)
- **I = \frac{\pi}{16} (2.66)^4 = 302.76 \text{ in}^4/\text{LF} 2.66''\)

- **Steel R_{f,\text{allow}} = 1.06 (42) = 21.7 K**
- **T_{\text{concrete}} = 0.184 + 0.0275 = 0.2115 \text{ in}^4**

- **Nail Head**
  - **D_e = 8 + 4 = 12'' (4'' shank)**
  - **V_f = 0.58 \sqrt{\frac{400}{1.61}} (12)(0.38) = 39.2 K**
  - **T_{\text{nail}} = 39.2 / 1.5 = 26.1 K**

- **Steel R_{f,\text{allow}} = 1.75 \frac{1}{1.8} = 41.7 K**
Titan 40/16

\[ A = 879 \text{ mm}^2 \]
\[ = 1.36 \text{ in}^2 \]
\[ F_y = 590 \text{ MPa} \]
\[ = 85.55 \text{ ksi} \]
\[ F_y = 0.3 (85.55) = 25.7 \text{ ksi} \]
\[ V_{allow} = 34.9 \text{ k} \]

\[ A_5 = \pi (4) (12) = 226.2 \text{ in}^2/\text{LF} \]

1.5% w/ #6 Epoxy Coated Bar - Williams 75 ksi

\[ 1\frac{1}{8}'' \text{ OD} \quad 1\frac{1}{4}'' \text{ ID} \]

\[ A_{tc} = 0.54 \text{ in}^2 \]
\[ F_{tc} = 2.8 \times 10^6 \text{ psi} \]
\[ I_{tc} = 0.13 \text{ in}^4 \]

\[ A_3 = \frac{1.5\pi (12)}{\text{LF Bar}} = 56.55 \text{ in}^2/\text{LF Bar} \]

\[ T_{allow} = 56.55 \text{ ft-lb/Bar} \]

\[ A_{5\%} = \frac{56.55}{5} \Rightarrow \text{ FE Bond} = \frac{56.55(10)}{2(4)} = 70.67 \text{ psi/lfm} \]

\[ I_{composite} = 0.13 + \frac{21}{28} (1.44 \times 10^{-5}) = 0.13077 \text{ in}^4 = 2.0413 \times 10^{-10} \text{ ft}^4 \]

\[ A_{composite} = 0.54 \text{ in}^2 + \frac{27}{28} (0.44) = 5.977 \text{ in}^2 = 0.3354 \text{ ft}^2 \]
Slide Analysis Information

Document Name:
File Name: Lower_Slope_Repair_01_PROB_Composite_Surcha
ge250_K019.sli

Project Settings

Project Title: SLIDE - An Interactive Slope Stability Program
Failure Direction: Right to Left
Units of Measurement: Imperial Units
Pore Fluid Unit Weight: 62.4 lb/ft³
Groundwater Method: Finite Element Analysis
Tolerance (groundwater): 1e-006
Maximum number of iterations (groundwater): 500
Data Output: Standard
Calculate Excess Pore Pressure: Off
Allow Ru with Water Surfaces or Grids: Off
Random Numbers: Pseudo-random Seed
Random Number Seed: 10116
Random Number Generation Method: Park and Miller v.3

Analysis Methods

Analysis Methods used:
Spencer

Number of slices: 25
Tolerance: 0.005
Maximum number of iterations: 50

Surface Options

Surface Type: Circular
Search Method: Slope Search
Number of Surfaces: 10000
Upper Angle: Not Defined
Lower Angle: Not Defined
Composite Surfaces: Disabled
Reverse Curvature: Create Tension Crack
Minimum Elevation: Not Defined
Minimum Depth: Not Defined

Loading

Seismic Load Coefficient (Horizontal): 0.19
1 Distributed Load present:
Distributed Load Constant Distribution, Orientation:
Vertical, Magnitude: 250 lb/ft²

Groundwater Analysis

Maximum Number of Iterations: 500
Iteration Tolerance: 1e-006
Mesh Element Type: 3 noded triangles
Number of Elements: 1273
Number of Nodes: 701

Material Properties

Material: Fill - Silts and Sands
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft³
Cohesion: 32 psf
Friction Angle: 32 degrees

Unsaturated Shear Strength Angle: 0 degrees
Air Entry Value: 0 psf
Ks: 3.28e-007
K2/K1: 1
K Angle: 0

Model: Simple

Material: Colluvium
Strength Type: Mohr-Coulomb
Unit Weight: 110 lb/ft³
Cohesion: 100 psf
Friction Angle: 28 degrees
Unsaturated Shear Strength Angle: 0 degrees
Air Entry Value: 0 psf
Ks: 3.28e-008
K2/K1: 1
K Angle: 0

Model: Simple

Material: Puente Formation - Sandstone - Siltstone
Strength Type: Mohr-Coulomb
Unit Weight: 117 lb/ft³
Cohesion: 270 psf
Friction Angle: 27 degrees
Unsaturated Shear Strength Angle: 0 degrees
Air Entry Value: 0 psf
Ks: 1e-009
K2/K1: 1
K Angle: 0

Model: Simple

Material: Shotcrete
Strength Type: Undrained
Unit Weight: 150 lb/ft³
Cohesion: 288000 psf
Unsaturated Shear Strength Angle: 0 degrees
Air Entry Value: 0 psf
Ks: 1e-007
K2/K1: 1
K Angle: 0

Model: Simple

Support Properties

Support: Soil Nails - Bond
Soil Nail: Bond
Support Type: Soil Nails
Force Application: Passive
Out-of-Plane Spacing: 4 ft
Tensile Capacity: 24700 lb
Plate Capacity: 24700 lb
Bond Strength: 1508 lb/ft

Global Minimums

Method: spencer
FS: 1.028000
Center: 129.117, 798.517
Radius: 22.432
Left Slip Surface Endpoint: 121.021, 777.597
Right Slip Surface Endpoint: 144.957, 782.633
Resisting Moment= 49017.4 lb-ft
Driving Moment= 47682.5 lb-ft
Resisting Horizontal Force= 2080.1 lb
Driving Horizontal Force= 2023.45 lb
Valid / Invalid Surfaces

Method: Spencer
Number of Valid Surfaces: 7374
Number of Invalid Surfaces: 3126
Error Codes:
Error Code -100 reported for 26 surfaces
Error Code -101 reported for 1 surface
Error Code -105 reported for 4 surfaces
Error Code -106 reported for 314 surfaces
Error Code -107 reported for 39 surfaces
Error Code -108 reported for 522 surfaces
Error Code -111 reported for 1551 surfaces
Error Code -112 reported for 669 surfaces

Error Codes

The following errors were encountered during the computation:

-100 = Both surface / slope intersections are on the same horizontal surface. In general, this will give a very high or infinite factor of safety (zero driving force), if calculated.

-101 = Only one (or zero) surface / slope intersections.

-105 = More than two surface / slope intersections with no valid slip surface.

-106 = Average slice width is less than 0.0001 * (maximum horizontal extent of soil region). This limitation is imposed to avoid numerical errors which may result from too many slices, or too small a slip region.

-107 = Total driving moment or total driving force is negative. This will occur if the wrong failure direction is specified, or if high external or anchor loads are applied against the failure direction.

