Slope Stability Evaluation
Ground Anchor Construction Area
White Point Landslide
San Pedro District
Los Angeles, California

July 1, 2014

SHANNON & WILSON, INC.

GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

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July 1, 2014

City of Los Angeles
Bureau of Engineering
1149 South Broadway Street, Suite 120
Los Angeles, California 90015

Attn: Mr. Gene Edwards

RE: SLOPE STABILITY EVALUATION, GROUND ANCHOR CONSTRUCTION AREA, WHITE POINT LANDSLIDE, SAN PEDRO DISTRICT, LOS ANGELES, CALIFORNIA

This report provides our conclusions and recommendations regarding the slope stability along the existing construction bench excavated by Hayward Baker, Inc. (HBI) for the construction of the ground anchors east of the White Point Landslide (Landslide). The Landslide is located as shown in the Vicinity Map, Figure 1. This report refers to Shannon & Wilson's previous conclusions and recommendations described in the following Landslide correspondence with the City of Los Angeles Bureau of Engineering (City):

- Final Geotechnical Report, August 15, 2012 (Final Report).
- Final Addendum Geotechnical Report No. 1, December 19, 2012 (Addendum-1 Report).
- Final Addendum Geotechnical Report No. 2, April 17, 2013.
- DRAFT Data Report for White Point Landslide, Boring B-12, dated April 24, 2013 (Boring B-12 Report).
- DRAFT Geotechnical Construction Observation Report, White Point Landslide Dewatering Drains, July 25, 2013.
- Field Activity Reports during ground anchor construction (FARs).

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SCOPE OF SERVICES

The purpose of our services was to evaluate the potential for earth movement downslope of open cracks exposed in the construction bench for the ground anchors. Details of our scope of services are provided in our proposal dated November 4, 2013, which the City authorized on November 5, 2013.[1] To evaluate potential earth movement in the construction bench, we performed the following geologic and geotechnical engineering services

- Geologic mapping of the existing bench excavation and bluff;
- Logging test pits excavated across the ground crack;
- Crack monitoring;
- Slope stability analyses, and;
- A summary of our findings in this report.

EXISTING CONDITIONS

Details of the existing ground conditions are provided in our field activity reports (FARs) for the ground anchor construction. Selected FARs that discuss the ground crack are provided in Appendix A and include:

- Review of ground crack and placement of first monitoring points on October 28, 2013 (FAR 1);
- Measurements of first monitoring points on October 29, 2013 (FAR 2);
- Measurements of first monitoring points and additional crack mapping on October 30, 2013 (FAR 3);
- Measurements of first monitoring points on October 31 and November 1, 2013 (FARs 4 and 5, respectively);
- Measurements of first monitoring points and additional crack mapping on November 4, 2013 (FAR 6);
- Measurements of first monitoring points and test pit excavations on November 5 and 6, 2013 (FARs 7 and 8, respectively);
- Installation of second monitoring points on November 7, 2013 (FAR 9);
- Measurements of second monitoring points on November 10, 2013 (FAR 10);

[1] City directed HBI, who we were under contract with, to authorize our proposal on this date.

- Installation of extensometers on December 4, 2013 (FAR 24);
- Review of second ground crack on January 14, 2014 (FAR 47), and:
- Additional crack mapping and test pit for second ground crack on February 12, 2014 (FAR 59).

HBI began excavating the bench for construction of the ground anchors during the week of October 21, 2013 at the location shown in the Site Plan, Figure 2. The completed temporary bench is bounded on the north by a 1 horizontal:1 vertical (1H:1V), 8-foot high slope. The bench is about 25 to 30 feet wide, 180 feet long and is at approximate Elevation +114 feet. The construction bench is relatively level. The natural bluff face below the bench descends to the beach with an average 1H:1V slope.

On October 28, 2013 following the excavation of the construction bench, Nancy Smith of HBI noted a linear crack or fracture approximately 6 feet in length and 2 to 3 inches in width that trended across the construction bench. Shannon & Wilson visited the site on the same day to measure and record the location and width of the fracture (FAR 1). During our initial observations (FARs 1 through 6), we measured the open portion of the crack, which was approximately 9 feet long and 0.5- to 1.5-inches wide. A portion of the crack was soil-filled and extended approximately 18 feet from the east end of the open crack, where it was obscured by surficial materials, as shown in Photograph 1 below.

