Final Addendum Geotechnical Report No.2
Ground Anchor Design
White Point Landslide
W.O. E1907483
Task Order Solicitation 11-087
San Pedro District
Los Angeles, California

April 17, 2013

SHANNON & WILSON, INC.

GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

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SAN PEDRO DISTRICT, LOS ANGELES, CALIFORNIA

1.0 INTRODUCTION

This report presents our geotechnical engineering recommendations for the proposed ground anchors at the White Point Landslide area. The White Point Landslide and surrounding area is shown on the Vicinity Map, Figure 1. The Site and Exploration Plan (Plate 1) shows the White Point Landslide including the proposed ground anchor area. The landslide area is composed of two distinct failures: the 2009 Landslide, and the 2011 Landslide. Additional details about the landslide area are provided in our previous reports described in Section 3.0 below.

In our final geotechnical report for the landslide, dated August 15, 2012 (Final Report), we recommended installing ground anchors to increase the stability of the eastern flank of the landslide (Eastern Flank Area). The proposed ground anchors will be installed on the south-facing bluff between the limits of the 2009 Landslide to the west, and the intersection of Paseo Del Mar and Weymouth Avenue to the east. The ground anchors will be connected to isolated concrete reaction pads or footings at the ground surface of the bluff face.

2.0 SCOPE OF SERVICES

Our scope of services is based on Task 2.5 of the Task Order Solicitation No. 11-087, dated June 18, 2012, and Subtask 2.1 of our proposal, dated August 3, 2012. The City of Los Angeles (City) Bureau of Engineering authorized our ground anchor (Task 2) scope of services on October 31, 2012. This report includes:

- Our recommendations and conclusions for the proposed slope anchor stability improvements.
- Incorporation of our previous reports (Section 3.0) by reference with the intention of using this report in combination with these previous documents.

This report is submitted to the City in conjunction with the draft 100 percent design plans and specifications. This report presents analyses results and anchor design recommendations. We retained Wagner Engineering & Survey, Inc. of Northridge, California (Wagner) to provide civil and survey support. We retained Cefali and Associates, Inc. of Pasadena, California (Cefali) to provide structural engineering support for the plans and specifications.

We issued a draft of this report with the 50 percent design plans and specifications on December 19, 2012. We incorporated comments from the City into this report, which supersedes the draft report in its entirety.

3.0 PROJECT DOCUMENTATAION

3.1 Shannon & Wilson Reports

This report is prepared in conjunction with our Final Report and our Final Addendum Geotechnical Report No. 1, dated December 19, 2012 (Addendum-1 Report). These reports include the following information relevant to the ground anchors:

3.2 Final Report:

- Research and geologic mapping;
- Subsurface explorations including logs for Borings B-1 through B-9;
- Instrumentation installed in Borings B-1 through B-9;
- Geotechnical and chemical laboratory testing of select soil and rock samples retrieved from the borings;
- Geologic and subsurface conditions encountered at the site including regional geology, descriptions of six geologic units, geologic structure and profiles, and groundwater readings to August 2012;
- Generalized subsurface profiles A-A' through J-J';
- Chronology of landslide events starting with the 2009 Landslide;
- Evaluation of six contributing factors to landslide initiation;
- Stability analyses of the 2011 Landslide;
- Preliminary recommendations for immediate improvements including dewatering, grading, and ground anchors; and,
- Five conceptual options for long-term repairs.

3.3 Addendum-1 Report:

- Research of City records on proposed residential properties at 1471, 1479 and 1481 Paseo del Mar immediately east of the ground anchor area;
- Additional subsurface explorations Borings B-10 and B-11, including logs;
- Updates of logs for Borings B-1 and B-7;
- Instrumentation installed in Borings B-10 and B-11;

- Additional instrumentation installed in Borings B-3, B-6 and B-8;
- Geotechnical and chemical laboratory testing of select soil and rock samples retrieved from Borings B-10 and B-11;
- Updates to geologic and subsurface conditions encountered at the site including revisions of geologic structure and additional profiles, and groundwater readings to December 2012;
- Generalized subsurface profiles K-K' through M-M';
- Updates to generalized subsurface profiles H-H' through J-J';
- Stability analyses of the Eastern Flank Area for dewatering and ground anchors; and,
- Dewatering evaluation and recommendations for horizontal directional drilling (HDD).

