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**Final Addendum Geotechnical Report No. 1**  
**White Point Landslide**  
**W.O. E1907483, Task Order Solicitation Number 11-087**  
**San Pedro District, Los Angeles, California**

1.0 INTRODUCTION

This addendum report presents geotechnical information for the White Point Landslide in the San Pedro District of Los Angeles (Figure 1). This addendum report summarizes field explorations we performed between July 16 and July 24, 2012, and dewatering recommendations. This report incorporates information presented in our Final Geotechnical Report dated August 15, 2012 (Final Report).


2.0 SCOPE OF SERVICES

Our scope of services is based on the Task Order Solicitation (TOS) No. 11-087, dated June 18, 2012, and our proposal, dated August 3, 2012. The City of Los Angeles (City) Bureau of Engineering (BOE) authorized portions of our scope with limited notice to proceeds (NTPs) for Task 2 (including portions of Subtasks 2.1 through 2.4) on July 2 and September 4, 2012. We received full NTP for our Task 2 scope of services on October 31, 2012. This report includes:

- File research and document review;
- Field exploration plan,
- Boring logs, borehole instrumentation construction logs, borehole geophysical logs,
- Instrumentation monitoring and results,
- Geotechnical and analytical laboratory test results,
- Geologic mapping results,
- Geologic cross sections,
- Stability analyses, and
- Dewatering analysis and recommendations.

We prepared a draft of this report dated November 16, 2012 for review by the City. We reviewed and incorporated comments from the City from their review of the draft report into this final report.
3.0 RESEARCH AND DOCUMENT REVIEW

3.1 General

The TOS requests review of residential structures in the vicinity of the study area and White Point Nike Missile Base. We previously reviewed documents for the Nike Missile Base as described in Section 2.4 of our Final Report and previously provided copies of these documents to the City. Per our discussions with the City, we limited our research and document review to the residences as described below for this report.

We reviewed City of Los Angeles Department of Building & Safety (LADBS) records on file for residential structures on the south side of Paseo del Mar immediately east of the landslide area: 1471, 1479 and 1481 Paseo del Mar (Appendix A). A summary of the documents reviewed and geotechnical observations is discussed for each residence below.

3.2 1471 Paseo del Mar

3.2.1 Documents Reviewed

- Geotechnical report by Lockwood-Singh & Associates (LSA) dated May 15, 1979 (report text and Plates B through E only).

3.2.2 Summary of Observations

LSA completed two test pits to a maximum depth of 6 feet. They recorded black, stiff, clay to depths ranging from 3.5 to 4 feet, underlain by moderately hard siltstone to 6 feet. LSA did not observe conditions of past geologic instability. LSA performed a slope stability analysis on a 100-foot, 1.5:1 (horizontal to vertical) slope south of the property and reported a factor of safety of 1.58.

3.3 1479 Paseo del Mar

3.3.1 Documents Reviewed

The LADBS records for 1479 Paseo del Mar we reviewed include (in chronological order):

- Geotechnical report by Richard Mills Associates, Inc. (RMA) dated October 13, 1978 (report text and Plate 3 only), and (RMA, 1978);
3.3.2 Summary of Observations

The RMA report assesses whether geologic conditions contributed to failure of a 5-feet tall retaining wall located along the northeast side of the residence and to prepare recommendations for a proposed replacement retaining wall (RMA, 1978). The RMA report includes a geologic map of the slope area above, below, and adjacent to both ends of the proposed retaining wall and four trenches (test pits) along the location of the proposed retaining wall. They encountered fill soil, related to the construction of an existing residence, consisting of brown, soft, clayey silt with sand and rock fragments to a depth of about 5 feet. Soil in the vicinity of the proposed retaining wall consisted of “rocky” silt and sand. The RMA report noted the slope above the proposed retaining wall is prone to surficial erosion and soil creep from sheetflow, mudflows, and surficial landslides, while the area near the toe of the slope is prone to beach erosion.

The RMA report mapped two potential landslides. One landslide damaged the retaining wall, upslope from the northeast part of the residence, approximately 32 feet wide, 18 feet long, and at least 8 feet deep. The other landslide was mapped along the west property line and is approximately 30 feet wide, 35 to 40 feet long, and of unknown depth. The RMA report did not indicate large landslides failures other than those noted on or adjacent to the residential site.

The RMA report performed a slope stability analysis of the surficial soil failures along the slope following the geometry of the mapped landslides, debris slides, and mudflows that occurred during the heavy rains between February and March, 1978. Approximately 5 to 10 yards of landslide debris accumulated along the northeast side of the existing house. Portions of the damaged retaining wall were collapsed or tilted during these failures, although some deformation had been noticed earlier by the property owner.

Although the RMA report found no indication of large failures at or near the site, they summarized that the available geologic data suggest that there is potential for future large instability due to adversely oriented bedding planes in the Altamira Shale underlying the existing slope. RMA concluded surficial failures in the form of shallow landslides, debris slides and mudflows can be expected to continue and may constitute a continuing maintenance problem.
The CKC Report completed additional field and laboratory testing in support of final
design of the new retaining wall. The CKC report did not provide additional geologic
information for the property.

3.4 1481 Paseo del Mar

3.4.1 Documents Reviewed

The LADBS records for 1481 Paseo del Mar we reviewed include (in chronological
order):

- Geotechnical report by Keith W. Ehlert, Consulting Engineering Geologist (KWE)
dated November 12, 1998 (report text and test boring only), and (KWE, 1998);
- City of Los Angeles review letter dated January 28, 1999 of KWE report above
(LADBS, 1999);
- City of Los Angeles update letter of project dated July 17, 2001 (LADBS, 2001a);
- City of Los Angeles review letter dated August 20, 2001 of KWE report dated July
12, 2001 (not included in LADBS file) (LADBS, 2001b);
- Geotechnical report by Dale Hinkle, P.E. Inc. (DH) dated November 12, 2001 (report
text only), and (DH, 2001);
- City of Los Angeles review letter dated December 27, 2001 of DH report dated
November 12, 2001 (LADBS, 2001c);
- DH geotechnical report dated January 12, 2002 (report text only) (DH, 2002a);
- City of Los Angeles review letter dated February 14, 2002 of DH report dated
January 12, 2002 (LADBS, 2002a);
- DH geotechnical report dated April 3, 2002 (report text and Figure 3 only) (DH,
2002b);
- City of Los Angeles review letter dated July 16, 2002 of DH report dated April 3,
2002 (LADBS, 2002b);
- DH geotechnical report dated August 15, 2002 (report text and Figures 1 and 4 only)
(DH, 2002a), and;
- City of Los Angeles approval letter dated September 10, 2002 of DH report dated
3.4.2 Summary of Observations

The KWE report was prepared for the proposed addition to a detached garage and eventual conversion into a two-story single-family guest house (KWE, 1998). The owner applied to have the proposed guest house addressed as 1475 Paseo del Mar, while the main house retained 1481 Paseo del Mar. KWE performed one boring to 80 feet. While downhole logging, they encountered brown, fill consisting of sandy silt from 0 to 2.5 feet, natural soil consisting of dark brown, silty clay with scattered pebbles and rock fragments from 2.5 to 6 feet, and bedrock from 6 to 80 feet. Bedding is variable down to a depth of 65 feet generally dipping at 4 to 14 degrees to the west-southwest. Groundwater was percolating out of the side of the boring at a depth of 51 feet and water was “squirting” out of the boring at depth of 59 feet. The geologist noted that when the bedrock is picked out, it is dry and water appears to be percolating through joints in the rock, not along bedding planes. At the completion of drilling, the water level in the borehole had risen to a depth of 67 feet.