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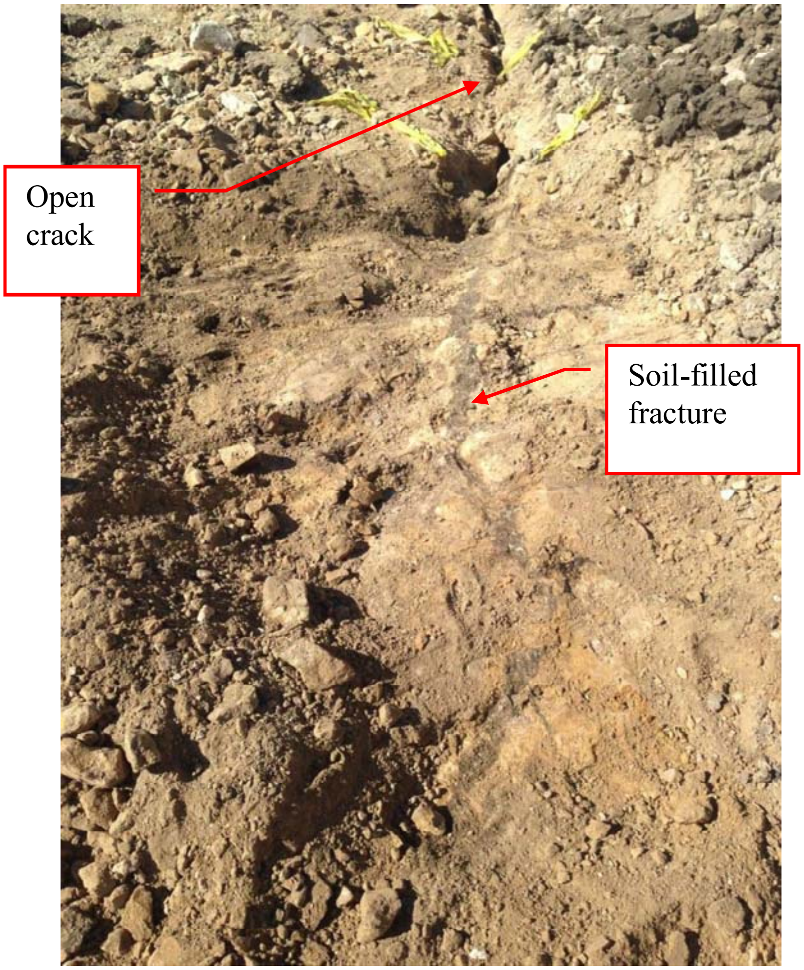

Photograph 1 – View to the west of the open crack on the construction bench.

A soil-filled portion of the crack was present on the western wall of the construction bench excavation as shown in Photograph 2.

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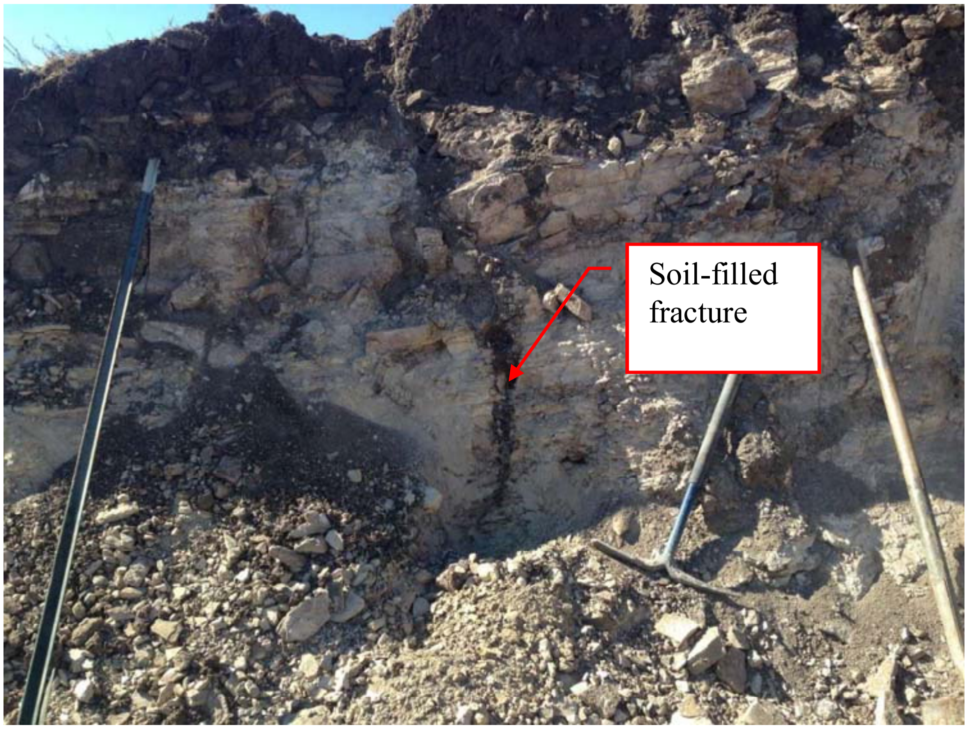

