Paseo Del Mar (White Point) Permanent Restoration Program
Table of Contents

1 Introduction ............................................................................................................................................. 1
2 Background ............................................................................................................................................. 1
  2.1 Existing Facility ............................................................................................................................. 1
3 Project Scope ......................................................................................................................................... 2
4 Design Criteria ........................................................................................................................................ 2
5 Options Analysis ..................................................................................................................................... 3
  5.1 OPTION 1 – Bridge to Span over Landslide ................................................................................. 4
    5.1.1 Aesthetics ........................................................................................................................ 5
    5.1.2 Cost .................................................................................................................................. 5
    5.1.3 Maintenance .................................................................................................................... 6
    5.1.4 Requirement for Shore Line Protection (Riprap) ............................................................. 6
  5.2 OPTION 2 – Anchored CIDH Piles with Buttress ......................................................................... 6
    5.2.1 Aesthetics ........................................................................................................................ 6
    5.2.2 Cost .................................................................................................................................. 6
    5.2.3 Maintenance .................................................................................................................... 7
    5.2.4 Requirement For Shore Line Protection (Riprap) ............................................................ 7
  5.3 OPTION 3 – Shear Pins with MSE Wall ....................................................................................... 7
    5.3.1 Aesthetics ........................................................................................................................ 7
    5.3.2 Cost .................................................................................................................................. 7
    5.3.3 Maintenance .................................................................................................................... 7
    5.3.4 Requirement for Shore Line Protection (Riprap) ............................................................. 8
6 Recommended Project ........................................................................................................................... 8
7 Schedule ............................................................................................................................................... 10
8 Construction Budget ............................................................................................................................. 10
9 QA/QC Program ................................................................................................................................... 10
10 Staffing Recommendation .................................................................................................................... 10
11 Design Development ............................................................................................................................ 11
  11.1 Roadway / Community Considerations ....................................................................................... 11
  11.2 Dredging / Location of Embankment Fill Sites ............................................................................ 11
  11.3 Haul Routes ................................................................................................................................ 11
  11.4 Permitting .................................................................................................................................... 12

List of Appendices

APPENDIX A – Option 1
APPENDIX B – Option 2
APPENDIX C – Option 3
APPENDIX D – Draft Geotechnical Memorandum
APPENDIX E – Structures Report
APPENDIX F – Wave Run-Up Study
APPENDIX G – Conceptual Striping Plans
APPENDIX H – Material and Dump Facilities
PROGRAM MANAGER APPROVAL:

This document hereby confirms the Paseo Del Mar (White Point) Permanent Restoration Project of the BUREAU OF ENGINEERING, GEOTECHNICAL ENGINEERING GROUP have discussed, reviewed, approved the direction of work, scope, alternative selection, and design parameters of this Pre-Design Report.

GEOTECHNICAL ENGINEERING GROUP
BUREAU OF ENGINEERING

[Signature]
Christopher F. Johnson, P.E., G.E. Group Manager

[Date]
6-23-16
Paseo Del Mar (White Point) 
Permanent Restoration Project 
PRE-DESIGN REPORT 
MAY 2016

This report is prepared under the direction of the following registered civil 
engineer:

Hilton L. Yee, P.E., 
AECOM 
915 Wilshire Blvd, Ste 700 
Los Angeles, CA 90017
1 Introduction

This Pre-Design Report will serve as a basis for the final design of the Paseo Del Mar (White Point) Permanent Restoration project. This report provides an overview of three options to restore the roadway that collapsed during the 2011 landslide event.

2 Background

On November 20, 2011 an approximately 400 feet stretch of roadway along the coast in the community of San Pedro in the City of Los Angeles at the Palos Verde Peninsula collapsed during the 2011 White Point landslide. This section of W Paseo Del Mar is approximately 120 feet above the sea along a steep bluff overlooking the Pacific Ocean. A large block of the bluff containing the roadway moved approximately 60 feet toward the ocean and left a large depression approximately 400 feet long by 60 feet wide by 40 feet deep where the roadway used to exist.

W Paseo Del Mar is a two lane scenic highway with shoulders on both sides, a bike path, and sidewalk along the bluff side. The City of Los Angeles initiated studies, cleanup, stabilization of the eastern adjacent slope and introduced a temporary street turn-around at the eastern end to close the road until a permanent solution is determined. The western end is fenced off.

City of Los Angeles Council District 15 formed a task force to identify and evaluate alternatives for a permanent solution. The task force recommended that the roadway be restored as opposed to being permanently closed or diverted into the White Point Nature Preserve.

Funding for the intermediate repairs was approved by City Council (09/18/2012 (CF -0600-S162) and 05/28/2013 (CF-13-0469). In July 2015, the City authorized an expenditure of up to $2 million for the Pre-Design phase of the permanent Restoration project utilizing funds from the Gas Tax Fund No. 206, Department No. 50, Appropriation Unit No. 50LKDM, “Paseo Del Mar at White Point Landslide.”

Funding for final design and construction has not yet been determined.

2.1 Existing Facility

West Paseo Del Mar, at this location between Weymouth Avenue to the east and Western Avenue to the west is designated by the City as a scenic highway. The 2035 Mobility Plan, which is still to be adopted, classifies this route as an Avenue 2 / scenic highway with a curb to curb roadway width of 46 feet and 13 feet wide sidewalks. The current Right of Way (ROW) street width is 70 feet. West Paseo Del Mar is bounded by the White Point Nature Preserve owned by the City of Los Angeles Department of Recreation and Parks to the north and the Pacific Ocean to the south. To the east is South Weymouth Avenue and to the west is Western Avenue. West of Western Avenue and east of South Weymouth Avenue are mostly residential buildings. Nearby along the south side of W Paseo Del Mar is the Royal Palms Beach Park and Fromhold Field, a Los Angeles County operated recreational baseball field. White Point County Beach south of the City of Los Angeles boundary is owned by the County of Los Angeles.
3 Project Scope

The purpose of this project is to restore the section of roadway that collapsed in the 2011 landslide event to its original location. URS (now merged with AECOM) is retained by the City of Los Angeles to identify options that would accomplish this task. The project consists of three main phases: Pre-Design, Design, and Construction. Currently, URS is only authorized for the Pre-Design Phase for which this Pre-Design Report is prepared. Other major tasks integrated with the Pre-Design Phase are geotechnical investigations, a Wave Run-Up Study, and public outreach.

Initially, it was thought that options could be derived with limited new geotechnical investigations and be based primarily on the past information contained in the reports provided by Shannon & Wilson, Inc. (S&W). However, as geotechnical investigations progressed in the areas that were not previously investigated by S&W such as the Graben, it became apparent that the options previously presented needed further refinement and new options were developed.

Seven options were developed overall and presented to the City. The City selected three options for inclusion in this report. From the three options presented, the City will select one option to move forward to final design.

4 Design Criteria

Roadway and drainage design will conform to the City of Los Angeles Bureau of Engineering standards. Signing and striping and any traffic signal modifications, if needed, will be per the Los Angeles Department of Transportation standards. Bridge design will be per AASHTO with supplements and amendments by Caltrans.

The posted speed limit in this area is 35 mph. Traffic lane widths were discussed with LADOT. Additionally, the bridge geometry was discussed in a meeting with the City’s Design Excellence Program, LADOT, and Council District 15 on May 17, 2015.

There are two primary configurations for the roadway based on whether a bridge or a reinforced embankment / retaining wall is used. The main difference between the two is that if a bridge is used, there will be a gap between the north sidewalk on the bridge and the existing ground and parking would not be allowed on the bridge. For the reinforced embankment / retaining wall configurations, there will be no gap between the roadway and the existing ground to the north and parking will be allowed on the south side of the road.

For the embankment / retaining wall alternatives, the following typical section is proposed (starting from north right of way line):

<table>
<thead>
<tr>
<th>Feet</th>
<th>Description</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>Shoulder</td>
<td>Must accommodate DWP Easement/poles</td>
</tr>
<tr>
<td>6</td>
<td>Bike lane</td>
<td>5 ft min</td>
</tr>
<tr>
<td>11</td>
<td>Traffic lane</td>
<td>min (12’-14’ possible)</td>
</tr>
<tr>
<td>4</td>
<td>Center median</td>
<td>striped double yellow</td>
</tr>
<tr>
<td>11</td>
<td>Traffic lane</td>
<td>min (12’-14’ possible)</td>
</tr>
<tr>
<td>6</td>
<td>Bike lane</td>
<td>5 ft min</td>
</tr>
<tr>
<td>8</td>
<td>Parking/Shoulder</td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>Sidewalk</td>
<td></td>
</tr>
<tr>
<td>64</td>
<td>Total</td>
<td></td>
</tr>
</tbody>
</table>
For the bridge Alternative, the following typical section is proposed (starting from north right of way line):

<table>
<thead>
<tr>
<th>Feet</th>
<th>Description</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>DWP easement</td>
<td>Depends on DWP needs</td>
</tr>
<tr>
<td>0</td>
<td>Sidewalk</td>
<td>Not required due to widened Sidewalk on south side. To be confirmed with COLA bridge requirements.</td>
</tr>
<tr>
<td>7</td>
<td>Bike lane</td>
<td>5 ft min next to curb</td>
</tr>
<tr>
<td>11</td>
<td>Traffic lane</td>
<td>min (12'-14' possible)</td>
</tr>
<tr>
<td>4</td>
<td>Center median</td>
<td>striped double yellow</td>
</tr>
<tr>
<td>11</td>
<td>Traffic lane</td>
<td>min (12'-14' possible)</td>
</tr>
<tr>
<td>7</td>
<td>Bike lane</td>
<td>5 ft min</td>
</tr>
<tr>
<td>0</td>
<td>Parking/Shoulder</td>
<td>No parking on bridge</td>
</tr>
<tr>
<td>15</td>
<td>Sidewalk</td>
<td></td>
</tr>
<tr>
<td>60</td>
<td>Total</td>
<td></td>
</tr>
</tbody>
</table>

It’s noted that these are minimum widths. Final widths will also depend on transitions at either end of the restoration.

5 Options Analysis

Restoration of the roadway requires a structural solution. Due to the geometry of the site, it is not possible to simply build an embankment on top of the landslide debris and reconstruct the asphalt roadway. The added weight of the earthwork would likely cause further instability by reactivating the landslide. To prevent further collapse of this area once the roadway is restored, the area below must be structurally reinforced or be bridged over.

The cause of the landslide can be attributed to several factors which apply to most of the coastal bluffs of southern California consisting of interbedded weak sedimentary rocks heavily distorted by tectonic movement. Our investigation was able to constrain the depth of the movement using geologic correlations and geotechnical back-analysis techniques. Previous landslide mitigation measures were based on the premise that a high groundwater level was a contributing factor to the landslide. However, since the general groundwater table did not appear to be unusually high at the time of failure, the actual trigger event can only be surmised. One possibility is water having been introduced into the bluff by leaking storm drains, since the slide seems to have started in the vicinity of a juncture of large storm drain pipes coming together at the west end. Another possibility is that a series of natural events including ongoing gradual reduction of stability by wave erosion and above-average rainfall triggering initial movement in June 2011 resulted in a reduction of shear strength to residual levels in November 2011 when the main event occurred. For additional information, see the Geotechnical Memo in Appendix D as well as past geotechnical studies.

The L.A. County storm drain noted above in Figure 2 has been re-routed and repaired. Additionally, subsurface drainage systems have been introduced at the eastern end of the landslide in conjunction with anchors to enhance slope stability.

As stated earlier, the focus of the structural solution was to either provide a roadway that can bridge over the existing landslide and future landslide events or to repair or reinforce the existing landslide area such that a new roadway embankment above it would be stable.

Several options were investigated. These options fall into either of two categories: Bridge options or Embankment options.
The bridge options are:

1A – A single span concrete double box girder bridge to span over the landslide;
1B – A single span concrete single box girder bridge to span over the landslide;
1C – A single span steel truss bridge to span over the landslide;
1D – A multiple span concrete slab bridge to span over and support the landslide.

The Embankment options are:

2A – Deep Buttress Fill;
2B – Anchored Cast-in-Drilled-Holes (CIDH) Pile with Buttress;
2C – Shear Pins with (Mechanically Stabilized Embankment (MSE) Wall.

A meeting was held with the Bureau of Engineering and Council District 15 on December 9, 2015 to present these 7 options with the goal of narrowing down the options to the three most feasible for discussion in this Pre-Design report. Another meeting on January 14, 2016 with the same goal was held with senior management from the Bureau of Engineering. Based on the meetings and internal discussions, the options were narrowed to these three:

1) Bridge to span over landslide (Previous Option 1A/1B);
2) Anchored CIDH Pile with Buttress (Previous Option 2B);
3) Shear Pins with MSE Wall (Previous Option 2C).

Alternatives that are not a part of the scope of this report as they were not recommended by the City's Task Force are:

1) Do nothing, which essentially is to close the road at the limits of the landslide;
2) Re-align road northerly into the White Point Nature Preserve.
3) Restore the road non-structurally; that is, restore the road by placing embankment fill above the landslide debris without mitigating for future landslides.

While the focus of the Pre-Design Report and the associated Geotechnical studies is to identify structurally feasible solutions for the restoration of the roadway, the selection of one option over another falls primarily on the following factors which are not listed in any particular order or priority:

- Aesthetics
- Cost
- Maintenance
- Requirement for shoreline protection (Riprap)

The following is a discussion of each of the three options selected by the City. A matrix for a summary evaluation of the options is presented in Section 6. A general plan of the concept for each of the options is found in the appendices.

5.1 OPTION 1 – Bridge to Span over Landslide

To limit major earthwork and remediation of the existing landslide, a single long span bridge supported on stable ground outside the limits of the landslide is proposed. A concrete box girder or double box girder bridge superstructure is utilized.

The bridge span is approximately 380 feet which is very long for a single span bridge. See the Preliminary Structures Report in Appendix E for further discussion.
5.1.1 Aesthetics

Aesthetics of the bridge has been discussed with the City’s Design Excellence Program. As the bridge is approximately 125’ above the beach level and the beach below is not a sandy public use recreational area, the view of the structure would normally be from a distance from either someone on a boat or an airplane. Detailed or intricate concrete patterning along the sides of the bridge would not make much sense since the visibility of the sides of the structure is from such a distance. Stone patterning would be considered for the bridge wingwalls if funding allows. Therefore the aesthetic treatment of the sides would most likely consist of utilizing colored concrete staining for more blending with the bluff or a false arch line to provide some roundedness to the straight line of the soffit. One recommendation from the City’s Design Excellence Program meeting was that the bridge presents an opportunity for the White Point Nature Preserve to create an environment normally found near or under bridges such as a ravine.

There is also a concern from the community about homeless encampments that could develop under the bridge. Fencing and access restrictions would be required to mitigate this from occurring.

Caltrans and the Coastal Commission collaborated on guidance for bridge barriers for structures along the coastline. The publication, Bridge Rails and Barriers, A Reference Guide for Transportation Projects in the Coastal Zone, recommends either the Type 80SW or the CA ST-40 bridge rail. Both of these barriers has a “see-thru” design and accommodates sidewalk.

It is feasible to enhance the standard Type 80 SW Barrier with additional architectural features such as concrete staining, cobble stone patterning, or decorative railing if desired. This detailing would be decided in final design.

5.1.2 Cost

Costs for this alternative also include additional slope stabilization of the bluff west of the landslide, as well as potential additional localized stabilization of the head scarp between the bridge abutments. The east side of the landslide, with its close proximity to homes, has already been mitigated with slope anchors, which is currently assumed to be sufficient for purposes of stabilizing the bridge abutment on that side. As there are no homes immediately west of the landslide, it was considered that stabilization of the west side of the landslide could be deferred. However, we now understand that it has been decided
that this area should also be mitigated as part of this project. We estimated approximately 80 anchors similar to the ones installed at the east side at an approximate cost of $2 million will suffice. Construction costs for this alternative with anticipated additional slope stability measures along the west abutment is approximately $16.9 million. For program budgeting purposes, a 30% contingency and 15% program and inspection cost is common at this pre-design level. Total programming cost for this alternative is $26.2 million.

5.1.3 Maintenance

Typical design life of asphalt roadways is around 20 years. Bridge design life is 75 years. A concrete bridge is the most common type bridge structure in California and has relatively low maintenance requirements. Because of the long span of the bridge, the abutments will bear a significant load which must be resisted by the abutment foundations. Any settlement must be minimized and significant settlement or movement would make the bridge non-functional. As indicated earlier, the western abutment will require further evaluation for stabilization. Measures similar to the slope anchors installed at the eastern side of the landslide may be utilized to reinforce the western embankment.

5.1.4 Requirement for Shore Line Protection (Riprap)

Because the abutment pile supports are set back approximately 150 feet from the shoreline, protection against erosion is not immediately required. Like any structure close to the shore, however, monitoring of the shoreline is needed and should significant erosion occur, protective measures would then be investigated. The remaining areas between the abutments would not affect the bridge should further erosion occur.

5.2 OPTION 2 – Anchored CIDH Piles with Buttress

A single row of large diameter Cast-in-Drilled Holes (CIDH) piles is proposed near the edge of the existing slope. After partial removal of the landslide debris to approximate elevation +75 feet, the piles are drilled and installed to below the basal shear interface layer. The tops of the piles will be connected with a reinforced concrete grade beam and tied back with anchors. A reinforced-earth buttress above the piles will stabilize the head scarp and support the new roadway.

See the Preliminary Structures Report in Appendix E for further discussion.

5.2.1 Aesthetics

Aesthetics of the embankment has been discussed with the City’s Design Excellence Program. The slope below elevation 75 ft +/- should be relatively undisturbed by the construction of the pile supports. The reconstructed slope above elevation 75 ft +/- will be reinforced but have a natural face. However erosion from wind and rain will tend to expose the reinforcing of the slopes. Mitigation of the erosion is usually accomplished with netting and native plantings.

A barrier is also required next to the sidewalk similar to the bridge alternative. Per the Caltrans and Coastal Commission guidance, either the Type 80SW or the CA ST-40 bridge rail would be used. See discussion in Section 5.1.

5.2.2 Cost

Primary costs for this alternative includes the piles and pile anchors, and the reinforced embankment. Similar to Option 1, the west side of the landslide will be stabilized with anchors. Construction costs for this alternative with limited riprap armoring and additional embankment stabilization west of the landslide is $28.8 million. For program budgeting purposes, a 30% contingency and 15% program and inspection cost is common at this pre-design level. Total programming cost for this alternative is $43.2 million.
5.2.3 Maintenance

Typical design life of asphalt roadways is around 20 years. Wall/Pile design life is 75 years. Like any structural system, annual or bi-annual monitoring and assessment is necessary. The embankment will be fitted with and underdrain system to prevent over saturation similar to what was done at the east side of the landslide. Drainage systems need to be inspected for damage and clogging.

Appearance of the reinforced slope may require some occasional maintenance especially if netting is used to prevent erosion.

5.2.4 Requirement For Shore Line Protection (Riprap)

Over time due to wave action and other erosive forces, the existing toe of slope will recede. Riprap protection up to elevation +25 is needed to protect the slope.

5.3 OPTION 3 – Shear Pins with MSE Wall

Rather than at the face of the existing slope, a row of large diameter piles and a grid of smaller diameter piles are constructed below the proposed roadway. The piles will handle the vertical loading of the Mechanically Stabilized Embankment (MSE) wall and mitigate lateral forces on the existing slope. Due to the tall height of the wall, traditional cantilever type walls would not be stable or cost effective. The MSE type wall utilizes a reinforcement strap tied to a segment of wall panel. The self-weight and friction of the compacted earth keeps the face panels in place. These wall types are proprietary. However, there are several pre-approved vendors of these types of walls by Caltrans such as the “Verdura” plantable wall system.

See the Preliminary Structures Report in Appendix E for further discussion.

5.3.1 Aesthetics

Aesthetics of the wall has been discussed with the City's Design Excellence Program. The slope below elevation 75 ft +/- should be relatively undisturbed by the construction of the pile supports. A concrete wall above elevation 75 ft +/- will be constructed. A vertical concrete faced wall would not be desirable. The tall concrete wall structure is massive and can be softened by stepping or using offset panels and planting. Alternatively, the Design Excellence Program recommended that the massive structure take on the look of a modern transportation element rather than blending its appearance in with stone and plantings. If this alternative is selected, additional architectural investigation is recommended to select a wall feature.

A barrier is also required next to the sidewalk similar to the bridge alternative. Per the Caltrans and Coastal Commission guidance, either the Type 80SW or the CA ST-40 bridge rail would be used. See discussion in Section 5.1.

5.3.2 Cost

Primary costs for this alternative includes the piles and shear pins, and the MSE wall. Similar to Options 1 and 2, costs for stabilizing the west embankment is included. Construction costs for this alternative with limited riprap armorng and additional embankment stabilization west of the landslide is $24.8 million. For program budgeting purposes, a 30% contingency and 15% program and inspection cost is common at this pre-design level. Total programming cost for this alternative is $37.3 million.

5.3.3 Maintenance

Typical design life of asphalt roadways is around 20 years. Wall/Pile design life is 75 years. Like any structural system, annual or bi-annual monitoring and assessment is necessary. The embankment will be fitted with drainage to prevent over saturation similar to what was done at the east side of the landslide. Drainage systems need to be inspected for damage and clogging.
Appearance of the wall surface may need maintenance for graffiti removal. If a plantable wall is used, native plantings that do not require watering should be used. During the plant establishment period, regular maintenance is required.

5.3.4 Requirement for Shore Line Protection (Riprap)

Over time due to wave action and other erosive forces, the existing toe of slope will recede. Riprap protection up to elevation +15 is needed to protect the slope. However, because the piles are set back more than 100 feet, this is not expected to occur for a significant period of time and an overall coastline management plan can be discussed with the Army Corp of Engineers.

6 Recommended Project

Ultimately, the selection of one of the three options will be by the City of Los Angeles in consultation with Council District 15 and the community.

Based on the four main factors identified above, the following qualitative evaluation is presented:

<table>
<thead>
<tr>
<th>Qualitative Analysis of Options</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Option</strong></td>
</tr>
<tr>
<td>Bridge</td>
</tr>
<tr>
<td>Anchored CIDH Pile</td>
</tr>
<tr>
<td>Shear Pin with MSE Wall</td>
</tr>
</tbody>
</table>

For a more quantitative analysis of the options, the following methodology can be used. Each of the 4 factors can be ranked in terms of importance. With four factors identified, the ranking will be from 1 to 4 with 4 being the highest in importance:

<table>
<thead>
<tr>
<th>Ranking of Importance (1-4; 4 is highest)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cost</td>
</tr>
<tr>
<td>Aesthetics</td>
</tr>
<tr>
<td>Maintenance</td>
</tr>
<tr>
<td>Riprap</td>
</tr>
</tbody>
</table>

Then, each of the options can be ranked within each factor from 1 to 3 with 1 being the worst of the three options and 3 being the best of the options within each factor.
### Scoring of Each Factor

1 – 3; 1 is worst; 3 is best

<table>
<thead>
<tr>
<th>Option</th>
<th>Cost</th>
<th>Aesthetics</th>
<th>Maintenance</th>
<th>Riprap (Requirement)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bridge</td>
<td>3</td>
<td>2</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>Anchored CIDH Pile</td>
<td>1</td>
<td>3</td>
<td>3</td>
<td>1</td>
</tr>
<tr>
<td>Shear Pin with MSE Wall</td>
<td>2</td>
<td>1</td>
<td>1</td>
<td>2</td>
</tr>
</tbody>
</table>

Finally, the ranking is multiplied by the factor score for a total score. The highest score indicates the best option.

### Overall Score

<table>
<thead>
<tr>
<th>Option</th>
<th>Cost</th>
<th>Aesthetics</th>
<th>Maintenance</th>
<th>Rip-Rap (Requirement)</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bridge</td>
<td>12</td>
<td>4</td>
<td>2</td>
<td>9</td>
<td>27</td>
</tr>
<tr>
<td>Anchored CIDH Pile</td>
<td>4</td>
<td>6</td>
<td>3</td>
<td>3</td>
<td>16</td>
</tr>
<tr>
<td>Shear Pin with MSE Wall</td>
<td>8</td>
<td>2</td>
<td>1</td>
<td>6</td>
<td>17</td>
</tr>
</tbody>
</table>

Based on this quantitative analysis, the Bridge Option has the highest score and is recommended.

