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1.0 INTRODUCTION

This report presents geotechnical and geological data and recommendations for the White Point Landslide in the San Pedro District of Los Angeles. The site is located as shown on the Vicinity Map, Figure 1. This report summarizes our field explorations, laboratory testing, geotechnical analyses, conclusions, and recommendations. With the exception of the Daily Field Activity Reports (FARs) and the Health and Safety Plan, this final geotechnical report supersedes our Preliminary Geotechnical Report, White Point Landslide, San Pedro District, Los Angeles, California, W.O. E1907483, dated January 6, 2012. Daily FARs and the Site Specific Health and Safety Plan are included in Appendices A and B of the Preliminary Geotechnical Report (Preliminary Report), respectively, and are not included in this report.

In this report, the name used for the project area is White Point, rather than Whites Point. While previous studies, maps, and reports have used both the singular and plural names, the name White Point is used for consistency with the adjacent White Point Nature Preserve (WPNP). Titles of previous work have not been changed in the references in this report.

1.1 Site and Landslide Description

The Palos Verdes Peninsula is well known for its large landslides due to geologic conditions that are conducive to slope instability. Those conditions include geologic structure and bedrock type, specifically the tuffaceous Altamira Shale Member of the Monterey Formation. The White Point Landslide is located along the south-facing shoreline of the Palos Verdes Peninsula, adjacent to the south edge of the WPNP, as shown in the Site and Exploration Plan, Plate 1, and the Geologic Map, Figure 2.

The WPNP consists of 102 acres that are delineated by Western Avenue to the west, Paseo del Mar to the south, Weymouth Avenue to the east, and the Los Angeles Air Force Base housing to the north. The WPNP also includes the remains of an abandoned Nike Missile base. In the southwest corner of the property, there is an approximately 1/4 acre parcel which belongs to the City Sanitation District of Los Angeles County and is used as a sanitary pump station.
Paseo del Mar is an east-west roadway at the top of a steep, approximately 120-foot-high, south-facing bluff overlooking the Pacific Ocean. The two-lane street is approximately 48 feet wide and is paved with asphaltic concrete. There are 4-foot and 9-foot-wide bike paths adjacent to the northern and southern sides of the street, respectively.

Reportedly, indications of movement consistent with a landslide were first noted by City of Los Angeles (City) representatives in June 2011, although indications of fresh ground cracks in the pavement of Paseo del Mar were observed in January 2010. These cracks became wider and more extensive, and were accompanied by minor vertical displacement of the roadway surface and adjacent areas in June 2011. Movement continued toward the beach over a period of five months. On November 20, 2011, a large section of the sea cliff reportedly failed in about 20 minutes, destroying an approximately 600-foot-long section of Paseo del Mar and associated utilities. It left a large, isolated block of roadway to the south of the upland terrace separated by a down-dropped graben. The block moved approximately 60 feet to the south as shown in Photograph 1 below (block is shown with detached pavement section). As described in subsequent sections of this report, there is another, earlier landslides in the immediate site vicinity; therefore, we refer to the November 20, 2011 event as the “2011 Landslide” in this report.

1.2 Landslide Types and Nomenclature

Many types of mass movement are referred to as landslides; however, in this report, we define a landslide as the movement of earth material along a distinct zone of weakness that separates the mass that has moved, from the more stable, underlying material. The following section defines the type of mass movement occurring at White Point. The nomenclature is taken from several sources, including USGS Fact Sheet 2004-3072 (2004), and Scullin (1986).

Landslide movement typically is either rotational or translational. Rotational landslides have a bowl-shaped, or concave-upward slide surface where the landslide mass rotates about an axis. Translational landslides have a roughly planar slide surface separating the landslide mass from the relatively stable material below. While the movement of a translational landslide is generally planar, blocks within the landslide mass commonly deform, including back-tilting and rotating. A subset of the translational landslide is the block landslide where the landslide mass consists of a single unit or a few closely related units that move downslope as a relatively coherent mass. The schematic block diagram in Figure 3 illustrates the general mode of mass movement observed at the White Point site and includes landslide terminology used in this report.

1.3 Scope of Services

The purpose of our geotechnical engineering services was to evaluate the landslide geometry and movement to assess risk of future landsliding, and develop repair options to restore Paseo del Mar. Our scope of services is based on the Task Order Solicitation (TOS) No. 11-087, dated November 16, 2011, and our proposal dated December 12, 2011. In accordance with our scope of services, this report presents data, conclusions and recommendations as described in our proposal and the TOS. This report includes:

- Literature research,
- Geologic mapping and air photo review results,
- Groundwater review,
- Historical activities at the site,
- Boring logs, borehole instrumentation construction logs, borehole geophysical logs,
- Instrumentation monitoring results to date,
- Geotechnical and analytical laboratory test results,
- Geologic and subsurface conditions with geologic cross sections,
- Stability analyses,
- Potential factors contributing to slope displacement, and
- Conclusions and recommendations.
Additional geotechnical engineering services requested by the City that were not part of the TOS or our proposal include:

- Purchasing and delivering survey markers (proposal addendum dated January 26, 2012),
- Review of the Nike Missile site (proposal addendum dated February 6, 2012),
- Animation of the landslide and conceptual repair options (proposal addendum dated March 20, 2012), and
- Civil engineering feasibility review of conceptual options (proposal addendum dated April 10, 2012).

We issued a draft of this report to the City on June 18, 2012. We responded to comments from the City and public on the draft report in our letter dated August 8, 2012. These comments were resolved in the letter and did not necessitate changes to our draft report.

2.0 RESEARCH DATA AND FIELD MAPPING

2.1 Literature Review

The literature reviewed for this study is presented in the reference section at the end of this report. The geology of the Palos Verdes Peninsula has been studied for about 80 years due to its urban location, the potential for oil-bearing sediments, marine terraces for tectonic uplift rates, and the propensity for large slope failures. Based on this geologic history of the area, the 2011 Landslide is not an unusual occurrence. Other large active landslides include the nearby Point Fermin landslide, the Portuguese Bend landslide, the Flying Triangle landslide, the Klondike Canyon landslide, the South Shores landslide, the Abalone Cove landslide, and the recent Trump National Golf Club landslide. The 2011 Landslide lies approximately 1.3 miles west and approximately 3.5 miles east of the Point Fermin and Portuguese Bend Landslide complexes, respectively. These landslides have occurred in the Monterey Formation, which is the same formation that underlies the White Point area.

2.2 Geologic Mapping

Geologic features in and near the landslide were mapped by Shannon & Wilson engineering geologists over the course of several site visits between November 2011 and March 2012. For two of these site visits, we were accompanied by City geologists. Our mapping included noting indications of slope instability such as groundwater seepage, hummocky ground, and ground cracks. Where rock outcropped in the bluff and intertidal zone, we mapped geologic data such as
lithology, bedding plane, and joint attitudes. The geologic data collected during the mapping is included on Plate 1.

### 2.3 Aerial Photograph Review

We reviewed stereo, vertical aerial photographs for the area, including 1928 Fairchild photographs, 1953 United States Department of Agriculture photographs, and oblique aerial photographs from the California Coastal Records Project website (2012). We also reviewed online historical aerial photographs on Google Earth with images to 1994 and Google Maps oblique and Street View photographs. The historic aerial photographs reviewed were chosen in part because they depict the residential development and the construction of the White Point Military Reservation. The 1928 photographs were the earliest set of aerial photographs that were completed for Los Angeles County and typically predate most of the development present today. The 1953 photographs were chosen because they were closest to the construction date of the nearby Nike Missile Base.

In our aerial photographic review, we did not find evidence of pre-existing features that would indicate slope instability in the area upslope (north) of the 2011 Landslide. However, both 1928 and 1952 photograph years show a re-entrant (i.e., erosional feature) south of, and adjacent to Paseo del Mar on the east side of the 2011 Landslide. The aerial photographs show the re-entrant was approximately 150 feet wide with the head approximately 50 feet north of the north side of Paseo del Mar. Figure 4 shows the 1928 and 1953 aerial photographs of the re-entrant. In the 1928 aerial photograph, a series of parallel gullies are shown upslope and coalesce at the re-entrant. The gullies are absent from the 1953 aerial photograph, possibly from grading associated with the military base. Paseo del Mar originally crossed the head of this re-entrant on a timber trestle. From review of storm drain plans (City Drawing Number C-1855) on Navigate LA (2012), the re-entrant was partially filled sometime after construction of the timber trestle, and before 1982 when the concrete retaining wall was constructed to support the roadway fill. It is not clear as to why the additional fill was placed below the trestle.

We also noted that the Google Maps Street View photos show a series of en echelon fractures within the pavement of Paseo del Mar in the area of the western boundary of the 2011 Landslide. The date of the photo is shown as May 2011.

### 2.4 Groundwater Review

As part of this study, we reviewed several documents related to groundwater of the White Point area, including:
- Site Investigation Report, Whites Point Nike Missile Site, San Pedro, California, by International Technology Corporation (IT) (1996), and

Additionally, we reviewed groundwater well data recorded by the Los Angeles County Department of Public Works. The nearest groundwater well on this database, Well No. 322S, is located approximately 8.5 miles northeast of the 2011 Landslide near the intersection of Figueroa Street and West “C” Street in San Pedro. WCC (1997) indicates that no industrial or potable water supply wells were completed in the Monterey Formation on Palos Verdes Peninsula, and that no known operating water supply wells are within 2 miles of the White Point site.

Review of the State of California GeoTracker website (California State Water Resources Control Board, 2012) did not indicate observation wells in the immediate area; however, at least nine deep borings (seven with observation wells) have been drilled at the WPNP since 1986 (WCC, 1997). Observation well and boring logs for three borings proximal to the 2011 Landslide, MW-5 through MW-7, are included in Appendix A, under Previous Exploration Logs. The locations of the previous borings, including current and historic groundwater data, are shown on the Groundwater and Instrumentation Map, Plate 2. Note that we were unable to locate these wells during our field studies, and that they may be abandoned.

3.0 FIELD EXPLORATIONS

3.1 Health and Safety Plan

Shannon & Wilson prepared a Site-Specific Health and Safety Plan (SSHSP) for this project prior to initiation of the drilling and geologic mapping program. The City-approved SSHSP is included in our Preliminary Report. The purpose of the SSHSP was to protect the health and safety of the field personnel from physical and chemical hazards associated with the work. The plan identified the anticipated hazards at the site as well as possible hazards related to subsurface structures and utilities. Field personnel received a copy of the SSHSP and were required to read the plan and comply with its requirements. No accidents or recordable injuries were reported by our personnel or reported by our subcontractors during the fieldwork.
3.2 Soil Borings

We advanced nine borings near the 2011 Landslide between November 25 and December 20, 2011. The borings are designated B-1 through B-9 and are shown on Plates 1 and 2. The geotechnical explorations consisted of three rotary core borings (B-1, B-7, and B-9), four 24-inch-diameter bucket auger borings (B-2 through B-5), and two rotosonic borings (B-6 and B-8). The locations of the borings are shown on Plate 1. Details of the explorations and the logs of the borings are presented in Appendix A.