The DH reports consisted of communications between KWE and LADBS in addition to performing geologic mapping and slope stability analyses. DH noted less than 3 degrees of dip into the slope and the observed failures are likely due to over-the-slope drainage. In the surficial stability analysis, DH considered the bedding with no more than 5 degrees and found factors of safety of 1.56 for the terrace deposits, 1.14 for the slope slough, and 3.7 for the exposed bedrock. The DH report suggested removing loose soil on the slope or adding a concrete slough wall to protect structures below the slope (DH, 2001). The DH report noted the house has a short wall which is likely not adequate to prevent occasional mudflow.

The DH response letter to LADBS comments noted that there is an approximately 12-feet tall concrete plug at the top of the slope extending beneath 1481 Paseo del Mar which tapers down to zero at the street (DH, 2002c). The plug is 20 to 30 feet wide at the bluff and underlies the house. The DH response letter recommended a caisson system to support the proposed guest house.

4.0 FIELD EXPLORATIONS

We completed two additional borings for this phase of work designated B-10 and B-11. Both borings were completed on the east side of the 2011 Landslide along the Paseo Del Mar right-of-way. Refer to our Final Report for information and location of the previous borings B-1 through B-9, including boring logs. Revisions to the B-1 and B-7 boring logs are also included in this
report. Locations of the borings are shown in Plates 1 and 2. The techniques used to advance and sample borings B-10 and B-11 are described below.

4.1 Drilling

Gregg Drilling & Testing, Inc. (Gregg) of Signal Hill, California provided and operated a track-mounted CME 75 rotary drill rig to complete borings B-10 and B-11 between July 16 and July 24, 2012. An engineering geologist from Shannon & Wilson supervised the field exploration program. Shannon & Wilson field engineering geologists located the borings, observed the exploratory drilling, collected samples, and logged the borings. Our field activity reports during the drilling are included in Appendix B.

4.2 Health and Safety Plan

A Health and Safety Plan was prepared before initiation of the drilling and geologic mapping program. The plan identified known hazards at the site and possible hazards related to subsurface structures and utilities. The plan was submitted to City representatives for their review and approval. The field program was completed with no reportable injuries to Shannon & Wilson personnel or subcontractors. A copy of the Health & Safety Plan is included in Appendix B.

4.3 Rotary Coring

Continuous HQ3 coring was used in borings B-10 and B-11 to sample and advance through rock. Boring logs are provided in Appendix C. Core samples were visually classified and described in the field, then boxed for transport to our laboratory and storage facility for further examination. Photographs of the core are provided in Appendix C.

The rock core recovery shown on the boring logs was calculated by dividing the length of core recovered in the barrel by the length of each drilled run. This ratio is expressed as a percent. Recovery typically varied from 86 to 100 percent where core was recovered.

The rock quality designation (RQD) shown graphically on the boring logs is a modified core recovery percentage that includes only the total length of intact core pieces that are more than 4 inches long, divided by the total length of the core run. Rock core pieces shorter than 4 inches that are the result of close jointing, fracturing, or weathering in the rock mass were excluded from the RQD calculation. We distinguished natural fractures in the rock core from mechanical breaks due to drilling operations by shape and the texture of the fracture surfaces. While RQD
requires some interpretation, it does provide an estimate of rock mass quality and a comparison of rock quality in the borings. RQD varied from 0 to 100 for the samples collected from B-10 and B-11.

4.4 Packer Testing

Packer tests consist of isolating specific sections of a borehole with inflatable packers (bladders) to estimate the hydraulic conductivity of water-bearing zones. Monitoring groundwater levels in nearby wells or piezometers during the packer test can provide additional information regarding the soil and/or rock hydraulic conductivity and storativity in a larger area.

Packer tests consist of isolating specific sections of a borehole with inflatable packers (bladders) to estimate the hydraulic conductivity of water-bearing zones. Most packer testing is done by pumping water into the ground at a constant pressure and measuring flow. The hydraulic conductivity calculations are done using steady state well flow equations. For this study we also measured the pressure drop-off after each constant pressure test and calculated hydraulic conductivity. The packer testing methods and results performed in borings B-10 and B-11 are presented in Appendix D.

During the packer testing in boring B-11, piezometers in boring B-10 indicated a pressure response (See Figure J-5) suggesting hydraulic connectivity between the two boreholes. Using this data, we were able to make additional calculations of rock hydraulic conductivity and storativity between the packer test in boring B-11 and the piezometers in boring B-10. Section 8.0 and Appendix D present details of those calculations. The following table summarizes the packer test results:

<table>
<thead>
<tr>
<th>Boring</th>
<th>Test Interval</th>
<th>Calculated Hydraulic conductivity (cm/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Constant Pressure Test</td>
</tr>
<tr>
<td>B-10</td>
<td>72.0 – 85.5</td>
<td>5.0x10^-6</td>
</tr>
<tr>
<td></td>
<td>87.0 – 100.5</td>
<td>3.2x10^-3</td>
</tr>
<tr>
<td></td>
<td>102.0 – 115.5</td>
<td>3.0x10^-4</td>
</tr>
<tr>
<td></td>
<td>117.0 – 130.5</td>
<td>4.6x10^-6</td>
</tr>
<tr>
<td>B-11</td>
<td>72.5 – 85.5</td>
<td>4.0x10^-3</td>
</tr>
<tr>
<td></td>
<td>87.0 – 100.5</td>
<td>1.3x10^-6</td>
</tr>
<tr>
<td></td>
<td>102.0 – 115.5</td>
<td>3.8x10^-3</td>
</tr>
<tr>
<td></td>
<td>117.0 – 130.5</td>
<td>N/A</td>
</tr>
</tbody>
</table>

Notes:
1. Flow rate was zero.
2. Response was too rapid to measure drop-off.
4.5 **Borehole Logging**

GEOVision of Corona, California performed induction and natural gamma logging in borings B-1, B-3, B-5, B-6, B-7, B-8, B-9, B-10 and B-11, optical and acoustic television logging in boring B-10, and acoustic television logging in boring B-11. A summary of the testing procedures and a copy of the GEOVision report are attached in Appendix E.