One factor that was not included in the ranking above was the ability of the two embankment options to mitigate additional landslide movement of the area above the head scarp. In the event of movement due to landslides in this area, the White Point Nature Preserve could see additional impact since the bridge option is not intended to support the ground between the abutments. However, it should be noted that the land mass that already fell (from the Graben to beach level) appears to be stable. The head scarp area above the Graben is currently steep and is subject to localized failure. As part of the bridge option, this area will be graded to a shallower slope. If additional mitigation is desired, soil nailing or other localized mitigation could be utilized to mitigate future landslide damage at the White Point Nature Preserve. Potential costs for this added support would not significantly change the overall quantitative analysis and score.
7 Schedule

Final design is scheduled for completion in the 2016/17 fiscal year. An environmental document is being prepared. Notice to proceed for final design has not yet been issued by the City but is expected in mid-2016. Actual completion of Final Design is estimated to be 1 year post NTP, provided that no significant design concept changes are introduced due to the environmental document. Approximately 3 to 6 months should set aside for bid and award prior to construction. Bid through construction and post construction dates are unavailable until funding is secured for construction.

<table>
<thead>
<tr>
<th></th>
<th>Start</th>
<th>Finish</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pre-Design:</td>
<td>August 2015</td>
<td>April 2016</td>
</tr>
<tr>
<td>Design:</td>
<td>August 2016 (contingent)</td>
<td>August 2017 (contingent)</td>
</tr>
<tr>
<td>Bid &amp; Award:</td>
<td>August 2017 (contingent)</td>
<td>TBD</td>
</tr>
<tr>
<td>Construction:</td>
<td>TBD</td>
<td>2019 (contingent)</td>
</tr>
<tr>
<td>Post-Construction:</td>
<td>TBD</td>
<td>TBD</td>
</tr>
</tbody>
</table>

8 Construction Budget

The total programming and construction cost to complete the project as described above is estimated (Class “C”) to be in the range of $27 to $44 million depending on the option selected. Please see Appendices for the Class “C” cost estimates for each option.

9 QA/QC Program

QA/QC for the Project will be carried out by BOE staff. Reviews will be carried out by design team, project management, construction management, and the client at the 50% and 90% stages. Standard QC checklists will be used to record the results of the QC checking. All comments will be incorporated or addressed for Quality Assurance / Quality Control (QA/QC) purposes.

10 Staffing Recommendation

The following disciplines will be required for this project:

- Civil/Traffic/Drainage Engineering
- Structural Engineering
- Geotechnical Engineering
11 Design Development

11.1 Roadway / Community Considerations

The community has expressed via comments from the a community meeting held by the project team on October 28, 2015 and from various community groups such as the Coastal San Pedro Neighborhood Council that traffic is a major concern. Furthermore, the community would like to see traffic improvements made to intersections near the landslide. Specifically from the Coastal San Pedro Neighborhood Council, the following motion was put forward:

*Therefore be it resolved, the Coastal San Pedro Neighborhood Council requests that the City develop a detailed traffic plan including but not limited to the below traffic calming elements in the design of Paseo Del Mar between Gaffey Street and Western Ave and this plan be implemented in conjunction with the City's plan to rebuild Paseo Del Mar:*

1) Raised median strip between Roxbury Avenue and Weymouth Avenue.
2) Turnabouts or stop-signs on Paseo Del Mar at the following avenue intersections: Roxbury, Barbara, and Weymouth.
3) And other traffic calming devices such as illuminated crosswalks, bulb-outs, appropriate signage and speed humps.

The project limits for the landslide restoration work is currently limited to just what is necessary to re-establish the roadway that collapsed and that is generally from Weymouth Avenue on the east and approximately 1500 feet west of Weymouth Avenue. At the Paseo Del Mar and Weymouth Avenue intersection, LADOT has tentatively approved installing stop signs and a north-south pedestrian cross walk at the easterly leg of the intersection.

Speeding across this stretch of restored roadway is mitigated by narrower lanes and adjacent parking lanes. A bumpout can also be considered near the intersection of Weymouth Avenue. A conceptual striping plan is shown in Appendix G.

11.2 Dredging / Location of Embankment Fill Sites

For Options 2 and 3, Anchored CIDH piles with buttress and Shear pins with MSE wall, a large amount of embankment fill is required. One suggestion was to utilize dredged material from the Port of Los Angeles. However, The Port of Los Angeles completed its main channel deepening project to accommodate larger ships in year 2014. Remaining works being undertaken is mainly maintenance dredging. The quality of material obtained from maintenance dredging is generally silts, and sometimes contaminated, which are less suitable for use in the proposed project, which will require a high quality engineered fill. The use of and transport of dredged material would have additional environmental impacts such as large use of heavy diesel pumping equipment. Further this methodology would also have significant environmental permit implications (and potential time impacts).

11.3 Haul Routes

The major truck routes connecting to W Paseo Del Mar is S Gaffey Street to the east, Western Avenue to the west, and W 9th Street to the north. The closest concrete and paving plant is at 2521 E Artesia Boulevard, Long Beach, CA 90805 (Holliday Rock). Access will be from the east side via SR-47. Construction and demolition recycling facilities are located at 3031 East "I" Street, Wilmington, CA (Allied Waste Falcon) for Construction and Demolition Debris and 2850 California Avenue, Signal Hill, CA 90755.
(Hanson Aggregates) for Asphalt and Concrete. Access from both locations will be from the east side via the 47 freeway. See Appendix H for list of additional facilities.

11.4 Permitting

Other than the typical construction / traffic handling permitting with the City of Los Angeles, four other major agencies have jurisdiction within the construction impact area.

The Army Corp of Engineers (ACOE) permitting would be required depending on the area of impact near the high water elevation. Any grading or permanent changes such as bank stabilization under 1/3 acre or under 500 lineal feet of bank stabilization would qualify for a Nationwide Permit. Otherwise a specific permit would be required.

A coastal commission permit is required since this entire area / roadway is in the jurisdiction. The commission works with the City of Los Angeles on all projects along the coast and will co-review the environmental documents. A meeting with the Coastal Commission (Al Padilla) on December 21, 2015 indicated that the commission prefers not to utilize alternatives that would require significant bank stabilization (i.e. riprap, etc.) so as not to change the visual character of the shoreline. Also, the Commission expects to see a discussion of all alternatives in the environmental document, not just the ones that the Council District 15 task force approved.

Work that disturbs or intrudes upon the County beach area will require a permit from the Los Angeles County Beaches and Harbor.

Finally, the Regional Water Quality Board (RWQB) has jurisdiction over storm water discharge. Final drainage design will need to comply with any treatment BMPs that the RWQB will require. The City of Los Angeles Bureau of Sanitation administers and reviews projects for compliance with the mandates outlined in the National Pollutant Discharge Elimination System (or NPDES) Municipal Storm Water Permit (No. CAS004001). Depending on the quality of the storm water, it may need to be treated prior to entering the County’s storm drain system or outfall to the ocean. Groundwater captured by the underdrain system may contain metals which would require treatment prior to outfall to the ocean. Coordination with the County will be required for permitted connections to the County system.

The Wave Run-Up study in Appendix F identifies several other potential permitting agencies. However, for the three selected alternatives, we do not expect permitting requirements beyond the three listed above. The other permitting agencies would get involved for alternatives that require significant grading of the landslide area and changes to the shoreline.
Appendix A.
Option 1
# City of Los Angeles, California

## Paseo Del Mar (White Point) Restoration Project - Option 1A/B: Single-Span CIP/PS Concrete Box Girder Bridge

### CLASS "C" CONSTRUCTION COST ESTIMATE

May 2016 (Based on Conceptual Plans)

<table>
<thead>
<tr>
<th>ITEM</th>
<th>ITEM DESCRIPTION</th>
<th>QTY</th>
<th>UNIT</th>
<th>UNIT PRICE</th>
<th>SUB TOTAL</th>
<th>EXTENSION OF BASE COST</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Land Acquisition</strong></td>
<td>Land Acquisition</td>
<td>N/A</td>
<td>N/A</td>
<td>$0</td>
<td>$0</td>
<td><strong>$0</strong></td>
</tr>
<tr>
<td><strong>General Items</strong></td>
<td>Class A Field Office</td>
<td>1</td>
<td>LS</td>
<td>$60,000.00</td>
<td>$60,000</td>
<td><strong>$62,000</strong></td>
</tr>
<tr>
<td></td>
<td>Project Information Sign</td>
<td>1</td>
<td>LS</td>
<td>$2,000.00</td>
<td>$2,000</td>
<td><strong>$62,000</strong></td>
</tr>
<tr>
<td></td>
<td>Progress Schedule</td>
<td>1</td>
<td>LS</td>
<td>$0.00</td>
<td>$0</td>
<td><strong>$62,000</strong></td>
</tr>
<tr>
<td><strong>Demolition</strong></td>
<td>Hazardous Materials Abatement (assume none)</td>
<td>1</td>
<td>LS</td>
<td>$0.00</td>
<td>$0</td>
<td><strong>$62,000</strong></td>
</tr>
<tr>
<td></td>
<td>Demolition (concrete, asphalt concrete, storm drain structures, concrete retaining walls, trees)</td>
<td>1</td>
<td>LS</td>
<td>$20,000.00</td>
<td>$20,000</td>
<td><strong>$62,000</strong></td>
</tr>
<tr>
<td><strong>Grading and Earthwork</strong></td>
<td>Site Grading / Clearing and Grubbing</td>
<td>1</td>
<td>LS</td>
<td>$20,000.00</td>
<td>$20,000</td>
<td><strong>$62,000</strong></td>
</tr>
<tr>
<td><strong>Street Improvements</strong></td>
<td>Type &quot;A&quot; Concrete Curb (S-410-2)</td>
<td>1,132</td>
<td>LF</td>
<td>$18.00</td>
<td>$20,376</td>
<td><strong>$62,000</strong></td>
</tr>
<tr>
<td></td>
<td>Type &quot;C&quot; Concrete Curb and Gutter (S-410-2)</td>
<td>1,450</td>
<td>LF</td>
<td>$35.00</td>
<td>$50,750</td>
<td><strong>$62,000</strong></td>
</tr>
<tr>
<td></td>
<td>Concrete Sidewalk t = 4&quot; (S-444-0)</td>
<td>13,018</td>
<td>SF</td>
<td>$4.00</td>
<td>$52,072</td>
<td><strong>$62,000</strong></td>
</tr>
<tr>
<td></td>
<td>Concrete Ramps (S-442-3)</td>
<td>2</td>
<td>EA</td>
<td>$4,000.00</td>
<td>$8,000</td>
<td><strong>$62,000</strong></td>
</tr>
<tr>
<td></td>
<td>Concrete Alley Intersection, t = 8&quot;</td>
<td>160</td>
<td>SF</td>
<td>$12.00</td>
<td>$1,920</td>
<td><strong>$62,000</strong></td>
</tr>
<tr>
<td></td>
<td>Concrete Driveways, t = 6&quot;</td>
<td>1,000</td>
<td>SF</td>
<td>$10.00</td>
<td>$10,000</td>
<td><strong>$62,000</strong></td>
</tr>
<tr>
<td></td>
<td>Cold Plane AC Pavement</td>
<td>16,300</td>
<td>SF</td>
<td>$1.00</td>
<td>$16,300</td>
<td><strong>$62,000</strong></td>
</tr>
<tr>
<td></td>
<td>Asphalt Concrete Pavement, t = 6&quot;</td>
<td>2,419</td>
<td>TON</td>
<td>$120.00</td>
<td>$290,331</td>
<td><strong>$62,000</strong></td>
</tr>
<tr>
<td></td>
<td>Crushed Miscellaneous Base (CMB), t = 10&quot;</td>
<td>2,905</td>
<td>TON</td>
<td>$50.00</td>
<td>$145,273</td>
<td><strong>$62,000</strong></td>
</tr>
<tr>
<td></td>
<td>Crushed Miscellaneous Base (CMB), t = 4&quot; (under sidewalk, C&amp;G)</td>
<td>357</td>
<td>TON</td>
<td>$50.00</td>
<td>$17,829</td>
<td><strong>$62,000</strong></td>
</tr>
<tr>
<td><strong>Traffic and Lighting</strong></td>
<td>Electrical Pull Boxes/Conduit</td>
<td>1</td>
<td>LS</td>
<td>$5,000.00</td>
<td>$5,000</td>
<td><strong>$62,000</strong></td>
</tr>
<tr>
<td></td>
<td>Temporary Traffic Signals</td>
<td>1</td>
<td>LS</td>
<td>$0.00</td>
<td>$0</td>
<td><strong>$62,000</strong></td>
</tr>
<tr>
<td></td>
<td>Signage</td>
<td>1</td>
<td>LS</td>
<td>$5,000.00</td>
<td>$5,000</td>
<td><strong>$62,000</strong></td>
</tr>
<tr>
<td></td>
<td>Lighting (Bollard Lighting)</td>
<td>8</td>
<td>EA</td>
<td>$3,000.00</td>
<td>$24,000</td>
<td><strong>$62,000</strong></td>
</tr>
<tr>
<td></td>
<td>Permanent Striping</td>
<td>1</td>
<td>LS</td>
<td>$20,000.00</td>
<td>$20,000</td>
<td><strong>$62,000</strong></td>
</tr>
<tr>
<td></td>
<td>Traffic Control</td>
<td>1</td>
<td>LS</td>
<td>$15,000.00</td>
<td>$15,000</td>
<td><strong>$62,000</strong></td>
</tr>
<tr>
<td><strong>Planting &amp; Irrigation</strong></td>
<td>Soil Preparation and Fine Grading</td>
<td>17,400</td>
<td>SF</td>
<td>$0.20</td>
<td>$3,480.00</td>
<td><strong>$62,000</strong></td>
</tr>
<tr>
<td></td>
<td>24&quot; Native Box Tree</td>
<td>16</td>
<td>EA</td>
<td>$300.00</td>
<td>$4,800.00</td>
<td><strong>$62,000</strong></td>
</tr>
<tr>
<td></td>
<td>15&quot; BTH Native Washingtonia Palm Tree</td>
<td>0</td>
<td>EA</td>
<td>$600.00</td>
<td>$0.00</td>
<td><strong>$62,000</strong></td>
</tr>
<tr>
<td></td>
<td>Plant Establishment Work (90 Working Days)</td>
<td>1</td>
<td>LS</td>
<td>$7,500.00</td>
<td>$7,500.00</td>
<td><strong>$62,000</strong></td>
</tr>
<tr>
<td>Item</td>
<td>Description</td>
<td>Quantity</td>
<td>Unit</td>
<td>Base</td>
<td>Cont.</td>
<td>Base</td>
</tr>
<tr>
<td>------</td>
<td>-------------</td>
<td>----------</td>
<td>------</td>
<td>------</td>
<td>-------</td>
<td>------</td>
</tr>
<tr>
<td>Irrigation</td>
<td>LS</td>
<td>$25,000.00</td>
<td>$25,000.00</td>
<td>5</td>
<td>1</td>
<td>LS</td>
</tr>
<tr>
<td>Sub-Total</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>$40,780</strong></td>
</tr>
<tr>
<td>Storm Drain</td>
<td>LS</td>
<td>$50,000.00</td>
<td>$50,000</td>
<td>1</td>
<td>1</td>
<td>LS</td>
</tr>
<tr>
<td>1 Storm Drain Facilities (Pipe and Structures)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>$50,000</strong></td>
</tr>
<tr>
<td>2 Adjust Storm Drain Maintenance Holes to Grade</td>
<td>EA</td>
<td>$800.00</td>
<td>$2,400</td>
<td>3</td>
<td>5</td>
<td>EA</td>
</tr>
<tr>
<td>3 Storm Drain Catch Basins (Adjust or reconstruct)</td>
<td>EA</td>
<td>$6,000.00</td>
<td>$24,000</td>
<td>4</td>
<td>0</td>
<td>EA</td>
</tr>
<tr>
<td>Sub-Total</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>$76,400</strong></td>
</tr>
<tr>
<td>Sanitary Sewer</td>
<td>LS</td>
<td>$30,000.00</td>
<td>$30,000</td>
<td>1</td>
<td>5</td>
<td>EA</td>
</tr>
<tr>
<td>1 Sanitary Sewer Facilities (Pipe and Junction Structures)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>$30,000</strong></td>
</tr>
<tr>
<td>2 Adjust Sanitary Sewer Maintenance Holes to Grade</td>
<td>EA</td>
<td>$800.00</td>
<td>$4,000</td>
<td>5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sub-Total</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>$34,000</strong></td>
</tr>
<tr>
<td>Water Improvements</td>
<td>EA</td>
<td>$5,000.00</td>
<td>$5,000</td>
<td>1</td>
<td>2</td>
<td>EA</td>
</tr>
<tr>
<td>1 Relocate Fire Hydrants</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>$6,000</strong></td>
</tr>
<tr>
<td>2 Cap Existing Water Line Laterals</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>$6,000</strong></td>
</tr>
<tr>
<td>Sub-Total</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>$6,000</strong></td>
</tr>
<tr>
<td>Utility Relocations</td>
<td>EA</td>
<td>$25,000.00</td>
<td>$100,000</td>
<td>4</td>
<td>1</td>
<td>EA</td>
</tr>
<tr>
<td>1 Relocate Power Poles / Duct Bank underground</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>$103,000</strong></td>
</tr>
<tr>
<td>2 Cap / Reconnect Exist Gas</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>$3,000</strong></td>
</tr>
<tr>
<td>3 Adjust Existing Utilities to Grade (gas valves, water valves, etc.)</td>
<td>EA</td>
<td>$500.00</td>
<td>$5,000</td>
<td>10</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sub-Total</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>$108,000</strong></td>
</tr>
<tr>
<td>Bridge Option 1A &amp; 1B</td>
<td>LS</td>
<td>$9,900,000.00</td>
<td>$9,900,000</td>
<td>1</td>
<td>1</td>
<td>LS</td>
</tr>
<tr>
<td>1 Bridge (CIP/PS Box Girder)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>$11,900,000</strong></td>
</tr>
<tr>
<td>2 West Bank Stabilization</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>$2,000,000</strong></td>
</tr>
<tr>
<td>3 Earthwork (Minor Grading of Graben for working surface)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>$58,000</strong></td>
</tr>
<tr>
<td>4 Rip Rap Lining (not required for single-bridge options)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>$0</strong></td>
</tr>
<tr>
<td>5 Potential localized stabilization between abutments</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>$960,000</strong></td>
</tr>
<tr>
<td>Sub-Total</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>$12,916,000</strong></td>
</tr>
<tr>
<td>Miscellaneous / Supplemental Items</td>
<td>SY</td>
<td>$8.00</td>
<td>$16,178</td>
<td>2,022</td>
<td>1</td>
<td>LS</td>
</tr>
<tr>
<td>1 Whitepoint Nature Reserve Parking Lot Restoration/Improvements</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>$16,178</strong></td>
</tr>
<tr>
<td>2 Monumentation (Community Request)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>$10,000</strong></td>
</tr>
<tr>
<td>3 Mitigation Monitoring Program</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>$10,000</strong></td>
</tr>
<tr>
<td>4 SWPPP Preparation/Enforcement</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>$35,000</strong></td>
</tr>
<tr>
<td>5 Outreach Activities During Construction (Additional)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>$25,000</strong></td>
</tr>
<tr>
<td>6 Palm Tree (Relocation)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>$12,000</strong></td>
</tr>
<tr>
<td>Sub-Total</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>$118,178</strong></td>
</tr>
<tr>
<td>SUBTOTAL</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>$14,083,209</strong></td>
</tr>
<tr>
<td>A Time Related Overhead</td>
<td>LS</td>
<td>$1,408,320.89</td>
<td>$1,408,321</td>
<td>1</td>
<td>1</td>
<td>LS</td>
</tr>
<tr>
<td>B Mobilization (10% at Class C)</td>
<td>LS</td>
<td>$1,408,320.89</td>
<td>$1,408,321</td>
<td>1</td>
<td>1</td>
<td>LS</td>
</tr>
<tr>
<td></td>
<td>SUBTOTAL</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C Contingency (30% at Class C)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>$5,069,955.22</strong></td>
</tr>
<tr>
<td>D BUREAU OF ENGINEERING AND BUREAU OF CONTRACT ADMIN (15%)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>$2,534,977.61</strong></td>
</tr>
<tr>
<td>E CONSULTANT DESIGN AND CONSTRUCTION SUPPORT SERVICES</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>$1,700,000.00</strong></td>
</tr>
<tr>
<td>GRAND TOTAL (2017 DOLLARS)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>$26,204,784</strong></td>
</tr>
</tbody>
</table>

Assumptions:
1. Storm drain and Sanitary Sewer costs may change depending on the outcome of the CCTV inspections.
2. Mitigation Monitoring assumes 1 day/week for four months to monitor archeological conditions.
3. Community suggestion of one monument at each end.
Appendix B.
Option 2
## Paseo Del Mar (White Point) Restoration Project - Option 2B: Anchored CIDH with Reinforced Earth Buttress Fill

**City of Los Angeles, California**

**CLASS "C" CONSTRUCTION COST ESTIMATE**

May 2016 (Based on Conceptual Plans)

<table>
<thead>
<tr>
<th>ITEM</th>
<th>ITEM DESCRIPTION</th>
<th>QTY.</th>
<th>UNIT</th>
<th>UNIT PRICE</th>
<th>SUB TOTAL</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Land Acquisition</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>Land Acquisition</td>
<td>1</td>
<td>N/A</td>
<td>$0.00</td>
<td>$0.00</td>
</tr>
</tbody>
</table>

**General Items**

<table>
<thead>
<tr>
<th>ITEM</th>
<th>ITEM DESCRIPTION</th>
<th>QTY.</th>
<th>UNIT</th>
<th>UNIT PRICE</th>
<th>SUB TOTAL</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Class A Field Office</td>
<td>1</td>
<td>LS</td>
<td>$60,000.00</td>
<td>$60,000</td>
</tr>
<tr>
<td>2</td>
<td>Project Information Sign</td>
<td>1</td>
<td>LS</td>
<td>$2,000.00</td>
<td>$2,000</td>
</tr>
<tr>
<td>3</td>
<td>Progress Schedule</td>
<td>1</td>
<td>LS</td>
<td>$0.00</td>
<td>$0.00</td>
</tr>
</tbody>
</table>

**Sub-Total** | | | | | $62,000 |

**Demolition**

<table>
<thead>
<tr>
<th>ITEM</th>
<th>ITEM DESCRIPTION</th>
<th>QTY.</th>
<th>UNIT</th>
<th>UNIT PRICE</th>
<th>SUB TOTAL</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Hazardous Materials Abatement (assume none)</td>
<td>1</td>
<td>LS</td>
<td>$0.00</td>
<td>$0.00</td>
</tr>
<tr>
<td>2</td>
<td>Demolition (concrete, asphalt concrete, storm drain structures, concrete retaining walls, trees)</td>
<td>1</td>
<td>LS</td>
<td>$20,000.00</td>
<td>$20,000</td>
</tr>
</tbody>
</table>

**Sub-Total** | | | | | $20,000 |

**Grading and Earthwork**

<table>
<thead>
<tr>
<th>ITEM</th>
<th>ITEM DESCRIPTION</th>
<th>QTY.</th>
<th>UNIT</th>
<th>UNIT PRICE</th>
<th>SUB TOTAL</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Site Grading / Clearing and Grubbing</td>
<td>1</td>
<td>LS</td>
<td>$20,000.00</td>
<td>$20,000</td>
</tr>
</tbody>
</table>

**Sub-Total** | | | | | $20,000 |

**Street Improvements**

<table>
<thead>
<tr>
<th>ITEM</th>
<th>ITEM DESCRIPTION</th>
<th>QTY.</th>
<th>UNIT</th>
<th>UNIT PRICE</th>
<th>SUB TOTAL</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Type &quot;A&quot; Concrete Curb (S-410-2)</td>
<td>1,450</td>
<td>LF</td>
<td>$18.00</td>
<td>$26,100</td>
</tr>
<tr>
<td>2</td>
<td>Type &quot;C&quot; Concrete Curb and Gutter (S-410-2)</td>
<td>1,450</td>
<td>LF</td>
<td>$35.00</td>
<td>$50,750</td>
</tr>
<tr>
<td>3</td>
<td>Concrete Sidewalk, t = 4&quot; (S-444-0)</td>
<td>16,675</td>
<td>SF</td>
<td>$4.00</td>
<td>$66,700</td>
</tr>
<tr>
<td>4</td>
<td>Concrete Ramps (S-442-3)</td>
<td>2</td>
<td>EA</td>
<td>$4,000.00</td>
<td>$8,000</td>
</tr>
<tr>
<td>5</td>
<td>Concrete Alley Intersection, t = 8&quot;</td>
<td>160</td>
<td>SF</td>
<td>$12.00</td>
<td>$1,920</td>
</tr>
<tr>
<td>6</td>
<td>Concrete Driveways, t = 6&quot;</td>
<td>1,000</td>
<td>SF</td>
<td>$10.00</td>
<td>$10,000</td>
</tr>
<tr>
<td>7</td>
<td>Cold Plane AC Pavement</td>
<td>16,300</td>
<td>SF</td>
<td>$1.00</td>
<td>$16,300</td>
</tr>
<tr>
<td>8</td>
<td>Asphalt Concrete Pavement, t = 8&quot;</td>
<td>2,992</td>
<td>TON</td>
<td>$120.00</td>
<td>$359,066</td>
</tr>
<tr>
<td>9</td>
<td>Crushed Miscellaneous Base (CMB), t = 10&quot;</td>
<td>3,593</td>
<td>TON</td>
<td>$50.00</td>
<td>$179,667</td>
</tr>
<tr>
<td>10</td>
<td>Crushed Miscellaneous Base (CMB), t = 4&quot; (under sidewalk, C&amp;G)</td>
<td>441</td>
<td>TON</td>
<td>$50.00</td>
<td>$22,050</td>
</tr>
</tbody>
</table>