A Shannon & Wilson Certified Engineering Geologist supervised the field exploration program, and our field geologists and engineering staff located the borings, observed the exploratory drilling, collected samples, and logged the borings. Our daily activity during drilling was documented in our FARs included in our Preliminary Report. The original boring locations as identified by the City were modified in cooperation with the Palos Verdes Land Conservancy (Conservancy) to reduce potential impacts to the flora on the WPNP. Our biologist worked with Conservancy representatives to facilitate the equipment routes and final locations of the explorations.

3.3 Geophysical Borehole Surveys

GeoVision of Corona, California, performed acoustic televiewer logging in boring B-1 and optical televiewer logging in boring B-7. The selected geophysical logging techniques were based on the ability of the borehole to retain drilling fluids. Typically, drilling mud or water is required when using the acoustic televiewer method, where the optical televiewer method is best suited for clean boreholes or holes that cannot retain fluids. Details of the borehole televiewer surveys and the GeoVision report for borings B-1 and B-7 are included in Appendix B. Selected bedding and discontinuity attitudes obtained from the borehole televiewer surveys are on the logs of borings B-1 and B-7 and on Plate 1.

3.4 Instrumentation

3.4.1 General

Instrumentation, consisting of observation wells, inclinometers, and vibrating wire piezometers (VWPs), were installed at the boring locations shown on Plate 2. The locations and number of observation wells and inclinometers are based on the TOS and were modified in the field based on discussions with City representatives. The VWPs were installed based on experience with similar size and types of landslide projects. The wells, VWPs, and
inclinometers data along with the most recent measurements are shown in profile view on Plates 3 through 5. Observation well, VWP, and inclinometer installation details are presented in Appendix C. A brief summary of the instrumentation installation is provided below.

3.4.2 Observation Wells

A 2-inch-diameter PVC well casing was installed in borings B-6 and B-8, and an 8-inch PVC well casing was installed in boring B-3. Well development and well completion occurred between December 26 and 30, 2011.

3.4.3 Vibrating Wire Piezometers (VWPs)

VWPs were installed in borings B-1, B-5, B-7, and B-9 to monitor and record groundwater levels in the borings on a periodic basis. The VWPs can provide continuous monitoring of groundwater over time and may be coupled with inclinometer casing providing groundwater and deformation measurements in the same borehole. Additionally, VWPs provide groundwater measurements at a point, rather than throughout a depth interval such as in an observation well screen. For Boring B-1, we attached a datalogger to the VWP for the purpose of recording near-continuous groundwater levels at the boring. We plotted the readings as shown in Figure C-2 of Appendix C.

3.4.4 Inclinometers

Inclinometer casings were installed in borings B-1, B-5, B-7, and B-9 to permit periodic monitoring of the ground up-slope of the active landslide to detect lateral ground movements. Inclinometers are devices for monitoring deformation normal to the axis of a pipe by means of a portable probe passing through the pipe. One 2.75-inch outside diameter (O.D.) and two 3.34-inch ABS plastic inclinometer pipes, or casings, manufactured by the Durham Geo Slope Indicator Company were installed in vertical boreholes. The 2.75-inch O.D. casing was installed in boring B-1, and a 3.34-inch O.D. casing was installed in the remaining borings.

4.0 LABORATORY TESTING

4.1 Geotechnical Testing

Geotechnical laboratory tests were performed on selected samples retrieved from the borings. The testing included visual classifications, moisture content determinations, unit weights, hydrometer analyses, compressive strength, direct shear, torsional ring shear, Atterberg limits, expansion index, and corrosion. The moisture content is incorporated into the borings logs
presented in Appendix A. Descriptions of laboratory test procedures and results are presented in Appendix D.

4.2 Chemical Testing

To determine the possibility of in situ soil contamination characterization and appropriate disposal, three samples were collected from individual borings for analytical testing and submitted 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. Refer to Appendix E for environmental laboratory test results.

5.0 GEOLOGIC AND SUBSURFACE CONDITIONS

5.1 General

The Palos Verdes Peninsula is a northwest-trending, dome-like ridge, approximately 9 miles long and up to 5 miles wide. Its crest has a gently rolling upland ranging between elevations 1,100 and 1,480 feet above sea level. Below the upland, remnants of as many as 13 Pleistocene marine terraces recording previous marine shorelines ring the peninsula, indicating that the peninsula was an island for much of its geomorphic evolution. The peninsula is currently bounded by the Los Angeles coastal plain to the north, by Los Angeles Harbor on the east, and the Pacific Ocean on the south and west.

The 2011 Landslide is located along the southern coast of the Palos Verdes Peninsula. The subject landslide is located at the edge of a marine terrace with a south-facing slope of approximately 3 to 5 degrees. The headscarp (or crown) of the landslide is approximately 650-feet wide (east-west) and extends north from the bluff approximately 280 feet, encompassing a large section of Paseo del Mar (Plate 1).

5.2 Regional Geology

Geologic factors such as bedrock lithology, structure, and groundwater play a contributing role in the formation and occurrence of most large landslides on the Palos Verdes Peninsula. The
following sections present the general geologic and hydrologic conditions of the Palos Verdes Peninsula as well as detailed observations at the White Point site.

Approximately 2,000 feet of middle to upper Miocene sedimentary rocks, termed the Monterey Formation, unconformably overlie the 115 to 120 million-year-old (Ma) Catalina Schist. The Monterey Formation is widely exposed over the Palos Verdes Peninsula and is divided into three members: the Altamira Shale (15.5 to 13.0 Ma), the Valmonte Diatomite (13.0 to 6.9 Ma) and the Malaga Mudstone (6.9 to 3.5 Ma, Woodring and others, 1946). Woodring and others (1946) further divided the Altamira Shale into lower, middle and upper units characterized by a tuffaceous facies, cherty facies, and phosphatic facies, respectively. The tuffaceous lithofacies is the most aerially extensive of the three and the most important geologic unit from a slope stability standpoint (Haydon, 2007).

The lower or tuffaceous lithofacies of the Altamira Shale consists predominantly of silty shale in the lower part and porcelanite (silicified siltstone and claystone) in the upper part. It also contains the Portuguese Tuff and the Miraleste Tuff and intrusive basalt sills and dikes. The volcanic rock is exposed as jagged masses or sea stacks in the surf zone south of Point Vicente (Brown, 2008) and south of White Point (Dibblee and others, 1999). Most pyroclastic glass (volcanic tuff) within the study area has been diagenetically altered to bentonite clay comprised of both calcium and sodium montmorillonite (Hill and others, 2008), exhibiting low shear strength, and acting as barriers to groundwater flow (aquitards).

Younger Quaternary units include Pleistocene marine and non-marine terrace deposits, Holocene alluvium, recent beach deposits, and artificial fill.

Structurally, the Palos Verdes Peninsula is dominated by a broad northwest-trending doubly plunging anticline and the Palos Verdes Fault. The Palos Verdes Fault is part of a system of right-lateral strike-slip faults that extends over 125 miles southeast from Palos Verdes. At San Pedro, the fault bends to the west about 30 degrees, creating compressional motion rather than strike-slip motion. The anticline of the peninsula has been uplifted along a southwest dipping fault within this compressional bend. The axis of the anticline is generally coincident with the crest of the Palos Verdes Hills. The south limb of the Palos Verdes Anticline exhibits bedding dips within the Monterey Formation ranging between 5 and 35 degrees, with locally steep dips as much as 80 degrees (Woodring and others, 1946). Most of the large landslides on the Palos Verdes Peninsula are related to the existence of unfavorable geologic structure within the Altamira Shale.
Joints developed during folding of the Monterey Formation are closely related, geometrically, with the orientation properties of the folds with which they are associated. Figure 5 schematically illustrates the general orientation of three joint sets commonly generated during folding. The three classes of joints consist of:

- Cross joints which are generally aligned perpendicular to the axis of folding,
- Longitudinal joints which are generally subparallel to the axial surface of folds, and
- Oblique joints which are comprised of two conjugate sets that are symmetrically disposed to the hinge and axial surface of a fold. The axial surface of the fold bisects the obtuse angle (~120 degrees) of intersection of the oblique joint set.

Of the three joint sets generated during the deformation of brittle rock during folding, longitudinal and conjugate joint sets tend to be through-going, planar, continuous structures (Davis, 1984).

The California Geological Survey (CGS) has mapped about 180 landslides in the Palos Verdes Hills (Brown, 2008). Figure 6 shows the location of the landslides included on the Landslide Inventory Map of the Palos Verdes Peninsula, published by the CGS (Haydon, 2007). The tuffaceous lithofacies of the Altamira Shale hosts the highest density of mapped landslides on the Palos Verdes Peninsula. These landslides have been classified as historically active and dormant rock landslides with complex movement modes, typically translational block landslides. Landslides on the flank of the Palos Verdes Anticline are principally dip-slope landslides, with failure planes occurring on southwest- and seaward-dipping beds of altered tuff (bentonite).

5.3 Geologic Units

Site geology definitions are based on a review of published geologic maps, including the Geologic Map of Palos Verdes Peninsula and Vicinity by Dibblee (1999), the Geologic Map of the Long Beach 30’ X 60’ Quadrangle by Saucedo and others (2003), and the Seismic Hazard Evaluation of the San Pedro 7.5-minute quadrangle by the California Division of Mines and Geology (1998). The following geologic unit descriptions, listed youngest to oldest, are based on field observations during geologic mapping and subsurface explorations. The geologic abbreviation (e.g., Qt) used in this report is based on the published geologic maps and/or local convention and is listed to the right of the unit name. Refer to the boring logs for those units encountered during drilling and Plate 1 for units exposed at the ground surface in the vicinity of the 2011 Landslide.
5.3.1 Fill (af)

Fill has widely variable properties, depending on the material used as fill and whether the fill was placed in an engineered or non-engineered fashion. Fill soils were identified from the presence of irregular clasts of one soil type within soil of another type, disturbed appearance, and from the presence of debris such as asphalt, concrete, and wood. Fill soils were observed during geologic mapping within the headscarp and graben of the landslide. The fill soils represented backfill material for utility trenches and fill placed as backfill for the concrete block wall across the drainage at the east end of the landslide area. Aside from the subbase fill soils encountered in the explorations performed within Paseo del Mar, no fill soils were noted in the explorations. The fill soils in the former drainage behind the block wall appear to be silty to gravelly sand, with shale clasts in a silty sand matrix.