-108 = Total driving moment or total driving force < 0.1. This is to limit the calculation of extremely high safety factors if the driving force is very small (0.1 is an arbitrary number).

-111 = safety factor equation did not converge

-112 = The coefficient M-Alpha = \( \cos(\alpha) / (1 + \tan(\alpha) \tan(\phi) / F) \) < 0.2 for the final iteration of the safety factor calculation. This screens out some slip surfaces which may not be valid in the context of the analysis, in particular, deep seated slip surfaces with many high negative base angle slices in the passive zone.

Probabilistic Analysis Input

Project Settings:
Sensitivity Analysis: On
Probabilistic Analysis: On
Sampling Method: Latin-Hypercube
Number of Samples: 500

Analysis Type: Overall Slope

Material: FILL - Silt and Sand
Property: Cohesion
Distribution: Normal
Minimum: 2 (relative minimum: 30)
Mean: 32
Maximum: 62 (relative maximum: 30)
Standard Deviation: 10

Material: FILL - Silt and Sand
Property: Phi
Distribution: Normal
Minimum: 30 (relative minimum: 2.00001)
Mean: 32
Maximum: 34 (relative maximum: 2.00001)
Standard Deviation: 0.66667

Material: FILL - Silt and Sand
Property: Unit Weight
Distribution: Normal
Minimum: 94 (relative minimum: 6)
Mean: 100
Maximum: 106 (relative maximum: 6)
Standard Deviation: 2

Material: FILL - Sand - Silt - Medium Dense
Property: Cohesion
Distribution: Normal
Minimum: 92 (relative minimum: 60)
Mean: 112
Maximum: 172 (relative maximum: 60)
Standard Deviation: 20

Material: FILL - Sand - Silt - Medium Dense
Property: Phi
Distribution: Normal
Minimum: 30 (relative minimum: 2.00001)
Mean: 32
Maximum: 34 (relative maximum: 2.00001)
Standard Deviation: 0.66667

Material: FILL - Sand - Silt - Medium Dense
Property: Unit Weight
Distribution: Normal
Minimum: 97 (relative minimum: 6)
Mean: 103
Maximum: 109 (relative maximum: 6)
Standard Deviation: 2

Support: Soil Nail - Bond
Property: Bond Strength
Distribution: Normal
Minimum: 755 (relative minimum: 753)
Mean: 1508
Maximum: 2251 (relative maximum: 753)
Standard Deviation: 251

Support: Micropile
Property: Pile Shear Strength
Distribution: Normal
Minimum: 25 (relative minimum: 75)
Mean: 100
Maximum: 175 (relative maximum: 75)
Standard Deviation: 25

Support: Micropile - Tensile
Property: Adhesion
Distribution: Normal
Minimum: 720 (relative minimum: 720)
Mean: 1440
Maximum: 2160 (relative maximum: 720)
Standard Deviation: 240

**Probabilistic Analysis Results (Overall Slope)**

**Method:** spencer

- Factor of Safety, mean: 1.020880
- Factor of Safety, standard deviation: 0.115134
- Factor of Safety, minimum: 0.623146
- Factor of Safety, maximum: 1.200660
- Probability of Failure: 40.6000% (= 203 failed surfaces / 500 valid surfaces)

**Reliability index:** 0.18135 (assuming normal distribution)
**Reliability index:** 0.12761 (assuming lognormal distribution)

* best fit = Beta

**Critical Probabilistic Surface**

**Method:** spencer

- Normal Reliability Index: 0.24796
- Probability of Failure: 0.398
- Mean Factor of Safety: 1.02863
- Center: 129.117, 798.517
- Radius: 22.432
- Left Slip Surface Endpoint: 121.021, 777.597
- Right Slip Surface Endpoint: 144.957, 782.633
- Lognormal Reliability Index: 0.196312

**List of All Coordinates**

**Material Boundary**

<table>
<thead>
<tr>
<th>X</th>
<th>Y</th>
</tr>
</thead>
<tbody>
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**External Boundary**

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Appendix E
Hacienda Road (Upper) Site 2
Design Calculations
Slide Analysis Information

Document Name:
File Name: upper_slope_slumped_2-mpten_3nailedstdnd_30_k019_full_prob.sli

Project Settings
Project Title: SLIDE - An Interactive Slope Stability Program
Failure Direction: Right to Left
Units of Measurement: Imperial Units
Pore Fluid Unit Weight: 62.4 lb/ft³
Groundwater Method: Water Surfaces
Data Output: Standard
Calculate Excess Pore Pressure: Off
Allow Ru with Water Surfaces or Grids: Off
Random Numbers: Pseudo-random Seed 10116
Random Number Generation Method: Park and Miller v.3

Analysis Methods
Analysis Methods used:
Spencer

Number of slices: 25
Tolerance: 0.005
Maximum number of iterations: 50

Surface Options
Surface Type: Circular
Search Method: Slope Search
Number of Surfaces: 5000
Upper Angle: Not Defined
Lower Angle: Not Defined
Composite Surfaces: Enabled
Reverse Curvature: Create Tension Crack
Minimum Elevation: Not Defined
Minimum Depth: Not Defined

Loading
Seismic Load Coefficient (Horizontal): 0.19
Distributed Load present:
Distribution Load Constant Distribution, Orientation: Vertical, Magnitude: 250 lb/ft²

Material Properties
Material: Puente Formation - Sandstone - Siltstone
Strength Type: Mohr-Coulomb
Unit Weight: 117 lb/ft³
Cohesion: 270 psf
Friction Angle: 27 degrees
Water Surface: Water Table
Custom Hu value: 1

Material: Fill - Sand - Silt - Medium Dense
Strength Type: Mohr-Coulomb
Unit Weight: 103 lb/ft³
Cohesion: 112 psf
Friction Angle: 32 degrees
Water Surface: Water Table
Custom Hu value: 1

Material: Shotcrete
Strength Type: Undrained
Unit Weight: 150 lb/ft³
Cohesion Type: Constant
Cohesion: 280000 psf
Water Surface: None

Material: Slumped Fill - Upper
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft³
Cohesion: 6 psf
Friction Angle: 32 degrees
Water Surface: Water Table
Custom Hu value: 1

Material: Slumped Fill - Lower
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft³
Cohesion: 5 psf
Friction Angle: 32 degrees
Water Surface: Water Table
Custom Hu value: 1

Support Properties
Support: Soil Nail - Bond
Soil Nail - Bond
Support Type: Soil Nail
Force Application: Passive
Out-of-Plane Spacing: 3 ft
Tensile Capacity: 24700 lb
Plate Capacity: 24700 lb
Bond Strength: 1508 lb/ft

Support: Micropile - Tensile
Micropile - Tensile
Support Type: Grouted Tieback (with friction)
Force Application: Passive
Bond length: 100 percent
Out-of-Plane Spacing: 2.5 ft
Tensile Capacity: 10000 lb
Plate Capacity: 10000 lb
Pullout Strength Adhesion: 1440 lb/ft²
Pullout Strength Friction Angle: 0 degrees
Shear Strength Model: Linear
Grout Diameter: 0.5 ft