The information provided in these two reports are relevant to the ground anchor design and should be referred to for additional details as described above. This report provides details for the ground anchor design only. We did not perform additional subsurface explorations for the ground anchors design. We refer extensively to Borings B-3, B-7, B-8, B-10, and B-11 and generalized subsurface profiles L-L' and M-M' for this report.

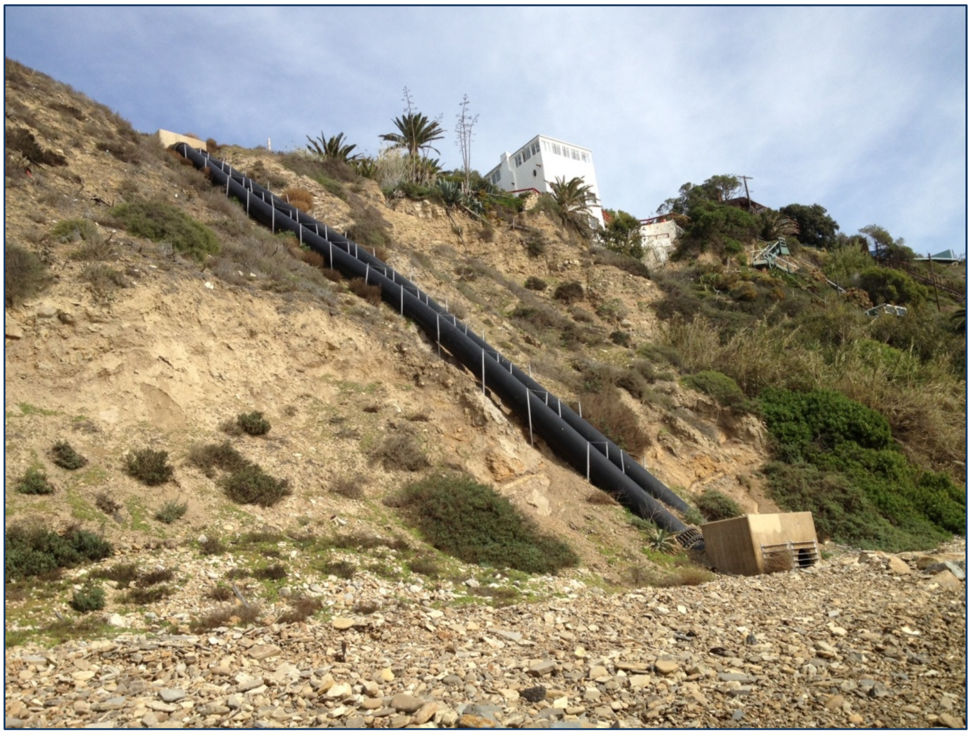
3.4 Plans and Specifications

Wagner and Cefali assisted us in preparation of the following ground anchor plan (Plan) sheets dated April 1, 2013:

- Sheet C-1.0 – General Notes and Survey Control Plan;
- Sheet C-2.0 – Specification for Ground Anchors;
- Sheet C-3.0 – Best Management Practices;
- Sheet C-3.1 – Erosion Control Plan;
- Sheet C-4.0 – Ground Anchor Plan;
- Sheet C-5.0 – Ground Anchor Cross Sections;
- Sheet C-6.0 – Ground Anchor Cross Sections, Details and Notes.

3.5 Existing County of Los Angeles Storm Drains

We obtained drawings of the existing storm drain at the site titled "Paseo Del Mar Project No. 655, Storm Drain Realignment II", dated November 1, 2011, from the County of Los Angeles Department of Public Works (County) on March 25, 2013. The drawings of the storm drains are attached in Appendix A. The storm drain on the slope at the proposed anchors consists of two 36-inch diameter high-density polyethylene pipes. The County installed the pipes during failure of the 2011 Landslide to divert the existing storm drain lines below Paseo del Mar. The new storm drain extends southward from below Paseo Del Mar, daylighting at the top of the slope and extending down the slope to the beach as shown in Photograph 1 below.



Photograph No.1: Storm Drain on Slope

At the slope crest, the pipes are embedded in a 9-foot wide by 6-foot deep reinforced concrete vault approximately 40 feet long below grade between Paseo del Mar. The invert of these pipes is at approximately El. +107' or approximately 13 feet below the ground surface. The vault connects to a new 54-inch diameter reinforced concrete pipe (RCP) below the ground surface. The 54-inch-diameter RCP extends northward into Paseo Del Mar and turns 90 degrees to the east to connect with a 72-inch-diameter RCP main storm drain at a manhole structure.