**Sub-Total** | | | | | $740,553 |

**Traffic and Lighting**

<table>
<thead>
<tr>
<th>ITEM</th>
<th>ITEM DESCRIPTION</th>
<th>QTY.</th>
<th>UNIT</th>
<th>UNIT PRICE</th>
<th>SUB TOTAL</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Electrical Pull Boxes/Conduit</td>
<td>1</td>
<td>LS</td>
<td>$5,000.00</td>
<td>$5,000</td>
</tr>
<tr>
<td>2</td>
<td>Temporary Traffic Signals</td>
<td>1</td>
<td>LS</td>
<td>$0.00</td>
<td>$0.00</td>
</tr>
<tr>
<td>3</td>
<td>Signage</td>
<td>1</td>
<td>LS</td>
<td>$5,000.00</td>
<td>$5,000</td>
</tr>
<tr>
<td>4</td>
<td>Lighting (Bollard Lighting)</td>
<td>8</td>
<td>EA</td>
<td>$3,000.00</td>
<td>$24,000</td>
</tr>
<tr>
<td>5</td>
<td>Permanent Striping</td>
<td>1</td>
<td>LS</td>
<td>$20,000.00</td>
<td>$20,000</td>
</tr>
<tr>
<td>6</td>
<td>Traffic Control</td>
<td>1</td>
<td>LS</td>
<td>$15,000.00</td>
<td>$15,000</td>
</tr>
</tbody>
</table>

**Sub-Total** | | | | | $69,000 |

**Planting & Irrigation**

<table>
<thead>
<tr>
<th>ITEM</th>
<th>ITEM DESCRIPTION</th>
<th>QTY.</th>
<th>UNIT</th>
<th>UNIT PRICE</th>
<th>SUB TOTAL</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Soil Preparation and Fine Grading</td>
<td>17,400</td>
<td>SF</td>
<td>$0.20</td>
<td>$3,480.00</td>
</tr>
<tr>
<td>2</td>
<td>24&quot; Native Box Tree</td>
<td>8</td>
<td>EA</td>
<td>$300.00</td>
<td>$2,400.00</td>
</tr>
<tr>
<td>3</td>
<td>15&quot; BTH Native Washingtonia Palm Tree</td>
<td>8</td>
<td>EA</td>
<td>$800.00</td>
<td>$4,800.00</td>
</tr>
<tr>
<td>4</td>
<td>Plant Establishment Work (90 Working Days)</td>
<td>1</td>
<td>LS</td>
<td>$7,500.00</td>
<td>$7,500.00</td>
</tr>
<tr>
<td>5</td>
<td>Irrigation</td>
<td>1</td>
<td>LS</td>
<td>$25,000.00</td>
<td>$25,000.00</td>
</tr>
</tbody>
</table>

**Sub-Total** | | | | | $43,180 |
<table>
<thead>
<tr>
<th>Storm Drain</th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Storm Drain Facilities (Pipe and Structures)</td>
<td>1 LS</td>
<td>$60,000.00</td>
<td>$50,000</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2 Adjust Storm Drain Maintenance Holes to Grade</td>
<td>3 EA</td>
<td>$800.00</td>
<td>$2,400</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3 Storm Drain Catch Basins (Adjust or reconstruct)</td>
<td>4 EA</td>
<td>$6,000.00</td>
<td>$24,000</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4 Construct new Storm Drain Manhole</td>
<td>0 EA</td>
<td>$6,000.00</td>
<td>$0</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Sub-Total</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>$76,400</strong></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Sanitary Sewer</th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Sanitary Sewer Facilities (Pipe and Junction Structures)</td>
<td>1 LS</td>
<td>$30,000.00</td>
<td>$30,000</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2 Adjust Sanitary Sewer Maintenance Holes to Grade</td>
<td>5 EA</td>
<td>$800.00</td>
<td>$4,000</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Sub-Total</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>$34,000</strong></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Water Improvements</th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Relocate Fire Hydrants</td>
<td>1 EA</td>
<td>$5,000.00</td>
<td>$5,000</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2 Cap Existing Water Line Laterals</td>
<td>2 EA</td>
<td>$500.00</td>
<td>$1,000</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Sub-Total</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>$6,000</strong></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Utility Relocations</th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Relocate Power Poles</td>
<td>4 EA</td>
<td>$25,000.00</td>
<td>$100,000</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2 Cap / Reconnect Exist Gas</td>
<td>1 EA</td>
<td>$3,000.00</td>
<td>$3,000</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3 Adjust Existing Utilities to Grade (gas valves, water valves, etc.)</td>
<td>10 EA</td>
<td>$500.00</td>
<td>$5,000</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Sub-Total</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>$108,000</strong></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Embankment Option 2B - Anchored CIDH</th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Wall</td>
<td>0 LS</td>
<td>$0.00</td>
<td>$0</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2 West Bank Stabilization</td>
<td>1 LS</td>
<td>$2,000,000.00</td>
<td>$2,000,000</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3 Earthwork &amp; Substructure</td>
<td>1 LS</td>
<td>$19,200,000.00</td>
<td>$19,200,000</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4 Rip Rap Lining</td>
<td>1 LS</td>
<td>$1,540,000.00</td>
<td>$1,540,000</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5 Potential localized stabilization between abutments (not req'd)</td>
<td>1 LS</td>
<td>$0.00</td>
<td>$0</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Sub-Total</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>$22,740,000</strong></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Miscellaneous / Supplemental items</th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Whitepoint Nature Reserve Parking Lot Restoration/Improvements</td>
<td>2,022 SY</td>
<td>$8.00</td>
<td>$16,178</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2 Monumentation (Community Request)</td>
<td>1 LS</td>
<td>$10,000.00</td>
<td>$10,000</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3 Mitigation Monitoring Program</td>
<td>1 LS</td>
<td>$20,000.00</td>
<td>$20,000</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4 SWPPP Preparation/Enforcement</td>
<td>1 LS</td>
<td>$35,000.00</td>
<td>$35,000</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5 Outreach Activities During Construction (Additional)</td>
<td>1 LS</td>
<td>$25,000.00</td>
<td>$25,000</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6 Palm Tree (Relocation)</td>
<td>1 LS</td>
<td>$12,000.00</td>
<td>$12,000</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Sub-Total</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>$118,178</strong></td>
</tr>
</tbody>
</table>

| **SUBTOTAL**                                      |   |   |   |   |   |   | **$24,037,311** |

| A Time Related Overhead (10%)                     | 1 LS | $2,403,731.08 | $2,403,731 |
| B Mobilization (10% at Class C)                   | 1 LS | $2,403,731.08 | $2,403,731 |
| **SUBTOTAL**                                      |   |   |   |   |   |   | **$28,844,773** |

| C Contingency (30% at Class C)                    |   |   |   |   |   |   | **$8,653,432** |
| D BUREAU OF ENGINEERING AND BUREAU OF CONTRACT ADMIN (15%) |   |   |   |   |   |   | **$4,326,715.95** |
| E CONSULTANT DESIGN AND CONSTRUCTION SUPPORT SERVICES |   |   |   |   |   |   | **$1,400,000.00** |
| **GRAND TOTAL (2017 DOLLARS)**                   |   |   |   |   |   |   | **$43,224,921** |

Assumptions:
1. Storm drain and Sanitary Sewer costs may change depending on the outcome of the CCTV inspections.
2. Mitigation Monitoring assumes 1 day/week for four months to monitor archeological conditions.
3. Community suggestion of one monument at each end.
Appendix C.
Option 3
### Paseo Del Mar (White Point) Restoration Project - Option 2C: Shear Pin with MSE Buttress Fill

**City of Los Angeles, California**

**CLASS "C" CONSTRUCTION COST ESTIMATE**

May 2016 (Based on Conceptual Plans)

<table>
<thead>
<tr>
<th>ITEM</th>
<th>ITEM DESCRIPTION</th>
<th>QTY.</th>
<th>UNIT</th>
<th>UNIT PRICE</th>
<th>SUB TOTAL</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Land Acquisition</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>Land Acquisition</td>
<td>N/A</td>
<td></td>
<td>$0</td>
<td>$0</td>
</tr>
<tr>
<td><strong>Sub-Total</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>General Items</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>Class A Field Office</td>
<td>1</td>
<td>LS</td>
<td>$60,000.00</td>
<td>$60,000</td>
</tr>
<tr>
<td>2</td>
<td>Project Information Sign</td>
<td>1</td>
<td>LS</td>
<td>$2,000.00</td>
<td>$2,000</td>
</tr>
<tr>
<td>3</td>
<td>Progress Schedule</td>
<td>1</td>
<td>LS</td>
<td>$0.00</td>
<td>$0</td>
</tr>
<tr>
<td><strong>Sub-Total</strong></td>
<td></td>
<td></td>
<td></td>
<td>$62,000</td>
<td></td>
</tr>
<tr>
<td><strong>Demolition</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>Hazardous Materials Abatement (assume none)</td>
<td>1</td>
<td>LS</td>
<td>$0.00</td>
<td>$0</td>
</tr>
<tr>
<td>2</td>
<td>Demolition (concrete, asphalt concrete, storm drain structures, concrete retaining walls, trees)</td>
<td>1</td>
<td>LS</td>
<td>$20,000.00</td>
<td>$20,000</td>
</tr>
<tr>
<td><strong>Sub-Total</strong></td>
<td></td>
<td></td>
<td></td>
<td>$20,000</td>
<td></td>
</tr>
<tr>
<td><strong>Grading and Earthwork</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>Site Grading / Clearing and Grubbing</td>
<td>1</td>
<td>LS</td>
<td>$20,000.00</td>
<td>$20,000</td>
</tr>
<tr>
<td><strong>Sub-Total</strong></td>
<td></td>
<td></td>
<td></td>
<td>$20,000</td>
<td></td>
</tr>
<tr>
<td><strong>Street Improvements</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>Type &quot;A&quot; Concrete Curb (S-410-2)</td>
<td>1,450</td>
<td>LF</td>
<td>$18.00</td>
<td>$26,100</td>
</tr>
<tr>
<td>2</td>
<td>Type &quot;C&quot; Concrete Curb and Gutter (S-410-2)</td>
<td>1,450</td>
<td>LF</td>
<td>$35.00</td>
<td>$50,750</td>
</tr>
<tr>
<td>3</td>
<td>Concrete Sidewalk t = 4&quot; (S-444-0)</td>
<td>16,675</td>
<td>SF</td>
<td>$4.00</td>
<td>$66,700</td>
</tr>
<tr>
<td>4</td>
<td>Concrete Ramps (S-442-3)</td>
<td>2</td>
<td>EA</td>
<td>$4,000.00</td>
<td>$8,000</td>
</tr>
<tr>
<td>5</td>
<td>Concrete Alley Intersection, t = 8&quot;</td>
<td>160</td>
<td>SF</td>
<td>$12.00</td>
<td>$1,920</td>
</tr>
<tr>
<td>6</td>
<td>Concrete Driveways, t = 6&quot;</td>
<td>1,000</td>
<td>SF</td>
<td>$10.00</td>
<td>$10,000</td>
</tr>
<tr>
<td>7</td>
<td>Cold Plane AC Pavement</td>
<td>16,300</td>
<td>SF</td>
<td>$1.00</td>
<td>$16,300</td>
</tr>
<tr>
<td>8</td>
<td>Asphalt Concrete Pavement, t = 8&quot;</td>
<td>2,992</td>
<td>TON</td>
<td>$120.00</td>
<td>$359,066</td>
</tr>
<tr>
<td>9</td>
<td>Crushed Miscellaneous Base (CMB), t = 10&quot;</td>
<td>3,593</td>
<td>TON</td>
<td>$50.00</td>
<td>$179,667</td>
</tr>
<tr>
<td>10</td>
<td>Crushed Miscellaneous Base (CMB), t = 4&quot; (under sidewalk, C&amp;G)</td>
<td>441</td>
<td>TON</td>
<td>$50.00</td>
<td>$22,050</td>
</tr>
<tr>
<td><strong>Sub-Total</strong></td>
<td></td>
<td></td>
<td></td>
<td>$740,553</td>
<td></td>
</tr>
<tr>
<td><strong>Traffic and Lighting</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>Electrical Pull Boxes/Conduit</td>
<td>1</td>
<td>LS</td>
<td>$5,000.00</td>
<td>$5,000</td>
</tr>
<tr>
<td>2</td>
<td>Temporary Traffic Signals</td>
<td>1</td>
<td>LS</td>
<td>$0.00</td>
<td>$0</td>
</tr>
<tr>
<td>3</td>
<td>Signage</td>
<td>1</td>
<td>LS</td>
<td>$5,000.00</td>
<td>$5,000</td>
</tr>
<tr>
<td>4</td>
<td>Lighting (Bollard Lighting)</td>
<td>8</td>
<td>EA</td>
<td>$3,000.00</td>
<td>$24,000</td>
</tr>
<tr>
<td>5</td>
<td>Permanent Striping</td>
<td>1</td>
<td>LS</td>
<td>$20,000.00</td>
<td>$20,000</td>
</tr>
<tr>
<td>6</td>
<td>Traffic Control</td>
<td>1</td>
<td>LS</td>
<td>$15,000.00</td>
<td>$15,000</td>
</tr>
<tr>
<td><strong>Sub-Total</strong></td>
<td></td>
<td></td>
<td></td>
<td>$69,000</td>
<td></td>
</tr>
<tr>
<td><strong>Planting &amp; Irrigation</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>Soil Preparation and Fine Grading</td>
<td>17,400</td>
<td>SF</td>
<td>$0.20</td>
<td>$3,480.00</td>
</tr>
<tr>
<td>2</td>
<td>24&quot; Native Box Tree</td>
<td>8</td>
<td>EA</td>
<td>$300.00</td>
<td>$2,400.00</td>
</tr>
<tr>
<td>3</td>
<td>15&quot; BTH Native Washingtonia Palm Tree</td>
<td>8</td>
<td>EA</td>
<td>$600.00</td>
<td>$4,800.00</td>
</tr>
<tr>
<td>4</td>
<td>Plant Establishment Work (90 Working Days)</td>
<td>1</td>
<td>LS</td>
<td>$7,500.00</td>
<td>$7,500.00</td>
</tr>
<tr>
<td>5</td>
<td>Irrigation</td>
<td>1</td>
<td>LS</td>
<td>$25,000.00</td>
<td>$25,000.00</td>
</tr>
<tr>
<td><strong>Sub-Total</strong></td>
<td></td>
<td></td>
<td></td>
<td>$43,180</td>
<td></td>
</tr>
<tr>
<td>Storm Drain</td>
<td>1</td>
<td>LS</td>
<td>$50,000.00</td>
<td>$50,000.00</td>
<td></td>
</tr>
<tr>
<td>------------------------------</td>
<td>---</td>
<td>-----</td>
<td>------------</td>
<td>------------</td>
<td></td>
</tr>
<tr>
<td>Adjust Storm Drain Maintenance Holes to Grade</td>
<td>3</td>
<td>EA</td>
<td>$800.00</td>
<td>$2,400.00</td>
<td></td>
</tr>
<tr>
<td>Storm Drain Catch Basins (Adjust or reconstruct)</td>
<td>4</td>
<td>EA</td>
<td>$6,000.00</td>
<td>$24,000.00</td>
<td></td>
</tr>
<tr>
<td>Construct new Storm Drain Manhole</td>
<td>0</td>
<td>EA</td>
<td>$6,000.00</td>
<td>$0.00</td>
<td></td>
</tr>
<tr>
<td><strong>Sub-Total</strong></td>
<td></td>
<td></td>
<td><strong>$76,400</strong></td>
<td><strong>$76,400</strong></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Sanitary Sewer</th>
<th>1</th>
<th>LS</th>
<th>$30,000.00</th>
<th>$30,000.00</th>
</tr>
</thead>
<tbody>
<tr>
<td>Adjust Sanitary Sewer Maintenance Holes to Grade</td>
<td>5</td>
<td>EA</td>
<td>$800.00</td>
<td>$4,000.00</td>
</tr>
<tr>
<td><strong>Sub-Total</strong></td>
<td></td>
<td></td>
<td><strong>$34,000</strong></td>
<td><strong>$34,000</strong></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Water Improvements</th>
<th>1</th>
<th>EA</th>
<th>$5,000.00</th>
<th>$5,000.00</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relocate Fire Hydrants</td>
<td>1</td>
<td>EA</td>
<td>$5,000.00</td>
<td>$5,000.00</td>
</tr>
<tr>
<td>Cap Existing Water Line Laterals</td>
<td>2</td>
<td>EA</td>
<td>$500.00</td>
<td>$1,000.00</td>
</tr>
<tr>
<td><strong>Sub-Total</strong></td>
<td></td>
<td></td>
<td><strong>$6,000</strong></td>
<td><strong>$6,000</strong></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Utility Relocations</th>
<th>1</th>
<th>EA</th>
<th>$25,000.00</th>
<th>$100,000.00</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relocate Power Poles</td>
<td>4</td>
<td>EA</td>
<td>$25,000.00</td>
<td>$100,000.00</td>
</tr>
<tr>
<td>Cap / Reconnect Exist Gas</td>
<td>1</td>
<td>EA</td>
<td>$3,000.00</td>
<td>$3,000.00</td>
</tr>
<tr>
<td>Adjust Existing Utilities to Grade (gas valves, water valves, etc.)</td>
<td>10</td>
<td>EA</td>
<td>$500.00</td>
<td>$5,000.00</td>
</tr>
<tr>
<td><strong>Sub-Total</strong></td>
<td></td>
<td></td>
<td><strong>$108,000</strong></td>
<td><strong>$108,000</strong></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Embankment Option 2C - Shear Pin</th>
<th>1</th>
<th>LS</th>
<th>$0.00</th>
<th>$0.00</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall</td>
<td>0</td>
<td>LS</td>
<td>$0.00</td>
<td>$0.00</td>
</tr>
<tr>
<td>West Bank Stabilization</td>
<td>1</td>
<td>LS</td>
<td>$2,000,000.00</td>
<td>$2,000,000.00</td>
</tr>
<tr>
<td>Earthwork &amp; Substructure &amp; MSE</td>
<td>1</td>
<td>LS</td>
<td>$16,900,000.00</td>
<td>$16,900,000.00</td>
</tr>
<tr>
<td>Rip Rap Lining</td>
<td>1</td>
<td>LS</td>
<td>$450,000.00</td>
<td>$450,000.00</td>
</tr>
<tr>
<td>Potential localized stabilization between abutments (not req'd)</td>
<td>1</td>
<td>LS</td>
<td>$0.00</td>
<td>$0.00</td>
</tr>
<tr>
<td><strong>Sub-Total</strong></td>
<td></td>
<td></td>
<td><strong>$19,350,000</strong></td>
<td><strong>$19,350,000</strong></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Miscellaneous / Supplemental Items</th>
<th>1</th>
<th>SY</th>
<th>$8.00</th>
<th>$16,178.00</th>
</tr>
</thead>
<tbody>
<tr>
<td>Whitepoint Nature Reserve Parking Lot Restoration/Improvements</td>
<td>2,022</td>
<td>SY</td>
<td>$8.00</td>
<td>$16,178.00</td>
</tr>
<tr>
<td>Monumentation (Community Request)</td>
<td>1</td>
<td>LS</td>
<td>$10,000.00</td>
<td>$10,000.00</td>
</tr>
<tr>
<td>Mitigation Monitoring Program</td>
<td>1</td>
<td>LS</td>
<td>$20,000.00</td>
<td>$20,000.00</td>
</tr>
<tr>
<td>SWPPP Preparation/Enforcement</td>
<td>1</td>
<td>LS</td>
<td>$35,000.00</td>
<td>$35,000.00</td>
</tr>
<tr>
<td>Outreach Activities During Construction (Additional)</td>
<td>1</td>
<td>LS</td>
<td>$25,000.00</td>
<td>$25,000.00</td>
</tr>
<tr>
<td>Palm Tree (Relocation)</td>
<td>1</td>
<td>LS</td>
<td>$12,000.00</td>
<td>$12,000.00</td>
</tr>
<tr>
<td><strong>Sub-Total</strong></td>
<td></td>
<td></td>
<td><strong>$118,178</strong></td>
<td><strong>$118,178</strong></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th><strong>SUBTOTAL</strong></th>
<th><strong>$20,647,311</strong></th>
<th><strong>$20,647,311</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>A Time Related Overhead</strong></td>
<td>1</td>
<td>LS</td>
</tr>
<tr>
<td><strong>B Mobilization (10% at Class C)</strong></td>
<td>1</td>
<td>LS</td>
</tr>
<tr>
<td><strong>SUBTOTAL</strong></td>
<td><strong>$24,776,773</strong></td>
<td></td>
</tr>
<tr>
<td><strong>C Contingency (30% at Class C)</strong></td>
<td></td>
<td><strong>$7,433,032</strong></td>
</tr>
<tr>
<td><strong>D BUREAU OF ENGINEERING AND BUREAU OF CONTRACT ADMIN (15%)</strong></td>
<td></td>
<td><strong>$3,716,516</strong></td>
</tr>
<tr>
<td><strong>E CONSULTANT DESIGN AND CONSTRUCTION SUPPORT SERVICES</strong></td>
<td></td>
<td><strong>$1,400,000</strong></td>
</tr>
<tr>
<td><strong>GRAND TOTAL (2017 DOLLARS)</strong></td>
<td><strong>$37,326,321</strong></td>
<td><strong>$37,326,321</strong></td>
</tr>
</tbody>
</table>

Assumptions:
1. Storm drain and Sanitary Sewer costs may change depending on the outcome of the CCTV inspections.
2. Mitigation Monitoring assumes 1 day/week for four months to monitor archeological conditions.
3. Community suggestion of one monument at each end.
Appendix D.
Draft Geotechnical Memorandum
APPENDIX D

Pre-Design Geotechnical Memorandum

Paseo Del Mar Permanent Restoration Project

Prepared by: Wolfgang Roth, PhD, PE, GE and Christopher Goetz, PG, CEG
5/17/2016
# TABLE OF CONTENTS

## 1.0 INTRODUCTION

## 2.0 INVESTIGATIONS

- 2.1 Literature/Data Review
- 2.2 Geologic Mapping
- 2.3 Supplemental Exploratory Borings
- 2.4 Field Instrumentation and Testing
- 2.5 Laboratory Testing

## 3.0 SUMMARY OF GEOLOGIC CHARACTERIZATION

- 3.1 Rock Mass Quality
- 3.2 Landslide Morphology
  - 3.2.1 The Slidescarp (Landslide Boundary)
  - 3.2.2 The Island
  - 3.2.3 The Peninsula
  - 3.2.4 The Graben
  - 3.2.5 The 2009 Landslide
- 3.3 Key Observations from Geologic Mapping
  - 3.3.1 The Slidescarp
  - 3.3.2 The Island
  - 3.3.3 Outside the Landslide
- 3.4 Key Observations from Drilling Investigations
  - 3.4.1 Boring Cluster 1
  - 3.4.2 Boring Cluster 2
  - 3.4.3 Boring Cluster 3
  - 3.4.4 Boring Cluster 4
  - 3.4.5 Boring Cluster 5
  - 3.4.6 Boring Cluster 6
  - 3.4.7 Boring Cluster 7
  - 3.4.8 Boring Cluster 8
- 3.5 Estimation of Basal Shear Surface
1.0 INTRODUCTION

This memorandum presents the results of our preliminary geotechnical evaluations in support of the White Point Landslide Restoration Project (Project). This report provides our Pre-Design Level findings based on our review of existing geologic and geotechnical data, subsurface explorations and laboratory testing, our interpretation of the geologic and geotechnical conditions encountered in our recent field exploration program, and recommendations for the foundation design of the proposed structural elements to restore the portion of Paseo del Mar that was destroyed and lost following the 2011 White Point landslide.

As part of the current Pre-Design Task, we reviewed existing data, explored the surface and subsurface conditions at the site, and developed geotechnical recommendations for the design of several bridge and retaining (MSE) wall schemes for restoring Paseo del Mar.