Additional fill deposits were also identified by ground-surface topography in the vicinity of existing structures. The large, flat paved area surrounding the Nike missile silos consists of a partial fill of approximately 5 to 6 feet at the southeast corner. Other topographical evidence of fill soils include the sidecast fill along the south side of the beach access road, which was not mapped in detail.

5.3.2 Landslide Deposits (Qls)

Landslide deposits consist of soil and rock previously mobilized by various types of mass movement processes. Landslide deposits consisting of both soil and rock were identified during geologic mapping along the bluff directly above the active beach. Landslide deposits were not encountered within our explorations drilled north of the present landslide headscarp. The bulk of the landslide deposits extend south of the existing landslide crown (headscarp) to the beach and above an elevation of between 0 and 15 feet above mean seal level (MSL) as shown in Photograph 2. Landslide deposits within the study area are characterized as a heterogeneous mixture of silt, sand, gravel, and boulders comprised principally of material derived from the Altamira Shale. Blocks of intact Altamira Shale and terrace soil are also present within the main body of the landslide mass. The Altamira Shale blocks observed within the landslide debris are characterized by numerous, dilated, orthogonal fractures.
Photograph 2 – View of the main block of the 2011 Landslide, showing graben and Palm Tree “island”. View to west.

5.3.3 Beach Deposits (Qb)

At the shoreline, steep to near-vertical cliffs exist above a less than 30-foot-wide, discontinuous beach deposits resulting from ongoing wave action at the toe of the bluff. Beach deposits consist primarily of sand, gravel, cobbles and few boulders principally confined to the toe of the slope as shown in Photograph 3. Once landslide debris accumulations at the toe of the slope are reworked by wave action in the intertidal zone, they are transformed into beach deposits. Outward of the slope toe, the deposit pinches out to nearly absent, with exposed bedrock predominating below MSL. Due to the relatively low strength of the Altamira Shale, rapid deterioration of larger clasts contribute significantly to the turbidity of the water in the intertidal zone, especially during high tide when wave action is lapping at the toe of the bluff. No beach deposits were encountered within our subsurface explorations performed at the site.
5.3.4 Alluvial Fan Deposits (Qal)

Alluvial fan deposits were identified by ground-surface topography at the mouth of the drainage located along the northern portion of the site (Plates 1 and 2). The fan-shaped, gentle slope at the bottom of the drainage is inclined approximately 12 degrees and extends from approximate elevation 142 up to 185 feet MSL. The alluvial fan soils are deposited by intermittent surface water flows within the drainage. The drainage was dry during our site visits for this study. No alluvial fan deposits were encountered within our explorations performed at the site.

5.3.5 Terrace Deposits (Qt)

The topographic bench extending landward from the sea cliffs to the base on the slope, approximately 500 to 600 feet to the north of the landslide, is blanketed with Quaternary marine and non-marine terrace deposits as shown in Photograph 4. These deposits were encountered in all the explorations performed at the site and range in thickness between 4.5 and 9.0 feet. The deposits consist of medium stiff to very stiff, dark olive-brown to brownish-black, slightly gravelly to gravelly, slightly sandy to sandy, silty clay with brownish-yellow angular siltstone...
clasts to 6-inch-diameter that increase in abundance with depth. Scattered clayey silt and silty sand zones also exist within the terrace deposits. The soils are dry to slightly moist and exhibit desiccation cracks indicative of expansive, high-plasticity clay.

![Photograph 4 – View of dark gray terrace deposits overlying light brown Monterey Formation in the headscarp of the main landslide. View to east.](image)

### 5.3.6 Altamira Shale (Tma)

The Altamira Shale member of the Monterey Formation underlies the terrace deposits at the site and was encountered in all the explorations performed at the site. Within the explorations, the Altamira Shale comprises clayey siltstone, silty sandstone, silty claystone, limey to silicified siltstone, sandstone derived from glaucophane schist (Catalina Schist), and bentonite beds. The rock is thinly bedded to laminated and contains some tar along fractures and in brecciated zones. Gypsum, caliche, and minor sulfur deposits exist along fractures within the upper oxidized zone. The oxidized horizon is mottled yellowish-brown and grayish-orange, and exists in the upper 40 to 45 feet of the formation. Below the oxidized horizon, the Altamira Shale is generally unweathered (fresh) and exhibits colors ranging from brownish-black to olive gray. Many light bluish-gray sandstone beds were also noted in the borings and intertidal zone.

From a rock/soil strength standpoint, the weakest zone observed in the borings consists of the bentonite clay beds. Two- to five-inch-thick bentonite beds were observed in borings B-2,
B-3, B-7, and B-9 at depths between 88 and 97 feet. Additional bentonite beds were encountered in the same borings between depths between 10 and 39 feet. Of particular note, the bentonite beds encountered between 88 and 97 feet are highly polished, soft, wet, and generally slightly discordant to bedding. Bentonite beds encountered above 39 feet were folded; however, they did not exhibit polished and slickensided surfaces.

Post- and syn-depositional deformation was also noted within the borings and in outcrop along the shoreline as shown in Photograph 5. Specifically, the finer-grained layers exhibit significant soft sediment deformation in the form of tight folds and flame structures. The more competent sandstone beds were not generally deformed. Abundant fine-grained “rip-up” clasts were also noted within the sandstone beds.

Photograph 5 – Beds of the Altamira Shale member of the Monterey Formation showing tight isoclinals folding. Exposure located in inter-tidal zone immediately west of the 2011 Landslide. View to southwest.

5.4 Geologic Structure

Structurally, the Altamira is the only geologic unit encountered at the site to exhibit rock structure measurable in the field. Bedding and discontinuity characteristics such as attitude, filling, roughness, and type were obtained using various methods from all of the explorations performed for this study. Oriented bedding and discontinuity attitudes, obtained in borings B-1
through B-5 and B-7 are shown in the boring logs. Measured bedding and discontinuity attitudes obtained from our geologic mapping along with those obtained from borings B-1 through B-5 and B-7 were used to develop the approximate bedding attitudes shown on the Generalized Subsurface Profiles A-A' through I-I', Plates 3 through 5.

Based on geologic mapping and structural data recovered from the borings, the bedding attitudes along the beach and outside of the landslide mass exhibit a broad synclinal structure with an axis oriented approximately north to south, and plunging south toward the Pacific Ocean. The syncline appears to roughly control the location of the 2011 Landslide. The synclinal fold limbs generally dip between 14 to 31 degrees SE on the western limb and 9 to 14 degrees SW on the eastern limb. Evidence of relatively persistent discontinuities (or joints) oriented at N70ºE, 75-90ºS/N and tight folding with variable bedding attitudes ranging between N90ºE, 10ºN and N70ºW, 17ºS exists along the beach at the west edge of the landslide headscarp.

Within the borings performed at the site, bedding strike attitudes trended from N44ºW at the east side of the landslide headscarp (boring B-7), to between N40ºW and N64ºW toward the center of the headscarp, to between N73ºE and N27ºE at the west portion of the landslide (boring B-1). Average southerly dips within the explorations ranged between 12 and 35 degrees.

West of the landslide, the geologic structure becomes dominated by tight isoclinals folds generally with east-west to northwest-southeast-trending fold axes. Many of these folds are exposed along the sea cliff slope face west of the main landslide mass. The geologic structure separating these two structural regimes appears to be a series of northeast-trending shears or faults that are exposed in the Altamira Shale outcrop in the interdial zone. These shears or faults roughly parallel the western boundary of the active landslide scarp.

### 5.5 Subsurface Profiles

We reviewed the subsurface samples and data collected at the project site and prepared generalized subsurface profiles. The locations of the subsurface profiles are shown on Plate 1. Eight subsurface profiles were prepared roughly perpendicular to the 2011 Landslide headscarp shown in Plates 3 through 5. Using the translation direction of the displaced palm tree, Profile C-C' was oriented approximately parallel to and through the main landslide mass and along the suspected direction of movement. Additional subsurface profiles were prepared along the approximate pre-landslide centerline of Paseo del Mar (Profile I-I') and the intact eastern flank of the landslide (Profile J-J'). For profile locations where explorations were located off the profile, geologic information was extrapolated along strike and the apparent dip in the line of
section shown on the profile. The interpreted geologic structure and unit distributions are shown in the subsurface profiles.

5.6 Groundwater

5.6.1 General

The presence of water can cause seasonal or shallow mass wasting because of excessive rainfall and infiltration, and long term or deep-seated persistent landslide movement because of increased porewater pressure. To help characterize the role of groundwater on the 2011 Landslide, we reviewed historical information (IT, 1996, and WCC, 1997), groundwater observations during drilling, and the groundwater measurements obtained from our instrumentation since installation.

5.6.2 Historical Data

Based on our literature review described previously, excess groundwater pressure conditions exist within the bedrock in the vicinity of the 2011 Landslide. Excess groundwater pressure conditions are found where groundwater is under sufficient hydrostatic pressure (or head) to rise above the point at which it is encountered in wells. Such excess hydrostatic head commonly contributes to instability by creating uplift pressures, reducing effective overburden stress, and decreasing shear strength in the underlying rock and soil.

Artesian groundwater conditions are described within the IT (1996) and WCC (1997) reports. Groundwater was initially encountered during drilling in boring MW-5, located north of the landslide, at 110 feet below ground surface (bgs). A subsequent measurement indicated groundwater at 16 feet bgs in boring MW-5, showing substantial rise of groundwater in the well. Similarly, groundwater was encountered during drilling in MW-7 at 138 feet bgs and subsequently measured at 14 feet bgs. The excess hydrostatic heads for MW-5 and MW-7 are therefore 94 and 124 feet, respectively.

Similar evidence of excess groundwater pressures were noted in MW-1 and MW-2 by WCC in 1997 (boring logs were not available). One WCC boring encountered flowing artesian conditions at approximately 27 feet MSL (IT, 1996). Flowing artesian conditions are where hydrostatic pressures are sufficient to force groundwater above the ground surface. Borings MW-1 and MW-2 are located along the eastern portion of the site, outside the landslide are shown in Plate 1.
5.6.3 Post-2011 Landslide Observations

Groundwater observations during field explorations consisted of geologic mapping, drilling, down-hole logging, and laboratory observations of saturated samples, ranging from outcrop to sample scales. In general, groundwater seepage observed in borings B-1 through B-5 revealed relatively light fracture flow between 48 and 68 feet bgs and relatively heavy fracture flow at various depths between 78 and 109 feet bgs. Additional drilling observations and groundwater seepage details are included on the boring logs. In addition, locations of surface seepage at the toe of the bluff observed during the geologic mapping are shown in Plates 1 and 2.