Photograph 2 - Soil-filled fracture in the western cut for the construction bench. The view is looking west with the construction bench access slope to the right of the photograph and the ocean to the left.

A second crack was exposed during excavation of anchor pad E-1 on January 13, 2014. We reviewed this crack the following day as described in FAR 47. We completed additional explorations of this ground crack on February 12, 2014 as described in FAR 59.

FIELD EXPLORATIONS

Shannon & Wilson completed additional geologic mapping of the first ground crack along the bench. On November 6, 2013, two 12-foot deep test pits (designated Test Pits TP-1 and TP-2) were excavated with a backhoe across the crack on the construction bench surface as shown in the Site and Exploration Plan, Figure 3. Details of the test pit excavations are provided in FAR 8 and the logs are presented herein (Appendix B). Our engineering geologist logged the test pits and noted the crack and associated were nearly vertical in and below the base of both test pits. Altamira Shale bedrock comprised the test pit walls.

Much of the crack we observed was filled with soil. The soil appears to be derived from the terrace deposits that overlie the Altamira Shale bedrock. Relative displacement of the bedrock units on both sides of the fracture indicate movement was parallel to bedding planes and towards the ocean. We did not observe evidence of vertical displacement. Where soil was noted in the crack or fractures, the soil appears to be relatively dense and in close contact with the adjacent bedrock. Plant roots were present in the soil-filled crack within test pit TP-1 to a depth of approximately 14 feet below the original ground surface.

Stability Analyses

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.

The slope stability analyses for this project should be used to understand changes in stability of the slope below the construction bench from previous and current construction activities for the ground anchors. In our opinion, the bluff stability has improved because of the installed dewatering drains that are described in our Addendum-1 Report. The stability is being further improved with the ground anchors installation,

The landslide area is composed of two distinct failures: the 2009 Landslide and the 2011 Landslide. Our Final Report describes the chronology and failure modes of the 2009 and 2011 Landslides. Our Final and Addendum-1 Reports used the 2011 Landslide failure as a model to complete the stability analyses. For the construction bench stability analyses of the ground cracks, it is our opinion that the 2009 Landslide is a better model for the stability analyses presented below based on:

- Proximity to the 2009 Landslide;
- Similar geometry of the ground cracks relative to the scarp of the 2009 Landslide;

- Similar geologic conditions, and;
- Similar failure mode.

There may be a causal relationship between the 2009 Landslide and the ground cracks observed in the construction bench. However, there is insufficient data to make a conclusive statement. From our geologic interpretation, the ground cracks likely formed during or prior to the 2009 Landslide.

We used the computer program SLOPE/W version 7.17 (Geo-Slope International, 2007) to perform two-dimensional, limit equilibrium stability analyses of potential failure modes that could form the ground cracks. The following sections describe the assumptions we made to model the landslide, including geology, landside geometry, and soil parameters. We developed our stability models using our interpretation of the surface and subsurface data presented in this report and in our Final and Addendum-1 Reports.

Geology and Hydrogeology

Following our field explorations, we revised generalized subsurface profiles from our Addendum-1 Report, designated J-J' and K-K' as shown in Figures 4 and 5, respectively (see location, Figure 3). The profile legend is shown in Figure 6. For the analyses, we assumed that bedding dips out of the slope as shown in Profiles J-J' and K-K', and that bentonite clay is present on the failure surface. We assumed potential new landslides would fail in a similar manner to the 2009 Landslide.

We assumed that the ground cracks form because of small ground movement when the groundwater level was higher, and because of stress relief caused by coastal bluff retreat. Our groundwater data demonstrates a complex system of confined and unconfined zones is present as described in our Final Report. That report shows historical groundwater data as high as 14 feet below the ground surface (bgs). During drilling following the 2011 Landslide, we installed vibrating wire piezometers (VWPs) to monitor groundwater levels. Groundwater in the VWPs recorded water pressures as high as about +107 feet at Boring B-10. The piezometric head recorded at this VWP, and historically suggests the groundwater could fill a ground crack to this elevation, assuming hydraulic connection to the aquifer. We modeled one piezometric surface representing an unconfined zone condition similar to our back analysis in our Final and Addendum-1 Reports.