2.0 INVESTIGATIONS

2.1 LITERATURE/DATA REVIEW

The initial tasks of the site investigation consisted of a “desk top study” which included a review of the published literature regarding the geology of the Palos Verdes Peninsula, pertinent site specific reports and data regarding the White Point Landslide, and historic aerial and land based photographs and images.

The geology of the Palos Verdes Peninsula is well documented in the published literature due to its susceptibility for large landslides, its oil bearing deposits, and its conspicuous flight of emergent marine terraces that record tectonic uplift along the active Palos Verdes fault. The published geologic literature provided the background information necessary to understand the geologic information and data that was gathered during site specific field investigations. Key sources of published literature for this project were Dibblee (1992), Woodring and Others (1946), Ehlig (1982), and Bryant (1982).
Shortly after the White Point landslide occurred in 2011, the City of Los Angeles (COLA) engaged Shannon and Wilson, Inc. (S&W) to perform a geotechnical investigation, implement a monitoring program to measure ground-movements and groundwater levels, and to recommend preliminary stabilization measures. The remedial measures recommended by S&W were implemented by COLA in 2012, and these consisted of installing hydraugers and ground anchors to the east of the landslide and interim grading work within the landslide to remove debris, trim slopes and make the landslide free draining. During the course of their work S&W submitted several reports documenting their findings including a Final Geotechnical Report (S&W, 2012a), a Geotechnical Design Report, Interim Grading Plan (S&W, 2012b) and a Construction As-Built Report (S&W, 2012c). These key references provided site specific information regarding the landslide.

Analysis of pre and post landslide photographs of the project site provided important insight into the geometry of the landslide and the mechanism of sliding. Available images and photographs included aerials from Google Earth, historic aerial photographs available from NETR Online (http://www.historicaerials.com), aerial oblique photographs taken prior to and after the November 2011 failure (provided by Los Angeles Department of Water & Power), land based photographs taken during the 2012 grading operations (S&W Construction As-Built Report) and ground and aerial photographs taken by various news agencies.

2.2 GEOLOGIC MAPPING

The White Point Landslide and surrounding area was mapped on a 1:480 scale, topographic/aerial photographic base by a California Certified Engineering Geologist (CEG) on several occasions between late October and late December of 2015. The topography on the base map was prepared by Wagner Engineering & Survey Inc. using aerial photography with a flight date of August 29, 2015. The geologic mapping focused on identifying the surface morphology of the landslide and the distribution and orientation of the various geologic features (e.g. fractures, shears, bedding, folds) that are revealed in outcroppings within and outside the landslide. The findings of the geologic mapping are summarized on Figure 1 (Landslide Morphology Map) and Figure 2 (Geologic Map and Exploration Site Plan). A discussion of the key findings from the geologic mapping is presented in Section 3.2 and 3.3.
2.3 SUPPLEMENTAL EXPLORATORY BORINGS

Seventeen (17) geotechnical/geologic exploratory borings were drilled and logged within and immediately adjacent to the White Point Landslide between October 27 and November 16, 2015 to further define the subsurface geometry of the slide. These included eight large diameter bucket auger borings (AECOM-15-BA-1 through AECOM-15-BA-8) that were drilled by Tri-Valley Drilling of Ventura, California, and nine mud rotary core borings (AECOM-15-CB-1 through AECOM-15-CB-8 and AECOM-15-CB-3A) drilled by Woodward Drilling Inc. of Rio Vista, California.

The bucket auger borings were sampled at 5-foot intervals utilizing a Modified California drive sampler and were down hole logged by a CEG. The location of the exploratory borings are shown on Figure 2 (Geologic Map and Exploration Site Plan), and a discussion of the key findings from the drilling investigation is presented in Sections 3.4 and 3.5. Key data from the borings are also summarized on the interpretive geologic sections which are presented as Figures 6 through 9 and which are discussed in Section 3.6.

2.4 FIELD INSTRUMENTATION AND TESTING

Inclinometer casing was installed in eight (8) of the nine (9) mud rotary rock core borings within and adjacent to the White Point Landslide. As of the date of this geotechnical memorandum, baseline and secondary readings have been recorded in all slope inclinometer installations. These initial readings serve as the baseline comparison for determining ground movement with future readings. AECOM will record inclinometer readings periodically and after significant rainfall and seismic events, which are catalysts of slide displacement in this region.

Two vibrating wire piezometers (VWP’s) were installed in each of seven (7) mud rotary core borings (See Table 1 Summary of Geotechnical Instrumentation). In each of these borings one VWP was installed below the groundwater table and the other VWP was installed at shallower depths (~30 to 55 feet below ground surface) where anomalously high ground temperature from geothermal activity was encountered in some of the borings.
One of the mud rotary core borings (AECOM-15-CB-3) was converted to a standpipe piezometer well. The well consists of 2” PVC with a screened portion from 29–109.5 feet below ground surface (bgs). A summary of the geotechnical instrumentation is provided in the following Table 1.

<table>
<thead>
<tr>
<th>Boring</th>
<th>Depth of Boring (ft. bgs)</th>
<th>Depth of Vibrating Wire Piezometer (ft. bgs)</th>
<th>Inclinometer Casing (ft. bgs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>AECOM-15-CB-1</td>
<td>109</td>
<td>30</td>
<td>103</td>
</tr>
<tr>
<td>AECOM-15-CB-2</td>
<td>109.5</td>
<td>30</td>
<td>104</td>
</tr>
<tr>
<td>AECOM-15-CB-3</td>
<td>110</td>
<td>*standpipe piezometer</td>
<td>-</td>
</tr>
<tr>
<td>AECOM-15-CB-3a</td>
<td>79</td>
<td>35</td>
<td>75</td>
</tr>
<tr>
<td>AECOM-15-CB-4</td>
<td>134.5</td>
<td>58.5</td>
<td>118.5</td>
</tr>
<tr>
<td>AECOM-15-CB-5</td>
<td>132.5</td>
<td>60</td>
<td>115</td>
</tr>
<tr>
<td>AECOM-15-CB-6</td>
<td>124.5</td>
<td>55</td>
<td>105</td>
</tr>
<tr>
<td>AECOM-15-CB-7</td>
<td>139</td>
<td>55</td>
<td>90</td>
</tr>
<tr>
<td>AECOM-15-CB-8</td>
<td>131</td>
<td>-</td>
<td>105</td>
</tr>
</tbody>
</table>

Pressuremeter tests (PMT) were performed in two (2) select mud-rotary borings to provide a more accurate correlation of the in-situ stress-strain relationship of the underlying materials at the site. In principle, the PMT is performed by applying pressure to the sidewalls of a borehole and observing the corresponding deformation. All PMT tests were conducted in "soft" rock conditions and in general accordance with ASTM Standard Test Method D-4719.

2.5 LABORATORY TESTING

Engineering properties of select subsurface soil and rock materials were evaluated by geotechnical laboratory testing. Sample types taken in the fill/alluvium portion of the borings included, large bulk, small bulk, and relatively undisturbed ring samples. Intact cores were taken in the bedrock materials using a NX core barrel. Initial index tests including moisture and density and sieve analyses were performed for correlation with existing laboratory data.
Ring-shear testing was done on clay gouge material from the bedding parallel shear encountered in BA-2 @ ~48 ft.

3.0 SUMMARY OF GEOLOGIC CHARACTERIZATION

3.1 ROCK MASS QUALITY
General rock conditions are best exposed in the slidescarp of the landslide. The rock mass is Altamira Shale member of the Monterey Formation. The Altamira Shale bedrock at the site is generally characterized by rhythmically bedded siltstones with occasional fine to medium sandstone beds, and “chaotic zones” consisting of soft sediment folding and rip-up clasts. In general, the rock is weak to very weak with moderate to widely spaced discontinuities. The fractures exposed in the unsupported slidescarp are generally dilated 1/8”-1/4”, and up to an inch or so in some cases. The exposed rock face is in the oxidized zone, and weathering is moderate. Bedding thickness varies from thin (< 1inch) to wide (1’ to 3’). The Geological Strength Index (GSI, Hoek and Marinos, 2007) of the rock mass is at the lower end of the very blocky scale with fair to good surface conditions (GSI ~ 35-55).

3.2 LANDSLIDE MORPHOLOGY
The principal morphologic components of the White Point Landslide are informally named here and shown on Figure 1. The displaced mass of ground comprising the landslide forms a relatively coherent block, referred to as the “The Island” that is separated from “The Slidescarp” by a surficial trough referred to as “The Graben”. Near the eastern side of The Slidescarp, projecting towards the graben and island is a block of ground that is referred to as “The Peninsula”. East of the 2011 White Point Landslide is the “2009 Landslide”. Each of these landslide features is discussed in further detail below.
3.2.1 The Slidescarp (Landslide Boundary)

The Slidescarp of the landslide extends from its western margin at the beach along an approximately N50°E trend for about 370 feet where it gradually bends towards a trend of about N60°W. Approaching The Peninsula it bends further to a trend of about N50°W.

It is difficult to delineate The Slidescarp to the south and east of The Peninsula. It appears to approach and generally merge with the well-defined slidescarp that is along the northern and eastern edge of the 2009 Landslide. However it’s not evident that the 2009 landslide had any displacement or reactivation in 2011. Therefore it seems more appropriate to delineate The Slidescarp approximately along the western edge of the 2009 Landslide, which is at the eastern edge of The Island (see Figure 1). However it is not truly a slidescarp in this area as it does not form a distinctive topographic scarp. Nevertheless, it does appear that the landslide boundary is located in this general area.

The interpretation that the landslide boundary is in this area is corroborated by observations that are shown on the annotated photograph of the sea cliff that is presented as Figure 3 (Photograph of Eastern Margin of White Point Landslide). On Figure 3 the displaced Altamira Shale that forms the southeast side of The Island is on the left side of the photograph and an exposure of the 2009 landslide debris is on the right side of the photograph. Adjacent to the Altamira Shale is a swath of very coarse grained (cobble and boulder) deposit that is either landslide debris or alternatively is a surficial slope wash. At beach level this material is in contact with Altamira Shale Bedrock along a surface that is oriented N50°W, 20°SW. This contact has some characteristics (i.e. subtle slickensides) that suggest it might be a basal shear surface of the landslide.

To the east of the swath of coarse grained debris is a light reddish brown layer of gravel to sand size debris that is about 15 to 20 feet thick, is crudely stratified and which appears to project beneath and underlie the eastern side of the island. The layering in this material is oriented approximately N26°E, 33°NW, which is a reasonable orientation of the eastern margin of the White Point Landslide. Although no slickensided shears, clay gouge,
striations or other obvious indicators of shear displacement were observed, the eastern (lower) contact of this debris layer is interpreted to be the eastern margin of the White Point Landslide.

3.2.2 The Island
The Island is a coherent block of Altamira Shale bedrock that separated from the Slidescarp and moved approximately 60 feet along a bearing of about S15°W (based on the before and after position of a palm tree located on The Island). The basic morphology of The Island consists of an upper, gently south sloping surface, in the vicinity of the palm tree, a steep south facing coastal bluff along its south side, an approximately 20 foot high cliff along its northwestern side, and a shallow to moderately steep northeast sloping surface along its northeast side.

3.2.3 The Peninsula
The Peninsula is a S20°W trending block that projects into the graben area from the eastern side of The Slidescarp at Paseo Del Mar Drive. Photographs that were taken immediately after the November 20 failure show that the peninsula was a coherent block of ground with vertical walls that experienced some vertical and horizontal displacement. Based on inspection of these photographs it is estimated that the vertical displacement of the peninsula is no more than about 5 feet and the horizontal displacement is no more than about 25 feet. The Peninsula, which appears to have been approximately 30 feet high immediately after the slide, was lowered about 20 feet during the 2012 grading operation and now remains as a low protuberance into the graben area.

3.2.4 The Graben
The Graben is a surficial trough or depression that is located between The Slidescarp and The Island. It is approximately 45 feet deep, 470 feet long and up to about 90 feet wide. The Graben can be subdivided into the “main graben” and the “southeast finger” of The Graben. The main graben extends from the beach between The Slidescarp and the west side of The Island and extends towards the northeast to where it tapers between the north side of The Peninsula and The Slidescarp. The southeast finger of The Graben, which is located between the south side of The Peninsula and the northeast side of The Island, appears to be a slightly higher (less deep) depression than the main graben.
The Graben was extensively graded in 2012 to promote surface drainage. The landslide debris that fell into The Graben (i.e., asphalt and concrete, underground utilities, guard rails, etc.) was removed and then the native soil and rock debris was graded so the surface drains towards the southwest.

3.2.5 The 2009 Landslide
The 2009 Landslide is located along the east side of The Island and the eastern end of the finger graben. It has a well-defined slidescarp along its northern and eastern side. Its western side is ambiguous presumably because it was modified by, and is coincident with, the eastern side of the White Point Landslide.

3.3 KEY OBSERVATIONS FROM GEOLOGIC MAPPING
Exposures of the Altamira Shale bedrock occur in The Slidescarp, around the perimeter of The Island, and along exposures in the intertidal zone and in roadcuts outside the limits of the landslide. This section summarizes the key structural and stratigraphic observations from these geologic exposures.

3.3.1 The Slidescarp
The Slidescarp provides a nearly continuous exposure of Altamira Shale that is not displaced by landsliding. Two separate exposures occur in The Slidescarp. From about the west side of the detached Paseo Del Mar Drive to about the location of The Peninsula there exists a continuous exposure that is about 400 feet long and on average about 20 feet high. Bedding orientations measured from this exposure demonstrate that the Altamira Shale consistently strikes northwesterly (from about N30 to 80°W) and dips from about 10 to 25° towards the southwest. A calcareous marker bed that is tentatively classified as a dolostone is exposed along most of this exposure as shown on Figure 2 (Geologic Map and Site Exploration Plan) and Figure 4 (Photograph of Dolostone Marker Bed in Slidescarp Exposure).

From about the south side of the County Road to the beach, along the western side of the slidescarp is an approximately 100 foot long, 20 foot high exposure. This exposure shows that the Altamira Shale in this area is tightly folded. Just below the County Road is an exposure of a large, tight, recumbent fold that closes towards the south, has a nearly
horizontal to shallow north dipping lower limb, and steeply south dipping (~60°) upper limb (see Figure 5 Recumbent Fold in Slidescarp Exposure).

### 3.3.2 The Island

The margins of The Island provide exposures of the Altamira Shale that has been displaced by landsliding. A nearly continuous exposure of Altamira Shale occurs along the northwest side of the island. This exposure is about 200 feet long and up to about 20 feet high. Bedding orientations measured from this exposure demonstrate that the Altamira Shale along the northwest side of the island mostly strikes northwesterly (from about N30° to 80°W) and dips from about 15 to 20° towards the southwest, which is similar to the orientation of bedding in the eastern exposure of The Slidescarp. A nearly continuous exposure of Altamira Shale that is approximately 300 feet long and up to about 100 feet high exists along the coastal bluff (sea cliff) that forms the south side of the island.

The basal shear at the toe of the landslide is poorly and discontinuously exposed at the base of the sea cliff on the south side of The Island. A very narrow shear that is oriented N80°W, 5°SW and which had striations that rake steeply was found near the southwest side of the island. At the southeast side of the island the surface that is oriented N50°W, 20°SW (noted above in the Slidescarp landslide boundary section, and shown on Figure 3) is also interpreted to be basal shear surface at the landslide toe. Although these exposures provide only subtle evidence for landslide shearing, it is evident that the landslide basal shear must daylight at the base of the sea cliff based on comparison of pre- and post-landslide photographs and Google Earth imagery. The photographs and images unequivocally demonstrate that the base of the sea cliff was displaced southward at least 40 feet during the catastrophic failure on November 20, 2011.

There are two key marker beds that correlate with beds recognized in the borings exposed on this bluff face, as delineated on Figure 2. One of these beds is a cemented siltstone that is characterized by tar filled fractures (designated marker bed 8 on Figure 2). It crops out near the base of the bluff, at an elevation of about 8 to 13 feet for a length of about 120 feet. The other marker bed is the dolostone that was also recognized in The Slidescarp (designated marker bed 11 on Figure 2). The dolostone bed crops out from about elevation 90 feet on the southeast side of The Island to about elevation 60 feet on the southwest side.
of The Island. It also is exposed in a small outcropping on the north side of The Island as shown on Figure 2.

3.3.3 Outside the Landslide

Structural data (mostly bedding attitudes) were also measured from bedrock exposures outside the limits of the landslide to provide some insight into the geologic structure of the Altamira Shale that might be controlling the location of the western limits of the landslide. The data were mostly collected from the base of the bluff to the west of the landslide and from bedrock exposures in the intertidal zone. Bedding exposed in the intertidal zone mostly dips from about 5 to 20° degrees towards the southwest except near the western end of the landslide where there are shallow to steeply east dipping, and near vertical bedding. This deviation from the shallow southwest dipping bedding that is prevalent in most of the landslide area appears to be a manifestation of the same folding that is exposed along the western exposure of The Slidescarp (the fold shown on Figure 5). The bedding orientations from the intertidal zone and the observed fold at the west end of The Slidescarp suggest that a large (map scale size), northwest trending, recumbent fold comes on shore at the west end of the landslide. This suggests that the landslide developed in the shallow out-of-slope dipping beds that form the lower limb of this recumbent fold and the portion of the bluff immediately to the west of the landslide (which did not fail) is characterized by steeply in-to and out-of-slope dipping and folded beds that are less susceptible to instability due to the orientation of the beds. However, further west bedding generally returns to shallow out-of-slope orientation, which raises concern about the possibility of an incipient landslide in this area, as discussed in section 3.6.1 below.

3.4 KEY OBSERVATIONS FROM DRILLING INVESTIGATIONS

The seventeen borings that were drilled within and near the landslide were located in 8 boring clusters that consisted of one bucket auger boring and 1 or more adjacent core borings. The borings encountered five basic geologic materials that are herein referred to as:

1. in-place bedrock;
2. displaced bedrock;
3. landslide debris;
4. Quaternary terrace deposits, and
5. artificial fill.

In place bedrock (designated as Tma on Figures 2 and 6 through 9) is that portion of the Altamira Shale that is outside the limits of the landslide Slidescarp and below the landslide basal shear. Conversely, displaced bedrock (designated as Qls/Tma on Figures 2 and 6 through 9) is that portion of the Altamira Shale that is inside the limits of the landslide Slidescarp and above the basal shear. Landslide debris (designated as af/Qls on Figures 6 through 9) is obviously disrupted blocks of rock and rubble that has in part been reworked by the grading operations within the graben. Quaternary terrace deposits (designated as Qt on Figures 2 and 6 through 9) form an approximately 5 to 10 foot thick surficial layer above bedrock outside of the landslide and on the crest of The Island. Artificial fill (af) was encountered outside the landslide slidescarp on the east side in the vicinity of Paseo Del Mar Drive. The following subsections summarize the key finding of each boring cluster.

3.4.1 Boring Cluster 1

Boring Cluster 1 consists of a bucket auger boring (AECOM-15-BA-1) to a depth of 72 feet and an adjacent core boring (AECOM-15-CB-1) to a depth of 109 feet. Bucket Auger Boring AECOM-15-BA-1, encountered landslide debris (af/Qls) down to a depth of 14 feet, and in place bedrock (Tma) below the debris. The Altamira Shale bedrock in the boring was intact interbedded siltstone and less common sandstone that is essentially devoid of fractures. Neither the landslide basal shear nor any other discontinuity related to either landsliding or tectonic activity was found in AECOM-15-BA-1. Bedding consistently dipped from about 10 to 25 degrees towards the southwest to the maximum depth that was logged. The Altamira Shale bedrock encountered in AECOM-15-BA-1, as well as all other borings and exposures, is characterized by mostly rhythmically bedded siltstones with occasional sandstone beds, and “chaotic zones” consisting of soft sediment folding and rip-up clasts. As a consequence of the geothermal activity that characterizes the White Point Area, the atmosphere of the bucket auger boring was characterized by elevated temperatures (approximately 97° F) and high humidity from depths of about 14 to 44 feet bgs.
Core boring AECOM-15-CB-1, which was drilled about 23 feet to the northwest of AECOM-15-BA-1, encountered landslide debris (af/Qls) down to a depth of 12 feet and Altamira Shale bedrock (Tma) below that. Following completion of the core drilling, the boring was equipped with an inclinometer and two vibrating wire pressure transducers, one at 30 feet bgs and the other at 103 feet bgs. The deeper of the two transducers indicates a groundwater table at about 54 feet bgs (el. 25 feet). The upper transducer at 30 feet bgs recorded an elevated ground temperature of the geothermal zone as 106°F.

3.4.2 Boring Cluster 2
Boring Cluster 2 consists of a bucket auger boring (AECOM-15-BA-2) to a depth of 87.5 feet and an adjacent core boring (AECOM-15-CB-2) to a depth of 109.5 feet. Bucket Auger Boring AECOM-15-BA-2, encountered landslide debris (af/Qls) down to at least 14 feet below ground surface, displaced bedrock (Qls/Tma from 14 to 48 feet, and in place bedrock below that. From 14 to about 26 feet depth, the Altamira Shale bedrock was highly weathered, and/or disrupted and deformed by landsliding (af/Qls). Bedding was noticed from about 18.5 to 20 feet and its orientation was measured as anomalously steep (N80°W, 50°N). However below this depth to about 26 feet bgs the bedding was indistinct and it’s uncertain if this material is displaced bedrock (Qls/Tma) or landslide debris (af/Qls). Elevated temperatures and high humidity existed in AECOM-15-BA-2 from about 18 to 45 feet bgs. These inhospitable conditions prevented detailed downhole logging and therefore an unequivocal and precise elevation of the top of displaced bedrock (Qls/Tma) was not determined. The material below about 26 feet is considered to be displaced bedrock (Qls/Tma) with a high level of confidence. As discussed below, based on stratigraphic correlations the rock below a depth of 48 feet in this boring is believed to be in-place bedrock (Tma).

Bedding orientation data, which was recorded from 48 feet to the lower limit of down hole logging at 78 feet, consistently dipped from about 10 to 15° towards the southwest. At 48 feet bgs the boring encountered a bedding parallel shear that was oriented N40°W, 12°SW. This shear, which was about ½ inch thick and consisted of a moderately plastic, slickensided, clay gouge, might be the basal shear of the landslide, as discussed below in Section 3.6.
Core Boring AECOM-15-CB-2, which was drilled about 10 feet to the east of AECOM-15-BA-2, encountered landslide debris (af/Qls) down to at least 13.5 feet bgs, displaced bedrock (Qls/Tma) from 13.5 feet to about 44 feet and in-place bedrock below that. Core sampling in CB-2 began at 9.5 feet depth but recovery was only about 55% above 29 feet depth. Thus the nature of the materials encountered in the upper part of the boring (i.e., the depth of the contact between af/Qls and Qls/Tma) is open to some interpretation. Steeply dipping beds (65°) were encountered at depths of 18-19.5 feet. However, with poor recovery above and below this core sample it is uncertain whether this material is displaced bedrock (Qls/Tma), or if these steeply dipping beds represent a large boulder within the landslide debris (af/Qls). Following completion of the core drilling the boring was equipped with an inclinometer and two vibrating wire pressure transducers, one at 30 feet bgs and the other at 104 feet bgs. The deeper of the two transducers indicates a groundwater table at about 82.5 feet bgs (el. 27 feet). The upper transducer at 30 feet bgs recorded an elevated ground temperature of the geothermal zone as 118°F.

3.4.3 Boring Cluster 3
Boring Cluster 3 consists of a Bucket Auger boring (AECOM-15-BA-3) to a depth of 87 feet, an adjacent core boring (AECOM-15-CB-3) to a depth of 109.5 feet, and another adjacent core boring (AECOM-15-CB3A) to a depth of 79 feet. Bucket Auger boring AECOM-15-BA-3, encountered landslide debris (af/Qls) down to a depth of at least 23 feet bgs, displaced bedrock (Qls/Tma) from 23 feet to approximately 44 feet and in place bedrock (Tma) below that. From 23 to about 30 feet depth the Alta mira Shale bedrock is highly weathered, and/or disrupted and deformed. Bedding was noticed from about 23 to 26 feet and its orientation was measured as dipping anomalously steep (70°). However from 26 to 30 feet depth the bedding is indistinct, the material is soil like and it’s uncertain if it is displaced bedrock (Qls/Tma) or Unit landslide debris (af/Qls). Elevated temperatures and high humidity were experienced from about 10 to 40 feet, with peak temperatures of 100°F. These inhospitable conditions prevented detailed downhole logging and therefore an unequivocal and precise determination of the top of displaced bedrock (Qls/Tma). The material below about 30 feet depth was interpreted to be displaced bedrock (Qls/Tma) with a high level of confidence. Stratigraphic correlations discussed in a later section suggest that below a depth of about 44 feet, the rock is in-place (Tma) and was not involved in the landslide. Bedding orientation data was recorded from about 42 feet to the lower limit of
downhole logging at 78 feet. The bedding consistently dipped from about 10 to 20° towards the southwest.