Below is a list of groundwater observations for each of the nine borings during drilling and logging operations at the site and in our laboratory:

- **Boring B-1**: Water level in the borehole could not be maintained during flushing above 63 feet and televiewer logging. Borehole was dry during drilling to 109 feet bgs.
- **Boring B-2**: Slight fracture seepage observed during down-hole logging at 68 feet bgs. Strong (1/2 gallon per minute [gpm]) seepage observed streaming approximately 6-inches out into borehole in 1/8-inch-diameter stream at 88 feet bgs.
- **Boring B-3**: Pervasive fracture seepage observed during down-hole logging at 48 feet bgs. Strong (1/2 gpm) seepage issuing from brecciated silicified zone at 97 feet bgs.
- **Boring B-4**: Moderate seepage observed during down-hole logging along fractures at 62 feet bgs. Strong groundwater flow issuing from fractures below 78 feet bgs.
- **Boring B-5**: Seepage observed during down-hole logging issuing from fractured siliceous siltstone layer at 66 feet bgs. Strong fracture seepage noted below 86 feet bgs.
- **Boring B-6**: Groundwater observations not noted during drilling due to sonic drill action.
- **Boring B-7**: Groundwater observations not noted during drilling due to the use of wet rotary methods (HQ3 coring), however circulation loss and difficult drilling was noted as the cause for termination at 117 feet bgs.
- **Boring B-8**: During laboratory clear plastic tube review, abundant free water was present between a depth of 97 and 98 feet bgs (Groundwater observations not noted during drilling due to sonic drill action). The free water saturated a brecciated (1/2- to 2-inch-diameter, angular fragments), siliceous siltstone zone. The water was trapped between relatively plastic claystone layers.
- Boring B-9: Groundwater observations not noted during drilling due to the use of wet rotary methods (HQ3 coring), however approximately 1,000 gallons of water was used to drill the upper 81.5 feet.

5.6.4 Groundwater Measurements

Groundwater was most recently measured in the borings on May 29, 2012, as shown in Table 1. Groundwater was recorded during drilling, and was measured weekly for the first month and monthly thereafter. Appendix C contains details of well and VWP installation, chronologic groundwater measurements, and groundwater measurements by others. In general, during the period of monitoring groundwater levels on site from December 2011 through March 2012 there has been a slight decrease in water levels across the site of approximately 1 to 6 feet, with the exception of boring B-1. Boring B-1 has noted an increase of approximately 5 feet (Plate 2).

### TABLE 1

<table>
<thead>
<tr>
<th>ELEVATION OF INSTRUMENTATION AND GROUNDWATER</th>
<th>B-1</th>
<th>B-2</th>
<th>B-3</th>
<th>B-4</th>
<th>B-5</th>
<th>B-6</th>
<th>B-7</th>
<th>B-8</th>
<th>B-9</th>
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<tr>
<td>Ground Surface Elevation (ft)</td>
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<td>124.5</td>
<td>124.1</td>
<td>130</td>
<td>123.9</td>
<td>136.7</td>
<td>122.6</td>
<td>127.6</td>
<td>128.1</td>
</tr>
<tr>
<td>Well Screen/VWP Elevation (ft)</td>
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<td>ND</td>
<td>14.1 to 84.1</td>
<td>ND</td>
<td>6.41</td>
<td>26.7</td>
<td>7.3</td>
<td>17.6 to 98.6</td>
<td>14.0</td>
</tr>
<tr>
<td>Baseline Groundwater Elevation (ft) and Date</td>
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<td>1/4/12</td>
<td>25.0</td>
<td>12/30/11</td>
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<td>60.1</td>
<td>1/4/12</td>
<td>86.1</td>
<td>1/4/12</td>
</tr>
<tr>
<td>Groundwater Elevation (ft) 5/29/12</td>
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<td>26.4</td>
<td>-</td>
<td>64.1</td>
<td>90.2</td>
<td>26.3</td>
<td>51.6</td>
<td>35.8</td>
</tr>
</tbody>
</table>

Notes:
1) Elevation datum is MSL.
2) ND = Not Developed

6.0 WHITE POINT SITE CHRONOLOGY

6.1 General

Early development of the site was observed in the 1907 USGS topographic map and 1928 aerial photographs that showed a two-lane road along the current Paseo del Mar alignment (Figure 4). We have limited information on when the underground utilities were installed along Paseo del
Mar. The timber trestle (referred to by the City as the “sidehill bridge”) was replaced by a retaining wall, which according to the City plans is dated 1982.

Rainfall records prior to the 2011 Landslide initiation show below-average rainfall after the heavy rains of the 2004-2005 season until the 2009-2010 and 2010-2011 above-average rainfall seasons. Details of the rainfall records are provided in Appendix F.

6.2 2009 Landslide

The initial report of ground movement that could be associated with the 2011 Landslide is a small landslide that occurred in December 2009 (2009 Landslide) following a significant precipitation event (see Figure F-3). The 2009 Landslide is shown in Plate 1 as below the retaining wall constructed in 1982. Reportedly, City representatives in January 2010 noted cracks along Paseo del Mar on the western scarp of the 2011 Landslide. The City set up a regular survey monitoring of Paseo del Mar in this area. We have been provided with a letter by the City describing the pre-landslide survey data in Appendix F.

On June 29, 2011, the City survey crew noticed a depression in the road near the crack observed in January 2010. Monitoring continued through the summer, and according to the City survey representatives, the movement was classified as a landslide.

6.3 Street Closure

On September 19, 2011, the City closed a stretch of Paseo del Mar between Weymouth and Western Avenues due to ground cracking and approximately 6 inches of displacement (C.A.R.E., 2012). A perimeter chain-link fence was constructed around the landslide area starting on November 11, 2011.

6.4 Failure

On November 20, 2011, failure of the landslide occurred in the afternoon at about 1630 hours. The translation of the landslide was approximately 60 feet based on survey of the isolated palm tree on the main landslide block. The failure occurred during a heavy rain event where about ½-inch of precipitation was recorded (see Appendix F).

In mid-November, the City scheduled a meeting on November 21, 2011 with their geotechnical on-call consultants for proposals to explore and repair the 2011 Landslide. Following the landslide event the day before, City representatives requested Shannon & Wilson attend a site meeting on November 21, 2011. During this and subsequent site meetings over the following
days, the scope of our services described previously was developed and subsurface explorations started that week.

7.0 EVALUATION OF POTENTIAL LANDSLIDE CONTRIBUTING FACTORS

7.1 General

In addition to the influence of the primary factors (i.e., geology and soil/rock strengths), we evaluated the potential for several external factors to have contributed to the activation of the 2011 Landslide. The factors that we determined may have contributed to the landslide are (in alphabetical order):

- Irrigation
- Coastal Bluff Erosion
- Precipitation
- Residential Development
- Road Construction
- Underground Utilities

Additionally, we determined that the Nike Missile Base likely did not contribute to the activation of the landslide.

With the available information, we were unable to determine the causal relationship between the 2011 Landslide and construction of the “sidehill bridge” retaining wall or the 2009 Landslide. The impacts of these contributions are discussed in both this section and our “Stability Analyses” following this section.

7.2 Contributing Factors

In our opinion, the following factors could have contributed to the activation of the 2011 Landslide.

7.2.1 Irrigation

Irrigation of the ground in the vicinity of the 2011 Landslide could affect the groundwater conditions at the site, specifically if the irrigation is conducted up-gradient and/or up-dip of the site. Sources of irrigation could be from the WPNP or from residential development, primarily north and east of the site.
In conjunction with the redevelopment of the White Point Military Base as a park, Los Angeles County Department of Public Works completed an engineering geologic and seismic report in 1975 as part of an Environmental Impact Report (EIR) to determine the feasibility of the site for park development. One of the five recommendations contained in the EIR stated, “Irrigation should be controlled to avoid creating geologic instability, particularly in the southern one-half of the site, as south-dipping strata may conduct water toward the sea cliff” (County of Los Angeles, 1975).

While the functionality of the irrigation system at the WPNP was not reviewed, we observed drip-type irrigation equipment throughout the vegetated area north, northwest and northeast of the landslide crown/headsarp. We observed sprinkler-type irrigation north of the WPNP trail. City representatives reported to us that the irrigation system was installed in 2003.

The residential areas east of Weymouth Avenue and the U.S. Air Force housing north of the site in the vicinity of Grissom Drive are situated up-dip and up-gradient of the 2011 Landslide. While residential irrigation could be a contributing factor to landsliding, we do not currently know volumetric information and usage rates for these areas.

We understand the City continues to review and compile LADWP water records of WPNP and these surrounding properties in an effort to quantify irrigation, including the start date of irrigation, the shut-off date, the volumetric information, and the water usage rates.

7.2.2 Long-Term Coastal Bluff Erosion

The steep bluffs at the site and the surrounding area are a result of ongoing coastal bluff erosion. The following discussion of coastal bluff erosion is based on our professional experience and existing literature, including Johnsson (2003), Johnson and Marcum (2007), and Pipkin and Ploesel (1973).

Coastal bluff erosion occurs where wave energy is sufficient to erode the bluff face, resulting in landward migration of the shoreline over time. This process tends to occur episodically, with greater regression during periods of increased wave activity and precipitation and is exacerbated by the absence of a protective beach, such as exhibited at the site. In general, the erosion of coastal bluffs depends upon several factors, including:

- Bedrock lithology or soil type,
- Surface runoff (rilling and gullying),
- Groundwater (promoting creep, reducing soil effective shear strength because of porewater pressure),
- Wave action (erodes cliffs to a steep, unstable condition and removes mass wasting debris which would otherwise accumulate at the base of the slope, increasing stability), especially during storm events, and
- Other factors such as effects of wetting and drying, chemical weathering and wind.

To the extent that coastal erosion by the mechanisms listed above has resulted in steep bluff faces, coastal erosion can be considered a contributing factor to the 2011 Landslide. We are not aware that significant amounts of coastal erosion occurred between the 2004-2005 rainfall season and the activation of the 2011 Landslide. As discussed in the Stability Analyses section, we modeled the effects of erosion at the toe of the slope and found that it had a relatively insignificant effect on the deep-seated slope stability compared to elevated groundwater levels. However, coastal erosion may to have contributed to the smaller 2009 Landslide.

7.2.3 Precipitation

The precipitation record for the San Pedro area is summarized in Appendix F. Figures F-1, F-2, and F-3 in Appendix F illustrated the annual precipitation, the average monthly precipitation and the daily precipitation records, respectively. High precipitation levels in this area usually occur between November and March. Prior to the 2011 Landslide failure, the precipitation record indicates nearly two inches of precipitation fell between October 2011 and November 20, 2011.