4.0 SUBSURFACE CONDITIONS

4.1 General

The soil and bedrock materials likely to be encountered consist of colluvium, terrace deposits and bedrock of the Upper Monterey Formation. Colluvium is described below. A detailed description of the Upper Monterey Formation is provided in our Final and Add-1 Reports. A brief description of the geologic units relevant to their probable behavior for the ground anchors is presented below.

4.2 Colluvium (Qc)

Colluvium is the down-slope accumulation of topsoil, weathered bedrock and other organic materials under the influence of gravity and moisture. These deposits are Quaternary age (Pleistocene and Holocene) and usually overlie bedrock and landslide debris. The colluvium is generally loose silty sand and bedrock fragments that range in thickness from negligible up to 5 feet, with the thickest deposits generally at the slope toe. Colluvium is generally confined to the bluff face, but is not shown on our geologic maps due to the localized presence and thickness of the unit.

4.3 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. 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.

4.4 Altamira Shale Member-Monterey Formation (Tma)

Where encountered by our borings, the Altamira Shale member of the Monterey Formation comprises clayey siltstone, silty sandstone, silty claystone, limey to silicified siltstone, sandstone, and bentonite beds. The rock is typically thinly bedded to laminated, and contains some tar along fractures and in brecciated zones. Gypsum, caliches, and minor sulfur deposits exist along fractures within the upper oxidized zone. The upper, exposed shale is highly weathered and the weathering decreases with depth. Siliceous layers are present at depths 50 feet below the road surface grades or deeper (about Elevation +70 feet). Note that the siliceous beds are considerably harder than the surrounding shale beds. The siliceous beds caused a significant reduction in drilling rates during our exploration program.

From a rock/soil strength standpoint, the weakest material observed in the borings consists of the bentonite clay beds. Two- to five-inch-thick bentonite beds were observed in borings at depths between 10 and 39 feet and between 88 and 97 feet. The bentonite beds encountered between 88 and 97 feet are highly polished, soft, wet, and generally slightly discordant to bedding. The bentonite beds encountered above 39 feet were folded; however, they did not exhibit polished and slickensided surfaces.

4.5 Groundwater

A detailed description of the groundwater conditions in the Eastern Flank area was completed in our Addendum-1 Report. The groundwater piezometric surface consists of a variable surface generally ranging from about 14 feet to 105 feet below the ground surface (see Plate 2). We have installed vibrating wire piezometers (VWPs) as part of our instrumentation installations at borings B-7, B-10, and B-11. The VWPs are attached to dataloggers that record groundwater levels every hour. Plots of the groundwater elevations recorded in the VWPs are provided in Figure 2. Refer to our Final and Addednum-1 Reports for additional information on the VWPs.

5.0 GROUND ANCHORS FOR LANDSLIDE STABILIZATION

5.1 General

Ground anchors are commonly used in combination with walls, horizontal beams, or anchor blocks to improve stability of slopes and landslides. We proposed anchor blocks as shown in the Plans. An example of anchor blocks is shown in Photograph 2 below. Pre-stressed ground anchors act against the thrust of the potential slip surface and increase the normal stress on the potential slip surface. Both of these actions contribute to increase stability of the slope. Anchored slopes and landslide stabilization systems are designed to restrain forces associated with unstable ground masses. Limit equilibrium analyses are used to evaluate ground anchor loads for anchored slopes and landslide stabilization systems.

Photograph No. 2: Anchored concrete panels (blocks),
Stone Point Landslide, Spring Valley, California
(Source: FHWA-WY-03/03F)

5.2 Design Approach

The target slope stability factor of safety for anchored slope systems is typically 1.5 for static condition. Higher values, although not common, may be required depending on the criticality of the structure, requirements with respect to deformation control, and confidence in the selected shear strength parameters. When analyzing slopes and landslides, the factor of safety should be calculated for critical potential failure surfaces since several surfaces (both planar and circular) may have factors of safety less than the target value.