Core Boring AECOM-15-CB-3, which was drilled about 5 feet to the southeast of AECOM-15-BA-3, encountered landslide debris (af/Qls) down to a depth of at least 18.5 feet, displaced bedrock (Qls/Tma) from 18.5 feet to approximately 44 feet and in place bedrock (Tma) below that. Core sampling in AECOM-15-CB-3 began at 4.5 feet depth but recovery was only about 23% above 36 feet depth. Thus the subsurface conditions in the upper part of the boring are open to some interpretation. For example, it is uncertain whether the rock encountered at 18.5 feet is displaced bedrock (Qls/Tma), or if the recovered core is from large blocks of bedrock encompassed within the landslide debris (af/Qls). Furthermore, the no recovery zones can be considered possible locations where the landslide basal shear could cut through the boring. From about 23 feet to the bottom of the hole the bedding consistently dipped from about 5 to 20 degrees. Following completion of the core hole the boring was equipped with a standpipe piezometer to 109 feet bgs.

Core Boring AECOM-15-CB-3A, which was located about 7 feet to the west of AECOM-15-BA-3, was drilled with a tri-cone rock bit and neither sampled nor logged from zero to 36 feet depth. Core sampling in CB-3A was done from 36 feet to the bottom of the boring at 79 feet. Following completion of core drilling, the boring was equipped with an inclinometer and two vibrating wire pressure transducers, one at 35 feet bgs and the other at 75 feet bgs. The deeper of the two transducers indicates a groundwater table at about 48.64 feet bgs (el. 27 feet). The upper transducer at 35 feet bgs recorded an elevated ground temperature of the geothermal zone as 104°F.

3.4.4 Boring Cluster 4

Boring Cluster 4 consists of a bucket auger boring (AECOM-15-BA-4) to a depth of 106 feet and an adjacent core boring (AECOM-15-CB-4) to a depth of 134.5 feet. Bucket Auger boring AECOM-15-BA-4, encountered Quaternary terrace deposits to a depth of 5 feet and Altamira Shale bedrock below that. From 5 feet to the maximum depth observed (approximately 98 feet) the bedding consistently dipped from about 5 to 20 degrees towards the southwest.
Core Boring AECOM-15-CB-4, which was drilled about 5 feet east of AECOM-15-BA-4, encountered Quaternary terrace deposits to a depth of 5.5 feet and Altamira Shale bedrock below that. Following completion of the core drilling, the boring was equipped with an inclinometer and two vibrating wire pressure transducers, one at 58.5 feet bgs and the other at 118.5 feet bgs. The deeper of the two transducers indicates a groundwater table at about 80 feet bgs (el. 44 feet). The upper transducer at 58.5 feet bgs is recording normal (not elevated) temperature of the ground as 73°F.

3.4.5 Boring Cluster 5

Boring Cluster 5 consists of a bucket auger boring (AECOM-15-BA-5) to a depth of 112 feet and a core boring (AECOM-15-CB-5) to a depth of 132.5 feet. The core boring was drilled 50 feet to the east of the bucket auger boring. Bucket Auger Boring AECOM-15-BA-5, encountered Quaternary terrace deposits (Qt) to a depth of 6 feet and Altamira Shale bedrock (Tma) below that. From 6 feet to the maximum depth observed (approximately 110 feet) the bedding consistently dipped from about 5 to 20 degrees towards the southwest.

Core Boring AECOM-15-CB-5 encountered Quaternary Terrace deposits (Qt) to a depth of 9.5 feet and Altamira Shale bedrock (Tma) below that. The bedding consistently dipped from about 10 to 20 degrees. Following completion of the core drilling the boring was equipped with an inclinometer and two vibrating wire pressure transducers, one at 60 feet bgs and the other at 115 feet bgs. The deeper of the two transducers indicates a groundwater table at about 78 feet bgs (el. 48 feet). The upper transducer at 60 feet bgs recorded a ground temperature of 80°F.

3.4.6 Boring Cluster 6

Boring Cluster 6 consists of a bucket auger boring (AECOM-15-BA-6) to a depth of 122 feet and an adjacent core boring (AECOM-15-CB-6) to a depth of 124.5 feet. Bucket Auger boring AECOM-15-BA-6, encountered artificial fill (af) to a depth of 6 feet and Altamira Shale bedrock (Tma) below that. Bedding consistently dipped from about 5 to 20° towards the southwest to a depth of about 106 feet. From 105 feet to the lowest observed portion of the boring at 117 feet, the bedding steepens to about 50 to 60° to the southwest.
Core Boring AECOM-15-CB-6, which was drilled about 10 feet southeast of AECOM-15-BA-6, encountered artificial fill to a depth of 10.2 feet (including concrete from 6 to 10.2 feet) and Altamira Shale bedrock (Tma) below that. From the top of the rock to the bottom of the hole at 124.5 feet the bedding consistently dipped from about 5 to 20 degrees. The steeper bedding noted in the bottom of the adjacent bucket auger boring, below 105 feet was not evident. Following completion of the core hole the boring was equipped with an inclinometer and two vibrating wire pressure transducers, one at 55 feet bgs and the other at 105 feet bgs. The deeper of the two transducers indicates a groundwater table at about 82 feet bgs (el. 41 feet). The upper transducer at 55 feet bgs recorded a ground temperature of 73°F.

3.4.7 Boring Cluster 7

Boring Cluster 7 consists of a bucket auger boring (AECOM-15-BA-7) to a depth of 117 feet and an adjacent core boring (AECOM-15-CB-7) to a depth of 139.0 feet. Bucket Auger boring AECOM-15-BA-7, encountered Quaternary terrace deposits (Qt) to a depth of 5.5 feet and Altamira Shale bedrock (Tma) below that. Bedding orientations in AECOM-15-BA-7 are slightly more variable than observed in the other borings. Bedding mostly dipped from about 10 to 20° in a southerly direction to the bottom of the boring. The strike of these south dipping beds ranged from about N60-80°W and from N40-80°E. Thus, both southeasterly and southwesterly dips were common. Also there were a couple of relatively anomalous bedding orientations measured at depths between about 14 and 25 feet (N10°E, 6°NW, and N2°E, 13°NW) that suggests the presence of folding.

Core Boring AECOM-15-CB-7, which was drilled about 6 feet northeast of AECOM-15-BA-7, encountered Quaternary terrace deposits (Qt) to a depth of 8.2 feet and Altamira Shale bedrock (Tma) below that. Following completion of the core hole the boring was equipped with an inclinometer and two vibrating wire pressure transducers, one at 55 feet bgs and the other at 90 feet bgs. The deeper of the two transducers indicates a groundwater table at about 78 feet bgs (el. 45 feet). The upper transducer at 55 feet bgs recorded a ground temperature of 73°F.
3.4.8 Boring Cluster 8

Boring Cluster 8 consists of a bucket auger boring (AECOM-15-BA-8) to a depth of 120 feet and an adjacent core boring (AECOM-15-CB-8) to a depth of 139.0 feet. Bucket Auger boring AECOM-15-BA-8, encountered Quaternary terrace deposits (Qt) to a depth of 8 feet and Altamira Shale bedrock (Tma) below that. Bedding orientations in AECOM-15-BA-8 are similar to those in AECOM-15-BA-7 and slightly more variable than observed in the other borings (AECOM-15-BA-1 to AECOM-15-BA-6). Bedding mostly dipped from about 0 to 25° in a southerly direction but the strike varied. Most strikes were northwesterly, ranging from N30-88°W. Less common strikes were northeasterly, ranging from N40-80°E. Also there was an anomalous bedding orientation of N30°W, 20°NE measured at a depth of 25 feet, which might be the limb of a tight, recumbent fold that was intersected between 23 and 27 feet depth. At 48 feet bgs the boring encountered a shear zone that was oriented N80°W, 33°SW. This shear consists of about a 2 to 10 inch wide zone of numerous slickensided and striated shear planes.

Core boring AECOM-15-CB-8, which was drilled about 16 feet northeast of AECOM-15-BA-8, encountered Quaternary terrace deposits (Qt) to a depth of 7.2 feet and Altamira Shale bedrock (Tma) below that. Bedding in the upper 29 feet was moderately steep, and possibly could be a manifestation of the possible fold that was encountered in AECOM-15-BA-8. Below about 29 feet depth bedding consistently dipped from about 5 to 25°. The shear zone that was noted above in AECOM-15-BA-8 at 48 feet was intersected at 45.4 to 46.8 feet and was about 12 inches wide and dipping about 30 to 45°. Following completion of the core hole the boring was equipped with an inclinometer to a depth of 105 feet.

3.5 ESTIMATION OF BASAL SHEAR SURFACE

The upslope extent of the landslide movement is constrained by the Slidescarp, and based on geologic mapping and analysis of photographs taken before and after the landslide, the toe of failure is likely at or very near the base of the bluff. A primary objective of the subsurface drilling was to locate the landslide’s basal shear beneath The Graben and The Island. Unfortunately, only negligible to no direct evidence of the basal shear was discovered in the bucket auger and core borings that were drilled into the landslide. The basal shear was not evident in Boring Cluster 1 or Boring Cluster 3, and only a thin shear
that may be the basal shear was logged at a depth of 48 feet in AECOM-15-BA-2 of Boring Cluster 2. Fortunately the Altamira Shale is well stratified with several distinguishable “marker beds” that are laterally continuous and can be correlated between borings. Consequently, the depth of the basal shear was estimated by correlating the stratigraphy of the Altamira Shale between borings (on cross sections) and using the premise that the basal shear must be located between the stratigraphically highest layer that is not displaced by landsliding and the stratigraphically lowest layer that is. This section summarizes the stratigraphy that was developed by a review of the core samples and supplemented by recognition of some of these strata in geologic exposures.

Eleven “marker beds” were recognized in the core and designated Unit 1 through Unit 11, with Unit 1 being the deepest, stratigraphically lowest, marker bed and Unit 11 being the stratigraphically highest. These units are summarized on Table 2, including a description of each unit and their elevation in each boring. These beds are shown on geologic cross sections B-B’, C-C’ and the east side of D-D’ as discussed in the following section. The west side of D-D’, including borings AECOM-15-CB-7 and AECOM-15-CB-8 were not included in this analysis because of the structural complexity (folding) of the west side which makes correlation impractical.
3.6 GEOLOGIC CROSS SECTIONS

Geologic Cross Sections A-A’, B-B’, C-C’ and D-D’, which are presented as Figures 6 through 9, provide the geologic interpretations of the landslide subsurface geometry based on consideration of the available data. The locations of these sections are shown on the Geologic Map and Site Exploration Plan presented as Figure 2. The following paragraphs describe the key elements of the subsurface interpretations shown on each section.

3.6.1 Section A-A’

Section A-A’ (Figure 6) is a slope perpendicular section located well to the west of the landslide, looking towards the southeast, and crossing through the Cluster 8 borings. On section A-A’ the principal feature of interest is the shear zone that was encountered in AECOM-15-BA-8 at 48 feet bgs and in AECOM-15-CB-8 at about 46 feet bgs. The shear
zone is about 10 to 12 inches wide and oriented N80°W, 33°SW. Of related interest is an approximately 3 foot high, arcuate break in slope of the ground surface that is located about 40 feet to the north of Boring Cluster 8, and which is delineated by the 125 foot elevation contour on Figure 2. It’s plausible that this slope break is the result of incipient landslide movement that passes through the shear zone at 48 feet. Furthermore, directly downslope of BA-8 is a small landslide that is located between the County Road and the beach. Section A-A’ illustrates our speculation that an incipient landslide might be located in this area, with the slidescarp located at the arcuate break in slope north of Boring Cluster 8, and the toe of the slide at beach level.

3.6.2 Section B-B’

Section B-B’ (Figure 7) is a slope perpendicular section through the central portion of the White Point Landslide, looking towards the southeast, and crossing through Boring Clusters 2, 3, and 4 as well as S&W borings B-5 and B-9. Stratigraphic correlation between borings along Section B-B’ suggests that the basal shear of the landslide beneath the graben is located above unit 7 and below Unit 8. Units 1 through 7 do not exhibit discernable vertical displacement from landsliding. Whereas Unit 8 in AECOM-15-CB-3 is intersected at an elevation that is about 8 to 10 feet lower than would be expected if it were not displaced by the landslide. Direct evidence of a basal shear surface between units 7 & 8 in AECOM-15-CB-3 was not found but could be interpreted to cut through a 3 to 4 inch no recovery zone that is located between these layers. Likewise direct evidence of a basal shear was not observed in the adjacent bucket auger boring (AECOM-15-BA-3) that was down-hole logged. However, the geothermal zone encountered in the upper part of this boring created an atmosphere that was inhospitable for detailed down-hole logging, so careful inspection of that portion of the boring could not be accomplished. In AECOM-15-CB-2 unit 8 was encountered higher than in AECOM-15-CB-3 and does not appear to be obviously displaced. As illustrated on Figure 7, this might be explained by a north dipping shear that borders the north side of The Island between AECOM-15-CB-2 and AECOM-15-CB-3. Boring AECOM-15-BA-2 encountered what might be the only direct evidence of a basal shear in any of the borings that were drilled into the landslide. A bedding parallel shear that is oriented N40°W, 12°S was encountered at 48.7 feet below ground surface (elevation 29 feet). In the adjacent core boring (AECOM-15-CB-2) there was a no recovery zone and a report by the driller that the boring drilled very soft between 43 and 45 feet bgs (elevation ~34 feet). Collectively, the apparently displaced Unit 8 in AECOM-15-CB-3, the
shear encountered in AECOM-15-BA-2, and the no recovery/soft drilling zone in AECOM-15-CB-2 suggest that the basal shear for the White Point Landslide is at an elevation of about 30 to 40 feet beneath The Graben surface along Section B-B’.

Units 9 and 10 were not recognized in any of the borings on Section B-B’ but Unit 11 (the dolostone bed) was intersected in AECOM-15-CB-4 and AECOM-15-BA-4. It was also exposed in The Slidescarp (Figure 4), in a small outcropping along the north side of the island, and across the bluff face that forms the south side of the island. The base of Unit 11 is at elevation 113.19 in CB-4, at about elevation 110 feet in The Slidescarp exposure, at about 83 feet elevation along the north side of The Island and about 65-75 feet elevation on the south side of the island along Section B-B’. Considering these elevations, it appears that the Unit 11 exposures on The Island have been down-dropped by landslide movement about 15 vertical feet.

A perplexing aspect of Section B-B’ is that the drop in the topographic surface in the graben area which is about 45 feet, far exceeds the interpreted vertical displacement of Unit 8 in AECOM-15-CB-3, which is about 8-10 feet. One explanation is that The Graben is not a true graben in the geologic sense, rather it’s a topographic chasm produced by the pulling apart of The Island from The Slidescarp along a basal shear that is located approximately at, or very near, the surface of the graben. If this is correct, then it’s conceivable the basal shear was excavated out during site grading operations, and this would explain the absence of obvious evidence for a basal shear in the seven borings that were drilled in the landslide. A possible explanation for the apparent offset of Unit 8 is that this bed is not a primary stratigraphic bed, rather its recognizable features (it is silicified and contains tar filled fractures) have a secondary origin that can cut across primary stratigraphic layering. An example of this can be observed at the base of the bluff (seacliff) near the southwest end of The Island, where a silicified layer with tar filled fractures (interpreted to be Unit 8) abruptly steps stratigraphically higher to the east, about 3 feet. Thus, Unit 8 might not represent a primary stratigraphic layer that can be confidently utilized for constraining displacement.

\[1\] A graben is a depressed block of land bordered by parallel faults. A graben is the result of a block of land being downthrown producing a valley with a distinct scarp on each side.
Considering the above discussion, there is some uncertainty regarding the depth of the basal shear below the graben. The lower basal shear shown as the red dashed line on Section B-B’ is the deepest estimated location based on stratigraphic correlation and assuming that Unit 8 is displaced by landsliding. However, the basal surface may be shallower if unit 8 is not displaced by landsliding. An interpretation of a higher basal shear is shown as the red dotted line on Section B-B’. Future monitoring of the inclinometers that were installed in the landslide may provide data that can further reduce or eliminate this uncertainty.

3.6.3 Section C-C’
Section C-C’ (Figure 8) is approximately slope perpendicular through the eastern portion of the White Point Landslide, looking towards the southeast, and crossing through core borings CB-1 and CB-5. The borings along Section C-C’ provide stratigraphic evidence that suggests that the basal shear of the landslide beneath the easternmost margin of the graben is located about 3 to 12 feet bgs depending on how the geology of the uppermost portion of the core from boring AECOM-15-CB-1 is interpreted. Boring AECOM-15-CB-1 encountered unequivocal landslide debris from 0-3 feet, a hard dolostone that dips 60° from about 3-6 feet, no recovery from 6-12 feet, and Altamira Shale bedrock that dips from about 10-20° from 12 feet and below. This core record is compatible with two alternative geologic interpretations. One is that the dolostone at 3-6 feet is the top of in-place bedrock and the other interpretation is that the dolostone is a boulder that is encompassed within landslide debris that also includes the no recovery zone down to 12 feet below ground surface. If the dolostone at 3-6 feet is the top of bedrock, then it would appear that Units 1 through 7 and Units 9 & 10 do not exhibit discernable vertical displacement from landsliding. Therefore the basal shear would presumably be above unit 10 (the lower dolostone).

Although direct evidence of a basal shear surface above Unit 10 in CB-1 was not found, it could have been excavated away during the post-landslide grading operations. The alternative interpretation, which is shown on Figure 8, is that the dolostone encountered in AECOM-15-CB-1 is actually a boulder that is encompassed within the landslide debris. This allows the possibility that the basal shear could cut through the no recovery zone about 6-
12 feet below ground surface. The adjacent bucket auger boring, which was located approximately 27 feet to the east encountered landslide debris to a depth of 14 feet and intact bedrock with no apparent basal shear below that. Considering the information from both the bucket auger boring and the core boring, it is likely that the basal shear is at about 12 to 14 feet bgs, corresponding to an elevation that is no lower than about 64 feet. It is conceivable that the basal shear may have been excavated out in the graben area where AECOM-15-BA-1 and AECOM-15-CB-1 were located. With this alternative interpretation, Unit 9 is the highest stratigraphic layer that is not displaced by landsliding and Unit 10 is the lowest layer that presumably is displaced. For Section C-C’, this second alternative was assumed, and the estimated basal shear is interpreted to be slightly steeper than bedding and daylights at the base of the bluff (i.e. at the possible basal shear exposure shown on Figure 3 that is oriented N50°W, 20°SW).

Upslope of Boring AECOM-15-CB-1, Section C-C’ cuts through The Peninsula and as noted previously, The Peninsula experienced relatively minor vertical and horizontal displacement. Thus, the primary basal shear surface likely daylights between the south side of The Peninsula and CB-1.

The basal shear of the landslide is presumably exposed at the base of the bluff very near section C-C’ (the basal shear exposure shown on Figure 3 that is oriented N50°W, 20°SW). Thus, the geometry of the slide at C-C’ would appear to consist of a basal shear that dips slightly steeper than bedding between AECOM-15-CB-1 and its exposure at the base of the bluff.

3.6.4 Section D-D’
Section D-D’ (Figure 9) is mostly a slope parallel section that cuts through the White Point Landslide, looking towards the north, and crossing through or near Boring Clusters 8, 7, 2, 1, and 6. It also intersects sections A-A’, B-B’ and C-C’ and therefore affords an important check to verify that those sections provide a reasonable interpretation in three dimensions. Due to the structural complexity caused by folding on the west side (which would make correlation between borings uncertain), the “marker bed” strata that are discussed above in Section 3.5 were only applied to the portion of the section at and east of Boring Cluster 2. From CB-6 to Boring Cluster 2 the strata have westward dips, which is compatible with
observations of bedding in the nearby and approximately parallel Slidescarp exposure. Thus, the surface data from geologic mapping corroborates these subsurface correlations of strata.

As shown on Section D-D’ the landslide basal shear is interpreted to be deepening towards the west, approximately parallel to, or slightly steeper than the bedding in the Altamira Shale. Based on the data, the deepest portion of the basal shear surface is estimated to be west of Boring Cluster 2.

The shear zone noted above that was encountered in AECOM-15-BA-8 at 48 feet bgs and in AECOM-15-CB-8 at about 46 feet bgs is shown at the west side of the section in the immediate vicinity of Boring Cluster 8. Although it is plausible that this shear surface could be evidence for an incipient landslide, as shown on Section A-A’ (Figure 6) its geometry is not adequately understood to speculate on the lateral extent of this feature on Section D-D’.

4.0 GEOTECHNICAL ENGINEERING ANALYSIS

The data obtained from subsurface exploration, geologic mapping, and laboratory and in-situ field testing were utilized for evaluating the most likely geologic conditions leading to the 2011 landslide. This was accomplished by performing a series of back-analyses simulating a range of possible geologic conditions in an attempt to trigger slope failure (Safety Factor, FS=\(<1.0\)). The set of parameters (and the distribution of rock-mass vs. bedding-plane strength) which resulted in FS=\(<1.0\) was subsequently used for forward-analyzing the structural-support demand of remedial options. The objective was to achieve a minimum Safety Factor of FS=1.5 for remedial options under static conditions.
4.1 BACK-ANALYSIS OF 2011 LANDSLIDE

4.1.1 Analysis Tools

Shear-Strength Reduction Method of Slope Stability Analysis

All slope stability analyses were performed with FLACv7.0 (Itasca, 2011) utilizing the shear-strength reduction technique (Matsui & San, 1992). With this method, the soil/rock-material strength is reduced in stages until failure occurs, and the factor of safety is the ratio of the material’s actual shear strength to the reduced shear strength triggering failure. One important advantage of this technique over conventional limit-equilibrium analysis is the ability to find the critical failure mode automatically and without the geometric or kinematic inconsistencies associated with rigid-body trial-failure surfaces. Dawson et al (1999) demonstrated that the definition of the safety factor obtained with this technique is identical to that used in conventional limit-equilibrium analyses.

Ubiquitous Joint Model

The coastal buffs at the site consist of relatively weak, fractured sedimentary rocks with distinct, generally out-of-slope dipping bedding planes. For analyzing slope stability in such complex geologic materials it is important to consider the relationship between rock mass (cross-bedding) and bedding plane shear strength. To take this relationship into account, FLAC’s ubiquitous joint model for analyzing the stability of a rock mass containing planes of weakness was employed. This anisotropic-plasticity model simulates bedding planes, weak layers, or joints embedded in a Mohr-Coulomb solid material.

The model accounts for sliding as well as tensile separation on the weak planes, while incorporating the strength of the rock mass between the weak planes. The ubiquitous joint model allows for sliding along slip surfaces without requiring the placement of actual interface elements in the numerical mesh. Sensitivity analyses covering a range of possible bedding configurations can thus be analyzed quickly by simply specifying different dip angles and locations of the weak planes, rather than creating a new numerical mesh with sliding interfaces at new orientations.
Rock-Mass Strength Model

Rock-mass strength was estimated using the Hoek-Brown methodology (Hoek and Marinos, 2007). Corresponding Mohr-Coulomb strength parameters were computed for each element of the FLAC model mesh with the program RocLab (Hoek et al., 2002). Key input parameters required for the Hoek-Brown evaluation are the Unconfined Compressive Strength (UCS) of intact rock, the Geological Strength Index (GSI), and a disturbance factor D. Guidelines for estimating the empirical values of GSI and D are provided in Figures 10 and 11, respectively.

4.1.2 Back-Analyses of Cross Section B-B’

Back-analyses of Section B-B’ were performed with FLAC using the model mesh shown in Figure 12. Rock-mass and bedding-plane strength values were varied within ranges estimated based on laboratory test results and geologic evaluations. The groundwater table and orientation of bedding planes were assumed to be the same for all cases analyzed. The analyses were conducted using the strength-reduction method combined with the ubiquitous-joint model and rock-mass strength formulation described above.