4.6 **Instrumentation**

We installed inclinometers to measure horizontal ground movement, and six vibrating wire piezometers (VWPs) to measure groundwater pressure at different elevations in borings B-10 and B-11. We also installed VWPs in previously constructed wells in borings B-3, B-6 and B-8. As VWPs were previously installed in borings B-1, B-5, B-7, and B-9, groundwater level or pressure is now being monitored in nine borings (B-1, B-3, B-5, B-6, B-7, B-8, B-9, B-10 and B-11). We installed dataloggers to continuously record the VWP data at each boring. Appendix F presents details about the inclinometer, VWP, and dataloggers. The boring logs located in Appendix A of this report and in the Final Report show the VWP and well depths, and details of the inclinometer installations.

5.0 **LABORATORY TESTING**

5.1 **Geotechnical Testing**

The table below shows the geotechnical laboratory testing performed on samples from borings B-10 and B-11. Appendix G describes the geotechnical laboratory testing methods and presents the results. The boring logs in Appendix C show water content results at the selected sample locations.

<table>
<thead>
<tr>
<th>Number of Tests</th>
<th>Geotechnical Laboratory Tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>In-Place Density and Field Moisture (ASTM D2216 and D2937)</td>
</tr>
<tr>
<td>8</td>
<td>Atterberg Limits (ASTM D4318)</td>
</tr>
<tr>
<td>8</td>
<td>Sieve plus Hydrometer (ASTM D422)</td>
</tr>
<tr>
<td>6</td>
<td>Chemical Tests for corrosion potential (CA DOT Method)</td>
</tr>
<tr>
<td>6</td>
<td>Uniaxial Compressive Strength (ASTM D7012)</td>
</tr>
</tbody>
</table>
5.2 Chemical Testing

To evaluate the possibility of in situ soil contamination and to characterize the boring spoils and mud for appropriate disposal, one sample was collected from each boring and submitted for analytical testing to American Environmental Testing Laboratory, Inc. of Burbank, California. Samples were analyzed by the following methods:

- Volatile organic compounds by Method 8260B (in situ and for disposal purposes)
- Semi-volatile organic compounds by Method 8270C (in situ and for disposal purposes)
- Metals (for disposal purposes)

Samples were collected and delivered to the laboratory and analyzed following chain-of-custody procedures. The boring spoils and mud were collected by Soil Safe of California on October 26, 2012. The laboratory results for the samples and disposal manifest form is presented in Appendix H.

6.0 GROUNDWATER CONDITIONS

6.1 Groundwater Level and Piezometric Head

Groundwater level or piezometric head measurements at the site began in December 2011 during the original phase of work for the landslide study. Previous wells completed by IT Corporation for the U.S. Air Force for the Nike Missile Base had been read in the vicinity of the landslide but were unable to be located during our study. Currently, observation wells and VWPs are installed in borings B-3, B-6, and B-8; one VWP installed in borings B-1, B-5, B-7 and B-9; and three VWPs installed directly into each of borings B-10 and B-11. Each boring contains a datalogger that records hourly groundwater level or piezometric head.

Figures F-2 through F-11 show the recorded groundwater levels or piezometric head recorded between since December 2011.

Observation wells are poly-vinyl chloride (PVC) standpipes installed with a 10 foot section of screened pipe in a water bearing strata; the remainder of the pipe is blank. Sand pack is installed in the screened section of pipe and typically extends two feet above and below the screened section. Therefore, the height of water measured in the observation well indicates the highest piezometric head in a zone up to 4 feet higher than the length of the screened section. Conversely, VWPs are installed in a boring with 2 to 3 feet of sand pack surrounding the
instrument. The groundwater pressure measured by the VWP represents the piezometric head at the filter tip of the VWP. In this report, we use the term “groundwater level” to indicate the elevation of groundwater found in the subsurface. Because groundwater conditions at White Point are complex, no single or simple groundwater level or gradient exists in the landslide area. Rather, the subsurface contains several confined zones that are separated by intervening low permeable confining material or offset and displaced by faulting. We use the term “piezometric head” to describe the groundwater fluid pressure in these confined zones. The following piezometric head data exemplifies the hydrostratigraphic complexity:

- VWPs installed at elevations from 2 to 13 feet in borings B-1, B-5, B-7 and B-8 had piezometric heads of about 57, 61, 24, and 33 feet, respectively. Therefore, VWPs at similar elevations had substantially different piezometric heads.
- The piezometric head in the deepest VWP in boring B-10 (at elevation -7 feet) increased since installation in July 2012. In late September 2012, the piezometric head in this VWP was above elevation 100 feet. The piezometric heads in the two shallower B-10 VWPs (at elevations 10.9 and 40.9 feet) were progressively lower at elevations 46 and 40 feet, respectively. The decreasing head with increasing elevation at B-10 indicates an upward hydraulic gradient.
- In boring B-11, the three VWPs show piezometric heads that varied by less than 10 feet, and indicated an apparent downward hydraulic gradient.

6.2 Precipitation and Recharge

The hydrographs for the 14 piezometers (Appendix F) also present daily precipitation data for the San Pedro area. Most of the annual precipitation occurs between November and March. Of the 14 piezometers, we only have groundwater level data for one datalogger (B-1) before the end of the 2011-12 rainy season, which ends on June 30, 2012. Monitoring in the remaining 13 piezometers commenced during late July 2012.

We started recording groundwater levels in boring B-1 during December 2011. Between January and May 2012, the groundwater level in this boring increased by 15 feet (from elevation 60 to 75 feet) in what appears to be response to approximately four inches of precipitation. Between the end of the rainy season and October 2012, the groundwater level in B-1 declined by 25 feet. Although we do not have a complete year of data, the annual fluctuation magnitude at B-1 may be as much as 30 feet. The groundwater levels in the remaining seven single-completion piezometers declined by between 1.5 and 4 feet between mid-July and late October, indicating a smaller (up to 10 feet) likely annual fluctuation range than at boring B-1.
Since late July 2012, measurable precipitation occurred only on three days, with the highest daily amount of 0.44 inches on November 30, 2012. Groundwater levels responded relatively quickly (less than one day) following the precipitation events, suggesting that the water-bearing zones are reasonably well connected to the point(s) of recharge.

6.3 Hydrogeology

In our opinion, the hydrostratigraphy in the landslide area is characterized by a complex sequence of variably-fractured siltstone and sandstone units that contain a water table (unconfined) aquifer and multiple confined aquifers. The variability in piezometric head indicates that confined aquifers may be thin. A shallow and deeper sequence of groundwater units exists, separated by a low permeability bentonite unit. Figure F-11a (Appendix F) shows the measured piezometric levels in five VWPs completed at similar elevation (6.4 to 14.1 feet above mean sea level)) between July and October 2012. These piezometric levels range between 65 feet elevation (in B-5) and 25 feet elevation (in B-7), indicating that some degree of offset-faulting of water-bearing zones and/or local fracture zone drainage likely exists.

The packer testing in boreholes B-10 and B-11 identified a relatively transmissive zone (evidenced by pressure test flows between 2 and 12 gpm) between elevations 6.5 and 20 feet, which is below the inferred bentonite confining layer. A moderately transmissive zone (evidenced by pressure test flows between 1 and 2 gpm) exists in both boreholes above the inferred bentonite layer.