The effect from precipitation on the groundwater levels at the site has been observed and found to be significant. The groundwater levels recorded by the piezometer in boring B-1 did not rise during a period of relatively heavy precipitation on December 13 and 14, 2011, but subsequently rose about 12 feet from the level observed during drilling in the six-month period between January through May 2012 (Figure C-3 in Appendix C).

The rise of the groundwater level from precipitation is believed to increase hydrostatic pressure (driving force) in the soil mass and contribute to the mobilization of the 2011 Landslide.

7.2.4 Residential Development

Residential development around the perimeter of the site can be defined as two general zones:
• Development east of Weymouth Avenue, and
• Development to the north between the site and West 25th Street.

Development east of Weymouth Avenue generally took place between 1928 and 1958, based on a review of historical aerial photographs and City Sewer plans on the Navigate LA (2012) website. Approximately 13.5 acres at the northern edge of the White Point site, south of West 25th Avenue, was developed by the U.S. Air Force sometime after 1987 and before 1994, based on historical imagery. The 13.5 acres were developed except for approximately 15 houses in the vicinity of Grissom Drive, which were constructed sometime between 1994 and 2002 based on historical imagery. The Grissom Drive area is north of the project area (Plate 1), generally up-dip of the 2011 Landslide.

The closest residential structures to the landslide range from approximately 300 feet to the east of the landslide to approximately 1,100 feet north of the landslide headscarp. Based on the geologic structure of the underlying Altamira Shale, infiltration from landscape irrigation and/or leaking underground utilities related to the residential development could be directed along bedding planes to the general area of the 2011 Landslide. Therefore, localized irrigation from development could be influencing groundwater at the landslide, a contributing factor to instability. Grading related to residential development is unlikely to have contributed to slope instability at the 2011 Landslide site.

7.2.5 Road Construction

While information regarding the initial construction of the roadway was not located, aerial photographs, on-line City information (Navigate LA, 2012), and field observations provided the following information on the roadway in the vicinity of the landslide. Paseo del Mar is shown on the 1907 USGS topographic map of the San Pedro Hills and thus was constructed sometime prior to that year. On-line information from Navigate LA (2012) indicates the southern portion of the roadway extended over an approximately 35-foot-deep drainage re-entrant that extended to the centerline of the roadway. The south side of the road was supported by a timber trestle as shown in Appendix F. In 1982, a concrete block wall was constructed across the drainage and the drainage backfilled. Wall details are also provided in Appendix F. Portions of the timber trestle were observed in the fill soils behind the broken concrete block wall within the landslide mass during our geologic mapping. We also noted broken underground irrigation piping exists along the south margin of Paseo del Mar. We assume the irrigation line was for the landscaping along the south side of the roadway and is a separate system from the nearby WPNP irrigation described previously.
Observations of the Paseo del Mar pavement section indicate at least three separate paving layers. Of particular note, the upper most pavement section thickens to the south from 1.5 inches at the centerline to 5 inches at the south curb (Figure 7, Photo 1), perhaps indicative of past movement or settlement of the southern portion of the roadway; alternatively the thicker pavement section could be related to utility installation and routine pavement maintenance. We do not have information on the most recent pavement overlay date for this section of Paseo del Mar.

To the extent that roadway grading may have redistributed the slope mass and paving would have altered the infiltration patterns of surface runoff, road construction may have been a contributing factor to the 2011 Landslide.

7.2.6 Underground Utilities

An underground sewer pipe was installed along Paseo del Mar by the City of Los Angeles in 1940. The as-built drawings indicate the pipe is an 8-inch vitrified clay pipe (VCP). The pipe invert is about six feet below street level. We understand that the sewer pipe serviced the restrooms for the WPNP.

On the east side of Paseo del Mar, a storm drain pipe (reportedly 54 to 84 inches in diameter) was installed below the street according to the design plans prepared sometime after September 1969 by the City of Los Angeles for the Los Angeles County Flood Control District. The as-built drawings indicate the pipe is a variable diameter precast concrete pipe (PCP). The pipe invert is about 12 to 15 feet below street grade. The inverts of the storm drain pipes exposed in the 2011 Landslide headscarp are within the Altamira Shale (see Photographs 2 and 4 above). In 2011, the storm drain outfalls were reconstructed down slope to the shoreline east and west of the observed 2011 Landslide movement. The locations of the new storm drain outfalls are shown on Plate 1.

After cracks were discovered on the roadway, the storm drains were surveyed with video cameras for damage on July 13, August 19, and September 20, 2011. The survey indicated some damages to the storm drain pipes. The details of the survey and the pre-landslide storm drains are included in Appendix F (see Figure F-4).

The construction drawings from September 1969 also indicate a 4-inch gas and 6-inch water lines in the street. The depths of these lines are about 3 to 4 feet below street level.
Similar to the precipitation, the leakage of water from sewer, storm drain, or water lines may have contributed to the increase in groundwater level that could have led to mobilization of the 2011 Landslide.

7.3 Non-Contributing Factors

We concluded that it is unlikely that the Nike Missile Base contributed to the activation of the 2011 Landslide. A history of the Nike Missile base is provided in Appendix F. During February 2012, a tour of the missile silos was conducted with City representatives to observe possible evidence of ponded water, either past or present. Evidence of ponded water inside the Nike Missile silos was not observed during the tour. A summary of the tour, including sketches of existing cracks within the silos, was presented to the City in a letter, dated February 27, 2012. A copy of the letter is attached to Appendix F.

Based on our observations, it is our opinion that the presence of the Nike Missile Base did not contribute to activation of the 2011 Landslide.

7.4 Inconclusive Factors

With the information from our subsurface explorations, geologic mapping and interpretation of the site, historical review of the site, insight gained from our stability analyses (described in the following section) and information provided to us by the City, it is our opinion that a definitive statement cannot be made regarding the causal relationship between the 2009 Landslide and the 2011 Landslide.

As described previously in the Chronology and Road Construction sections, a retaining wall was built to replace the timber trestle at the slope re-entrant location sometime in the early 1980’s (slope re-entrant visible in Figure 4). This location represents a topographic low point of the site and appears to be the area where the natural waterway that drains the upper terrace originally crossed the roadway (the alluvial fan of the natural waterway is visible in Plate 1). To our knowledge, no measures were taken to capture the flow of the natural waterway and convey it to the ocean or existing stormwater infrastructure during construction of the retaining wall, which included placing fill in the natural waterway depression. It is possible that surface runoff infiltration continues to flow through the subsurface along its original drainage path, towards the retaining wall, and this water might have saturated the retaining wall backfill. The saturated backfill may have contributed to the 2009 Landslide below the retaining wall.
We understand City representatives observed cracking in the Paseo del Mar roadway in January 2010 in the area that would eventually be the west flank of the 2011 Landslide. The cracks were observed during survey monitoring of the 2009 Landslide area. This suggests that the larger landslide mass that would eventually become the 2011 Landslide initially mobilized at approximately the same time as the 2009 Landslide. However, there is insufficient data to determine whether the 2009 Landslide occurred prior to or after the formation of the cracks that were observed in January 2010. As discussed in the following Slope Stability section, we were not able to directly capture the effect of the 2009 Landslide on the 2011 Landslide with our two-dimensional slope stability analysis because the 2009 Landslide occurred east of the main portion of the 2011 Landslide mass.

8.0 STABILITY ANALYSES

8.1 General

The stability of natural slopes is a complex, three-dimensional relationship between the driving forces of mass, water, 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, many of the complexities inherent to real slopes are greatly simplified in order to perform the analyses. Accordingly, the results of slope stability analyses are approximate and should be treated as such. Regardless, slope stability analysis is a useful tool for developing an understanding of factors that affect the stability of a slope and for making decisions regarding repair, mitigation, or avoidance of unstable areas.

We used the computer program SLOPE/W version 7.17 (Geo-Slope International, 2007) to perform two-dimensional, limit equilibrium stability analyses of the 2011 Landslide and potential future landslides. 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.

Our stability analyses are not supported by subsurface explorations directly within the existing landslide mass. The post-landslide field conditions were deemed too dangerous and inaccessible for drilling or other exploration equipment. Therefore, we have developed our stability model based on the geologic mapping and extrapolation of the subsurface data from our borings around the landslide perimeter.
8.2 Geology

8.2.1 General

Geology typically plays a critical role in coastal landslides. The geologic and hydrogeologic conditions are discussed previously in the report. For the analyses, we assumed that bedding dips out of the slope as shown in Plates 3 through 6, and that bentonite clay is present on the failure surface. We assumed that the weak bentonite clay contributed significantly to the development of the landslide. Figure 8, based on Cross Section C-C’, shows a simplified cross section of the 2011 Landslide as modeled for our stability analyses.

We assumed that the structural discontinuities discussed previously and shown in Plate 1 and Figure 4 could have functioned as a release surface for the 2011 Landslide. Specifically, we assumed that conjugate oblique joints trending northwest and northeast and/or faults trending northeast served as vertical planes of weakness near the head of the landslide. We considered cases with and without the discontinuity surfaces for the slope stability modeling. We modeled the discontinuities as extending vertically from the ground surface to the failure plane. As described in the following sections, rock discontinuities likely played a significant role in the landslide.

Erosion at the toe of a slope could be a destabilizing effect because the mass of the material acts as a buttress against the weight of the soil and rock above. As discussed above, beach deposits consisting of boulder-sized fragments of Altamira Shale in the White Point region and lack of sand deposits on the beach are evidence of ongoing erosion of the coastal bluff, indicating that this may have been a contributing factor to the landslide. We modeled toe erosion in the slope stability analyses by incrementally removing material from the toe of the slope. The results suggested that recent toe erosion had a minimal effect on the stability of the slope. However, it should be noted that the steep bluffs on the Palos Verdes Peninsula are generally shaped by long-term coastal erosion, resulting in conditions conducive to landsliding.

8.2.2 Hydrogeology

To capture the effect of elevated porewater pressures acting on the failure plane, we modeled two separate piezometric surfaces, one for the confined aquifer conditions (i.e., artesian) and one for unconfined aquifer conditions. Applying the confined piezometric surface to the bentonite clay layer modeled at the base of the landslide and the unconfined piezometric surface to the remaining materials resulted in failure surfaces that resembled the observed (actual) failure surface, as described below.
8.3 Analyses and Results

8.3.1 General

The primary goal of the stability analyses was to evaluate the stability of the 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 feasible options for reconstruction of Paseo del Mar.