Pre-Sliding Groundwater Table

Since the available GW data had been recorded sometime after the slide event, best-estimate assumptions were made about the GW conditions which may have led up to the Nov 20, 2011 landslide. The thinking was that the GW table would have likely dropped immediately after the slide occurred, due to drainage through the exposed headscarp. Hence, the pre-sliding GW table assumed for back-analysis was selected with a bias towards data on the high side of those measured after the slide event. The following outlines our reasoning for selecting GW EL +75’ for back-analysis:

1. GW EL. +75’ at SW's B-4 was measured during drilling on 12/8/11, less than 3 weeks after the slide. Being some distance away from the head scarp, it was thought to have been less affected by drainage through the headscarp; and

2. GW EL. +74.5’ at SW's B-1 was derived from pore-pressure recorded by a vibrating-wire piezometer (VWP) some 6 months after the slide event. Having recovered steadily since its installation about 5.5 months earlier (~1.5 months after the slide), it was thought possible that the sudden breakaway of the 'island' may have created zones of expansion which resulted in a temporary drop of pore pressure behind the headscarp. Thus, locations
close to the headscarp may have experienced an apparent drop in GW level, followed by slow recovery.

For the final-design phase we plan on further investigating the effect of the GW table on back-calculated strength values. It should be noted, however, that the GW assumptions for back-analysis are somewhat “self-correcting” when it comes to forward analysis of remedial measure, provided the GW table is maintained at, or below, the level assumed for back-analysis.

**Bedding-Plane Strength**

Bedding-plane shear strength as a function of effective normal stress was derived from ring-shear test results. The upper graph of Figure 14 shows the test results reported by S&W (2012a). This sample, which contained bentonite, had been concluded by S&W to be representative of the basal shear plane of the 2011 landslide. Our recent investigation, however, did not confirm the existence of any continuous bentonite layer(s).

As part of the current investigation, a new set of ring-shear tests was conducted on gouge material encountered in a bucket-auger boring drilled in the graben area. The test results, which show significantly higher bedding-plane strength, are presented in the bottom graph of Figure 13.

All but the first trial case of back-analysis were conducted with the higher bedding-plane strength. The switch to higher strength was prompted by the outcome of the first analysis case which produced a back-calculated safety factor, FS, significantly less than unity, even when assigning a high-end Geologic Strength Index, GSI=60, for the rock mass throughout the entire model.

**Rock-Mass Strength**

The following rock-mass properties used for back-analysis are shown in Figure 14:

1. Unconfined Compressive Strength (UCS) of intact rock was based on a median value of unconfined compression test data reported by S&W (2012a) as shown in Figure 13. UCS and the elastic properties of the rock mass, also shown in this figure, were kept constant for all back-analysis cases;

2. Geological Strength Index (GSI) was varied within a range of GSI=30-60, with zones of lower GSI values in the upper zones of the bluff, while GSI for rock below El +/-0.0’, was
kept constant at GSI=60.

3. Disturbance Factor, D, was varied between D=0 and D=0.7. The latter value was applied in accordance with the bottom two categories described in Figure 11. Since the bluff face, being exposed to wave action, has been eroding over time, it is conceivable that stress relief with tension cracks forming caused a degree of damage comparable to that suffered by “good blasting” or “mechanical excavation.”

The Hoek-Brown yield surfaces are shown in Figure 15.

Analysis Results
Analysis results are presented in Figures 16 through 20, in terms of critical failure zones which are overlain on top of the best-estimate geologic interpretation of shear planes activated in the 2011 landslide event. The contours of maximum shear-strain rates shown in the figures indicate the critical failures zones. These zones of distress develop at the instant when equilibrium is lost during the gradual strength-reduction process involved in the method of slope stability analysis used. The following Table 3 summarizes assumptions, findings, and conclusions based on the back-analysis results.

Table 3 – Summary of Back-Analysis Results

<table>
<thead>
<tr>
<th>Case</th>
<th>FS</th>
<th>Bedding Strength</th>
<th>Rock-Mass Strength</th>
<th>Comparison of critical failure zone(s) with Geologic Interpretation</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.91</td>
<td>#1</td>
<td>GSI=60 D=0</td>
<td>Good match with deep-basal shear interpretation and head scarp; but no failure on ‘island’ side of graben –bedding strength #1 is believed to be too low/not representative</td>
</tr>
<tr>
<td>2</td>
<td>1.47</td>
<td>#2</td>
<td>GSI=30-45-60 D=0</td>
<td>Reasonable match with deep-basal shear interpretation and head scarp; slight indication of potential failure on ‘island’ side of graben</td>
</tr>
<tr>
<td>3</td>
<td>1.37</td>
<td>#2</td>
<td>GSI=30-60 D=0</td>
<td>Similar to above; strong indication of potential failure on ‘island’ side graben</td>
</tr>
<tr>
<td>4</td>
<td>1.26</td>
<td>#2</td>
<td>GSI=45-60 D=0.7</td>
<td>Good match with downslope portion of deep basal shear, with indication of shallow-basal shear in the graben area; slight indication of potential failure on ‘island’ side of graben</td>
</tr>
<tr>
<td>5</td>
<td>1.01 Fail</td>
<td>#2</td>
<td>GSI=30-60 D=0.7</td>
<td>Similar to above; FS ~ 1.0 indicating failure</td>
</tr>
</tbody>
</table>
All cases produced potential failure planes/zones which match quite well with the basal shear plane interpretations of Section B-B’ (Figure 7), which were independently developed based on geologic evaluations. Case #3 produced the best overall match with the deep-basal shear interpretation, with failure even occurring on the ‘island’ side of the graben. Both Cases #4 and #5 produced failure planes/zones that are shallower in the graben area, which would be more consistent with the shallow-basal plane interpretation also shown in Figure 7.

However, only Case #5 produced actual failure conditions (i.e. FS~1.0). Therefore, for the purpose of this Pre Design Report, the geologic conditions assumed for Case #5 were used for forward-analyzing remedial options. Additional parametric back-analyses are intended to be performed in the final-design phase in order to fine-tune the design parameters.

4.2 REMEDIAL OPTION #1 – SINGLE LONG SPAN BRIDGE

Option #1 is a single-span bridge across the landslide area. Structurally, this bridge concept consists of a concrete box girder supported on large-diameter cast-in-drilled-hole (CIDH) piles at the abutments. With the exception of some site regrading to support the bridge formwork, conceptually, this option requires very little modification of the existing landslide topography and is designed to be relatively unaffected should future re-triggering of the landslide occur along the original slide planes.

Anticipated pile dimensions are approximately 8 to 10 feet diameter by 100 feet deep, established in the competent bedrock to achieve adequate bearing and lateral support. With regards to abutment stability, the City has already invested considerable time and effort to stabilize the bluff east of the landslide (adjacent to existing residential properties) through installation of permanent anchors and horizontal drains. Since the proposed installation of a large-diameter CIDH pile, if anything, would further increase stability, no additional investigation of the east abutment stability is planned at this time. The stability of the east side of the landslide will need to be confirmed during design. Additional stabilization measures will need to be performed, as necessary, as part of the permanent restoration project.

On the west side, our initial review of the existing S&W (2012a) data had suggested the bluff to be relatively stable. This assessment has essentiaality been confirmed by our current investigation of the area immediately adjacent to the western scarp of the main slide (see Section 3.3.3). However, as discussed in Section 3.6.1, further to the west, approaching Boring Cluster 8, there is concern of an incipient landslide possibly jeopardizing the stability of the bluff. While this is unlikely to directly impact the western
abutment of the proposed bridge, we recommend that stabilization measures be implemented to enhance the stability of the bluff supporting the road west of the abutment. Figures 21 and 22 show a conceptual layout of anchors based on reasonably conservative engineering judgment aimed at enhancing the stability of the bluff in this area. The rough order of magnitude (ROM) cost estimate for installing this slope stabilization measure is $2 million. Additional stability analyses of the bluff to the west of the landslide will be performed as part of the final-design phase.

4.3 REMEDIAL OPTIONS #2 AND #3-EMBANKMENT/RETAINING WALL

Forward stability analyses of remedial options were performed using the distribution of rock-mass and bedding-plane strength of Back-Analysis Case 5. The target factor of safety to achieve under gravity conditions was FS=1.5. Slopes at this level of safety factor under gravity conditions are generally expected to perform satisfactory under seismic conditions as well. The stability under seismic conditions will be analyzed in more detail as part of the final design by performing pseudo-static slope stability analysis.

4.3.1 Option #2 - Anchored CIDH Piles with Buttress Fill

A conceptual section of remedial Option #2 is shown in Figure 23 with a list of the major construction stages necessary for implementation. The FLAC model mesh used for analyzing this option is shown in Figure 24 which also contains an overlay of the geologic section for reference. The analysis was conducted by simulating the construction stages as follows:

- Gravity turn-on to establish in-situ stress state after partial removal of the ‘island’ by excavation to El +75 ft (Figure 24);
- Installation of CIDH piles, grade beam, and anchors (this stage merely required rechecking equilibrium under gravity, as no external forces were applied);
- Placement of reinforced buttress fill (Figure 25); and after reaching equilibrium. Initiation of strength-reduction process. This process is continued until equilibrium is lost, at which point strain-rate contours are plotted indicting the critical shear zone(s).
- These shear zones and the calculated safety factor (FS) are shown in Figure 26.

Figures 27 through 29 show the force demand on the structural members. Since the critical failure is developing on the downslope side of CIDH (see Figure 26), the overall safety factor of
the Anchored CIDH with buttress fill is actually greater than 1.5.

4.3.2 Option #3 – Shear Pins with MSE Wall

A conceptual section of remedial Option #3 is shown in Figure 30 with a list of the major construction stages necessary for implementation. The FLAC model mesh used for analyzing this option is shown in Figure 31 which also contains an overlay of the geologic section for reference. The analysis was conducted by simulating the construction stages as follows:

- Gravity turn-on to establish in-situ stress state after partial removal of the ‘island’ by excavation to El +75 ft (Figure 31);
- Installation of CIDH piles, grade beam, and anchors (this stage merely required rechecking equilibrium under gravity, as no external forces were applied);
- Installation of shear pins;
- Placement of reinforced buttress fill (Figure 32); and after reaching equilibrium:
- Initiation of strength-reduction process. This process is continued until equilibrium is lost, at which point strain-rate contours are plotted indicting the critical shear zone(s). These shear zones and the calculated safety factor (FS) are shown in Figure 33.

Figures 34 through 36 show the force demand on the structural members. Since the critical failure is developing on the downslope side of CIDH (see Figure 33), the overall safety factor of the MSE wall-supported road embankment is actually greater than 1.5.

5.0 DESIGN AND CONSTRUCTION CONSIDERATIONS

5.1 PILE INSTALLATION

Based on observations during our field exploration and our past project experience with similar subsurface materials, the potential of caving should not be precluded during the installation of CIDH piles. Standard Specifications (Caltrans, 2010) and Special Provisions
should be used for the installation of CIDH piles at the project site. The Contractor is expected to follow the Project Plans and Specifications.

- It is anticipated that the bottom of the drilled holes will be BELOW groundwater. Therefore, CIDH piles WILL require construction using the wet methods.

- If temporary casing is used, provisions and installation methods in Section 49-3.02, “Drilled Holes” of the Caltrans Standard Specifications (Caltrans, 2010) shall be followed. Casing must be removed after placement of concrete to allow mobilization of the estimated skin friction capacity.

- Final Project Plans and Specifications should be reviewed prior to construction to confirm that the full intent of the recommendations presented in the Foundation Report has been applied to the design and that the recommendations presented are applicable to the final scope of the project. Following review of Plans and Specifications, sufficient and timely observations during construction should be performed to correlate findings of the investigation with actual subsurface conditions exposed during the construction. Observation and testing by a qualified geotechnical consultant should be performed during construction.

- A contamination study is not within the scope of services of this investigation. The soils encountered during the geotechnical investigation are not considered to be contaminated. If contamination is observed in soil cuttings during construction, the excavated soils should be removed and disposed of properly in accordance with appropriate environmental protocols.

- If the piling center-to-center spacing is less than 3 pile diameters, do not drill holes or drive casing for an adjacent pile until 24 hours have elapsed after concrete placement in the preceding pile and your prequalification test results for the concrete mix design show that the concrete will attain at least 1800 psi compressive strength at the time of drilling or driving.

- The Contractor should be made aware that utility lines are known to be immediately adjacent to the project alignment. These utility lines may be non-yielding and their tolerance of soil movement during excavation may be low. Consequently, it is the responsibility of the Contractor to notify and coordinate with the Underground Services Alert (USA) and to obtain all available as-built
utility plans before any proposed earthwork. All active or inactive utilities within the construction limits should be identified for relocation, abandonment, or protection prior to grading. Any pipelines greater than 2 inches in diameter to be abandoned in-place should be filled with sand/cement slurry after their locations are reviewed and approved by the Resident Engineer.

- During CIDH pile excavations, erosion and surficial sloughing may occur. Excavations during wet seasons will require erosion protection.

- Based on our subsurface exploration, the subsurface materials to be excavated are fill and alluvium consisting of both granular and cohesive materials, and sedimentary bedrock.

- Prior to any site work and excavations, conditions of existing structures and improvements should be surveyed and photo documented.

5.2 MSE WALLS

Conventional earth moving equipment (dozers, scrapers, etc.) should be able to excavate the artificial fill and native soil units with no unusual difficulty. Excavation of some of the more competent bedrock units may require special excavators or backhoes.

5.2.1 Unsuitable Material

Suitable material is that which is free from contamination, organics or deleterious materials and is appropriate for planned use. The on-site material is suitable except for the top 2 feet, which may contain asphalt and other debris.

5.2.2 Reuse as Fill

Existing artificial fill and native soils and some of the bedrock units are expected to be suitable for reuse as Structure Backfill.

5.2.3 Groundwater Control

Groundwater control may be required. It may be necessary to "dewater" the excavation, which is the removal of water from the excavation by gravity drainage, sump pumping, or other similar means. The Contractor is responsible for groundwater control.
5.3.4 Temporary Slopes
The design and construction of temporary excavation support systems (e.g. shoring or soil nailing) and temporary slopes, as well as the maintenance and monitoring of these works during construction, is the responsibility of the Contractor. The Contractor should have a competent person evaluate the soil conditions encountered during excavation to determine permissible temporary slope inclinations and other measures as required by California OSHA (Cal/OSHA).

Existing infrastructure that is within a 1.5:1 (H:V) line projected up from the bottom edge (toe) of temporary slopes should be monitored during construction. The Contractor should note that the materials encountered in construction excavations could vary significantly across the site. A competent person as defined by Cal/OSHA should observe and map excavations and assess the stability of temporary slopes and shoring systems.

5.3.5 Temporary Loading Condition
Overall stability of the MSE wall was evaluated for permanent loading condition. Temporary loading conditions of the MSE wall will follow Caltrans Standard Specifications (Caltrans, 2010). The use of heavy construction equipment for construction of the reinforced soil mass should be avoided and any temporary stockpile material should be stored away from the reinforced soil mass. Per Caltrans Bridge Design Aids 3-8, the compaction clearance should be 4 feet where hand-operated compaction equipment is expected to be used and 8 feet where wheel-mounted compaction equipment is to be used (Caltrans, 2011).
6.0 REFERENCES


Shannon & Wilson, (2012b) Interim Grading Plan (Submitted to COLA on October 9, 2012)


7.0 SUPPORTING FIGURES AND ATTACHMENTS

The following are attached and complete this Pre-Design Geotechnical Memorandum.

- Figure 1: Landslide Morphology Map
- Figure 2: Site Geologic Map and Exploration Plan
- Figure 3: Photograph of Eastern Margin of White Point Landslide
- Figure 4: Photograph of Dolostone Bed in Slidescarp Exposure
- Figure 5: Photograph of Recumbent Fold in Slidescarp Exposure
- Figure 6: Geologic Cross section A-A’
- Figure 7: Geologic Cross section B-B’
- Figure 8: Geologic Cross section C-C’
- Figure 9: Geologic Cross section D-D’
- Figure 10: Geologic Strength Index Chart
- Figure 11: Disturbance factor Chart
- Figure 12: Slope Stability Back-Analyses-Section B-B’
- Figure 13: Bedding-Plane Strength
- Figure 14: Rock Mass Properties
- Figure 15: Hoek-Brown Yield Surfaces
- Figure 16: Case 1 Back Analysis
- Figure 17: Case 2 Back Analysis
- Figure 18: Case 3 Back Analysis
- Figure 19: Case 4 Back Analysis
- Figure 20: Case 5 Back Analysis
- Figure 21: Option 1 West Side Slope Stabilization: Site Plan
<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>22</td>
<td>Option 1 West Side Slope Stabilization: Cross Section</td>
</tr>
<tr>
<td>23</td>
<td>Option 2 Anchored CIDH with Buttress Fill</td>
</tr>
<tr>
<td>24</td>
<td>Option 2 Stage #1</td>
</tr>
<tr>
<td>25</td>
<td>Option 2 Stage #2, #3 and #4</td>
</tr>
<tr>
<td>26</td>
<td>Option 2 Stage #4: Factor of Safety</td>
</tr>
<tr>
<td>27</td>
<td>Option 2 Stage #4: Anchor Force</td>
</tr>
<tr>
<td>28</td>
<td>Option 2 Stage #4: Pile Bending Moments</td>
</tr>
<tr>
<td>29</td>
<td>Option 2 Stage #4: Pile Shear Surfaces</td>
</tr>
<tr>
<td>30</td>
<td>Option 3 Shear Pins with MSE Wall</td>
</tr>
<tr>
<td>31</td>
<td>Option 3 Stage #1</td>
</tr>
<tr>
<td>32</td>
<td>Option 3 Stage #2: #3, #4 and #5</td>
</tr>
<tr>
<td>33</td>
<td>Option 3 Stage #5: Factor of Safety</td>
</tr>
<tr>
<td>34</td>
<td>Option 3 Stage #5: Anchor Force</td>
</tr>
<tr>
<td>35</td>
<td>Option 3 Stage #5: Pile Bending Moments</td>
</tr>
<tr>
<td>36</td>
<td>Option 3 Stage #5: Pile Shear Surfaces</td>
</tr>
</tbody>
</table>
Figure 1. Landslide Morphology Map

The Island
The Graben
The Peninsula
Southeast Finger
2009 Landslide

Scale in Feet

0 50 100

Project: White Point Landslide
Project Number: 60441707
Date: April 2016
Figure 1
DISPLACED ALTAMIRA SHALE OF THE ISLAND PARTIALLY COVERED BY VENEER OF SLOPE WASH

MASSIVE COBBLE & BOULDER LANDSLIDE DEBRIS AND/OR SLOPEWASH

CRUDELY LAYERED, NORTH WEST DIPPING GRAVEL LANDSLIDE DEBRIS

LAYERING ORIENTED N26E, 33NW

2009 LANDSLIDE DEBRIS

BASEL SHEAR EXPOSURE (N50W, 20SW)

ALTAMIRA SHALE BEDROCK

BEACH BOULDER/COBBLE DEPOSIT

ALTAMIRA SHALE BEDROCK

Photograph of Eastern Margin of White Point Landslide

Project No. 60441707

Project: White Point Landslide

AECOM

Figure 3
PHOTOGRAPH OF DOLOSTONE BED IN SLIDESCARP EXPOSURE

QUATERNARY TERRACE DEPOSIT

DOLOSTONE BED

SHALLOW SOUTHWEST DIPPING BEDDING

REGRADED LANDSLIDE DEBRIS

LANDSLIDE DEBRIS
Photograph of Recumbent Fold in Slidescarp Exposure

BEDDING ORIENTED
N36W, 29N

BEDDING ORIENTED
N40W, 60S

HORIZONTAL BEDDING
Pre-slide ground surface from NavigateLA 2006

Landslide shear; dashed where uncertain
Alternative interpretation of landslide shear
Marker bed discussed in text
Apparent dip of bedding measure in boring
Apparent dip of shear measured in boring

Af / Qls Artificial fill/graded landslide debris
Qls Landslide; bedrock formation involved in slide
Qt Terrace deposits; clays
Tma Altamira Shale; interbedded siltstone and sandstone

0 40 80 feet
1" = 40’
Pre-slide ground surface from NavigateLA 2006
Graded surface of Peninsula; upper ~20' removed

(*BA-5 projected ~54' from NW)
(*BA-1 projected ~22' from SE)

BA-1*/CB-1

Landslide shear; dashed where uncertain
Marker bed discussed in text
Apparent dip of bedding measured in boring
Apparent dip of shear measured in boring

Artificial fill/graded landslide debris
Landslide; bedrock formation involved in slide
Terrace deposits; clays
Altamira Shale; interbedded siltstone and sandstone
Geological Strength Index Chart

(Geological Strength Index for Jointed Rocks) (Hoek and Marinos, 2007)

From the lithology, structure and surface conditions of the discontinuities, estimate the average value of GSI. Do not try to be too precise. Quoting a range from 33 to 37 is more realistic than stating that GSI = 35. Note that the table does not apply to structurally controlled failures. Where weak planar structural planes are present in an unfavourable orientation with respect to the excavation face, these will dominate the rock mass behaviour. The shear strength of surfaces in rocks that are prone to deterioration as a result of changes in moisture content will be reduced if water is present. When working with rocks in the fair to very poor categories, a shift to the right may be made for wet conditions. Water pressure is dealt with by effective stress analysis.

<table>
<thead>
<tr>
<th>STRUCTURE</th>
<th>SURFACE CONDITIONS</th>
<th>DECREASING SURFACE QUALITY</th>
</tr>
</thead>
<tbody>
<tr>
<td>INTACT OR MASSIVE - intact rock specimens or massive in situ rock with few widely spaced discontinuities</td>
<td>VERY GOOD Very rough, fresh unweathered surfaces</td>
<td>90</td>
</tr>
<tr>
<td>BLOCKY - well interlocked undisturbed rock mass consisting of cubical blocks formed by three intersecting discontinuity sets</td>
<td>GOOD Rough, slightly weathered, iron stained surfaces</td>
<td>80</td>
</tr>
<tr>
<td>VERY BLOCKY - interlocked, partially disturbed mass with multi-faceted angular blocks formed by 4 or more joint sets</td>
<td>FAIR Smooth, moderately weathered and altered surfaces</td>
<td>70</td>
</tr>
<tr>
<td>BLOCKY/DISTURBED/SEAMY - folded with angular blocks formed by many intersecting discontinuity sets. Persistence of bedding planes or schistosity</td>
<td>POOR Slicksided, highly weathered surfaces with compact coatings or fillings</td>
<td>60</td>
</tr>
<tr>
<td>DISINTEGRATED - poorly interlocked, heavily broken rock mass with mixture of angular and rounded rock pieces</td>
<td>VERY POOR Slicksided, highly weathered surfaces with soft clay coatings or fillings</td>
<td>50</td>
</tr>
<tr>
<td>LAMINATED/SHEARED - Lack of blockiness due to close spacing of weak schistosity or shear planes</td>
<td>N/A</td>
<td>40</td>
</tr>
</tbody>
</table>

Figure 10
Table 1: Guidelines for estimating disturbance factor $D$  
(Hoek et al., 2002)

<table>
<thead>
<tr>
<th>Appearance of rock mass</th>
<th>Description of rock mass</th>
<th>Suggested value of $D$</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image1.png" alt="Image 1" /></td>
<td>Excellent quality controlled blasting or excavation by Tunnel Boring Machine results in minimal disturbance to the confined rock mass surrounding a tunnel.</td>
<td>$D = 0$</td>
</tr>
<tr>
<td><img src="image2.png" alt="Image 2" /></td>
<td>Mechanical or hand excavation in poor quality rock masses (no blasting) results in minimal disturbance to the surrounding rock mass. Where squeezing problems result in significant floor heave, disturbance can be severe unless a temporary invert, as shown in the photograph, is placed.</td>
<td>$D = 0$</td>
</tr>
<tr>
<td><img src="image3.png" alt="Image 3" /></td>
<td>Very poor quality blasting in a hard rock tunnel results in severe local damage, extending 2 or 3 m, in the surrounding rock mass.</td>
<td>$D = 0.8$</td>
</tr>
<tr>
<td><img src="image4.png" alt="Image 4" /></td>
<td>Small scale blasting in civil engineering slopes results in modest rock mass damage, particularly if controlled blasting is used as shown on the left hand side of the photograph. However, stress relief results in some disturbance.</td>
<td>$D = 0.7$ Good blasting $D = 1.0$ Poor blasting</td>
</tr>
</tbody>
</table>
| ![Image 5](image5.png) | Very large open pit mine slopes suffer significant disturbance due to heavy production blasting and also due to stress relief from overburden removal.  
In some softer rocks excavation can be carried out by ripping and dozing and the degree of damage to the slopes is less. | $D = 1.0$ Production blasting $D = 0.7$ Mechanical excavation |

Figure 11
Slope-Stability Back-Analyses - Section B-B’

FLAC Model Mesh

Bedding Dip = 12 deg.

GW +75 ft

Transition

Bedding Dip = 6 deg.