Global Minimums
Method: spencer
FS: 0.678567
Center: -23.546, 813.375
Radius: 52.334
Left Slip Surface Endpoint: -0.000, 766.637
Right Slip Surface Endpoint: 16.125, 779.243
Left Slope Intercept: -0.000 769.898
Right Slope Intercept: 16.125 779.243
Resisting Moment = 67212.6 lb-ft
Driving Moment = 128525 lb-ft
Resisting Horizontal Force = 1366.85 lb
Driving Horizontal Force = 2014.32 lb

Valid / Invalid Surfaces

Method: spencer
Number of Valid Surfaces: 3333
Number of Invalid Surfaces: 2067
Error Codes:
Error Code -100 reported for 9 surfaces
Error Code -105 reported for 9 surfaces
Error Code -106 reported for 110 surfaces
Error Code -107 reported for 7 surfaces
Error Code -108 reported for 371 surfaces
Error Code -109 reported for 2 surfaces
Error Code -110 reported for 1308 surfaces
Error Code -112 reported for 250 surfaces
Error Code -113 reported for 1 surface

Error Codes

The following errors were encountered during the computation:

-100 = Both surface / slope intersections are on the same horizontal surface. In general, this will give a very high or infinite factor of safety (zero driving force), if calculated.

-105 = More than two surface / slope intersections with no valid slip surface.

-106 = Average slice width is less than 0.0001 * (maximum horizontal extent of soil region). This limitation is imposed to avoid numerical errors which may result from too many slices, or too small a slip region.

-107 = Total driving moment or total driving force is negative. This will occur if the wrong failure direction is specified, or if high external or anchor loads are applied against the failure direction.

-108 = Total driving moment or total driving force < 0.1. This is to limit the calculation of extremely high safety factors if the driving force is very small (0.1 is an arbitrary number).

-109 = Soiltype for slice base not located. This error should occur very rarely, if at all. It may occur if a very low number of slices is combined with certain soil geometries, such that the midpoint of a slice base is actually outside the soil region, even though the slip surface is wholly within the soil region.

-111 = Safety factor equation did not converge

-112 = The coefficient M-Alpha = cos(alpha)(1+tan(alpha)tan(phi)/F) < 0.2 for the final iteration of the safety factor calculation. This screens out some slip surfaces which may not be valid in the context of the analysis, in particular, deep seated slip surfaces with many high negative base angle slices in the passive zone.

-113 = Surface intersects outside slope limits.

Probabilistic Analysis Input

Project Settings:
Sensitivity Analysis: On
Probabilistic Analysis: On
Sampling Method: Latin-Hypercube
Number of Samples: 400
Analysis Type: Overall Slope

Material: FILL - Silt and Sand
Property: Cohesion
Distribution: Normal
Minimum: 10 (relative minimum: 60)
Mean: 70
Maximum: 130 (relative maximum: 60)
Standard Deviation: 20

Material: FILL - Silt and Sand
Property: Phi
Distribution: Normal
Minimum: 30 (relative minimum: 2.00001)
Mean: 32
Maximum: 34 (relative maximum: 2.00001)
Standard Deviation: 0.66667

Material: FILL - Silt and Sand
Property: Unit Weight
Distribution: Normal
Minimum: 94 (relative minimum: 6)
Mean: 100
Maximum: 106 (relative maximum: 6)
Standard Deviation: 2

Material: FILL - Sand - Silt - Medium Dense
Property: Cohesion
Distribution: Normal
Minimum: 52 (relative minimum: 60)
Mean: 112
Maximum: 172 (relative maximum: 60)
Standard Deviation: 20

Material: FILL - Sand - Silt - Medium Dense
Property: Phi
Distribution: Normal
Minimum: 30 (relative minimum: 2.00001)
Mean: 32
Maximum: 34 (relative maximum: 2.00001)
Standard Deviation: 0.66667

Material: FILL - Sand - Silt - Medium Dense
Property: Unit Weight
Distribution: Normal
Minimum: 97 (relative minimum: 6)
Mean: 103
Maximum: 109 (relative maximum: 6)
Standard Deviation: 2

Support: Soil Nail - Bond
Property: Bond Strength
Distribution: Normal
Minimum: 755 (relative minimum: 753)
Mean: 1506
Maximum: 2281 (relative maximum: 753)
Standard Deviation: 251

Support: Micropile
Property: Pile Shear Strength
Distribution: Normal
Minimum: 25 (relative minimum: 75)
Mean: 100
Maximum: 175 (relative maximum: 75)
Standard Deviation: 25

Support: Micropile - Tensile
Property: Adhesion
Distribution: Normal
Minimum: 720 (relative minimum: 720)
Mean: 1440
Maximum: 2160 (relative maximum: 720)
Standard Deviation: 240

Water Table Location
Distribution: Lognormal
Normalized Mean: 0.5
Normalized Standard Deviation: 0.15

Probabilistic Analysis Results (Overall Slope)
Method: spencer
Factor of Safety, mean: 0.674984
Factor of Safety, standard deviation: 0.004050
Factor of Safety, minimum: 0.663932
Factor of Safety, maximum: 0.678650
Probability of Failure: 100.000% (= 400 failed surfaces / 400 valid surfaces)
Reliability index: -80.25574 (assuming normal distribution)
Reliability index: -65.51709 (assuming lognormal distribution)
* best fit = Beta

Critical Probabilistic Surface
Method: spencer
Normal Reliability Index: -47102.1
Probability of Failure: 1
Mean Factor of Safety: 0.702148
Center: -25.4425, 864.92
Radius: 98.9003
Left Slip Surface Endpoint: 8.29484, 774.083
Right Slip Surface Endpoint: 45.2267, 798.622
Lognormal Reliability Index: -39264
Probability of Failure: 1
Mean Factor of Safety: 0.702148
Center: -25.4425, 864.92
Radius: 98.9003
Left Slip Surface Endpoint: 8.29484, 774.083
Right Slip Surface Endpoint: 45.2267, 798.622

List of All Coordinates

Min Water Table
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14.360 758.689
34.583 759.044
58.248 760.582
91.969 764.800
134.092 773.598
176.688 787.441
213.013 804.835
220.040 813.945

Max Water Table
-0.000 766.302
14.176 772.189
20.494 773.985
27.655 775.618
37.689 777.404
46.100 778.040
85.170 782.980
100.400 785.880
128.020 792.500
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Mean Water Table
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Material Boundary
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Min Water Table
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Max Water Table
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|                           | 184.901 | 825.680 |
|                           | 179.453 | 827.230 |
|                           | 172.158 | 829.609 |
|                           | 171.817 | 829.566 |
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| Support                   | 162.435 | 827.179 |
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|                           | 112.705 | 826.837 |
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|                           | 67.198  | 796.868 |
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|                           | 124.413 | 826.671 |
|                           | 126.474 | 826.650 |

| Support                   | 80.250  | 810.110 |
|                           | 80.250  | 765.110 |

| Support                   | 83.271  | 810.172 |
|                           | 94.897  | 766.783 |

| Support                   | 92.084  | 812.400 |
|                           | 121.061 | 804.635 |

| Support                   | 92.613  | 815.400 |
|                           | 121.591 | 807.635 |

| Support                   | 80.250  | 810.110 |

<p>| Support                   | 80.250  | 765.110 |</p>
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<td></td>
</tr>
<tr>
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<td>810.635</td>
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**Support**