7.0 LOCAL GEOLOGY

7.1 Geologic Mapping

A Shannon & Wilson engineering geologist performed geologic mapping along the slopes east of the 2011 and 2009 Landslides, and on the beach below for this study. We also collected additional data from bedrock exposures in the intertidal surf area, and on the slope area between the slope crest and the beach. The geologic data collected is included in the Site and Exploration Plan (Plate 1) along with data collected from our original study.

7.2 Geologic Structure

The interpreted geologic structure in the area east of the landslide (near borings B-7, B-10 and B-11) is shown on the Generalized Subsurface Profiles, Plates 3 through 8, which include our interpretations of surface mapping and subsurface exploration data. A legend for the profiles is shown on Plate 9. Descriptions of the geologic units are provided in our Final Report. The
Generalized Subsurface Profiles H-H’ through L-L’ show the orientations of sedimentary bedding and fracture surfaces. We measured these rock discontinuities during our original study and current field mapping and using boring televiwer surveys. Bedding and discontinuity characteristics such as attitude, filling, roughness, and type were obtained using several methods. Appendix E presents the boring televiwer surveys and describes the methods used to calculate discontinuity orientations. Appendix C summarizes bedding and fracture data on each boring.

Stereographic projections of the fractures and bedding attitudes observed in borings B-1 through B-5, B-7, B-10, and B-11 are shown on Plate 10. These projections are presented as poles to planes. An illustration and description of the presentation method (equal-angle stereographic projection) is included on the plate.

We also developed structural contours of a particular sedimentary layer (marker bed) identified using natural gamma surveys displayed in Appendix E. The natural gamma surveys are made by lowering an instrument into each borehole and measuring the intensity of natural gamma radiation. Natural gamma radiation in rock typically increases with increasing clay content. The specific marker bed identified using the gamma surveys that was used to develop the structure contours on Plate 10 is designated with an arrow on each gamma log. While several groups of high and low gamma radiation were correlated between borings at the site, the correlated marker bed (and interpreted structural contours) selected for illustration was chosen because it is close in elevation to the interpreted 2011 Landslide failure surface.

8.0 STABILITY ANALYSES

The stability of natural slopes is a complex, three-dimensional relationship between the driving forces of soil and rock mass, groundwater, and external forces such as earthquakes and surcharge loading, and the resistance offered by the strength of the soil or rock. When performing slope stability analyses, the complexities inherent in real slopes must be simplified to perform the analyses. Accordingly, the results of slope stability analyses describe in this report are approximate. The slope stability analyses for this project should be used to understand changes in stability for alternative improvements being considered. The primary goal of the stability analyses was to evaluate the stability of the eastern coastal bluff that remains following the 2011 Landslide to:

- Estimate the potential for further regression that could affect adjacent, existing development and infrastructure; and
- Provide recommendations to improve slope stability through dewatering.
We used the computer program SLOPE/W version 7.17 (Geo-Slope International, 2007) to perform two-dimensional, limit equilibrium stability analyses of potential future landslides in the area east of the 2011 Landslide. The stability analyses were in addition to the analyses completed in our original Final Report. The following sections describe the assumptions we made to model the landslide, including the geology of the site, the geometry of the landslide, the model input parameters and results of the analyses.

We developed our stability models based on the geologic mapping, extrapolation of the subsurface data from our borings around the landslide, and the geologic and hydrogeologic conclusions describe in this Addendum and in our Final Report.

8.1 Geology

Geology typically plays a critical role in coastal landslides. For the analyses, we assumed that bedding dips out of the slope as shown in Plates 5 through 10, and that bentonite clay is present on the failure surface. We assumed potential new landslides would fail along the same assumed bentonite surface as the 2011 Landslide.

We assumed that vertical or near-vertical rock discontinuities exist near the landslide headscarp, which is consistent with our observations of the 2011 Landslide, and that unstable conditions would occur when the water level in these discontinuities is approximately half-way between the ground surface and the failure surface. The stereonets for borings B-7, B-10 and B-11 (Plate 10) east of the 2011 Landslide show conjugate oblique joints trending northwest and northeast and/or faults trending northeast that could form nearly vertical planes of weakness near the head of a potential future landslide. Thus, we only considered cases where the discontinuities extend vertically from the ground surface to the failure plane.

8.2 Hydrogeology

Our groundwater data demonstrates a complex system of confined and unconfined zones. During drilling, the groundwater was observed at a lower elevation than what was recorded by the VWPs after drilling. Two VWPs, one in B-10 and one in B-11, are located at an elevation within the potential failure plane. The piezometric head recorded at those two locations suggests the groundwater could fill 30 percent of a connecting tension crack. To capture the effect of elevated porewater pressure acting on the failure plane, we modeled one piezometric surface representing an unconfined zone condition, similarly to our back analysis on cross section C-C’ in our Final Report.
8.3 Analyses and Results

For the analyses, we defined the surface geometry based on the 2011 survey contours for the after-sliding conditions. We interpreted the subsurface geometry and geology as described previously. For our stability analyses, we used the cross sections K-K’ and L-L’ shown on Plates 8 and 9.

Properties of the geomaterials used in the slope stability analyses are presented in Table 3, below, and also discussed in Appendix I. We used the mean value of index properties for soil as discussed in our Final Report. Input parameters for rock were modified based on our findings in borings B-10 and B-11 and subsequent geotechnical testing of samples. For materials that could not be adequately characterized by laboratory testing performed at discrete sampling intervals, and in the case of qualitative rock properties such as geologic strength index (GSI), an engineering geologist estimated the parameters needed for slope stability analyses. Shear strength parameters are further discussed below.

| Table 3: Material Properties Used in Slope Stability Analyses |
|---------------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|
| Strength Model       | Total Unit Weight | Friction Angle | Cohesion | Uniaxial Compressive Strength | Geologic Strength Index | Intact Rock Parameter, $m_i$, | Disturbance Factor, $D$ |
| Terrace Deposits (Qt) | Mohr-Coulomb     | 103             | 34      | 0   | NA | NA | NA |
| Altamira Shale (Tma) | Hoek-Brown$^5$  | 118             | See Note 5 | See Note 5 | 740 | 35 | 7 | 0 |
| Weathered Tuff (Bentonite Clay; Tma) | User-Defined Nonlinear Function$^6$ | 118             | See Note 6 | See Note 6 | NA | NA | NA |

Notes:
1. pcf = Pounds per cubic foot
2. psf = Pounds per square foot
3. psi = Pounds per square inch
4. As described by Hoek and Marinos (2000).
5. The generalized Hoek and Brown (1997) model is nonlinear and does not correspond to single values of friction angle and cohesion; the full nonlinear curve is presented in Appendix I.
6. The shear strength envelope used to model the bentonite clay is nonlinear and does not correspond to single values of friction angle and cohesion. The full nonlinear curve is presented in Appendix I.
As discussed in our Final Report, we found that the stability of the slope was strongly influenced by the shear strength of the bentonite clay layers, and that it was appropriate to assign residual shear strength values based on the results of our ring shear tests. As before, we used a nonlinear shear strength envelope to model rock in the slope, as described by the Generalized Hoek-Brown Strength Criterion (Hoek and Brown, 1997; Hoek and Marinos, 2000). This nonlinear model is discussed further in Appendix I.