Information on groundwater (i.e. porewater pressures) is necessary for slope stability analyses of anchored systems. The available piezometric data for each water-bearing zone must be evaluated if possible; however, these hydraulic heads are likely to change as a result of seasonal changes in precipitation, construction activities, or irrigation that could change or interrupt water flow paths. We used the computer program SLOPE/W version 7.17 (Geo-Slope International, 2007) to perform two-dimensional, limit equilibrium stability analyses of potential future slope failures in the Eastern Flank Area. Details of our initial stability analyses including dewatering are provided in our Final and Addendum-1 Reports. We have used these analyses to evaluate the increase in stability of the Eastern Flank Area after installation of ground anchors. The analyses were used to develop selection of anchor type, loading, and configuration.

5.3 Geology and Hydrogeology

The geologic and hydrogeologic conditions are described in our Final and Add-1 Reports. For the analyses, we assumed that bedding dips out of the slope as shown in the generalized subsurface profiles L-L' and M-M' (Plates 3 and 4) and that bentonite clay is present on the failure surface. We assumed potential new slope failures in the Eastern Flank Area would move along similar bentonite surfaces as the 2011 Landslide.

We also assumed that vertical or near-vertical rock discontinuities exist near the potential landslide headscarp, which is consistent with our observations of the 2011 Landslide. Unstable conditions would occur when the water level in these discontinuities has risen to approximately half-way between the ground surface and the basal failure surface.

The groundwater data collected from our previous studies demonstrates a complex system of confined and unconfined zones in the Eastern Flank Area. For our slope stability analyses, we modeled the groundwater at the level of the horizontal dewatering wells that are currently being installed as of this report date.

5.4 Slope Geometry

We defined the surface geometry based on the ground contour survey following the 2011 Landslide, by the City of Los Angeles, dated December 20, 2011. We interpreted the subsurface geometry and geology as described in our Final and Addendum-1 Reports. For our stability analyses, we used the generalized subsurface profiles L - L' and M - M'.

We performed forward analyses for generalized subsurface profile L-L' to assess current and future stability after installation of the dewatering and ground anchor systems. Profile L-L' is oriented approximately perpendicular to the slope. Profile M-M' is oriented approximately parallel to the true dip direction of the bedding. The results of the stability analysis and direction of the bedding suggest a failure along profile L-L' is slightly more critical than a failure along profile M-M'. Therefore, we consider the results of the L-L' analyses to be more representative of the potential for future landsliding in the Eastern Flank Area.

5.5 Soil and Rock Properties

Properties of the geo-materials used in the slope stability analyses are presented in our Addendum-1 Report and are reproduced in Table 1. We used the mean value of index properties for soil. Input parameters for rock were modified based on our findings in borings B-10 and

B-11 and subsequent geotechnical testing of samples. For materials that could not be adequately characterized by laboratory testing performed at discrete sampling intervals, and in the case of qualitative rock properties such as geologic strength index (GSI), a certified engineering geologist estimated the parameters needed for slope stability analyses. As discussed in our Final Report, the slope stability was strongly influenced by the shear strength of the bentonite clay layers. We assigned residual shear strength values based on the results of our ring shear tests. For the Altamira Shale, we used a nonlinear shear strength envelope to model rock in the slope, as described by the Generalized Hoek-Brown Strength Criterion (Hoek and Brown, 1997; Hoek and Marinos, 2000).

5.6 Seismicity

For seismic analyses, the maximum horizontal acceleration (MHA) at a 475-year hazard level (10 percent probability of exceedance in 50 years) and the pseudo-static horizontal seismic coefficient (k_{eq}) were determined using the procedure from "Recommended Procedures for Implementation of CDMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazard in California" by the Southern California Earthquake Center (2002). We determined the maximum horizontal acceleration at the site to be equal to 0.27g (g is acceleration due to gravity) which corresponds to an earthquake with a moment magnitude (Mw) of 7.2 at a distance of 6.0 kilometers (4 miles). The maximum horizontal acceleration was then adjusted to correspond to a pseudo-static horizontal seismic coefficient (k) of 0.16g for slope stability analysis.

5.7 Ground Anchors

Ground anchors, consisting of seven-wire strands designated as ASTM A416 (Standard Specification for Steel Strand, Uncoated Seven-Wire for Prestressed Concrete) were assumed to be used in slope stabilization. The ultimate tensile strength of strand is 270 kips per square inch (ksi). We assumed the horizontal spacing between anchors was 20 feet. We analyzed two cases of anchor configurations. Case 1A consists of two rows of anchors on the slope at El.+110 and El. +90. Case 1B consists of two rows, one at El. +110 on the slope and another on the level ground surface on the top of slope. Calculations for design of the ground anchors are provided in Appendix B.