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**Support**

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**Document Name**
File Name: Upper_Slope_Slumped_2-MP100_3NailedSTND_30_k019_PROB.sli

**Project Settings**
Project Title: SLIDE - An Interactive Slope Stability Program
Failure Direction: Right to Left
Units of Measurement: Imperial Units
Pore Fluid Unit Weight: 62.4 lb/ft3
Groundwater Method: Water Surfaces
Data Output: Standard
Calculate Excess Pore Pressure: Off
Allow Ru with Water Surfaces or Grids: Off
Random Numbers: Pseudo-random Seed
Random Number Seed: 10116
Random Number Generation Method: Park and Miller v.3

**Analysis Methods**
Analysis Methods used:
Spencer

Number of slices: 25
Tolerance: 0.005
Maximum number of iterations: 50

**Surface Options**
Surface Type: Circular
Search Method: Slope Search
Number of Surfaces: 5000
Upper Angle: Not Defined
Lower Angle: Not Defined
Composite Surfaces: Enabled
Reverse Curvature: Create Tension Crack
Minimum Elevation: Not Defined
Minimum Depth: Not Defined

**Loading**
Seismic Load Coefficient (Horizontal): 0.19

**Material Properties**
Material: FILL - Sand - Silt - Medium Dense
Strength Type: Mohr-Coulomb
Unit Weight: 103 lb/ft3
Cohesion: 112 psf
Friction Angle: 32 degrees
Water Surface: Water Table
Custom Hu value: 1

Material: Slumped FILL-Upper
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 5 psf
Friction Angle: 32 degrees
Water Surface: Water Table
Custom Hu value: 1

Material: Slumped FILL-Lower
Strength Type: Mohr-Coulomb
Unit Weight: 100 lb/ft3
Cohesion: 5 psf
Friction Angle: 32 degrees
Water Surface: Water Table
Custom Hu value: 1

Support Properties
Support: Soil Nail - Bond
Soil Nail - Bond
Support Type: Soil Nail
Force Application: Passive
Out-of-Plane Spacing: 3 ft
Tensile Capacity: 24700 lb
Plate Capacity: 24700 lb
Bond Strength: 1508 lb/ft

Support: Micropile
Micropile
Support Type: Micro-Pile
Force Application: Passive
Out-of-Plane Spacing: 2.5 ft
Pile Shear Strength: 100 lb

**Global Minimums**
Method: spencer
FS: 0.760757
Center: -2.124, 863.595
Radius: 81.809
Left Slip Surface Endpoint: 29.658, 788.212
Right Slip Surface Endpoint: 48.775, 799.548
Resisting Moment=100344 lb-ft
Driving Moment=131900 lb-ft
Resisting Horizontal Force=1046.28 lb
Driving Horizontal Force=1375.31 lb

**Valid / Invalid Surfaces**
Method: spencer
Number of Valid Surfaces: 3466
Number of Invalid Surfaces: 1934
Error Codes:
- Error Code -100 reported for 13 surfaces
- Error Code -105 reported for 20 surfaces
- Error Code -106 reported for 152 surfaces
- Error Code -107 reported for 2 surfaces
- Error Code -108 reported for 288 surfaces
- Error Code -109 reported for 1 surface
- Error Code -111 reported for 1202 surfaces
- Error Code -112 reported for 256 surfaces

Error Codes

The following errors were encountered during the computation:

-100 = Both surface / slope intersections are on the same horizontal surface. In general, this will give a very high or infinite factor of safety (zero driving force), if calculated.

-105 = More than two surface / slope intersections with no valid slip surface.

-106 = Average slice width is less than 0.0001 * (maximum horizontal extent of soil region). This limitation is imposed to avoid numerical errors which may result from too many slices, or too small a slip region.

-107 = Total driving moment or total driving force is negative. This will occur if the wrong failure direction is specified, or if high external or anchor loads are applied against the failure direction.

-108 = Total driving moment or total driving force < 0.1. This is to limit the calculation of extremely high safety factors if the driving force is very small (0.1 is an arbitrary number).

-109 = Solitary slice base not located. This error should occur very rarely, if at all. It may occur if a very low number of slices is combined with certain soil geometries, such that the midpoint of a slice base is actually outside the soil region, even though the slip surface is wholly within the soil region.

-111 = Safety factor equation did not converge

-112 = The coefficient M-Alpha = cos(alpha)/(1+tan(alpha)tan(phi)/F) < 0.2 for the final iteration of the safety factor calculation. This screens out some slip surfaces which may not be valid in the context of the analysis, in particular, deep seated slip surfaces with many high negative base angle slices in the passive zone.

Probabilistic Analysis Input

Project Settings:
Sensitivity Analysis: On
Probabilistic Analysis: On
Sampling Method: Latin-Hypercube
Number of Samples: 400

Analysis Type: Overall Slope
Material: FILL - Silt and Sand
Property: Cohesion
Distribution: Normal
Minimum: 10 (relative minimum: 60)
Mean: 70
Maximum: 130 (relative maximum: 60)
Standard Deviation: 20

Material: FILL - Silt and Sand
Property: Phi
Distribution: Normal
Minimum: 30 (relative minimum: 2.00001)
Mean: 32
Maximum: 34 (relative maximum: 2.00001)
Standard Deviation: 0.66667

Material: FILL - Silt and Sand
Property: Unit Weight
Distribution: Normal
Minimum: 94 (relative minimum: 6)
Mean: 100
Maximum: 106 (relative maximum: 6)
Standard Deviation: 2

Material: FILL - Sand - Silt - Medium Dense
Property: Cohesion
Distribution: Normal
Minimum: 52 (relative minimum: 60)
Mean: 112
Maximum: 172 (relative maximum: 60)
Standard Deviation: 20

Material: FILL - Sand - Silt - Medium Dense
Property: Phi
Distribution: Normal
Minimum: 30 (relative minimum: 2.00001)
Mean: 32
Maximum: 34 (relative maximum: 2.00001)
Standard Deviation: 0.66667

Material: FILL - Sand - Silt - Medium Dense
Property: Unit Weight
Distribution: Normal
Minimum: 97 (relative minimum: 6)
Mean: 103
Maximum: 109 (relative maximum: 6)
Standard Deviation: 2

Support: Soil Nail - Bond
Property: Bond Strength
Distribution: Normal
Minimum: 755 (relative minimum: 753)
Mean: 1508
Maximum: 2261 (relative maximum: 753)
Standard Deviation: 251

Support: Micropile
Property: Pile Shear Strength
Distribution: Normal
Minimum: 25 (relative minimum: 75)
Mean: 100
Maximum: 175 (relative maximum: 75)
Standard Deviation: 25

Support: Micropile - Tensile
Property: Adhesion
Distribution: Normal
Minimum: 720 (relative minimum: 720)
Mean: 1440
Maximum: 2160 (relative maximum: 720)
Standard Deviation: 240

Water Table Location

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<th>Distribution</th>
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<tr>
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<td>Normalized Standard Deviation: 0</td>
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Probabilistic Analysis Results (Overall Slope)