We performed forward analyses for cross section L-L’, located east of the 2011 Landslide, to assess current stability and the effectiveness of dewatering to improve stability. Cross section L-L’ is oriented approximately perpendicular to the slope east of the 2011 Landslide, and passes through the approximate center of boring B-10. Cross section K-K’ is oriented approximately parallel to the true dip direction of the bedding, east of the 2011 Landslide, and passes through the approximate center of boring B-11. The results of the stability analysis and direction of the bedding suggest a failure along section L-L’ is slightly more critical than a failure along section K-K’. Therefore, we consider the results of the L-L’ analyses to be more representative of the potential for future landsliding east of the 2011 Landslide.

8.3.1 Forward Analyses

Using the geology, hydrogeology, geometry, and material property assumptions described above, we developed the stability models shown in Appendix I. We evaluated the stability of potential future failure surfaces. Because discontinuities partially filled with groundwater were shown to cause unstable conditions, we included similar hydrogeologic assumptions that were used in the back analyses for cross section C-C’ (see our Final Report), but adjusted for new groundwater data obtained in recently completed borings B-10 and B-11. We assumed that vertical or near-vertical discontinuities exist (i.e., they are structural geologic features caused by regional faulting and folding), and they extend over the entire extent of the model.

The term “tension crack” is often used in slope stability modeling to describe the presence of near-vertical discontinuities near the head of a landslide that form as a result of slope movement. However, as discussed in our Final Report, it is our opinion the discontinuities existed prior to the initial landslide. Therefore, they are not a result of tension forces that developed concurrently with the landslide, but are related to regional and local stresses.

We performed forward analysis for two conditions: static and seismic loading. We considered a number of groundwater levels in discontinuities, similar to the back analyses. For
static loading (i.e., not including vertical or horizontal accelerations caused by earthquakes), the results indicate that the eastern slope is currently stable given groundwater conditions to date. However, if the groundwater level in the discontinuities exceeds approximately 50 percent of the discontinuity height (i.e., an elevation MSL of approximately 75 feet), we found that the slope would be unstable under static conditions and could undergo a failure similar in nature to the 2011 Landslide. The highest groundwater piezometric surface that we have observed within the potential failure plane along the eastern flank is 62 feet elevation at B-11, or approximately 15 feet below the level found to cause instability for static conditions. Therefore, we conclude that severe hydrogeologic conditions that would raise the current piezometric surfaces could trigger new landsliding east of the 2011 Landslide.

We performed seismic analyses by representing ground accelerations as pseudo-static forces according to “Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California” (Blake and others, 2002). We assumed a level of ground shaking having a 10 percent probability of exceedance in 50 years. The analyses and design seismic parameters are described in Appendix I. We used a peak ground acceleration of 0.17 g (g is the acceleration due to gravity) corresponding to an earthquake with a moment magnitude ($M_w$) of 7.2 at a distance of 6.0 kilometers. Our analyses suggest that future landslides are likely during a design earthquake. The results of the stability analyses, including seismic loading, are shown in Table 4.

**TABLE 4**

<table>
<thead>
<tr>
<th>Cross Section</th>
<th>Water Level (% of Slope Height)</th>
<th>Static Factor of Safety (FS)</th>
<th>Seismic Factor of Safety (FS)</th>
<th>Water Elevation at B-11, ft.</th>
</tr>
</thead>
<tbody>
<tr>
<td>K-K'</td>
<td>0%</td>
<td>1.78</td>
<td></td>
<td>28.0</td>
</tr>
<tr>
<td></td>
<td>30%</td>
<td>1.47</td>
<td></td>
<td>56.1</td>
</tr>
<tr>
<td></td>
<td>40%</td>
<td>1.40</td>
<td></td>
<td>65.4</td>
</tr>
<tr>
<td></td>
<td>50%</td>
<td>1.32</td>
<td></td>
<td>74.8</td>
</tr>
<tr>
<td></td>
<td>60%</td>
<td>1.18</td>
<td></td>
<td>84.1</td>
</tr>
<tr>
<td></td>
<td>70%</td>
<td>0.95</td>
<td></td>
<td>93.5</td>
</tr>
<tr>
<td>L-L'</td>
<td>0%</td>
<td>1.48</td>
<td>0.60</td>
<td>28.0</td>
</tr>
<tr>
<td></td>
<td>30%</td>
<td>1.30</td>
<td>0.55</td>
<td>56.1</td>
</tr>
<tr>
<td></td>
<td>40%</td>
<td>1.26</td>
<td>0.54</td>
<td>65.4</td>
</tr>
<tr>
<td></td>
<td>50%</td>
<td>1.04</td>
<td>0.53</td>
<td>74.8</td>
</tr>
<tr>
<td></td>
<td>60%</td>
<td>0.84</td>
<td>0.53</td>
<td>84.1</td>
</tr>
</tbody>
</table>

Notes:
1. Given the applied groundwater conditions from C-C', the water in the tension cracks are at 30% of the tension crack height.
2. See Plates 8 and 9 for Cross Section K-K' and L-L' in Appendix I.
For Table 4, the FS is defined as the ratio of resisting forces to driving forces, or resisting moments to driving moments, whichever is more critical. For limit equilibrium slope stability analyses, a factor of safety (FS) less than 1.0 has no physical meaning because instability would have occurred prior to the slope reaching the modeled conditions (i.e., the inherent assumption of static equilibrium has been violated). Where FS values presented in Table 4 are less than one, the physical interpretation is that a landslide would have occurred prior to the indicated slope conditions being reached.

Our stability analysis for cross section L-L’ indicates that the area east of the 2011 Landslide is marginally stable at the current piezometric surface or groundwater levels, but it may become unstable during severe hydrogeologic conditions or during seismic activity. The inclinometer at borings B-1, B-5, B-9, B-10, and B-11 have not shown discernable movement since the 2011 Landslide to date. However, the boring B-7 inclinometer has detected ground movement at elevations between 59.5 and 61.5 feet MSL (61.1 and 63.1 feet depth) at a rate of about 0.1 inches per year. In our opinion, without implementation of dewatering, precipitation and infiltration could accelerate this movement. Further, we believe that continued slow movement could reduce the strength of bentonite clay that likely is present along the inferred failure surface.

9.0 DEWATERING EVALUATION

We developed a predictive groundwater flow model of the area to evaluate dewatering and groundwater control options that could reduce the risk of future failure east of the 2011 Landslide. We based the groundwater flow model on the existing conceptual understanding of the local geology and hydrogeology, described earlier. Appendix J describes the groundwater flow model, its development and presents the dewatering modeling evaluation results.