For the analyses, we defined the surface geometry based on the 2006 and 2011 survey contours for the before- and after-sliding conditions, respectively. We interpreted the subsurface geometry and geology as described previously. For our stability analyses, we used the cross sections shown on Plates 3 through 6.

Properties of the various geomaterials used in the slope stability analyses are presented in Table 2, below, and discussed in Appendix G. We performed sensitivity analyses to determine the influence of varying each of the input parameters (e.g., varying the material strengths presented below) and found that the groundwater level had the greatest influence on the stability of the slope, as discussed below. We used the mean value of index properties measured during laboratory testing. 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 2
MATERIAL PROPERTIES USED IN SLOPE STABILITY ANALYSES

<table>
<thead>
<tr>
<th>Unit</th>
<th>Strength Model</th>
<th>Total Unit Weight pcf</th>
<th>Friction Angle degrees</th>
<th>Cohesion psi²</th>
<th>Uniaxial Compressive Strength psi²</th>
<th>Geologic Strength Index m, mI</th>
<th>Intact Rock Parameter D</th>
<th>Disturbance Factor D</th>
</tr>
</thead>
<tbody>
<tr>
<td>Terrace Deposits (Qt)</td>
<td>Mohr-Coulomb</td>
<td>103</td>
<td>34</td>
<td>0</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>Altamira Shale (Tma)</td>
<td>Hoek-Brown⁵</td>
<td>118</td>
<td>See Note 5</td>
<td>See Note 5</td>
<td>690</td>
<td>45</td>
<td>9</td>
<td>0</td>
</tr>
<tr>
<td>Weathered Tuff</td>
<td>User-Defined Nonlinear Function⁶</td>
<td>118</td>
<td>See Note 6</td>
<td>See Note 6</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
</tbody>
</table>

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 G.
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 G.

We found that the stability of the slope was strongly influenced by the shear strength of the bentonite clay layers, and that assigning residual shear strength values based on the results of our ring shear tests (see Appendix D) resulted in failure surfaces that closely resembled the observed failure surface. Other major landslides on Palos Verdes Peninsula (e.g., Portuguese Bend, Abalone Cove) occurred along beds of bentonite clay in a similar rock-block-landslide fashion (Waltry and Lade, 2000).

For the analyses, we defined the shear strength of the bentonite clay using a nonlinear envelope as opposed to single friction angle and cohesion values (Note 6 of Table 2 above). However, it should be noted that the residual secant friction angle corresponding to the average normal stress acting on the failure surface was approximately 9 degrees, which is in the range reported by others (Stark and Eid, 1994; Watry and Lade, 2000) for bentonite clay layers within the Altamira Shale of 6 to 11 degrees. We also used a nonlinear shear strength envelope to model rock in the slope, the Generalized Hoek-Brown Strength Criterion (Hoek and Brown, 1997; Hoek and Marinos, 2000). Both nonlinear models are discussed further in Appendix G.

To “calibrate” the model of the post-landslide conditions, we back-analyzed the 2011 Landslide as discussed below. We performed stability back analyses for four cross sections,
A-A’, C-C’, E-E’, and H-H’. Additionally, we performed forward analyses for cross section J-J’, located east of the landslide, to assess the stability of the area east of the landslide. Cross section C-C’ is oriented such that it is parallel to the observed direction of landslide movement and passes through the approximate center of the landslide mass. These two conditions are inherent assumptions for limit equilibrium analysis. Therefore, we consider the results of the C-C’ analyses to be most representative of the actual landslide conditions. Likewise, cross section J-J’ is oriented approximately perpendicular to the slope east of the 2011 Landslide. Therefore, we consider the results of the J-J’ analyses to be most representative of the potential for future landsliding east of the 2011 Landslide.

Additional stability analyses for cross sections A-A’, E-E’, and H-H’ were also performed. However, these three cross sections are oriented oblique to the pre-landslide bluff slope face and the direction of landslide movement, and thus the inherent assumption of limit equilibrium slope stability analysis that the two-dimensional section be oriented parallel to the direction of movement does not apply for these sections. Hence, it is likely that a cross section passing through the same region but oriented perpendicular to the pre-landslide bluff slope face would result in a lower factor of safety (FS) than those presented herein for these three cross sections. For example, if a landslide were to occur near the area represented by cross section H-H’ (i.e., east of the 2011 Landslide), it is more likely that the movement would be perpendicular to the slope face as represented by cross section J-J’ than along the cross section H-H’ orientation.

We assumed target FS values of 1.5 and 1.1 for static and seismic conditions, respectively.

8.3.2 Back Analysis

Using the geology, hydrogeology, geometry, and material property assumptions described above, we developed models to represent the pre-landslide conditions. The models are shown in Appendix G.

Our back analyses suggest that vertical or near-vertical rock discontinuities exist near the landslide headscarp, which is consistent with our observations, and that unstable conditions could occur when the water level in these discontinuities is approximately half-way between the ground surface and the failure plane (i.e., the discontinuities are 50 percent full of groundwater), corresponding to a depth below the ground surface of approximately 50 feet. This is shallower than the post-landslide groundwater depths we have measured in our instrumented borings near
the headscarp (B-1, B-3, B-5, B-7, and B-9) of approximately 60 to 95 feet bgs (refer to Table C-1).

When the discontinuities are not included in the model, or when they are not sufficiently filled with groundwater, the results indicate marginally stable conditions. We also found that increasing the confined aquifer piezometric surface to an elevation of approximately 125 feet MSL results in instability. This is significantly higher than the highest post-landslide groundwater elevation measured in our instrumented borings (90 feet MSL in boring B-6), but is in the range reported by WCC (1997) of elevations 110 to 140 feet MSL in MW-5, MW-6, and MW-7. Post-landslide groundwater levels have likely dropped due to pressure relief from landslide movement and below average 2011-2012 rainfall season to date.

The results of our back analyses indicate that recent toe erosion did not have a significant influence on the stability of the slope compared to fluctuations in groundwater levels.

8.3.3 Forward Analyses

Using the material properties found to provide reasonable results for the back analyses with the post-landslide topography, we evaluated the stability of potential future failure surfaces. Because discontinuities partially filled with groundwater were shown to cause unstable conditions, we included the same hydrogeologic assumptions as used in the back analyses. We assumed that the discontinuities existed prior to the landslide movement and were not caused by the landslide itself (i.e., they are structural geologic features caused by regional faulting and folding), and they extend north 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, the analysis results presented above indicate that the discontinuities existed prior to the initial landslide, and not as a result of tension forces that developed concurrently with the landslide. Further, the discontinuities appear to have contributed significantly to instability. Based on the geologic information, it is likely that discontinuities similar in orientation and size exist in the subsurface north of the 2011 Landslide and could contribute to future sliding.

From our site observations, we assumed that the landslide debris mass will erode or be removed by wave action to the condition depicted in Figure G-7. 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 slope is currently more stable than prior to the landslide for the same groundwater conditions that caused instability in the back analyses. The increased stability can be attributed to the buttressing effect of the landslide debris. However, for discontinuity groundwater levels exceeding approximately 60 percent of the discontinuity height (i.e., a depth bgs of approximately 40 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 shallowest groundwater depth that we have measured in our instrumented borings is between 45 and 50 feet bgs in borings B-1 and B-6, or approximately 5 to 10 feet below the level found to cause instability for static conditions. The inclinometer at boring B-1 and inclinometers between boring B-6 and the 2011 Landslide have not shown significant movement.

We considered the stability of potential failure surfaces extending behind the forward-analysis critical failure surface in order to estimate the location of a boundary separating stable ground from potentially unstable ground. We found that the static failure surfaces extending at least 70 feet beyond the 2011 Landslide headscarp were shown to meet the target static FS of at least 1.5. However, the seismic FS required a further setback as described below.

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) for 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 G. We used a peak ground acceleration of 0.27 g (g is the acceleration due to gravity) corresponding to an earthquake with a moment magnitude (Mw) of 7.2 at a distance of 6.0 kilometers. Our analyses suggest that future landslides are likely during a design earthquake. For a design seismic event, our analyses suggest ground up to 80 feet upslope from the 2011 Landslide headscarp could move. Failure surfaces extending at least 170 feet upslope from the headscarp meet the target seismic FS of at least 1.1. The results of the stability analyses, including seismic loading, are shown in Table 3.
### TABLE 3
**SLOPE STABILITY ANALYSES RESULTS**

<table>
<thead>
<tr>
<th>Water Level in Discontinuities (% of Height)</th>
<th>Static Factor of Safety</th>
<th>Back Analysis</th>
<th>Forward Analysis, Seismic Factor of Safety&lt;sup&gt;1&lt;/sup&gt;</th>
<th>Forward Analysis, Seismic Factor of Safety&lt;sup&gt;1&lt;/sup&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cross Section A-A&lt;sup&gt;′&lt;/sup&gt;</td>
<td>No discontinuities modeled</td>
<td>1.5</td>
<td>Only back analyses performed for A-A&lt;sup&gt;′&lt;/sup&gt;.</td>
<td>Only back analyses performed for A-A&lt;sup&gt;′&lt;/sup&gt;.</td>
</tr>
<tr>
<td></td>
<td>0%</td>
<td>1.3</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>30%</td>
<td>1.3</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>50%</td>
<td>1.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cross Section C-C&lt;sup&gt;′&lt;/sup&gt;</td>
<td>No discontinuities modeled</td>
<td>1.1</td>
<td>1.3</td>
<td>NA</td>
</tr>
<tr>
<td></td>
<td>0%</td>
<td>1.1</td>
<td>1.3</td>
<td>NA</td>
</tr>
<tr>
<td></td>
<td>30%</td>
<td>1.1</td>
<td>1.3</td>
<td>0.8</td>
</tr>
<tr>
<td></td>
<td>50%</td>
<td>1.0</td>
<td>1.3</td>
<td>0.7</td>
</tr>
<tr>
<td></td>
<td>60%</td>
<td>0.9</td>
<td>1.0</td>
<td>0.6</td>
</tr>
<tr>
<td></td>
<td>100%</td>
<td>0.5</td>
<td>0.5</td>
<td>0.4</td>
</tr>
<tr>
<td>Cross Section E-E&lt;sup&gt;′&lt;/sup&gt;</td>
<td>No discontinuities modeled</td>
<td>1.1</td>
<td>Only back analyses performed for E-E&lt;sup&gt;′&lt;/sup&gt;.</td>
<td>Only back analyses performed for E-E&lt;sup&gt;′&lt;/sup&gt;.</td>
</tr>
<tr>
<td></td>
<td>0%</td>
<td>1.1</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>30%</td>
<td>1.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>50%</td>
<td>0.8</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cross Section H-H&lt;sup&gt;′&lt;/sup&gt;</td>
<td>No discontinuities modeled</td>
<td>1.4</td>
<td>1.4</td>
<td>Only static forward analyses performed for ( H-H^{'})</td>
</tr>
<tr>
<td></td>
<td>30%</td>
<td>1.3</td>
<td>1.4</td>
<td></td>
</tr>
<tr>
<td></td>
<td>50%</td>
<td>1.2</td>
<td>1.4</td>
<td></td>
</tr>
<tr>
<td></td>
<td>100%</td>
<td>1.0</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td>Cross Section J-J&lt;sup&gt;′&lt;/sup&gt;</td>
<td>No discontinuities modeled</td>
<td>N.A. (outside landslide limits)</td>
<td>1.1</td>
<td>Only static forward analyses performed for ( J-J^{'})</td>
</tr>
<tr>
<td></td>
<td>30%</td>
<td></td>
<td>1.1</td>
<td></td>
</tr>
<tr>
<td></td>
<td>40%</td>
<td></td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>100%</td>
<td></td>
<td>0.4</td>
<td></td>
</tr>
</tbody>
</table>

**Notes:**

1. For forward seismic analyses, we only considered cases where discontinuities were present and filled with groundwater to at least 30%, corresponding to the measured unconfined aquifer piezometric head.
2. See Figures G-7 and G-8 for Cross Sections C-C<sup>′</sup> and H-H<sup>′</sup>, respectively, in Appendix G.
3. NA = Not Applicable

For Table 3, 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 FS less than one 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 3 are less than one, the
physical interpretation is that a landslide would have occurred prior to the indicated slope conditions being reached.