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<th>Method: spencer</th>
<th>Factor of Safety, mean: 0.759482</th>
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<tr>
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<td>Factor of Safety, standard deviation: 0.011240</td>
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<td></td>
<td>Factor of Safety, minimum: 0.612524</td>
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<tr>
<td></td>
<td>Factor of Safety, maximum: 0.763256</td>
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<tr>
<td></td>
<td>Probability of Failure: 100.000% (= 400 failed surfaces / 400 valid surfaces)</td>
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<tr>
<td>Reliability Index: -21.39786 (assuming normal distribution)</td>
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<tr>
<td>Reliability index: -18.59781 (assuming lognormal distribution)</td>
<td>129.020</td>
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<td>* best fit = Triangular</td>
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Critical Probabilistic Surface

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<td>Probability of Failure: 1</td>
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<tr>
<td>Mean Factor of Safety: 0.932918</td>
<td>Center: 33.4486, 805.161</td>
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<tr>
<td>Radius: 15.7089</td>
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</tbody>
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Left Slip Surface Endpoint: 31.6986, 789.55
Right Slip Surface Endpoint: 48.1544, 799.638
Lognormal Reliability Index: -27562.5
Probability of Failure: 1
Mean Factor of Safety: 0.932918
Center: 33.4486, 805.161
Radius: 15.7089
Left Slip Surface Endpoint: 31.6986, 789.55
Right Slip Surface Endpoint: 48.1544, 799.638

List of All Coordinates

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<tr>
<td>126.474</td>
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</tr>
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</table>
Cohesion: 270 psf  
Friction Angle: 27 degrees  
Water Surface: None  

Material: FILL - Sand - Silt - Medium Dense  
Strength Type: Mohr-Coulomb  
Unit Weight: 103 lb/ft³  
Cohesion: 112 psf  
Friction Angle: 32 degrees  
Water Surface: None  

Material: Shotcrete  
Strength Type: Undrained  
Unit Weight: 150 lb/ft³  
Cohesion Type: Constant  
Cohesion: 288000 psf  
Water Surface: None  

Material: Slumped FILL-Upper  
Strength Type: Mohr-Coulomb  
Unit Weight: 100 lb/ft³  
Cohesion: 5 psf  
Friction Angle: 32 degrees  
Water Surface: None  

Material: Slumped FILL-Lower  
Strength Type: Mohr-Coulomb  
Unit Weight: 100 lb/ft³  
Cohesion: 5 psf  
Friction Angle: 32 degrees  
Water Surface: None  

Support Properties  
Support: Soil Nail - Bond  
Soil Nail - Bond  
Support Type: Soil Nail  
Force Application: Passive  
Out-of-Plane Spacing: 3 ft  
Tensile Capacity: 24700 lb  
Bond Strength: 1508 lb/ft  

Support: Micropile  
Micropile  
Support Type: Micro-Pile  
Force Application: Passive  
Out-of-Plane Spacing: 2.5 ft  
Pile Shear Strength: 100 lb  

Global Minimums  
Method: spencer  
FS: 1.326720  
Center: 68.974, 813.114  
Radius: 14.986  
Left Slip Surface Endpoint: 69.911, 798.175  
Right Slip Surface Endpoint: 83.652, 810.179  
Resisting Moment=60017.9 lb-ft  
Driving Moment=45237.7 lb-ft  
Resisting Horizontal Force=3030.9 lb  
Driving Horizontal Force=2284.5 lb  

Valid / Invalid Surfaces  
Method: spencer  
Number of Valid Surfaces: 3310  
Number of Invalid Surfaces: 2090  
Error Codes:  
Error Code -100 reported for 18 surfaces
Error Code -105 reported for 19 surfaces
Error Code -106 reported for 116 surfaces
Error Code -107 reported for 747 surfaces
Error Code -108 reported for 371 surfaces
Error Code -111 reported for 539 surfaces
Error Code -112 reported for 280 surfaces

Error Codes
The following errors were encountered during the computation:

-100 = Both surface / slope intersections are on the same horizontal surface. In general, this will give a very high or infinite factor of safety (zero driving force), if calculated.

-105 = More than two surface / slope intersections with no valid slip surface.

-106 = Average slice width is less than 0.0001 * (maximum horizontal extent of soil region). This limitation is imposed to avoid numerical errors which may result from too many slices, or too small a slip region.

-107 = Total driving moment or total driving force is negative. This will occur if the wrong failure direction is specified, or if high external or anchor loads are applied against the failure direction.

-108 = Total driving moment or total driving force < 0.1. This is to limit the calculation of extremely high safety factors if the driving force is very small (0.1 is an arbitrary number).

-111 = safety factor equation did not converge

-112 = The coefficient M-Alpha = cos(alpha)/(1+tan(alpha)tan(phi)/F) < 0.2 for the final iteration of the safety factor calculation. This screens out some slip surfaces which may not be valid in the context of the analysis, in particular: deep seated slip surfaces with many high negative base angle slices in the passive zone.

Probabilistic Analysis Input

Project Settings
Sensitivity Analysis: On
Probabilistic Analysis: On
Sampling Method: Latin-Hypercube
Number of Samples: 400
Analysis Type: Overall Slope

Material: FILL - Silt and Sand
Property: Cohesion
Distribution: Normal
Minimum: 10 (relative minimum: 60)
Mean: 70
Maximum: 130 (relative maximum: 60)
Standard Deviation: 20

Material: FILL - Silt and Sand
Property: Phi

Distribution: Normal
Minimum: 30 (relative minimum: 2.00001)
Mean: 32
Maximum: 34 (relative maximum: 2.00001)
Standard Deviation: 0.66667

Material: FILL - Silt and Sand
Property: Unit Weight
Distribution: Normal
Minimum: 94 (relative minimum: 6)
Mean: 100
Maximum: 106 (relative maximum: 6)
Standard Deviation: 2

Material: FILL - Sand - Silt - Medium Dense
Property: Cohesion
Distribution: Normal
Minimum: 52 (relative minimum: 60)
Mean: 112
Maximum: 172 (relative maximum: 60)
Standard Deviation: 20

Material: FILL - Sand - Silt - Medium Dense
Property: Phi
Distribution: Normal
Minimum: 30 (relative minimum: 2.00001)
Mean: 32
Maximum: 34 (relative maximum: 2.00001)
Standard Deviation: 0.66667

Material: FILL - Sand - Silt - Medium Dense
Property: Unit Weight
Distribution: Normal
Minimum: 97 (relative minimum: 6)
Mean: 100
Maximum: 109 (relative maximum: 6)
Standard Deviation: 2

Support: Soil Nail - Bond
Property: Bond Strength
Distribution: Normal
Minimum: 755 (relative minimum: 753)
Mean: 1508
Maximum: 2261 (relative maximum: 753)
Standard Deviation: 251

Support: Microplane
Property: Pile Shear Strength
Distribution: Normal
Minimum: 25 (relative minimum: 75)
Mean: 100
Maximum: 175 (relative maximum: 75)
Standard Deviation: 25

Support: Microplane - Tensile
Property: Adhesion
Distribution: Normal
Minimum: 720 (relative minimum: 720)
Mean: 1440
Maximum: 2160 (relative maximum: 720)
Standard Deviation: 240