5.8 Stability Analyses and Results

Using the geology, hydrogeology, geometry, and material property assumptions described above, we developed the input for slope stability models in SLOPE-W to evaluate the slope stability. We assumed that vertical or near-vertical discontinuities exist throughout the Eastern Flank Area (i.e., they are structural geologic features caused by regional faulting and folding). 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.

We performed forward analysis for two conditions: static and seismic loading for Sections L-L’ and M-M.’ The results are summarized in Table 2 and shown graphically for Case 1A on Figures 3 and 4 for Sections L-L’ and M-M’, respectively. For Case 1B, the results are shown graphically on Figures 5 and 6 for Sections L-L’ and M-M’, respectively. For static conditions, the results indicate that the factor of safety against landslide failure increases to above the target factor of safety of 1.5 for these sections.

For seismic loading, the slope displacement is about 24 cm (10 inches) for the current slope condition prior to the installation of horizontal dewatering system. After the dewatering system is installed, the estimated slope displacement would be reduced to 13 cm (5 inches). After the installation of the dewatering and the modeled ground anchor systems, the estimated slope displacements would range between 4 and 9 cm (2 to 3-1/2 inches). Based on the criteria from the CDMG Special Publication 117, the allowable slope displacements due to seismic loading are 6 cm (2½ inches) for slip surfaces that intersect engineered improvements (e.g., buildings), and 15 cm (6 inches) for slip surfaces that do not intersect engineered improvements (e.g. roads and landscaped areas). Hence, the proposed dewatering and ground anchors will limit the slope displacement due to the design seismic event within these criteria.

6.0 DESIGN RECOMMENDATIONS

6.1 Ground Anchors

Based on the results of our analyses, we recommend installing two rows of ground anchors to improve stability of the Eastern Flank Area. The design load on each anchor is 210 kips. Our design recommendations include:

- Each row consists of nine ground anchors with a center-to-center spacing of 20 feet.

- Each anchor should consist of 6 seven-wire strands designed in accordance with ASTM A416 (Standard Specification for Steel Strand, Uncoated Seven-Wire, Stress-Relieved Strand for Prestressed Concrete).
- All anchors will incorporate the PTI Class I double corrosion protection (highest protection). All wedge plates will be electro-zinc coated per ASTM B633. All bearing plates, trumpets and steel end caps will be either hot dip galvanized per ASTM A153 or epoxy coated per ASTM A775.
- A minimum diameter grout hole of 6 inches.
- The anchor inclination should be 45 degrees from horizontal.
- The first row of anchors shall be installed at the relatively level ground surface at the top of slope at elevation 120±. The second row shall be installed approximately El. +110 feet on the slope (Case 1B configuration).
- The minimum bonded length shall be 35 feet into the competent Altamira Shale below the assumed bentonite clay layer.
- The total lengths of anchor are at least 165 and 160 feet for the first (upper) and second (lower) rows, respectively.

To satisfy seismic conditions described previously, we assume engineered improvements such as buildings will not be constructed in the Eastern Flank Area. If there are structures (e.g. building, bridge, etc) planned to be constructed in the area in the future, the allowable ground displacement per CDMG Special Publication 117 will be 6 cm (2 ½ inches). These structures should be designed to accommodate the estimated seismic displacement.

6.2 Anchor Installation

Anchor holes should be drilled in a manner that would minimize ground loss and not endanger previously installed dewatering drains or undermine existing pavement, storm drains or, other underground utilities. The Contractor should be prepared to drill through and install anchors in the fractured bedrock with hard siliceous layers. Casing and down-the-hole hammer may be needed to penetrate localized harder beds.

In the anchor no-load zone, anchor holes could be filled with a material such as a sand pozzolan mixture that would not adhere to the anchor strand while preventing caving. We recommend that no-load zone lengths not be left open overnight. Alternatively, a bond breaker could be used around the strands in the no-load zone, and the zone could be filled with concrete or lean concrete backfill. Double corrosion protection will be required for the permanent anchors.

6.3 Anchor Load Testing

The frictional resistance of an anchor is dependent on many factors including, local bedrock strength, the Contractor's method, and care during installation. Consequently, the length of production anchors should be based on a series of test anchors. Anchor grout should be placed by tremie method. An allowable load transfer rate for a single-stage pressure-grouted anchor of 6 kips per linear foot (klf) is estimated for a 5- to 8-inch-diameter borehole.