9.1 Groundwater Model Development

We used the U.S. Geological Survey’s modeling code MODFLOW-NWT (Niswonger, 2011) to simulate groundwater flow and seepage in the landslide area. We used the software program GMS version 8.2 (Aquaveo, 2012) to facilitate model development and calibration, and perform dewatering simulations. The model domain covers 1,300 feet by 1,300 feet, roughly centered on the exploratory borings B-10 and B-11 (Appendix J, Figure J-1).

The upper model surface is defined by existing land surface (using LiDAR data). The base of the model is uniformly set at elevation -60 feet. The model computational mesh uses 5 feet by 5 feet cells in plan view (Figure J-2). The model uses 12 discrete layers to represent the
hydrostratigraphy. Layer thicknesses range from 5 feet (the constant thickness for layers 3 through 10, inclusive), to 140 feet (in layer 1 along the inland boundary). Layers 3 through 12 extend offshore, whereas layers 1 and 2 are truncated along the existing bluff line. Where layers are offshore, the uppermost active offshore layer (layer 3) has a surface elevation of zero. Table 5 below summarizes the model layers and the assigned hydraulic properties.

### Table 5
SUMMARY OF MODELED LAYERING AND HYDRAULIC PROPERTIES

<table>
<thead>
<tr>
<th>Model Layer(s)</th>
<th>Description</th>
<th>Layer Thickness (feet)</th>
<th>Kh, Kv (ft/day)</th>
<th>Sy (-), Ss (ft⁻¹)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>up to 140(1)</td>
<td>2.5, 0.5</td>
<td>0.05, 5e⁻⁵</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Moderate permeable zone</td>
<td>5</td>
<td>2.5, 0.5</td>
<td></td>
</tr>
<tr>
<td>3, 4 &amp; 5</td>
<td>Moderate permeable zone</td>
<td>5</td>
<td>5, 0.5</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>Bentonite clay layer</td>
<td>5</td>
<td>1e⁻⁴, 1e⁻⁵</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>Low permeable zone</td>
<td>5</td>
<td>1, 0.1</td>
<td>0.01, 5e⁻⁸</td>
</tr>
<tr>
<td>8 &amp; 9</td>
<td>High permeable zone</td>
<td>5</td>
<td>10, 1</td>
<td>0.05, 5e⁻³</td>
</tr>
<tr>
<td>10</td>
<td>Moderate permeable zone</td>
<td>5</td>
<td>5, 0.5</td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>Low permeable zone</td>
<td>10</td>
<td>1, 0.1</td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>Low permeable zone</td>
<td>10 to 95</td>
<td>1, 0.1</td>
<td></td>
</tr>
</tbody>
</table>

Notes:
1. This unit is not fully saturated. Kh, Kv – horizontal and vertical hydraulic conductivity; Ss, Sy – specific storage and specific yield (effective porosity).

The field explorations and testing identified the subsurface as a complex sequence of variably fractured and decomposed siltstone, sandstone and shale units. Groundwater flow occurs primarily in the secondary porosity (within fractures and between units) rather than in any primary porosity (which is expected to be low).

We assigned constant head to the upgradient (inland) boundary, ranging from elevation 160 feet in the upper layers to elevation 130 feet in the lower layers. We assigned an offshore (ocean) constant head of 5 feet to the upper surface of layer 3. The upgradient boundary represents groundwater inflow, whereas the down-gradient boundary heads represents groundwater discharge to the ocean. We used a series of head-dependent boundaries (MODFLOW’s “Drain” cells) to simulate the discharge of groundwater on the slope face where the water table in the uppermost layer intercepts land surface.

We assigned a uniform groundwater recharge rate of 2 inches per year (in/yr) to the surface of the uppermost active layer for onshore cells. This rate equates approximately 14 percent of the average annual precipitation for the area (14 inches). However, some of this recharge also represents infiltration of irrigation water applied towards the inland part of the model area.
9.2 Simulated Baseline Conditions

Figures J-3 and J-4 show the simulated static piezometric head in plan view for model layer 4, and a cross section southwest-northeast through the middle of the model. Table 6 summarizes the results of the static calibration using typical measured piezometric heads from the wells and piezometers. The observed “target” piezometric heads are typical measurements for each well and VWP during the period July to October 2012. The following summarizes the results:

- At the three B-11 VWPs, the model reproduces the observed piezometric heads reasonably well, with the target-simulated level differences between 2 and 7.5 feet.
- At the two shallow VWPs at B-10, the modeled piezometric levels are 13 and 15 feet higher than the observed levels. In terms of establishing a baseline condition on which to superimpose a dewatering system, this is a relatively conservative result. The model significantly underpredicts the anomalously high piezometric head in the deepest VWP at B-10.
- In the remaining four wells and VWPs (B-3, B-7, B-8 and B-9), the model also simulates a higher set of piezometric heads than those observed. This indicates the limitation of using uniform, homogeneous layers to represent the complex hydrostratigraphy. The observed piezometric heads in these wells and VWPs are markedly lower than in B-10 and B-11 at similar depths. As the focus on the dewatering is the area centered on B-10 and B-11, we do not consider the model’s inaccuracy elsewhere to be significant.

### Table 6

SUMMARY OF STATIC CALIBRATION MODEL RESULTS

<table>
<thead>
<tr>
<th>Boring/VWP</th>
<th>VWP/Screen Elevation (ft elev)</th>
<th>Assigned Model Layer</th>
<th>Target Head (ft elev)</th>
<th>Simulated head (ft elev)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-3 (well)</td>
<td>14.1</td>
<td>10</td>
<td>25</td>
<td>69</td>
</tr>
<tr>
<td>B-5 (VWP)</td>
<td>6.4</td>
<td>10</td>
<td>62.5</td>
<td>71.4</td>
</tr>
<tr>
<td>B-7 (VWP)</td>
<td>7.3</td>
<td>10</td>
<td>24</td>
<td>61</td>
</tr>
<tr>
<td>B-8 (well)</td>
<td>27.1</td>
<td>9</td>
<td>50</td>
<td>76</td>
</tr>
<tr>
<td>B-9 (VWP)</td>
<td>14.0</td>
<td>11</td>
<td>35</td>
<td>82</td>
</tr>
<tr>
<td>B-10 vwp1</td>
<td>32.5</td>
<td>5</td>
<td>45</td>
<td>60</td>
</tr>
<tr>
<td>B-10 vwp2</td>
<td>12.5</td>
<td>9</td>
<td>50</td>
<td>63</td>
</tr>
<tr>
<td>B-10 vwp3</td>
<td>-7.5</td>
<td>12</td>
<td>100</td>
<td>68</td>
</tr>
<tr>
<td>B-11 vwp1</td>
<td>40.9</td>
<td>3</td>
<td>60</td>
<td>53</td>
</tr>
<tr>
<td>B-11 vwp2</td>
<td>10.9</td>
<td>8</td>
<td>55</td>
<td>59</td>
</tr>
<tr>
<td>B-11 vwp3</td>
<td>9.1</td>
<td>11</td>
<td>52.5</td>
<td>60</td>
</tr>
</tbody>
</table>

Notes:
1. Mid-well screen and VWP tip elevation.
9.3 Dynamic Calibration

To calibrate the model dynamic (transient) conditions, we simulated the third packer test performed in boring B-11 at depths between 102 to 115.5 feet on July 23, 2012. This involved matching the observed response in VWPs in boring B-10 during the test. During the test, the pressure at boring B-11 was increased by 22 pounds per square inch (psi), or 51 feet of water head for 10 minutes. The measured piezometric head increase in borings B-10 VWP2 and VWP3 were about 1.7 feet and 0.7 feet, respectively. There was no apparent head increase in the shallower VWP1 (Figure J-5).