Probabilistic Analysis Results (Overall Slope)

Method: spencer
Factor of Safety, mean: 1.331389
Factor of Safety, standard deviation: 0.128161
Factor of Safety, minimum: 0.900257
Factor of Safety, maximum: 1.691830
Probability of Failure: 0.500% (= 2 failed surfaces /
Critical Probabilistic Surface

Method: spencer
Normal Reliability Index: 2.35747
Probability of Failure: 0.00617284
Mean Factor of Safety: 1.81368
Center: 61.4377, 814.359
Radius: 19.8953
Left Slip Surface Endpoint: 68.4886, 796.829
Right Slip Surface Endpoint: 78.1385, 807.748
Lognormal Reliability Index: 2.95786
Probability of Failure: 0.005
Mean Factor of Safety: 1.33182
Center: 68.9741, 813.114
Radius: 14.9684
Left Slip Surface Endpoint: 69.9114, 796.175
Right Slip Surface Endpoint: 83.6521, 810.179

List of All Coordinates

Max Water Table
-0.000 766.302
14.176 772.189
20.494 773.965
27.895 775.618
37.689 777.404
46.100 778.040
62.870 780.040
86.170 782.960
100.400 785.880
129.020 792.550
155.489 800.500
173.780 807.730
201.470 819.110
229.040 823.030

Material Boundary
26.730 786.468
50.852 783.996

Material Boundary
-0.000 766.302
3.201 766.143
4.202 766.355
5.263 767.173
9.013 767.542
10.965 767.715
12.134 767.487
15.103 769.107
16.684 769.688
17.256 769.888
19.399 770.403
22.614 772.189
24.067 772.888
25.900 773.189
27.758 773.689
29.901 774.404
31.888 774.975
33.474 775.618
35.831 776.618
37.689 777.404
41.475 779.119
42.589 779.528

45.429 780.943
47.976 782.333
50.406 783.834
50.852 783.966
52.907 786.335
56.282 787.481
58.622 788.978
60.599 790.390
63.481 792.622
64.809 793.765
66.048 794.731
66.992 795.637
68.196 796.552

Material Boundary
66.048 794.731
160.672 797.792

Material Boundary
12.134 767.487
31.791 767.807
56.223 770.922
82.403 776.750
110.319 783.498
128.145 787.807
147.270 793.430
160.672 797.792
175.900 803.884
186.464 808.110
197.191 812.901
204.796 816.465
218.039 823.541
229.040 828.337

Material Boundary
0.000764.800
11.840 755.090
32.252 765.192
56.681 769.010
82.583 774.797
110.940 781.653
128.616 785.927
147.240 791.430
160.898 795.802
176.159 801.804
187.020 806.076
197.032 810.514
205.190 814.373
217.879 821.154
229.040 827.017

Material Boundary
91.720 810.340
92.050 810.340
94.869 826.380

External Boundary
-0.000 766.302
0.000 764.800
0.000 740.550
229.040 740.650
229.040 827.017
229.040 829.337
229.040 835.939
222.354 834.847
195.432 827.596
186.324 826.020
187.984 828.706
184.901 828.680
179.453 827.230
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<tr>
<th>Distributed Load</th>
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<tbody>
<tr>
<td>94.550</td>
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<tr>
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<td>826.380</td>
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<td>98.034</td>
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<td>103.633</td>
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<td>112.705</td>
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<thead>
<tr>
<th>Support</th>
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</thead>
<tbody>
<tr>
<td>80.250</td>
<td>810.110</td>
</tr>
<tr>
<td>80.250</td>
<td>765.110</td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>Support</th>
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</tr>
</thead>
<tbody>
<tr>
<td>83.271</td>
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</tr>
<tr>
<td>94.897</td>
<td>766.783</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Support</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>92.084</td>
<td>812.400</td>
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<tr>
<td>121.061</td>
<td>804.635</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Support</th>
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</tr>
</thead>
<tbody>
<tr>
<td>92.613</td>
<td>815.400</td>
</tr>
</tbody>
</table>
Project Identification - Upper Hacienda Road - 16' Wall w/ Foreslope

--------- WALL GEOMETRY ---------

Vertical Wall Height = 16.0 ft
Wall Batter = 10.0 degree
Angle Length
(Deg) (Feet)
First Slope from Wallcrest. = 0.0 50.0
Second Slope from 1st slope. = 0.0 0.0
Third Slope from 2nd slope. = 0.0 0.0
Fourth Slope from 3rd slope. = 0.0 0.0
Fifth Slope from 3rd slope. = 0.0 0.0
Sixth Slope from 3rd slope. = 0.0 0.0
Seventh Slope Angle. = 0.0

--------- SLOPE BELOW THE WALL ---------

First Slope Angle below Toe. = 3.1 degrees
First Slope Distance from Toe. = 33.4 ft
Second Slope Angle. = 31.0 degrees
Second Slope Distance from Toe. = 84.0 ft
Vertical Depth of Search. = 30.0 ft
Number of Searches below wall Toe. = 5

--------- SURCHARGE ---------

THE SURCHARGES IMPOSED ON THE SYSTEM ARE:
Begin Surcharge - Distance from toe = 2.8 ft
End Surcharge - Distance from toe = 30.0 ft
Loading Intensity - Begin = 250.0 psf/ft
Loading Intensity - End = 250.0 psf/ft

--------- OPTION #1 ---------

Ultimate Punching shear, Bond & Yield Stress are used.

--------- SOIL PARAMETERS ---------

<table>
<thead>
<tr>
<th>Soil Layer</th>
<th>Unit Weight (pcf)</th>
<th>Friction Angle (Degree)</th>
<th>Cohesion Intercept (psf)</th>
<th>Bond Stress (psi)</th>
<th>Xs1 (ft)</th>
<th>Ys1 (ft)</th>
<th>Xs2 (ft)</th>
<th>Ys2 (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>110.0</td>
<td>32.0</td>
<td>70.0</td>
<td>10.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
</tbody>
</table>

* Ultimate bond Stress values also depend on BSF (Bond Stress Factor.)
NO Water Table defined for this problem.

-------- SEARCH LIMIT --------

The Search Limit is from 105.0 to 50.0 ft

You have chosen NOT TO LIMIT the search of failure planes to specific nodes.