Prior to installing production anchors within a particular soil stratum, performance tests should be accomplished for each anchor type and/or installation method that would be used. The need to install pre-production test tiebacks would depend on the location, quantity, and number of tieback anchors. The number of strands in the selected anchors should be increased as required to complete the performance tests. Approximately 5 to 10 percent of production anchors, randomly selected, should be performance tested by loading in 25 percent (0.25P) increment to 133 percent of design capacity (1.33P). The 133 percent load should be held constant for minimum of at least 10 minutes. We recommend all anchors be locked off at 110 percent of the design load to compensate for long-term anchor relaxation. Anchors that do not meet the testing criteria recommended herein should be locked off at 50 percent of the failing load and replaced with additional anchors, as required.

Load testing for all ground anchors and acceptability should be as recommended by the Post Tensioning Institute Manual, Chapter 4, Recommendations for Prestressed Rock and Soil Anchors (2005). As described in this manual, the following tests should be accomplished.

6.3.1 Initial Lift-off Readings

After transferring the load to the stress anchorage and prior to removing the jack, a lift-off reading should be made. The load determined from the lift off reading should be within 5 percent of the specified lock-off load. If the load is not within 5 percent of the lock-off load, the end anchorage should be reset and another lift-off reading should be made.

6.3.2 Lift-off Test

Lift-off tests should be conducted on selected anchors, both during and after construction, to check the magnitude of seating and transfer load losses and to determine whether long-term losses are occurring.

6.3.3 Acceptance Criteria

The results of each anchor test should be evaluated by Shannon & Wilson in order to determine anchor acceptability. An anchor would be acceptable provided:

- The total movement obtained from a performance and proof test exceeds 80 percent of the theoretical elastic elongation of the design free-stressing length.
- The creep rate during the final test load does not exceed 0.080 inch per log cycle of time and is a linear or decreasing creep rate, regardless of tendon length and load. Otherwise, the anchor should be held for an additional 60 minutes at the required test load.

6.4 Instrumentation

We recommend instrumentation to monitor force in the strands along the length of the anchor. We have reviewed several load measuring instruments (e.g. strain gage, etc) for this application. We recommend the DYNA Force® electro-magnetic sensors be installed along the anchor to monitor the load distribution in the anchor during the performance test and for long-term monitoring of the anchor performance and future slope movement. The extended length of the unbonded zone of the proposed anchors could result in frictional losses that may affect the performance testing results. The load sensors will help in determining the amount of losses, so the anchor can be properly evaluated during the performance tests. Once testing is complete, the load sensors could be used to monitor for the long-term performance of the anchors and impacts from possible slope movements. The detailed description and application of DYNA Force® sensor is attached in Appendix C of this report.

6.5 Toe of Slope Protection

We understand the City intends that the toe of slope be protected from erosion from wave action by rip-rap. We recommend the rip-rap consist of quarried rock with an average unit weight of 162 lbs per cubic foot, sub-angular, in double layer with an average diameter of 3 feet (D_{50} size). The rip-rap should be keyed below the beach deposits (consisting of cobbles) to a depth of at least two feet or firm soil/bedrock, whichever is shallower.

Alternatively, a Reno mattress may be used in lieu of rip-rap. The Reno mattress should consist of woven wire mesh filled with rock with the dimensions and specifications similar to the rip-rap noted above. The Reno mattress should be constructed as a continuous section, designed in accordance with ASTM A975-97, and supplied by a qualified and experienced manufacturer.

Rip-rap or a Reno mattress should be designed to resist wave action in accordance with the U.S. Army Corps of Engineers Coastal Engineering Manual by a civil engineer registered in the State of California. The design shall be submitted to the City of Los Angeles Bureau of Engineering and Shannon & Wilson for review and approval prior to construction.

7.0 GEOTECHNICAL OBSERVATION

During the construction of the ground anchors, our field representative should be present at the site to observe the construction activities. Our representative should be on site full-time to observe the installation, testing and locking-off of the ground anchors. Our representative should be assigned the following duties:

- Observe the drilling operation;
- Observe the placement of steel tendons and grout;
- Observe subgrade for concrete bearing panels and anchorages; and,
- Observe the performance tests, proof tests and locking-off loads of ground anchors.

8.0 LIMITATIONS

This report was prepared for the exclusive use of the City of Los Angeles for specific application to this project.

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

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

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

SHANNON & WILSON, INC.

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