Back-Analysis
Bedding-Plane Strength

Ring Shear Test Results = B-6 @ depth 76 ft [El 60 ft] (S&W, 2012)

\[ \text{Shear Stress} = -0.00000397(\text{esyy})^2 + 0.18272935(\text{esyy}) + 217.93901298 \]
\[ R^2 = 1.00000000 \]

Ring Shear Test Results = BA-2 @ depth 48 ft [El 29 ft] (current investigation)

\[ \text{Shear Stress} = -0.0000095(\text{esyy})^2 + 0.4084343(\text{esyy}) + 209.4422291 \]
\[ R^2 = 1.00000000 \]
Rock-Mass Properties

**Unconfined Compressive Strength (S&W, 2012)**

<table>
<thead>
<tr>
<th>Boring (S&amp;W, 2012)</th>
<th>Depth (ft)</th>
<th>Rock Type</th>
<th>UCS (psi)</th>
<th>UCS (ksf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1 44</td>
<td>Siltstone</td>
<td>980</td>
<td>141</td>
<td></td>
</tr>
<tr>
<td>B1 50</td>
<td>Clayey Siltstone</td>
<td>892</td>
<td>128</td>
<td></td>
</tr>
<tr>
<td>B7 83</td>
<td>Siltstone</td>
<td>383</td>
<td>55</td>
<td></td>
</tr>
<tr>
<td>B7 97</td>
<td>Siltstone</td>
<td>461</td>
<td>66</td>
<td></td>
</tr>
<tr>
<td>B9 103</td>
<td>Clayey Siltstone</td>
<td>581</td>
<td>84</td>
<td></td>
</tr>
<tr>
<td>B9 43</td>
<td>Siltstone - Sandstone</td>
<td>1053</td>
<td>152</td>
<td></td>
</tr>
<tr>
<td>B9 58</td>
<td>Clayey Siltstone</td>
<td>447</td>
<td>64</td>
<td></td>
</tr>
</tbody>
</table>

Median: 581 84

**Elastic Properties**

- Shear velocity, $V_s$ (estimated): 1000 ft/sec
- Unit Weight (S&W, 2012): 120 pcf
- Shear Modulus (derived from $V_s$): 6,200 ksf
- Poisson Ratio (-): 0.25

**Hoek-Brown Parameters**

<table>
<thead>
<tr>
<th>GSI</th>
<th>UCS (ksf)</th>
<th>D</th>
<th>-&gt;</th>
<th>Averg C (ksf)</th>
<th>Averg PHI (degrees)</th>
</tr>
</thead>
<tbody>
<tr>
<td>30</td>
<td>84</td>
<td>0</td>
<td>1.7</td>
<td>1.7</td>
<td>29.5</td>
</tr>
<tr>
<td>60</td>
<td>84</td>
<td>0</td>
<td>3.2</td>
<td>3.2</td>
<td>38</td>
</tr>
<tr>
<td>30</td>
<td>84</td>
<td>0.7</td>
<td>0.975</td>
<td>0.975</td>
<td>19.5</td>
</tr>
<tr>
<td>60</td>
<td>84</td>
<td>0.7</td>
<td>2.259</td>
<td>2.259</td>
<td>32.38</td>
</tr>
</tbody>
</table>
Hoek-Brown Yield Surfaces

Figure 15

Back-Analysis
Back-Analysis

CASE 1 - FS = 0.91

FLAC (Version 7.00)

GSI=60 (D=0)

Max. shear strain-rate
- 0.00E+00
- 1.00E-06
- 2.00E-06
- 3.00E-06
- 4.00E-06
- 5.00E-06

Contour interval = 1.00E-06
Extrap. by averaging

JOB TITLE: .
Case 2 - FS = 1.47

GSI=30 (D=0)
GSI=45 (D=0)
GSI=60 (D=0)

Max. shear strain-rate
- 0.00E+00
- 1.00E-06
- 2.00E-06
- 3.00E-06
- 4.00E-06
- 5.00E-06
- 6.00E-06
- 7.00E-06

Contour interval = 1.00E-06
Extrap. by averaging

Back-Analysis
Rock Mass GSI=30, D=0
Bedding-Plane Strength #2

FLAC (Version 7.00)

LEGEND

29-Dec-15 17:23
step 23934
-4.163E+02 < x < 1.649E+02
-2.772E+02 < y < 3.040E+02

Max. shear strain-rate
0.00E+00
2.50E-07
5.00E-07
7.50E-07
1.00E-06
1.25E-06
1.50E-06
1.75E-06
2.00E-06

Contour interval = 2.50E-07
Extrap. by averaging

CASE 3 - FS = 1.37

GSI=30 (D=0)
GSI=60 (D=0)
FOS results GSI=45–60 (D=0.7-0.0)
Bedding-Plane Strength #2

Case 4 - FS = 1.26

Figure 19
FOS results GSI=30–60 (D=0.7 – 0.0)  
Bedding-Plane Strength #2

CASE 5 - FS = 1.01

GSI=30 (D=0.7)  
GSI=60 (D=0)

FLAC (Version 7.00)

LEGEND

21-Jan-16 13:38
step 29536
-4.356E+02 <x< 2.756E+02
-3.454E+02 <y< 3.657E+02

Factor of Safety 1.01
Max. shear strain-rate
0.00E+00
2.50E-06
5.00E-06
7.50E-06
1.00E-05
1.25E-05
1.50E-05
1.75E-05

Contour interval= 2.50E-06
Extrap. by averaging
Boundary plot

Water Table
Figure 22

WHITE POINT LANDSLIDE
April 2016

Proj. No.: 60441707

Date: April 2016

Figure: 22

LEGEND

- Landslide shear; dashed where uncertain

- Qls: Landslide; bedrock formation involved in slide

- Qt: Terrace deposits; clays

- Tma: Altamira Shale; interbedded siltstone and sandstone

Slope zones:

- 2.5:1 walking

- Cliffs (right bank) 60° SW

- Precise ground surface from NavigaLA 2008
Option 2
Anchored CIDH with Buttress Fill

1. Excavate ~El +75 ft
2. Install 8-ft CIDH piles, 12ft o.c.
3. Install grade beam & anchors
4. Construct buttress fill

Option 2
Anchored CIDH with Buttress Fill

1. Excavate ~El +75 ft
2. Install 8-ft CIDH piles, 12ft o.c.
3. Install grade beam & anchors
4. Construct buttress fill

![Diagram of Option 2: Anchored CIDH with Buttress Fill with Basal shear annotations.](image-url)
Stage #1: Excavate to ~El +75 ft

- GSI=30, D=0.7
- GSI=60, D=0.0
Stage #2: Install CIDH piles
Stage #3: Install grade beam & anchors
Stage #4: Construct buttress fill
Stage #4: Factor of Safety

**Option 2**

**CIDH Figure 26**

**Legend**

- Date: 24-Jan-16 22:34
- Step: 32654
- -4.320E+02 <= x <= 2.720E+02
- -3.420E+02 <= y <= 3.620E+02

**Factor of Safety 1.50**

Max. shear strain-rate:
- 0.00E+00
- 5.00E-08
- 1.00E-07
- 1.50E-07
- 2.00E-07
- 2.50E-07
- 3.00E-07

Contour interval = 5.00E-08
Extrap. by averaging
Boundary plot

Water Table
Stage #4: Anchor Force

Option 2

> Capacity=650 kips

Axial Force on
Structure Max. Value
# 2 (Cable) -6.474E+05

Legend

24-Jan-16 22:36
step 32654
-1.307E+02 < x < 2.482E+02
-1.951E+02 < y < 1.838E+02

Grid plot

Cable Plot

Axial Force on Structure Max. Value
# 2 (Cable) -6.474E+05

< Capacity=650 kips

Figure 27
Stage #4: Pile Bending Moments

FLAC (Version 7.00)

LEGEND

24-Jan-16  22:37
step     32654
-1.461E+02 <x<  2.513E+02
-1.965E+02 <y<  2.009E+02

Grid plot

Pile Plot

Moment on
Structure  Max. Value
# 1 (Pile ) -1.566E+07

< Capacity=25,000 kip-ft
Stage #4: Pile Shear Forces

**FLAC (Version 7.00)**

**LEGEND**

24-Jan-16 22:38
step 32654
-1.120E+02 < x < 2.492E+02
-1.807E+02 < y < 1.806E+02

Grid plot

Pile Plot
- Shear Force on Structure Max. Value
# 1 (Pile) -1.013E+06

< Capacity=1,900 kips

Option 2
Option 3
Shear Pins with MSE Wall

1. Excavate to ~El +75 ft
2. Install 4-ft diameter CIDH piles 20 ft o.c.
3. Install grade beam & anchors
4. Install 3-ft diameters shear pins on 20 ft x 20 ft grid
5. Construct MSE wall

Figure 30
Stage #1: Excavate to ~El +75 ft

GSI=30, D=0.7

GSI=60, D=0.0

Option 3
Stage #2: Install CIDH piles
Stage #3: Install grade beam & anchors
Stage #4: Install pin piles
Stage #5: Construct MSE wall
Stage #5: Factor of Safety

Option 3

Figure 33

FS=1.43
Stage #5: Anchor Force

Option 3

< Capacity=650 kips
Stage #5: Pile Bending Moments

Option 3

Stage #5: Pile Bending Moments

-990 kip-ft
-Capacity=1,900 kip-ft

< Capacity=1,900 kip-ft
Stage #5: Pile Shear Forces

Option 3

< Capacity = 400 Kips

< 270 kips
Appendix E.
Structures Report
WHITE POINT LANDSLIDE
Paseo Del Mar Permanent Restoration Project
At White Point Landslide, Los Angeles, California

STRUCTURES REPORT

Prepared for:
City of Los Angeles
Department of Public Work
Bureau of Engineering - Geotechnical Engineering Group
1149 S. Broadway, Suite 120
Los Angeles, CA 90015-2213

Prepared by:
URS
999 W. Town & Country Road
Orange, CA 92868

URS and AECOM have joined together as one company

May 2016
# TABLE OF CONTENTS

1.0 INTRODUCTION .................................................................................................................. 1

1.1 BACKGROUND .................................................................................................................... 2
1.2 EXISTING CONDITION ......................................................................................................... 2
1.3 SITE WORK COMPLETED TO DATE .................................................................................... 4
1.4 OPTIONS EVALUATED ........................................................................................................... 5
1.5 OPTIONS CONSIDERED BUT NOT EVALUATED ................................................................. 5

2. BRIDGE OPTION (#1) .............................................................................................................. 7

2.1 DESCRIPTION ..................................................................................................................... 7
2.2 SITE CONSTRAINTS .............................................................................................................. 8
2.3 DESIGN CRITERIA ............................................................................................................... 8
2.4 GEOTECHNICAL CONSIDERATIONS .................................................................................. 9
2.5 CONSTRUCTION CONSIDERATIONS ................................................................................... 9
2.6 UTILITY CONSIDERATIONS ............................................................................................... 10
2.7 AESTHETIC CONSIDERATIONS ......................................................................................... 11
2.8 MISCELLANEOUS CONSIDERATIONS .............................................................................. 11
2.9 ENVIRONMENTAL CONSIDERATIONS ............................................................................. 12
2.10 EROSION CONSIDERATIONS ........................................................................................... 12
2.11 PERMITS AND AGREEMENTS .......................................................................................... 13
2.12 RIGHT OF WAYS AND TEMPORARY CONSTRUCTION EASEMENTS .............................. 13
2.13 CONSTRUCTION COST ...................................................................................................... 13
2.14 OPTION SUMMARY ......................................................................................................... 14

3. FILL / EMBANKMENT OPTION (#2) – ANCHORED CIDH WITH BUTTRESS ..... 15

3.1 DESCRIPTION ...................................................................................................................... 15
3.2 SITE CONSTRAINTS ............................................................................................................. 16
3.3 DESIGN CRITERIA ................................................................................................................. 16
3.4 GEOTECHNICAL CONSIDERATIONS ................................................................................ 16
3.5 CONSTRUCTION CONSIDERATIONS .................................................................................. 17
3.6 UTILITY CONSIDERATIONS ............................................................................................... 17
3.7 AESTHETIC CONSIDERATIONS ........................................................................................ 18
3.8 MISCELLANEOUS CONSIDERATIONS .............................................................................. 18
3.9 ENVIRONMENTAL CONSIDERATIONS ............................................................................. 18
3.10 EROSION CONSIDERATIONS ........................................................................................... 19
3.11 PERMITS AND AGREEMENTS .......................................................................................... 19
3.12 RIGHT OF WAYS AND TEMPORARY CONSTRUCTION EASEMENTS .............................. 20
3.13 CONSTRUCTION COST ...................................................................................................... 20
3.14 OPTION SUMMARY ......................................................................................................... 20

4. WALL / EMBANKMENT OPTION (#3) - SHEAR PINS WITH MSE WALL ........... 21

4.1 DESCRIPTION ...................................................................................................................... 21
4.2 SITE CONSTRAINTS ............................................................................................................. 22
4.3 DESIGN CRITERIA ................................................................................................................. 22
4.4 GEOTECHNICAL CONSIDERATIONS ................................................................................ 23
4.5 CONSTRUCTION CONSIDERATIONS .................................................................................. 23
4.6 UTILITY CONSIDERATIONS ............................................................................................... 23
4.7 AESTHETIC CONSIDERATIONS ........................................................................................ 24
4.8 MISCELLANEOUS CONSIDERATIONS .............................................................................. 24
4.9 ENVIRONMENTAL CONSIDERATIONS ................................................................. 24
4.10 EROSION CONSIDERATIONS ........................................................................ 25
4.11 PERMITS AND AGREEMENTS ...................................................................... 25
4.12 RIGHT OF WAYS AND TEMPORARY CONSTRUCTION EASEMENTS ................. 25
4.13 CONSTRUCTION COST .................................................................................. 26
4.14 OPTION SUMMARY ..................................................................................... 26

5. COMPARISON OF OPTIONS ............................................................................ 27

LIST OF FIGURES, PHOTOGRAPHS, AND TABLES
Figure 1: Project Vicinity Map ................................................................. 1
Figure 2: Project Location Map ................................................................. 2

Photograph 1: Aerial view at the start of 2011 Landslide (Dated Nov. 8, 2011) ......... 3
Photograph 2: Aerial view after the 2011 Landslide (Dated May 7, 2012) ................... 3
Photograph 3: Project site looking East (Dated Sep. 24, 2015) ................................. 4

Table 1: Structure Summary for Bridge Option .................................................. 14
Table 2: Structure Summary for Embankment Option with Anchored CIDH piles......... 20
Table 3: Structure Summary for Embankment Option with Shear Pins and MSE Wall ........................................................................................................... 26
Table 4: Comparison of Options ......................................................................... 27

LIST OF APPENDICES
Appendix A - PRELIMINARY STRUCTURES CONSTRUCTION COST ESTIMATES
Appendix B - PRELIMINARY REPORT SUBMITTAL CHECKLIST
Appendix C - PRELIMINARY STRUCTURES PLANS
Appendix D - SELECTED AS-BUILT PLANS
1.0 INTRODUCTION

This Preliminary Structures Report is prepared for the permanent restoration project for reconstructing W Paseo Del Mar roadway to its original alignment at the White Point landslide area in San Pedro within the City of Los Angeles, California. This report supports the Pre-Design Report and provides an overview of various options considered and preliminary construction costs. After an evaluation of several feasible options, the study focused on three final options – a bridge, an embankment option utilizing anchored cast-in-drilled-hole piles, and a pile with shear pin option coupled with mechanically stabilized embankment walls.

Figure 1 shows the location of the landslide and Figure 2 marks the area of interest within the project limits.
1.1 BACKGROUND

On November 20, 2011, a landslide located near the White Point area of Palos Verde Peninsula destroyed an approximately 400 feet stretch of City of Los Angeles roadway in the community of San Pedro. The progression of the landslide can be seen on Photographs 1 and 3.

This section of roadway along Paseo Del Mar is a two-lane highway with shoulders on both sides, and a bike path and sidewalk along the cliff side. The City of Los Angeles initiated studies, cleanup, and stabilization of the adjacent eastern slopes and introduced a temporary street turn-around at the eastern end to close the road until a permanent solution is determined. Photograph 3 shows the current condition of the landslide area.

Council District 15 formed a task force to identify and evaluate alternatives for a permanent solution. The task force recommended that the roadway be restored as opposed to being permanently closed or diverted into the White Point Nature Preserve.

1.2 EXISTING CONDITION

The 2011 landslide created a large surface gap measuring about 400 feet along the centerline of Paseo Del Mar. This area is referred to as “graben” in this report (Photographs 2 and 3) and has been recently graded for drainage with its head scarp edges trimmed to avoid sloughing. The landslide area has been fenced off to the public.

West Paseo Del Mar is bounded by the White Point Nature Preserve owned by the City of Los Angeles Department of Recreation and Parks to the north and the Pacific Ocean to the south. To the east is S Weymouth Avenue and the west is Western Avenue. West of Western and east of S Weymouth are mostly residential and Paseo Del Mar right of way width is 70 feet in this reach. Nearby along the south side of W Paseo Del Mar is the Royal Palms Beach Park and Fromhold Field, a Los Angeles County operated baseball field.
Photograph 1: Aerial view at the start of 2011 Landslide (Dated Nov. 8, 2011).

Formation of initial surface cracks

Photograph 2: Aerial view after the 2011 Landslide (Dated May 7, 2012)
1.3 SITE WORK COMPLETED TO DATE

A summary of relevant site work completed to date (2011 through 2015) is as follows:

- Temporary and long-term closure of Paseo Del Mar at both ends of the landslide
- Closure and relocation of impacted utilities across the landslide
- Instrumentation and monitoring of selected borings – on-going
- Grading of graben area and head scarp
- Subterranean drainage system at east side of Landslide
- Slope stabilization of east bank using ground anchors
- Installation of surface drainage system (curb and gutter) at east end on Paseo Del Mar
- Geotechnical exploratory borings (new work completed by URS in 2015)
- Cul-de-sac and drainage improvements at Weymouth and W Paseo Del Mar.
1.4 OPTIONS EVALUATED
The following three options are preferred for the permanent restoration of White Point Landslide:

1.4.1 Bridge Option (#1) - To Span over Landslide
This option involves building a single-span cast-in-place prestressed (CIP/PS) concrete box girder bridge that will span the current landslide.

1.4.2 Fill / Embankment Option (#2) – Anchored CIDH with Buttress
This option involves building an engineered fill (e.g.: reinforced earth embankment or reinforced soil system) with a front row of caissons (i.e.: cast-in-drilled holes concrete piles or CIDH) with ground anchors to support the engineered fill and to stabilize the current landslide.

1.4.3 Wall / Embankment Option (#3) – Shear Pins with MSE Wall
This option involves building a mechanically stabilized earth (MSE) wall over a series of caissons (i.e.: cast-in-drilled holes concrete piles or CIDH) that act as shear pins and anchors the wall base (using ground anchors) to stabilize the current landslide.

All three alternatives are described in detail in the next sections:

1.5 OPTIONS CONSIDERED BUT NOT EVALUATED
The following options were considered but abandoned due to the reasons cited below:

1.5.1 Other Bridge Types:
1.5.1.1 Single-Span
Other single-span bridge types such as: Concrete Tied Arch, Steel Tied Arch, Steel Truss, Steel Box Girders are also feasible for this site since they suit well to the required span range of approximately 400 feet. However, these bridges (superstructure in particular) will cost approximately 50% higher than the proposed bridge type, will require special inspection (fracture-critical elements), and will have higher maintenance costs (coastal environment). The Steel Truss option was originally developed and evaluated as “Option 1C – Single-Span Steel Truss”.

Signature style single-span bridge types such as cable-stayed or cable-extradose system were not considered since they will cost significantly more to build, inspect, maintain, and their high towers and back anchorages will not be suitable at this location from an aesthetic point of view.

1.5.1.2 Multiple-Span
A multiple-span continuous cast-in-place prestressed (CIP/PS) concrete slab bridge was considered as an alternate to the preferred single-span bridge, and was originally developed and evaluated as “Option 1D” – Multiple-Span Concrete Slab Bridge”. Other superstructure types such as: cast-in-place prestressed (CIP/PS) concrete box, reinforced concrete (RC) slab are also feasible, however, the required close proximity of intermediate bents better suites a
continuous concrete slab. Similarly, precast prestressed girders of shorter spans are also feasible at this location; however, continuity and load distribution requirements for superstructure eliminate this superstructure type. There is also little or no benefit in any precast or prefabricated construction at this site since falsework can be easily constructed for a conventional cast-in-place concrete construction.

The continuous multiple-span superstructure system entails about double large concrete bent columns (8’ diameter) founded on CIDH concrete pilings spaced at about 40’-50’ on centers. The large diameter CIDH acts as shear pins through the basal layer (location of landslide slip layer) to stabilize the current landslide and the continuous superstructure slab serves as a “whaler” to distribute the CIDH forces to a larger interconnected system. The higher overall cost, vulnerability of continuous multiple-span bridge superstructure to CIDH movements from a potentially active landslide zone, and relatively poor aesthetics (about 18 large columns at tight spacing) were the reasons for abandoning this option.

Two or three-span bridges were not considered feasible at this location due to excessive forces on foundation from the potential movement of the existing landslide.

### 1.5.2 Other Wall Types:

#### 1.5.2.1 Gravity

Gravity type walls are not considered due to height of wall, stability of the landslide area, and steep terrain.

#### 1.5.2.2 Cantilever

Cast-in-place concrete cantilever wall was not considered due to steep terrain geometry, stability of the landslide area, and constructability of placing a spread footing type foundation.

#### 1.5.2.3 Solider Piles

A solider pile wall with tiebacks was not considered due to steep terrain geometry, excessive height of wall, extremely long pile lengths required to penetrate the slide zone, and constructability of anchorages.

#### 1.5.2.4 Soil Nails

A soil nail or ground anchor wall is not feasible at this location since a vast majority of the walls is located in “fill” section.

#### 1.5.2.5 Secant Piles

Secant pile wall with closely spaced cast-in-drilled-hole (CIDH) was considered but not evaluated due to steep terrain and depth of pile required for stability. A tieback system on CIDH was also considered but abandoned due to excessive cost due to excavation required for the installation of anchorages.
1.5.3 Other Fill Types:

1.5.3.1 Full Depth Engineered Fill

A full depth embankment fill that will intercept and remove the existing landslide slip layer was developed and evaluated originally as “Option 2A - Deep Buttress Fill.” This option required excavating to approximately -3 feet elevation and replacing the entire fill with an engineered fill (i.e.: reinforced soil system or reinforced earth geotechnical systems) to support the steep embankment slopes (1H:1V or steeper). It also required building of a large rip-rap revetment at embankment toe to prevent wave erosion which can jeopardize the embankment stability. This option was abandoned primarily due to its high cost which can be attributed to permitting requirements for working in the shoreline, extensive subsurface drainage system requirements, high embankment construction cost requiring a large quantity of imported fill, potential of causing adverse impacts to adjacent shoreline, and maintenance efforts required for toe protection against wave action and sea rise.

1.5.3.2 Partial Depth Engineered Fill

Several partial fill options were considered; however, the stability of overall fill without a positive measure to prevent the existing slide from future movements was a factor in elimination of most partial fill options.

1.5.3.3 Traditional Dirt Fill

The traditional fill option of re-grading the existing landslide area to its original embankment plan and profile (using a combination of existing and imported fills) is not considered feasible since such systems will be inherently instable.

2. BRIDGE OPTION (#1)

2.1 DESCRIPTION

The proposed bridge option will be a single-span cast-in-place / prestressed (CIP/PS) concrete box girder bridge that will be 380’ long and 58’-5” wide (final geometry to be determined). Its superstructure will consists of 16’-6” deep double boxes with deck overhangs on each side. It will have a sloped girder faces with architectural finish, 15 feet wide sidewalk on south side and no sidewalk on north side, and barrier on both sides.

The new superstructure will be supported by seat abutments founded on two large diameter CIDH piles at each end. Preliminary estimates indicate that the piles will 12 feet in diameter with pile tips extending to -20 feet elevation.

West bank in the vicinity of west bridge abutment will be stabilized using deep abutment foundation piles and ground anchors and subsurface drainage (similar to recent work on east bank.) The purpose of stabilizing west abutment is to prevent west abutment from moving/tilting during a potential new landslide involving west bank. The remaining portion of west bank will also be stabilized.
The grading work will also require the re-shaping of graben area and removal of Palm Tree Island. This grading is necessary to provide a flat work surface for falsework construction under the bridge and CIDH construction at abutments. North head scarp will be slightly extended to provide a suitable work platform for the construction of abutments and wing walls. The top edge of north head scarp will be graded to a flatter slope to prevent any future landslide from hitting the bridge edge. The grading will also assist in providing full inspection/maintenance access to all primary bridge components.

No shoreline protection measures are required for the bridge option.