-------- REINFORCEMENT PARAMETERS --------

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of Reinforcement Levels</td>
<td>5</td>
</tr>
<tr>
<td>Horizontal Spacing</td>
<td>3.0 ft</td>
</tr>
<tr>
<td>Diameter of Reinforcement Element</td>
<td>1.250 in</td>
</tr>
<tr>
<td>Yield Stress of Reinforcement</td>
<td>75.0 ksi</td>
</tr>
<tr>
<td>Diameter of Grouted Hole</td>
<td>4.0 in</td>
</tr>
<tr>
<td>Punching Shear</td>
<td>39.2 kips</td>
</tr>
</tbody>
</table>

-------- (For ALL Levels) --------

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reinforcement Lengths</td>
<td>30.0 ft</td>
</tr>
<tr>
<td>Reinforcement Inclination</td>
<td>15.0 degrees</td>
</tr>
<tr>
<td>Vertical Spacing to First Level</td>
<td>2.0 ft</td>
</tr>
<tr>
<td>Vertical Spacing to Remaining Levels</td>
<td>3.0 ft</td>
</tr>
<tr>
<td>Depth Below Wall Toe (Ft)</td>
<td>Minimum Safety Factor</td>
</tr>
<tr>
<td>--------------------------</td>
<td>-----------------------</td>
</tr>
<tr>
<td>Toe</td>
<td>3.44</td>
</tr>
</tbody>
</table>

Reinf. Stress at Level 1 = 9.164 ksi (Pullout controls...)
2 = 9.274 ksi (Pullout controls...)
3 = 9.383 ksi (Pullout controls...)
4 = 9.493 ksi (Pullout controls...)
5 = 9.603 ksi (Pullout controls...)

<table>
<thead>
<tr>
<th>Depth Below Wall Toe (Ft)</th>
<th>Minimum Safety Factor</th>
<th>Distance Behind Wall Toe (Ft)</th>
<th>Lower Failure Plane Angle (Deg)</th>
<th>Lower Failure Plane Length (Ft)</th>
<th>Upper Failure Plane Angle (Deg)</th>
<th>Upper Failure Plane Length (Ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.00</td>
<td>2.60</td>
<td>26.4</td>
<td>18.4</td>
<td>13.9</td>
<td>53.1</td>
<td>22.0</td>
</tr>
</tbody>
</table>

Reinf. Stress at Level 1 = 5.041 ksi (Pullout controls...)
2 = 5.743 ksi (Pullout controls...)
3 = 6.445 ksi (Pullout controls...)
4 = 7.147 ksi (Pullout controls...)
5 = 7.849 ksi (Pullout controls...)

<table>
<thead>
<tr>
<th>Depth Below Wall Toe (Ft)</th>
<th>Minimum Safety Factor</th>
<th>Distance Behind Wall Toe (Ft)</th>
<th>Lower Failure Plane Angle (Deg)</th>
<th>Lower Failure Plane Length (Ft)</th>
<th>Upper Failure Plane Angle (Deg)</th>
<th>Upper Failure Plane Length (Ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>12.00</td>
<td>2.47</td>
<td>35.8</td>
<td>32.0</td>
<td>21.1</td>
<td>43.2</td>
<td>24.6</td>
</tr>
</tbody>
</table>

Reinf. Stress at Level 1 = 2.415 ksi (Pullout controls...)
2 = 3.487 ksi (Pullout controls...)
3 = 4.559 ksi (Pullout controls...)
4 = 5.631 ksi (Pullout controls...)
5 = 6.988 ksi (Pullout controls...)

<table>
<thead>
<tr>
<th>Depth Below Wall Toe (Ft)</th>
<th>Minimum Safety Factor</th>
<th>Distance Behind Wall Toe (Ft)</th>
<th>Lower Failure Plane Angle (Deg)</th>
<th>Lower Failure Plane Length (Ft)</th>
<th>Upper Failure Plane Angle (Deg)</th>
<th>Upper Failure Plane Length (Ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>18.00</td>
<td>2.39</td>
<td>45.3</td>
<td>36.9</td>
<td>56.6</td>
<td>89.9</td>
<td>0.0</td>
</tr>
</tbody>
</table>

Reinf. Stress at Level 1 = 0.000 ksi
2 = 1.034 ksi (Pullout controls...)
3 = 2.393 ksi (Pullout controls...)
4 = 3.751 ksi (Pullout controls...)
5 = 5.110 ksi (Pullout controls...)

<table>
<thead>
<tr>
<th>Depth Below Wall Toe (Ft)</th>
<th>Minimum Safety Factor</th>
<th>Distance Behind Wall Toe (Ft)</th>
<th>Lower Failure Plane Angle (Deg)</th>
<th>Lower Failure Plane Length (Ft)</th>
<th>Upper Failure Plane Angle (Deg)</th>
<th>Upper Failure Plane Length (Ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>24.00</td>
<td>2.26</td>
<td>45.3</td>
<td>41.5</td>
<td>60.4</td>
<td>89.9</td>
<td>0.0</td>
</tr>
</tbody>
</table>

Reinf. Stress at Level 1 = 0.000 ksi
2 = 0.039 ksi (Pullout controls...)
3 = 1.280 ksi (Pullout controls...)
4 = 2.520 ksi (Pullout controls...)
5 = 3.761 ksi (Pullout controls...)

<table>
<thead>
<tr>
<th>Depth Below Wall Toe (Ft)</th>
<th>Minimum Safety Factor</th>
<th>Distance Behind Wall Toe (Ft)</th>
<th>Lower Failure Plane Angle (Deg)</th>
<th>Lower Failure Plane Length (Ft)</th>
<th>Upper Failure Plane Angle (Deg)</th>
<th>Upper Failure Plane Length (Ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>30.00</td>
<td>2.21</td>
<td>45.3</td>
<td>45.5</td>
<td>64.5</td>
<td>89.9</td>
<td>0.0</td>
</tr>
</tbody>
</table>

Reinf. Stress at Level 1 = 0.000 ksi
2 = 0.000 ksi
3 = 0.288 ksi (Pullout controls...)
4 = 1.393 ksi (Pullout controls...)
5 = 2.499 ksi (Pullout controls...)

******************************************************************************
* For Factor of Safety = 1.0
* Maximum Average Reinforcement Working Force: 1.879 Kips/level
******************************************************************************
Project Identification - Upper Hacienda Road - 16' Wall w/ Foreslope k=0.19

--------- WALL GEOMETRY -------

Vertical Wall Height = 16.0 ft
Wall Batter = 10.0 degree

First Slope from Wallcrest. = 0.0 50.0
Second Slope from 1st slope. = 0.0 0.0
Third slope from 2nd slope. = 0.0 0.0
Fourth Slope from 3rd slope. = 0.0 0.0
Fifth Slope from 3rd slope. = 0.0 0.0
Sixth slope from 3rd slope. = 0.0 0.0
Seventh Slope Angle. = 0.0

--------- SLOPE BELOW THE WALL -------

First Slope Angle below Toe. = 1.1 degrees
First Slope Distance from Toe. = 33.4 ft
Second Slope Angle. = 31.0 degrees
Second Slope Distance from Toe. = 84.5 ft
Vertical Depth of Search. = 30.0 ft
Number of Searches below wall Toe. = 5

--------- SURCHARGE -------

THE SURCHARGES IMPOSED ON THE SYSTEM ARE:

Begin Surcharge - Distance from toe = 2.8 ft
End Surcharge - Distance from toe = 30.0 ft
Loading Intensity - Begin = 250.0 psf/ft
Loading Intensity - End = 250.0 psf/ft

--------- OPTION #1 -------

Ultimate Punching shear, Bond & Yield Stress are used.