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Analyses and Results

We assumed the critical failure surfaces begin in the newly-mapped cracks, extend down to and then slide along one of beds shown in the profiles. From the orientation of the ground crack, it appears that the movement is closer to down slope or towards the ocean (Profile J-J') than parallel to geologic dip direction (Profile K-K'). The following assumptions were made for the analyses:

- Current groundwater levels were modeled using VWP data following installation of the horizontal directional drilled (HDD) drain pipes.
- The geologic structure was updated with our recent geologic mapping.
- Bentonite beds are present at or near the locations shown on the attached cross sections.
- For shear strength of bentonite, we assumed that drained conditions existed because of the relatively slow moving nature of the landslide, and that the residual shear strength conditions had been reached due to recent movement and previous displacement to accommodate inter-layer slip during folding. We modeled the clay using a nonlinear envelope defined by a series of torsional ring shear tests (ASTM D7012).
- The temporary construction bench is present.

Our analyses show factors of safety for current conditions that range from 1.14 to 2.39. The factor of safety is affected mostly by the depth of the failure plane (sliding surface), inclination of the failure plane, and the groundwater elevation. Details of the slope stability analyses are included in Appendix C.

Grading for the construction bench removed approximately 8 feet of overburden above the ground cracks. Comparing stability models with and without the construction bench showed a 5 percent increase in factor of safety caused by excavation of the bench. HBI constructed temporary drilling platforms that will cantilever over the bluff slope to construct the lower row of ground anchors. Our stability analyses show a 1 to 2 percent decrease in factor of safety caused by the weight of the temporary drilling platform. We modeled the weight of the temporary platform only, and did not include shear resistance of the four 50-foot long micropiles that will support each temporary platform. Therefore, we consider our analyses conservative, because they neglect the deeper load transfer and shear resistance the micropiles create.

CONCLUSIONS AND RECOMMENDATIONS

Past Movement

It is our opinion that the slope adjacent to the ground cracks has not moved recently. The presence of soil filling the cracks and roots in the soil shows movement has not occurred in the time required for the soil fill to occur plus the time for root growth. The 14-foot root depth suggests movement has not occurred for a decade or more. If recent movement had occurred, we would expect to see cracks in and adjacent to the soil. The extensometers and monitoring stakes placed across the crack (described in FAR 24) have not detected dilated or tensional movement across the crack since installation.

Current Stability

We believe that the overall factor of safety has increased because of the HDD drain construction and excavation of the construction bench. These activities raised the factor of safety by: 1) lowering the groundwater levels; and 2) reducing the driving force from soil and rock removal. The current drought conditions have further reduced potential driving forces associated with groundwater.

The two-dimensional stability analyses used to estimate the factor of safety for the overburden and platform underestimates three-dimensional differences between the overburden and smaller platform. The overburden removal created by the construction bench excavation relieved driving pressure over almost 180 linear feet of the slope. The drilling platforms will add temporary drive pressures over an area that is approximately 10 feet wide. Therefore, the additional driving pressures from the platform are confined to a localized area of the slope around the platform. The driving pressures from the removed overburden applied to an area of the slope about 50 times larger than the platform.

In our opinion, the probability of slope failure from the ground cracks during ground anchor construction is low.

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.

Please call if you have any questions.

Sincerely,

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Enc: References
Figure 1 – Vicinity Map
Figure 2 – Site Plan
Figure 3 – Site and Exploration Plan
Figure 4 – Generalized Subsurface Profile J-J'
Figure 5 – Generalized Subsurface Profile K-K'
Figure 6 – Generalized Subsurface Profile Legend
Appendix A – Selected Field Activity Reports
Appendix B – Logs of Test Pits
Appendix C – Slope Stability
Appendix D - Important Information About Your Geotechnical/Environmental Report

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REFERENCES

Shannon & Wilson, Inc., 2012a, Final Geotechnical Report, White Point Landslide, San Pedro District, Los Angeles, California, W.O. E1907483, Task Order Solicitation 11-087, prepared for City of Los Angeles Geotechnical Engineering Group, Los Angeles, California., report dated August 15, 2012.

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