Figure J-6 shows the modeled response in boring B-10 VWP1 and VWP2s. The model showed a peak head increase of about 3.5 feet resulted in B-10 VWP2, and no head increase in VWP1 or VWP2. Although the modeled head increase is larger than that observed, the VWP recording frequency during the test was hourly. Therefore, the actual head increase in B-10 VWP2 likely was greater than 1.6 feet. The model’s inability to reproduce the observed piezometric head increase in the deepest VWP3 indicates that the simulated connectivity between the units below the bentonite layer may be too low.

9.4 Dewatering Simulation

The objective of the dewatering simulation was to evaluate the number, spacing and geometry of drains that will be needed to reduce and maintain a lower piezometric head to improve slope stability. We considered drain spacing ranging from about 20 to 50 feet.

The simulation involved using the MODFLOW Drain function to represent parallel drains that would extend from the near access road (near B-8) to the bluff slope toe. The Drain function is a head-dependent sink that removes groundwater from the model at a rate that is calculated using (a) the hydraulic gradient in the cells surrounding the Drain cell, and (b) the conductance of the cell. Therefore, a higher discharge rate will occur for a Drain cell with a relatively steep local hydraulic gradient and a high conductance. We assigned the Drain elevation at one foot above the base of the cell, and calculated the Drain conductance based on the cell hydraulic conductivity. The simulated drains pass through model layers 1 (at the water table, elevation 83 feet) to layer 4 (below the toe of the slope, at elevation 7.5 feet). No other changes were made to the Baseline version of the model described above.

We ran the model to both steady-state and transient conditions (for one year). Figures J-7, J-8 and J-9 show the results for long-term (steady-state) piezometric head in model layers 1, 4 and 7 with the dewatering drains installed 20 feet apart and operating at 100 percent efficiency. The
water table falls below the base of layer 1 (and layer 2) across an area approximately 150 feet wide at the edge of the bluff, and approximately 300 feet wide in layer 3 (base elevation of 26 feet). Figure J-10 shows the resulting piezometric head and water table surface in section view through the middle of the drain array. The water table drops below the bluff face. Figures J-11, J-12 and J-13 show how the piezometric levels in hypothetical monitoring wells located at the bluff toe and the bluff edge, and at well B-11 would decline with time once the drains have been installed. The piezometric head responds relatively quickly, and achieves a near-steady state condition near the bluff and outfall in layer 4 after about 14 days. The predicted steady-state discharge from the 11 drains ranges between 25 and 35 gpm, or 2.3 to 3.2 gpm per drain.

**10.0 RECOMMENDATIONS**

In Section 9.2 of our Final Report, we recommended three immediate stability improvements of the 2011 Landslide and vicinity, in particular the eastern flank:

- Dewatering with directional drains,
- Cleaning and shaping of the landslide mass, and
- Slope anchor system.

The recommendations provided in this section are for the dewatering with directional drains. We previously provided recommendations for cleaning and shaping of the landslide mass in our letter dated October 9, 2012. Our geotechnical recommendations for the slope anchor system will be issued separately.

Considering the results of the stability and dewatering analyses above, we recommend immediate repair work include construction of the dewatering system described in the following sections to reduce the potential for future landsliding east of the 2009 and 2011 Landslide. Our recent observations and analyses for the slope conditions east of the 2011 Landslide indicate the slope could become unstable during severe hydrogeologic conditions.

Future landslide movements, triggered by high groundwater conditions, could occur during heavy rainfall; as a result of excessive irrigation; and/or from design-level seismic events. Our recommendations for dewatering are discussed in the following section.

**10.1 Dewatering Recommendations**

Based on the groundwater flow modeling, we recommend installing arrays of parallel gravity-fed drains to intercept groundwater and reduce porewater pressure. The drains should be spaced...
about 20 feet apart, in an upper and lower configuration. The drains could be constructed using horizontal directional drilling (HDD), which would allow most drilling operations to occur north of Paseo del Mar. The HDD drains should target the higher permeable zones identified by the boreholes above the bentonite clay zone, and daylight near the toe of the existing coastal bluff.

The expected long-term flow rate from the cumulative drains (maximum of 22) is between 25 and 35 gpm. Initial discharge rates could be as high as 100 gpm. Due to the heterogeneity of the bedrock permeability and fracture pattern, the individual drains will experience differing discharge rates, and some drains may discharge virtually no groundwater. As a collective sink, the drain array should develop a groundwater capture zone up to 300 feet wide at the top of the bluff slope.

10.2 Horizontal Direction Drilling (HDD)

We recommend the drains proposed in the above section be installed using HDD techniques. HDD is a trenchless excavation method consisting of drilling a small-diameter pilot hole along a designed path. Depending on the contactors means and methods, the pilot hole might be enlarged by reaming to a diameter suitable for pipe installation. HDD is a specialized drilling technique, which is capable of steering the drilling head and producing a curved drilling alignment.

10.2.1 Drainage Alignment

We selected the drain locations shown in Plate 1 to satisfy drainage criteria previously presented. Two drains will be installed at each location in an upper and lower configuration as shown in Plate 11 and Plate 12. The upper drain alignment is intended to alleviate perched groundwater conditions and to increase the probability of intersecting fractures. The lower drain alignment is intended to lower the piezometric head in confined zones by 15 to 20 feet to reduce groundwater pressure on critical potential landslide failure surfaces.

The actual design path should be the responsibility of the selected contractor. The contract documents provide required tolerances to the alignment shown in Plate 11 and Plate 12 for use by the Contractor in developing the design drill path. The documents also limit HDD launch, retrieval, and contractor work to designated construction zones.
10.2.2 Method

The HDD method requires two staging areas. A staging area at the top of the slope would be required for the drilling machine and a mud pit. This staging area will typically be 150 feet long by 100 feet wide for a single HDD bore. Because the drainage plan will require multiple HDD bores, the staging area may be as large as 150 feet long by 200 feet wide. A temporary pipe assembly area will also be required. Generally, contractors prefer an area with a length equal to the total drill length, with a width of approximately 12 to 15 feet to lay out and pre-assemble the pipe for rapid installation as soon as the hole is drilled. However, if an area with a length equal to the length of the hole is not available, the pipe assembly area can be as short as one-third the length of the hole. Ramps and rollers can be used to guide the pipe over driveways and other obstructions in the laydown area.