--------- SOIL PARAMETERS -------

<table>
<thead>
<tr>
<th>Soil Layer</th>
<th>Unit Weight (Psf)</th>
<th>Friction Angle (Degree)</th>
<th>Cohesion Intercept (Psi)</th>
<th>Bond Stress (Psi)</th>
<th>Coordinates of Boundary Xs1 (ft)</th>
<th>Ys1 (ft)</th>
<th>Xs2 (ft)</th>
<th>Ys2 (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>110.0</td>
<td>32.0</td>
<td>70.0</td>
<td>10.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
</tbody>
</table>

* Ultimate bond Stress values also depend on BSF (Bond Stress Factor.)
--------- EARTHQUAKE ACCELERATION ---------

Horizontal Earthquake Coefficient = 0.19 (a/g)
Vertical Earthquake Coefficient = 0.00

--------- WATER SURFACE ---------

NO Water Table defined for this problem.

--------- SEARCH LIMIT ---------

The Search Limit is from 105.0 to 50.0 ft

You have chosen NOT TO LIMIT the search of failure planes to specific nodes.

--------- REINFORCEMENT PARAMETERS ---------

Number of Reinforcement Levels = 5
Horizontal Spacing = 3.0 ft
Diameter of Reinforcement Element = 1.250 in
Yield Stress of Reinforcement = 75.0 ksi
Diameter of Grouted Hole = 4.0 in
Punching Shear = 39.2 kips

--------- (For ALL Levels) ---------

Reinforcement Lengths = 30.0 ft
Reinforcement Inclination = 15.0 degrees
Vertical Spacing to First Level = 2.0 ft
Vertical Spacing to Remaining Levels = 3.0 ft
<table>
<thead>
<tr>
<th>DEPTH (ft)</th>
<th>MINIMUM WALL TOE FACTOR</th>
<th>MINIMUM WALL TOE ANGLE (deg)</th>
<th>LOWER FAILURE ANGLE (deg)</th>
<th>LOWER FAILURE LENGTH (ft)</th>
<th>UPPER FAILURE ANGLE (deg)</th>
<th>UPPER FAILURE LENGTH (ft)</th>
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<tr>
<td>12.00</td>
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<td>21.7</td>
<td>33.2</td>
<td>35.8</td>
</tr>
<tr>
<td>18.00</td>
<td>1.35</td>
<td>50.0</td>
<td>28.5</td>
<td>28.5</td>
<td>39.2</td>
<td>32.3</td>
</tr>
<tr>
<td>24.00</td>
<td>1.32</td>
<td>50.0</td>
<td>38.7</td>
<td>64.0</td>
<td>89.9</td>
<td>0.0</td>
</tr>
<tr>
<td>30.00</td>
<td>1.35</td>
<td>50.0</td>
<td>42.6</td>
<td>67.9</td>
<td>89.9</td>
<td>0.0</td>
</tr>
</tbody>
</table>

Reinf. Stress at Level

1. For Factor of Safety = 1.0
2. Maximum Average Reinforcement Working Force:
3. 3.415 Kips/level

* For Factor of Safety = 1.0
* Maximum Average Reinforcement Working Force:
* 3.415 Kips/level
Project Identification - Upper Hacienda Road - 16' Wall w/ Foreslope GWT

--------- WALL GEOMETRY ---------

Vertical Wall Height = 16.0 ft
Wall Batter = 10.0 degree

Angle Length (Deg) (Feet)
First Slope from Wallcrest. = 0.0 50.0
Second Slope from 1st slope. = 0.0 0.0
Third Slope from 2nd slope. = 0.0 0.0
Fourth Slope from 3rd slope. = 0.0 0.0
Fifth Slope from 3rd slope. = 0.0 0.0
Sixth Slope from 3rd slope. = 0.0 0.0
Seventh Slope Angle. = 0.0

--------- SLOPE BELOW THE WALL ---------

First Slope Angle below Toe. = 1.1 degrees
First Slope Distance from Toe. = 33.4 ft
Second Slope Angle. = 31.0 degrees
Second Slope Distance from Toe. = 84.0 ft
Vertical Depth of Search. = 30.0 ft
Number of Searches below wall Toe. = 5

--------- SURCHARGE ---------

THE SURCHARGES IMPOSED ON THE SYSTEM ARE:

Begin Surcharge - Distance from toe = 2.8 ft
End Surcharge - Distance from toe = 30.0 ft
Loading Intensity - Begin = 250.0 psf/ft
Loading Intensity - End = 250.0 psf/ft

Begin Second Surcharge - Distance from toe = 0.0 ft
End Second Surcharge - Distance from toe = 0.0 ft
Loading Intensity - Begin = 12.0 psf/ft
Loading Intensity - End = 0.0 psf/ft

--------- OPTION #1 ---------

Ultimate Punching shear, Bond & Yield Stress are used.

--------- SOIL PARAMETERS ---------

<table>
<thead>
<tr>
<th>Soil Layer</th>
<th>Unit Weight (pcf)</th>
<th>Friction Angle (Degree)</th>
<th>Cohesion (Psf)</th>
<th>Bond Stress (Psi)</th>
<th>Coordinates of Boundary</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>110.0</td>
<td>32.0</td>
<td>70.0</td>
<td>10.0</td>
<td>XS1 YS1 XS2 YS2</td>
</tr>
</tbody>
</table>

* Ultimate bond Stress values also depend on BSF (Bond Stress Factor.)
The Water Table is defined by three coordinate points.

X(1)-Coordinate = 100.00 ft   Y(1)-Coordinate = -45.00 ft
X(2)-Coordinate =  0.00 ft    Y(2)-Coordinate = -10.00 ft
X(3)-Coordinate =  50.00 ft   Y(3)-Coordinate =  4.00 ft

-------- SEARCH LIMIT --------

The Search Limit is from 105.0 to 50.0 ft

You have chosen NOT TO LIMIT the search of failure planes to specific nodes.

-------- REINFORCEMENT PARAMETERS --------

Number of Reinforcement Levels  = 5
Horizontal Spacing             = 3.0 ft
Diameter of Reinforcement Element  = 1.250 in
Yield Stress of Reinforcement   = 75.0 ksi
Diameter of Grouted Hole        = 4.0 in
Punching Shear                  = 39.2 kips

-------- (FOR ALL Levels) --------

Reinforcement Lengths  = 30.0 ft
Reinforcement Inclination  = 15.0 degrees
Vertical Spacing to First Level  = 2.0 ft
Vertical Spacing to Remaining Levels  = 3.0 ft
<table>
<thead>
<tr>
<th>Depth Below Wall Toe (ft)</th>
<th>Minimum Distance Behind Plane (ft)</th>
<th>Lower Failure Plane Angle (deg)</th>
<th>Lower Failure Plane Length (ft)</th>
<th>Upper Failure Plane Angle (deg)</th>
<th>Upper Failure Plane Length (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.00</td>
<td>2.60</td>
<td>26.4</td>
<td>18.4</td>
<td>13.9</td>
<td>53.1</td>
</tr>
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<td>45.3</td>
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<td>44.2</td>
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</tr>
</tbody>
</table>

Reinf. Stress at Level 1 = 5.041 Ksi (Pullout controls...)
2 = 5.743 Ksi (Pullout controls...)
3 = 6.445 Ksi (Pullout controls...)
4 = 7.147 Ksi (Pullout controls...)
5 = 7.849 Ksi (Pullout controls...)

For Factor of Safety = 1.0
Maximum Average Reinforcement Working Force: 4.691 Kips/level