The initial slope movement appears to have manifested in January 2010 with accelerated movement beginning in June 2011 during the dry season. Possible explanations for elevated groundwater levels during the dry season include:

- Groundwater levels at the site are strongly influenced by sources other than natural precipitation, including irrigation and pipe leakage from underground utilities, and
- Groundwater “mounds” and remains at elevated levels at the site for extended periods of time following above-average infiltration.

The groundwater monitoring instrumentation we installed at the site has shown moderate changes to the report date, which has included both moderate precipitation events and periods of no rain for several weeks. Most notably, the water level in boring B-1 has increased approximately 15 feet since the time of drilling (see Appendix C, Figure C-2). The amount of time required for the groundwater levels at the site to dissipate following an infiltration event that results in mounding could be on the order of several months to years. Hence, groundwater levels may have still been elevated when movement was detected in June 2011 as a result of the above average precipitation received during the 2010-2011 rainfall season. Supplementary infiltration from irrigation or underground utility leakage superimposed on the already elevated groundwater levels may have contributed to the slope instability as described in the previous section.

Based on cracks in Paseo del Mar observed by the City, it is possible that the 2011 Landslide mass initially mobilized at approximately the same time as the 2009 Landslide. As previously discussed, cracking in the road near the 2011 Landslide headscarp was originally reported by the City in January 2010. It is possible that undetected, minor movement occurred between the 2009 Landslide and the movement in June 2011. We have postulated three possible scenarios for the causal relationships between the 2009 and 2011 Landslides and potential impacts on the slope stability model. Additional scenarios, or variations in the scenarios we describe below, are also possible.

One possible scenario is that the 2009 Landslide occurred prior to the initial movement of the 2011 Landslide observed in January 2010. The 2009 Landslide event could have caused a decrease in the stability of the portion of the slope that would eventually be the 2011 Landslide by decreasing confinement and strength along the 2011 Landslide’s eastern flank. However, it
was not possible to directly capture this effect with the two-dimensional limit equilibrium slope stability analyses that we performed; such analyses are based on an assumption that the two-dimensional slice being analyzed represents an infinite three-dimensional prism of consistent shape.

A second possible scenario is that the initial movement of the 2011 Landslide occurred prior to the 2009 Landslide. The initial movement of the 2011 Landslide could have disturbed the slope face that then developed into the 2009 Landslide. This possible scenario has the same modeling limitations as the first possible scenario and therefore could not be analyzed directly.

A third possible scenario is that the 2009 Landslide and cracking observed in January 2010 are unrelated. The cracks in January 2010 could have occurred from an unrelated coastal bluff failure along the adjacent slope face; however, we have no information on a recent coastal bluff failure in this area. As noted previously, long-term coastal bluff erosion is a contributing factor to the 2011 Landslide that could have resulted in both the 2009 Landslide and the street cracks observed in January 2010.

The concept that the slope was marginally stable under static conditions but became unstable at elevated groundwater levels matches the observed movement pattern of the 2011 Landslide, i.e., a gradual acceleration over the course of several months as infiltration continued. Groundwater in narrow discontinuities and in communication with near surface infiltration and/or a confined aquifer could quickly rise during significant precipitation and/or irrigation due to the small volume of the discontinuity. Because water pressure in a discontinuity is a function of the height of the groundwater column and does not depend on volume, a driving force can develop quickly and cause instability. However, movement of the landslide causes the discontinuity to dilate (i.e., crack width widens), lowering the groundwater level in the discontinuity, and potentially leading to a return to stable conditions until the next infiltration event. Also, continued discontinuity dilation would manifest at the surface as ground cracks, allowing additional surface and near-surface infiltration into the discontinuity. This pattern could have repeated several times at the 2011 Landslide between the initial movement noticed in January 2010 and the rapid movement on November 20, 2011.

Groundwater levels have likely fluctuated in response to past heavy rainfall seasons at the site. Prior to the chronology of landslide events presented in this report, the last heavy rainfall season occurred in 2004-2005. To our knowledge, no landslide movement occurred during the 2004-2005 rainfall season. Based on our review of the historical precipitation data, it is our opinion that changes in the area development between about 2005 and 2010 should be further
evaluated to determine potential influence on the groundwater from surface and near-surface sources.

As previously discussed, possible non-precipitation sources could include:

- Infiltration from irrigation sources in operation after 2005 upslope or up-dip of the landslide, and
- Underground utility line leaks (e.g., storm drains below Paseo del Mar).

The data provided to us by the City thus far does not include enough information or span a sufficient time range to allow us to evaluate the full impact of non-precipitation groundwater sources such as irrigation or utility leaks. We recommend that the City perform additional evaluations to determine the potential sources for additional groundwater other than rainfall after 2005.

The results of the slope stability modeling show that failure surfaces extending at least 70 feet beyond the headscarp of the 2011 Landslide (i.e., north of the headscarp) have FS values of about 1.5 for static conditions, and that failure surfaces extending at least 170 feet beyond the 2011 Landslide headscarp have FS values of about 1.1 for seismic conditions, even if discontinuities are modeled as completely filled with water. Mitigation alternatives are discussed further in the following sections.

Our stability analyses for cross section J-J’ indicate that the area east of the 2011 Landslide is marginally stable. The following section presents recommendations for improving the stability of this area.

## 9.0 RECOMMENDATIONS

### 9.1 General

Considering the results of the stability analyses above, we recommend immediate repair work including additional subsurface explorations, cleaning and shaping of the landslide headscarp area, and constructing the dewatering, and ground anchor improvements described in the following sections to reduce the potential for future deep-seated movement and movement close to the headscarp, particularly on the eastern flank of the 2011 Landslide. Although deep-seated movement has not been detected by the City survey monitoring or by our inclinometers installed at the site since the 2011 Landslide, our analyses indicate marginally stable conditions that could become unstable with a rise in groundwater levels along the east flank of the landslide.
From our research, subsurface explorations, analyses, and observations, the groundwater regime at the landslide site and surrounding vicinity is complex. Groundwater has either risen or remained relatively level to the end of our last reading for this report in May 28, 2012 (see Appendix C, Figure C-1). The general trend is rising groundwater to the north and west of the landslide, and relatively level to the east. Given the complexity of the groundwater and its influence on the stability, lowering the groundwater levels and maintaining these lowered levels is imperative to maintain stability on the east flank of the landslide. The proposed dewatering system described in the next section will be designed to perform this function.

Additionally, the scarp of the landslide will continue to erode with shallow, planar landslides or toppling failures, presenting a potential hazard. The recommended cleaning and shaping, dewatering, and ground anchor improvements are intended to reduce the destabilizing effects of groundwater on future deep-seated movement and to reduce failures in the near-vertical headscarp and eastern flank.

We evaluated five alternative long-term mitigation options to reduce the potential of larger landslide movement and restore or reroute Paseo del Mar. Future landslide movements, triggered by high groundwater conditions, could occur during heavy rainfall seasons, as a result of excessive irrigation, and/or from design-level seismic events. Our concept-level recommendations for long-term mitigation follow our immediate cleaning and shaping, dewatering, and ground anchor recommendations. Shannon & Wilson can provide design-level recommendations for the immediate repairs and the selected final long-term mitigation option at the request of the City. All of the proposed options are subject to environmental review.

9.2 Immediate Improvements

9.2.1 General

Our analyses and observations indicate that the slopes immediately behind (i.e., the scarp north) and the bedrock slope east of the 2011 Landslide are potentially unstable. Excess porewater pressure in the confined aquifers and the hydrostatic pressure acting on water-filled discontinuities could cause additional instability. To improve immediate stability of the existing slopes, we recommend:

- Installing a series of passive, horizontal or near-horizontal drains (also known as hydraugers) through the slope face and into the confined aquifers or discontinuities filled with groundwater to relieve excess porewater pressure;
- Cleaning and shaping surface topography in the graben of the landslide to reduce infiltration by eliminating depressions, and cleaning and shaping the landslide headscarp to reduce instabilities; and
- A slope anchor system to support the intact bedrock slope on the east flank of the landslide.

In addition to these improvements, we recommend that measures be taken to formally abandon the damaged segment of Paseo del Mar until a long-term repair option is implemented. Abandonment would include new signage, road striping, and barriers for a turnaround area. If the roadway is abandoned, we recommend that turnarounds be constructed at least 170 feet away from the 2011 Landslide limits as shown in Figure 9.

Recommendations are presented for these improvements below, and estimated costs are discussed in the following section.

### 9.2.2 Dewatering Recommendations

Based on our studies described above, we recommend that the immediate dewatering improvements described in this section be applied to the intact slope adjacent to the eastern flank of the landslide as shown in Figures 10 and 11. Drains installed to the confined aquifer will flow if the discharge point is located at an elevation below the hydrostatic head measured in the aquifer at the drain inlet point. In practice, however, drains should be installed such that the discharge point is at the lowest feasible elevation to promote maximum flow and relief of excess porewater pressure. Drains should be perforated as shown in Figure 10. We recommend two layers of drains, one that reaches below the slide plane and is targeted at the confined aquifers, and an upper layer intended to intercept discontinuities filled with groundwater.