The first step involves drilling a pilot hole along a designed path from the top of the slope to the exit point at the toe of the slope. The pilot hole is typically less than 1 foot in diameter and follows the design centerline of the proposed pipeline, within a specified horizontal and vertical tolerance. The slope of a typical design centerline, or drill path, starts at about 8 to 10 degrees below horizontal, uses large-radius arcs to achieve the desired line and grade. The exit angle typically is between 5 to 12 degrees. The allowable radius of a compound horizontal and vertical curve for the drill path is controlled by the diameter of the drill steel or by the diameter and type of produce pipe. A longer radius will facilitate installation and lower pullback loads. For design purposes, we recommend assuming the minimum radius of curvature a contractor may select is 600 feet.

The pilot hole is drilled using an HDD drill rig that pushes the directional drill bit and drill rods into the ground. Drilling is advance by either jetting a hole using high pressure drill fluid or by drilling the hole with a mud motor powered by high-pressure drill fluid. Steering is accomplished by aiming the jets at one quadrant of the bore to cut a hole in the specific direction or by aiming the mud motor drill bit toward the desired path. Mud motors have a bend in the housing that permits the drill bit to be directed at a pre-specified angle from the drill rod centerline path. The drill rods are then pushed in to the hole and the rods follow the desired direction of the drill bit.

Drilling fluid is pumped down the center of the drill rods. The fluid acts (a) as a coolant, (b) as a counter-acting fluid pressure that helps maintain the hole open, (c) as a transporting fluid to carry soil cuttings back to the surface, and (d) to form a “cake” around the borehole to help stabilize the opening. The fluid returns along the drilled path outside of the drill rods to the
ground surface to a collection area, located at the top of the slope. A fluid separation plane at the surface filters out the soil cuttings and re-circulates the bentonite mud back into the drill rods. The drilling fluid should consist of air, water, or polymer mixtures. **Bentonite is not a viable alternative for this project as it would tend to cake the soil around the borehole and limit the effectiveness of the drains.**

A directional monitoring device located behind the drill bit that registers angle, rotation, and direction (azimuth) measures the position of the pilot hole. Several systems are available to transmit and detect this data, including walkover, wireline, and downhole systems.

In general, HDD is most successful in soft or silty and clayey soil, and soft rock, all of which form relatively stable boreholes. In non-cohesive soil, such as sand and gravel that includes cobbles and boulders, HDD projects have experienced problems with caving holes and jammed and lost drill bits and rods. These holes usually require addition time and preparation procedures, such as pulling hole compactors through the completed bore. While the drilling fluid generally establish a more capable “cake” of mud-impregnated soil around the periphery of the drilled hole in silt and sand; gravel layers may be too coarse to accommodate and develop a stabilizing “cake,” particularly where groundwater is flowing into the drill hole.

Upon reaching the exit point, the drill bit is removed and a reaming tool is attached to the drill string if needed to progressively enlarge the hole. The drill rig rotates and pulls the reamer back into the bore to increase the borehole size, while the slurry circulation system is used to remove the soil cuttings. Tail rods are attached to the drill string and follow the reamer into the bore. If multiple reams are required, a larger reaming tool is attached to the tail rods, and a second reaming pass is made. The cycle is repeated until the bore reaches the design size. To reduce skin friction during pull-back of the produce pipe, the design bore size is typically larger than the product pipe. The Contractor should choose a final bore diameter that provides the best likelihood for successfully installing the product pipe, weighting the lower friction of the larger-diameter bore against the increased risks associated with constructing and maintaining an open and stable large-diameter bore.

Once the bore has been prepared and a hole opener has successful been pulled through the bore to verify that the hole is open and stable, the product pipe is attached to a pulling assembly at the pilot bore exit point and pulled into the hole. It is desirable to pull the product pipe in one continuous operation or in as few sections as possible to reduce friction in the borehole and the risk of caving ground, which could jam the pipe during the pulling process. The slurry used during drilling also acts to lubricate the product pipe during pull-back. The
Contractor must also monitor the integrity of the product pipe by maintaining a safe pullback fore to prevent tensile failure of the pipe. Pipe installation involves a rapid and continuous pull to reduce the risk of collapsing the hole.

10.2.3 Hydraulic Fracturing and Fluid Release

Hydraulic fracturing, or “frac-out,” is likely to occur when slurry pressure in excess of the total stress in the ground is applied to the walls of the bore. Fractures in the soil or rock can conduct slurry into the environment surrounding the borehole. Fractures extending upward toward the ground surface may cause a loss of slurry to the ground surface (inadvertent drilling fluid release). Frac-out should generally be expected to occur through highly permeable soil such as gravel; jointed or fractured clay; disturbed or loose soil such as fill; in areas with relatively shallow cover; and along sedimentary contacts where there is significant variation in density, such as between soil and rock. The typical areas where frac-outs and inadvertent drilling fluid release would likely occur include the entry, where loose colluvium may exist, or near the exit where landslide debris will be encountered and overburden depth is small. Frac-out may also occur near the HDD exit where there is a significant build-up in pressure due to elevation difference between the entry or exit points.

Contractors often excavate a pit and/or construct straw bale and sandbag dams with the surface return pipes and pumps at or downhill to collect and reuse the slurry from localized frac-outs. Alternatively, they may stage-grout a leaking hole or install conductor sleeves or casing to prevent frac-outs, caving holes, and erosion of sand and silt.

In general, due to the overburden and moderate rock strength in this area, we do not anticipate major frac-outs until near the exit point. Small frac-out volumes may occur along existing fractures in the rock. Because of the elevation difference between the entry and exit points is about 110 to 120 feet, significant frac-out may occur near the exit if the contractor does not take appropriate steps to mitigate the risk. The contract documents should require the Contractor to construct containment on the beach to contain frac-out, and to submit an exit plan to contain drilling mud during HDD break out.

11.0 LIMITATIONS

This report was prepared for the exclusive use of the City of Los Angeles for specific application to this project.
The analyses, conclusions, and recommendations contained in this report are based on site conditions as they presently exist and are subject to change as our services on this project progresses. We assume that the explorations made for this project are representative of the subsurface conditions throughout the project area (i.e., the subsurface conditions everywhere at the site are not significantly different from those disclosed by the explorations).

Within the limitations of the scope, schedule, and budget, the analyses, conclusions, and recommendations presented in this report were prepared in accordance with generally accepted professional geotechnical engineering principles and practices in this area at the time this report was prepared. We make no other warranty, either express or implied. These conclusions and recommendations were based on our understanding of the project as described in this report and the site conditions as interpreted from the current explorations.

Shannon & Wilson, Inc. has prepared the document, “Important Information About Your Geotechnical/Environmental Report,” in Appendix K to assist you and others in understanding the use and limitations of this report.

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