We recommend at least two additional borings (designated borings B-10 and B-11 in Figure 9) with VWPs located at specific depths within and above the confined aquifers would assist in the design and performance monitoring of the dewatering system. Shannon & Wilson’s dewatering specialists can provide design-level recommendations at the City’s request.

### 9.2.3 Cleaning and Shaping Recommendations

Erosion from rainfall and toppling- and shallow-translational-type landslides should be expected in the steep headscarp slopes. These failures can be sudden and could pose a safety hazard to individuals within the landslide mass or directly above the head scarp. We recommend that the steep headscarp on the perimeter of the landslide and on the north side of the slide mass
“island” be shaped to reduce the occurrence of such minor scarp failures. Additionally, we recommend cleaning and shaping to eliminate depressions in the graben of the landslide to reduce surface flow infiltration. Our conceptual cleaning and shaping recommendations are shown in Figure 12.

9.2.4 Slope Anchor System

To increase the stability of the eastern flank area, a slope anchor system post-tensioned tieback anchors with whalers or isolated reaction pads at the ground surface could be installed on the slope. By locating the bonded zone of the tieback anchors behind a potential failure plane, movement of the potential sliding mass would be resisted by the bond stress developed between the anchor and the stable rock.

We performed a conceptual design using the pre-landslide conditions for Cross Section C-C’, which are approximately representative of the current conditions on the eastern flank area. We determined that five rows of anchored whalers with tieback anchors spaced four feet on center would provide enough resisting force to increase the FS to 1.5 (assuming dewatering as discussed previously). As shown in Figure 12, the preliminary design consists of four strand anchors (7-wire strands per ASTM A-416) with a minimum 125-foot unbonded length and 35-foot bonded length, post-tensioned to 115 kips. Whaler beams would consist of 5.5-ft-wide continuous concrete beams embedded 2.5 feet into the ground surface to prevent a bearing capacity failure. The whaler beams would also need to be designed for structural resistance, which may govern their size.

9.3 Conceptual Long-Term Repair Options

We evaluated several long-term mitigation alternatives for restoring Paseo del Mar, including geotechnical feasibility and concept-level opinions of probable construction cost. Wagner Survey and Engineering provided a constructability review and approximate range of costs for the recommended immediate repairs discussed above and the long-term repair options discussed below as provided in Appendix H. Shannon & Wilson can provide design-level geotechnical recommendations and civil design plans for the preferred alternative at the request of the City.

Stability of the remaining slope will continue to decrease as the landslide debris is eroded by natural forces. We observed significant wave action and ongoing erosion at the base of the bluff following the landslide. Prior to implementing the proposed alternatives discussed below that involve grading (or cleaning and reshaping), we should perform further stability analyses to evaluate the influence of the removal or addition of mass on the stability of the remaining slopes.
9.3.1  **Restore Roadway**

To restore Paseo del Mar across the landslide area, the alternatives can be divided into two basic categories: replacing the road at its previous location or rerouting the road around the landslide and hazard zone.

9.3.1.1  **Reinforced Soil Slope**

To restore the road grade to its previous alignment and elevation, the void left by the landslide could be replaced with an engineered, reinforced soil slope (RSS), as shown in Figure 14. The RSS would consist of granular fill with steel or geosynthetic reinforcement layers capable of sustaining tensile stress. The reinforcement prevents internal deformation to provide internal stability, while the embankment mass acts as a resisting force to global failures passing through the upslope rock slope. Because granular fill material would be relatively free draining, hydrostatic pressures should not occur within the RSS and the fill should create a drainage boundary where constructed next to the upslope rock.

9.3.1.2  **Retaining Wall**

Another option for restoring the road grade to its previous elevation would be to construct a fill retaining wall. A feasible wall type could be an anchored soldier pile wall with both tiebacks and soldier pile foundation elements extending to competent, stable bedrock beyond the landslide failure surface as shown in Figure 15.

Temporary construction access roads would be required to facilitate equipment access to the graben area for soldier pile and tieback drilling and installation. Temporary excavation could be required below the final exposed wall base elevation to install additional rows of tieback anchors if needed for stability. The final slope should be restored as shown in Figure 15. Depending on the layout of the wall, rip rap could be required at the toe of the final slope for erosion protection against wave action.

9.3.1.3  **Bridge**

As an alternative to restoring the previous road grade, a bridge could be constructed to support the roadway across the landslide area. Locating the bridge piers outside the landslide limits could result in a span in excess of 500 feet, which would require special bridge support methods (e.g., suspension or cable-stayed bridges). A cheaper alternative would be a simply supported bridge as shown in Figure 16. The typical maximum standard bridge span is close to 300 feet, which would require intermediate piers in the landslide zone. The
foundations for intermediate piers would be designed to resist further landsliding. This would require that the foundations derive resistance from material below the landslide failure plane or outside the zone of influence of the landslide.

The California Department of Transportation (Caltrans) recently employed this mitigation technique at the Pitkins Curve Landslide on Highway 1 along the Big Sur Coast in Monterey County, California, shown in Photograph 6. A cantilevered, post-tensioned-concrete box-girder bridge was constructed to span approximately 310 feet across the active landslide at a cost of approximately $25 million (Turner and others, 2012).

![Photograph 6: Construction of 310-foot bridge spanning the Pitkins Curve Landslide, Highway 1, Big Sur Coast, California (photograph courtesy of the California Department of Transportation).]

9.3.1.4 Grading

The site could be regraded along the original alignment to accommodate the post-landslide topography as depicted in Figure 17. Excavated material from the cut slopes east and west of the 2011 Landslide could be used as fill in the graben area, resulting in a sag vertical curve. Movement of the landslide debris and corresponding settlement of the graben would likely cause continual damage to the roadway surface, requiring frequent maintenance similar to other landslide crossings on the Palos Verdes Peninsula (e.g., Palos Verdes Drive South across the Portuguese Bend landslide as shown in Photograph 7).
Photograph 7 – Palos Verdes Drive South across Portuguese Bend.
Photograph courtesy of CSU Long Beach, Department of Geological Services Website.

9.3.2 Reroute Roadway

An alternative to restoring the road to its original location is to reroute around the 2011 Landslide and the zone of potential future instability. We recommend a buffer of at least 170 feet away from the original landslide headscarp boundary (shown in Plate 1) measured perpendicular to the nearest point on the headscarp, as shown in Figure 18. We understand that gates may be constructed at each end of the proposed roadway section to allow the WPNP to close the road to vehicular access at night.

10.0 ADDITIONAL GEOTECHNICAL SERVICES

10.1 General

This report concludes our Geotechnical Design Report services as described in the TOS and our proposal. We are prepared to submit our proposal for final design of the selected repairs (including the immediate recommendations) for your review and approval. We also recommend continued monitoring of the landslide as described below.

10.2 Future Monitoring

To June 18, 2012, we have completed nine readings of our instrumentation (including the baseline reading). Per the TOS and our proposal, we have completed our monitoring services.
We recommend continued monitoring of the existing instrumentation and future instrumentation described above. Specifically, we recommend dataloggers be added to all VWPs for continuous monitoring of groundwater, particularly through the summer and the 2012-2013 rainfall season. Of particular interest is the rise in groundwater levels at Borings B-1 and B-6. We could provide results of our instrumentation monitoring in brief data reports for the City.

It is unlikely that the existing monitoring wells would contribute to future instability and therefore we do not anticipate a need to abandon the wells for this reason.

10.3 Final Design of the Repair

Shannon & Wilson should be involved in the final design engineering of the chosen alternative, consisting of the immediate repair and selected long-term mitigation option described in the previous section. We would also work with the City in the selection of the preferred mitigation option, and could review other mitigation options not included in this report, as requested.

10.3.1 Immediate Repair

For the immediate repair, our dewatering specialists can provide design-level recommendations for the drainage system and installation observation. Our engineers can also provide final design of the grading and slope anchor system. We also recommend additional subsurface explorations on the eastern flank of the landslide to support final design of the immediate repairs and to monitor their effectiveness upon construction. We recommend at least two additional borings on Paseo del Mar as shown in Figure 9:

- Future boring B-10 at or adjacent to the intersection of Weymouth Avenue, and
- Future boring B-11 midway between existing boring B-7 and future boring B-10.

These proposed borings should be drilled to a depth of at least 120 feet bgs and be instrumented with both inclinometers and VWPs. The borings should be installed and baseline readings of the instrumentation recorded prior to construction of the immediate repairs. Geologic structural data should also be obtained by either direct Downhole observation or by televiewer.

10.3.2 Selection of Long-Term Mitigation Option

Shannon & Wilson can provide design level geotechnical recommendations and civil design plans for the preferred mitigation option at the request of the City. Prior to implementing
the selected repair, we should perform further stability analyses to evaluate the influence of the removal or addition of mass on the stability of the remaining slopes.

11.0 LIMITATIONS

The analyses, conclusions, and recommendations presented in this report are based on the site conditions as observed during our reconnaissance and explorations. We assume that the soil and rock conditions observed in the explorations are representative of the subsurface conditions in all areas of the site; i.e., the subsurface conditions everywhere are not significantly different from those observed in the borings. If, during construction or additional explorations, subsurface conditions different from those described in our letter report are observed or appear to be present, we should be advised at once so that we can review these conditions and reconsider our recommendations where necessary. If there is a substantial lapse of time between the submission of this report and the start of construction, or if conditions have changed due to natural events or construction operations at or near the site, we recommend that this report be reviewed to determine the applicability of the conclusions and recommendations considering the changed conditions and time lapse.

This report was prepared for the exclusive use of the City Engineer in the design of conceptual roadway alternatives. It should be made available to prospective contractors and/or the Contractor for information on factual data only and not as a warranty of subsurface conditions. Unanticipated conditions are commonly encountered and cannot be fully determined by reconnaissance and subsurface explorations. Such unexpected conditions frequently require that additional expenditures be made to achieve a properly constructed project. Some contingency fund is recommended to accommodate such potential extra costs.

We recommend we be retained to review the geotechnical related portions of the plans and specifications to evaluate if they are in accordance with our recommendations. We also suggest a meeting with you and your contractor to discuss earthwork, drainage requirements, and other aspects of the roadway construction.
To assist you and others in understanding the use and limitations of our report, Shannon & Wilson, Inc. has prepared Appendix I, "Important Information About Your Geotechnical/Environmental Report."

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Geologic items related to geology, geologic setting, stratigraphy, and groundwater were prepared by or prepared under the direct supervision of Dean G. Francuch, P.G., C.E.G.

Geotechnical items related to the geotechnical laboratory testing, stability analyses, and conceptual engineering recommendations were prepared by or prepared under the direct supervision of R. Travis Deane, P.E., G.E.
12.0 REFERENCES


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