### 2.2 SITE CONSTRAINTS

The bridge option is designed with the following site constraints:

- The roadway width will be restored to 58’-0” width which includes: a 15’-0” sidewalk on south side (Ocean side), a no sidewalk on the north side and 5’-0” DWP easement on north side (Head scarp side), 7’-0” wide bicycle lanes on each side, two 11’-0” wide lanes separated by a 4’ striped median. The bridge section will be transitioned to match the existing roadway section on each end.

- No significant change to the current roadway centerline horizontal alignment or vertical profile grade will be made. Vertical and cross gradient will be provided for bridge roadway surface drainage during the final design phase.

- Re-constructed Paseo Del Mar will be confined to the existing City’s right-of-way width of 70’-0”. Any permanent relocation of the existing storm drains will be limited to this width.

- East bridge abutment will be located as such that it will not interfere with the recently installed drainage and slope stabilization anchor systems.

- West bridge abutment will be stabilized to prevent landslide movement on west bank since such excessive abutment movement can jeopardize the stability of the overall bridge system.

- Future landslide under the bridge (i.e.: between the two abutments) will safely pass under the proposed bridge. Localized mitigation of the area above the graben by grading or additional structural support such as soil nails will be reviewed.

### 2.3 DESIGN CRITERIA

#### 2.3.1 Load and Resistance Factor Design

In general, the design of the bridge will follow the AASHTO LRFD Design Specification, 6th edition (2012) with latest Caltrans Amendments (2015). Design live load shall include HL-93 with design tandem and permit load. Load combinations shall follow Caltrans Amendments to AASHTO LRFD as described in Section 3.4.1.

Project specific special load combinations to include landslide forces on the foundation may be required for the structure’s design during its final design phase.
2.3.2 Seismic Design

The new structure will comply with Caltrans Seismic Design Criteria (SDC) version 1.7 and Caltrans Memos to Designers (MTD) Chapter 20. The span of the proposed bridge exceeds 300 feet, and therefore, does not qualify as an “Ordinary Standard Bridge” as defined by SDC Article 1.1. The bridge will be classified as “Ordinary Non-Standard Bridge” and may require the development of project-specific seismic design criteria. However, the proposed structure being a single-span bridge, many provisions of SDC with minor modifications will still apply.

A seismic vulnerabilities evaluation of the proposed structure along with a summary of its seismic design evaluation and associated mitigation strategies will be included in its final design phase.

2.4 GEOTECHNICAL CONSIDERATIONS

2.4.1 Subsurface Conditions

See the Geotechnical Memo provided in Appendix D.

2.4.2 Geotechnical Considerations

See the Geotechnical Memo provided in Appendix D.

2.4.3 Foundation

Two 144 inch (or 12 feet) diameter Cast-in-Drilled-Hole (CIDH) concrete piles (also referred to as Caissons or Drilled Shafts) with tip elevations of - 20’ at each abutment are anticipated. The number, size and depth of CIDH piles will be adjusted in the final design phase based on the results of detailed analysis and design efforts.

Other foundation options such as spread footings, precast and steel piles, and smaller and shorter CIDH piles are not considered feasible due to the new landslide potential at this location. Long pile lengths required to penetrate the landslide zone and to address the overall bridge stability concerns to counteract excessive foundation movement.

2.5 CONSTRUCTION CONSIDERATIONS

General construction considerations are as follows:

- Roadway and sidewalks are permanently closed at both ends of the site; therefore, no staged construction or traffic (pedestrian/bike/vehicular) detour will be required.

- Access to the adjacent White Point Nature Preserve to be maintained during construction.

- During construction the area should be secured for public safety.
Construction access is available from both ends of the project site. A temporary dirt road to graben area is also available from west end which may require minor modifications for construction use.

Adequate construction laydown and storage area is available within the closed portion of the roadway.

The graben area will be graded and compacted for falsework placement for the construction of the cast-in-place box superstructure. Natural drainage of the graded area will be maintained throughout the construction and afterwards. It is recommended that falsework settlements, even though no excessive settlement is expected, be continuously monitored due to the large span length and potential landslide area.

Construction duration is anticipated to be a full construction season contingent upon any special permitting constraints.

2.6 Utility Considerations

The as-built drawings and topographic surveys indicate the presence of existing utilities at the project site. A few utilities have been partially abandoned due to the current landslide and some have been rerouted. New utilities, if any, to be carried on the bridge, will be determined during the final design phase. All existing utilities, their disposition, and new utilities, if any, will be verified in the final design process. The County has indicated that the partly abandoned 84” diameter storm drain will not be restored and across the landslide, and will not be carried by the bridge structure.

The following is a list of known existing utilities at or near the project site:

- 8” Sanitary Sewer – both approaches
- 8” Water – both approaches
- 3” Gas – both approaches
- Miscellaneous Gas – east approaches
- 8” Water – east approach – Abandoned
- 84” Storm Drain – east approach – partly abandoned
- 66” Strom Drain – east approach
- 36” Storm Drain – east approach
- 12” Sanitary Sewer – east approach
- 8” Sanitary Sewer – east approach
- 54” RCP Storm Drain – west approach – partly abandoned
- 42” RCP Storm Drain – west approach
• 24” RCP Storm Drain – west approach
• Overhead Power (on poles) – both approaches

2.7 AESTHETIC CONSIDERATIONS

General aesthetics considerations are as follows. The final aesthetic features will be developed in conjunction with the City and project Landscape Architect at the start of the final design phase. See the Pre-Design Report for additional discussion.

• This stretch of Paseo Del Mar is a designated scenic highway; therefore, careful considerations for its overall aesthetics as experienced by the traveling public will be necessary.

• The north fascia of the concrete bridge is 20 feet deep x 400 feet long (as compared to the iconic “Hollywood” sign which is 45 feet high and 350 feet long.) Treatment can be as simple as adding a parabolic arch profile with architectural concrete finish to a more elaborate scheme using curved soffit profile, colored concrete and texturing, and custom designed artistic treatment of entire bridge fascia and abutments.

• A bridge of this configuration – single-span cast-in-place concrete girder bridge (which is not built using segmental or cantilever construction methods) – will be a unique. The availability of natural dirt platform for easy falsework construction (very unique due to the landslide) makes this bridge type economical for this particular site.

• Since the bridge deck will provide a great vista of the open Pacific Ocean, surface treatment of ocean facing south 12 feet sidewalks along with an architectural railing, bridge end pilasters as an entrance, built-in architectural lightings on barriers to enhance the sidewalks and railings, etc. may be helpful to enhance experience of the travelling public and pedestrians.

• Open spaces around and under a bridge are necessary for inspection and maintenance but they can also create maintenance headaches (e.g.: graffiti, trash collection, vandalism, etc.) and public safety hazards (illicit activities, hiding/gathering places, etc.) Provisions for proper aesthetic treatment around and under the bridge, adequate lighting, locked access gates, and/or limiting access can help alleviate such concerns.

2.8 MISCELLANEOUS CONSIDERATIONS

The miscellaneous considerations associated with this alternate are as follows:

• Slope paving – not anticipated at either abutment.

• Approach slab – Approach slab Type N (30S), full width, will be provided at both abutments to avoid future settlements and to span minor bridge/approach movements.
• Surface drainage – Deck drains will be avoided if possible. The intent is to direct all surface drainage to drains outside the bridge limits. In general, all other surface drainage including the newly graded graben area should be directed away from the bridge foundation.

• Lighting – there are no existing street light in this reach of Paseo Del Mar, therefore, no street illumination provisions are provided at this time. Other lighting such as sidewalk illumination or accent lighting on the bridge will be determined during the final design phase.

• Instrumentation and Monitoring – since this is an active landslide zone, it is recommended that the bridge system, particularly bridge foundations, be monitored for potential movement so timely mitigation measures, if any, can be undertaken.

2.9 ENVIRONMENTAL CONSIDERATIONS
The known environmental constraints that may impact implementation of the proposed option are as follows. Additional considerations, if any, will be included in the final design process.

2.9.1 Caltrans Environmental Area
This project falls within Caltrans State Highway Environmental Area “Non-Freeze-Thaw Area” as defined in Caltrans Memo to Designers, Section 8-2. This area has mild climate where frost is rare and salt is applied infrequently. No special deck corrosion protection features are required.

2.9.2 Soil Corrosion
A corrosion study is not within the scope of services. The soil due to its proximity to the coast line can be considered corrosive at certain depths and locations. A final determination will be made during the final design phase. Most likely Type V cement will be utilized.

2.9.3 Hazardous Material
A contamination study is not within the scope of services. The soils encountered during the geotechnical investigations are not considered to be contaminated. The owner’s provided information does not indicate the presence of hazardous materials; however, the proximity to abandoned Nike Missile site may require additional verification prior to grading and excavation at the site.

2.10 EROSION CONSIDERATIONS
The bridge is not scour-critical, and therefore, no shoreline protection work such as rock armor (rip-rap) is proposed for Bridge option. The reason is that the proposed bridge is supported by deep foundation (CIDH piles) which is located at the outer limits of the current slide and at about
150 feet away from the shoreline. The stability of bridge foundation is unlikely to be adversely impacted by wave erosion of the shoreline. Although over a long run, the continuing erosion of Paseo Del Mar embankments at the shoreline may impact the bridge approaches and other roadway segments.

2.11 PERMITS AND AGREEMENTS
Various agencies with jurisdiction at or near the Paseo Del Mar landslide may require agreements, construction and/or encroachment permits in order to reconstruct the roadway. Some of these agencies include:

- City of Los Angeles
- California Coastal Commission
- Army Corp of Engineers
- Regional Water Quality Board

2.12 RIGHT OF WAYS AND TEMPORARY CONSTRUCTION EASEMENTS
No new permanent right of way is anticipated for the proposed alternate. The adjacent property to the north belongs to the City of Los Angeles. Formal temporary construction easement (TCE) should not be necessary on this adjacent property.

2.13 CONSTRUCTION COST
The total estimated construction cost for the Bridge Option is $26.5 Million (about $9.9 Million for the structural system alone). The total cost estimate (see pre-design report) includes the anticipated construction cost with 20% mobilization, 30% contingencies appropriate for this level of conceptual design (Class C) and a 15% allowance for BOE construction administration. A more detailed cost breakdown of structural system construction costs can be found in Appendix A.
2.14 OPTION SUMMARY

A summary of key features for this option are summarized in Table 1 below:

**Table 1: Structure Summary for Bridge Option**

<table>
<thead>
<tr>
<th>Structure Name</th>
<th>White Point Landslide Bridge.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structure Type</td>
<td>Cast-In-Place, Prestressed (CIP/PS) Concrete Box Girder.</td>
</tr>
<tr>
<td>Spans / Length</td>
<td>380’-0” (Single-Span).</td>
</tr>
<tr>
<td>Structure Depth / Height</td>
<td>16’-6” (Superstructure Depth).</td>
</tr>
<tr>
<td>Abutments / End Treatment</td>
<td>Short Seat-Type Abutment.</td>
</tr>
<tr>
<td>Vertical Clearance</td>
<td>Not applicable (inspection access only).</td>
</tr>
<tr>
<td>Temporary Minimum Vertical Clearance</td>
<td>Not applicable (falsework placement only).</td>
</tr>
<tr>
<td>Barriers / Railings</td>
<td>Type 80SW or ST-40 sidewalk railing (3’-6” total height) (modified per City of Los Angeles BOE standards).</td>
</tr>
<tr>
<td>Slope Paving / Landscaping</td>
<td>No slope paving / Landscaping will be determined later.</td>
</tr>
<tr>
<td>Structure Approach</td>
<td>Approach slab Type N (S-30).</td>
</tr>
<tr>
<td>Utilities</td>
<td>Utilities to be carried in the Bridge will be determined later.</td>
</tr>
<tr>
<td>Bank Stabilization</td>
<td>Required.</td>
</tr>
<tr>
<td>Slope Protection</td>
<td>Not Required.</td>
</tr>
<tr>
<td>Subsurface Drainage</td>
<td>Not Required.</td>
</tr>
</tbody>
</table>
3. **FILL / EMBANKMENT OPTION (#2) – ANCHORED CIDH WITH BUTTRESS**

3.1 **DESCRIPTION**

The proposed embankment option will be an engineered (reinforced) fill placed on the top of the graded area (at an average elevation of 75 feet). The fill toe will be stabilized using a single row of anchored cast-in-drilled-holes (CIDH) concrete piles connected at top by a concrete grade beam as a buttress. It is expected that CIDH piles will be 96 inch (8 feet) in diameter, spaced at about 12 feet, with its pile tips extending to elevation -20 feet. The grade beam will be about 10 feet by 10 feet (outer dimension) which will be tie-backed into a sound foundation layer (below the slip plane) using ground anchors spaced at 12 feet.

The top of the fill slope will be located about 10 feet offset from south sidewalk edge and the approach traffic (metal beam guardrail or modified rail system) barrier will be extended along the sidewalk for pedestrian and vehicle safety.

A tentative construction sequence for this option is as follows:

- Remove and grade Palm-Tree Island to a flat surface (of approximate elevation 75 feet) for CIDH pile installations.
- Drill 96” diameter CIDH concrete piles (buttressed piles) at the proposed base of the engineered fill with pile tips extending to elevation -20 feet.
- Build a 10’ wide by 10’ tall cast-in-place concrete grade beam (exact size to be determined in the final design phase) connecting the top of the CIDH piles.
- Install and tension ground anchors at 12 feet spacing at the grade beam anchoring to a solid foundation layer below the slip plane.
- Build an engineered fill behind and above the grade beam. Provide for the drainage of exposed scarp face within the limits of the new fill embankment. Provide a surface treatment of the engineered fill or install other means (e.g.: landscaping) to prevent surface erosion.
- Trim and shape existing slope to a flatter slope (from grade beam elevation to shoreline) and provide the specified shoreline protection at the toe.
- Construct railings, sidewalks, curb, gutters, etc.

This option will also require the stabilization of the entire west side. It should be noted that the stability of the banks is not as critical as compared to the bridge option since a substantial section of proposed engineered fill will remain intact should a new landslide in the adjacent bank occur. However, stabilization of entire west bank is essential to permanently restoring this segment of the roadway and not requiring any major rework in the future.
The grading work will require the re-shaping of graben area and removal of Palm Tree Island. This grading is necessary to provide a flat work surface for installation of anchored CIDH and grade beam.

3.2 SITE CONSTRAINTS

This embankment option is designed with the following site constraints:

- The roadway width will be restored to 64’-0” width which includes starting from south (Ocean) side: a 12’-0” sidewalk, 8’-0” parking lane / shoulder, 6’-0” bicycle lane, 11’-0” travel lane, 4’-0” median, 11’-0” travel lane, 6’-0” bicycle lane, and 6’-0” wide shoulder on north (head scarp) side. The embankment will be transitioned to match the existing roadway section on each end.
- No significant change to the current roadway centerline horizontal alignment or vertical profile grade will be made. Vertical and cross gradient will be provided for roadway surface drainage during the final design phase.
- Re-constructed Paseo Del Mar will be confined to the existing City’s right-of-way width of 70’-0”.
- CIDH piles, grade beam, reinforced embankment near east side will be located as such that they will not interfere with the recently installed drainage and slope stabilization anchor systems.
- Ends of the embankment system are susceptible to damage and local roadway connection can be interrupted again during a potential new landslide movement in either bank. Stabilize west bank to prevent such damage.
- Future landslide under the fill will be resisted by the proposed anchored CIDH system.

3.3 DESIGN CRITERIA

3.3.1 Design

In general, the design of the fill will follow the Caltrans Geotechnical Manual for Earth Retaining Systems (ERS) and Embankments along with other relevant industry publications for engineered fill. The proposed CIDH and grade beam will also comply with Caltrans Memos to Designers (MTD) for various components of this system for seismic design. Project specific design criteria to include seismic and landslide forces, if applicable, will be included during the final design phase.

3.4 GEOTECHNICAL CONSIDERATIONS

3.4.1 Subsurface Conditions

See the Geotechnical Memo provided in Appendix D.
3.4.2 Geotechnical Considerations
See the Geotechnical Memo provided in Appendix D.

3.4.3 Foundation
The foundation for the engineered fill system can be described as an anchored row of CIDH piles with grade beam and ground anchors. The fill toe is restrained from movement by a row of approximately 24 anchored 96 inch diameter CIDH piles, with pile tips to -20 feet. The tops of the piles are linked by a 10’ by 10’ grade beam with tie-backs at 12’ +/- spacing. The number, size and depth of CIDH piles and anchors will be adjusted in the final design phase based on the results of detailed analysis.

Other foundation options such as spread footings, precast and steel piles, and smaller and shorter CIDH piles are not considered feasible due to the new landslide potential at this location. Long pile lengths are required to penetrate the landslide zone and to address the overall wall stability concerns to counteract excessive foundation movement.

3.5 CONSTRUCTION CONSIDERATIONS
General construction considerations are as follows:

- Roadway and sidewalks are permanently closed at both ends of the site; therefore, no staged construction or traffic (pedestrian/bike/vehicular) detour will be required.
- Access to the adjacent White Point Nature Preserve to be maintained during construction.
- During construction the area should be secured for public safety.
- Construction access is available from both ends of the project site. A temporary dirt road to graben area is also available from west end which may require minor modifications for construction use.
- Adequate construction laydown and storage area is available within the closed portion of the roadway.
- The graben area will be graded and compacted for CIDH, grade beam, and anchor installation for reinforced embankment construction. Natural drainage of the graded area will be maintained throughout the construction and afterwards.
- Construction duration is anticipated to be a full construction season contingent upon any special permitting constraints.

3.6 UTILITY CONSIDERATIONS
In general, all existing and new utilities including the large diameter storm sewer can be easily accommodated in the embankment fill with minor adjustments to its reinforcement design. Refer to Section 2.6 (bridge option) for a list of the existing utilities and other considerations.
3.7 **AESTHETIC CONSIDERATIONS**

General aesthetics considerations are as follows. The final aesthetic features will be developed in conjunction with the City and project Landscape Architect at the start of the final design phase.

- This stretch of Paseo Del Mar is a designated scenic highway; therefore, careful considerations for its overall aesthetics as experienced by the traveling public will be necessary.

- The fill slope can be stepped, surfaced with natural rock patterns, and/or planted with native plants to provide continuity with the surrounding terrain.

- Since the embankment option will provide a great vista of the open Pacific Ocean, surface treatment of ocean facing south 12 feet sidewalks along with an architectural railing, end pilasters as an entrance, built-in architectural lightings on barriers to enhance the sidewalks and railings, etc. may be desirable to enhance the experience of the travelling public and pedestrians.

3.8 **MISCELLANEOUS CONSIDERATIONS**

The miscellaneous considerations associated with this alternate are as follows:

- Surface drainage – No special provisions for roadway drainage are required. In general, all surface drainage should be collected and directed away from the fill slope limits.

- Subsurface drainage – Special provisions for draining the fill and the land mass below should be provided to improve the stability of the embankment. Measures may include providing perforated pipes with pervious fill embedment.

- Lighting – there are no existing street lights in this reach of Paseo Del Mar, therefore, no street illumination provisions are provided at this time. Other lighting such as sidewalk illumination or accent lighting on the wall will be determined during the final design phase.

- Instrumentation and Monitoring – since this is an active landslide zone, it is recommended that the embankment system, particularly the CIDH piles, anchors, and grade beams, be monitored for potential movement so timely mitigation measures, if any, can be undertaken.

3.9 **ENVIRONMENTAL CONSIDERATIONS**

The known environmental constraints that may impact implementation of the proposed option are as follows. Additional considerations, if any, will be included in the final design process.
3.9.1 Caltrans Environmental Area
Refer to Section 2.9.1 (Bridge Option) for Caltrans environmental area.

3.9.2 Soil Corrosion
Refer to Section 2.9.2 (Bridge Option) for Soil Corrosion.

3.9.3 Hazardous Material
Refer to Section 2.9.3 (Bridge Option) for Hazardous Material.

3.9.4 Work in Coast Line
Installation of the CIDH piles is not expected to disturb the shoreline and any temporary disturbance can be restored to its current (post landslide) conditions. Rock armor (rip-rap) is recommended but the time period required to install this feature can be deferred. The (future) placement and maintenance of any rock armor (or sand toe) in the shoreline area will require special environmental considerations.

A higher level of rock armor (rip-rap) is required for this option at the slope toe to protect the anchored CIDH piles from soil erosion that can destabilize the fill over time. The work will also require trimming / reshaping of the existing slope to a much flatter slope and building of an access road for the placement and maintenance of rock armor. Such work involving a large area of shoreline (100’ wide x 800’ long) near mean sea level will require special environmental considerations.

3.10 EROSION CONSIDERATIONS
The system is susceptible to long term erosion due to wave action and other forces. A higher level of shoreline protection using rock armor (rip-rap) is proposed for this option. The reason is that the proposed fill toe is restrained by anchored CIDH piles within the limits of the current slide and depends on surrounding foundation soil for its lateral resistance and stability. In essence, the stability of the proposed fill can be adversely impacted by continual wave erosion of the shoreline. The proposed shoreline protection will measure 100’ wide and 800’ long and will involve placement of an 8’ thick 5 ton rock armor overlaid on 4’ thick filter blanket from -10’ elevation to 25’ elevation.

3.11 PERMITS AND AGREEMENTS
Refer to Section 2.11 (bridge option) for anticipated permits.
3.12 **RIGHT OF WAYS AND TEMPORARY CONSTRUCTION EASEMENTS**

Refer to Section 2.12 (bridge option) for TCEs.

3.13 **CONSTRUCTION COST**

The total estimated construction cost for the Embankment Fill Option with Anchored CIDH piles is $45.3 Million (about $19.2 Million for the structural fill system alone). The total cost estimate (see pre-design report) includes the anticipated construction cost with 20% mobilization, 30% contingencies appropriate for this level of conceptual design (Class C) and a 15% allowance for BOE construction administration. A more detailed cost breakdown of structural system construction costs can be found in Appendix A.

3.14 **OPTION SUMMARY**

A summary of key features for this option are summarized in the Table 2 below:

**Table 2: Structure Summary for Embankment Option with Anchored CIDH piles**

<table>
<thead>
<tr>
<th>Structure Name</th>
<th>White Point Landslide Embankment.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structure Type</td>
<td>Anchored CIDH piles with reinforced fill.</td>
</tr>
<tr>
<td>Spans / Length</td>
<td>530’-0” (Fill length).</td>
</tr>
<tr>
<td>Structure Depth / Height</td>
<td>Varies (60 feet maximum height).</td>
</tr>
<tr>
<td>Abutments / End Treatment</td>
<td>Special wall design treatment at both ends.</td>
</tr>
<tr>
<td>Vertical Clearance</td>
<td>Not applicable.</td>
</tr>
<tr>
<td>Temporary Minimum Vertical Clearance</td>
<td>Not applicable.</td>
</tr>
<tr>
<td>Barriers / Railings</td>
<td>Type 80SW or ST-40 sidewalk railing (modified per City of Los Angeles BOE standards).</td>
</tr>
<tr>
<td>Slope Paving / Landscaping</td>
<td>No slope paving / Landscaping will be determined later.</td>
</tr>
<tr>
<td>Structure Approach</td>
<td>Not Applicable.</td>
</tr>
<tr>
<td>Utilities</td>
<td>Utilities can be buried under the roadway as needed.</td>
</tr>
<tr>
<td>Bank Stabilization</td>
<td>Required.</td>
</tr>
<tr>
<td>Slope Protection</td>
<td>Required. Rock Toe Protection using 8’ thick 5 ton rock armor over 4’ thick ⅝ ton filter blanket, to elevation of 25’ at shoreline.</td>
</tr>
<tr>
<td>Subsurface Drainage</td>
<td>Required. Provide a drainage system containing filter blankets and perforated pipes at the interface between the exposed scarp face and new fill.</td>
</tr>
</tbody>
</table>
4. WALL / EMBANKMENT OPTION (#3) - SHEAR PINS WITH MSE WALL

4.1 DESCRIPTION

The proposed wall will be a stepped (about ¼ H: 1V slope) Mechanically Stabilized Earth (MSE) wall located on the top of a graded existing land mass stabilized by a grid of cast-in-drilled-holes (CIDH) piles acting as “shear pins” and a single front row of CIDH piles anchored using a concrete grade beam (also acting as a buttress). It is expected that the CIDH piles will be 36 inch in diameter and will be located in a grid pattern spaced approximately 20 feet in either direction with pile tip extending to -0 feet. The front row CIDH piles will be 48 inch in diameter with pile tips extending to elevation -10 feet and pile tops interconnected with a 5 feet x 5 feet continuous concrete grade beam which is tie-backed into a sound foundation layer below the slip plane using ground anchors spaced at 20 feet.

The top of MSE wall panels will be located under the exterior face of the south sidewalk slab that will also act as a moment slab. This 12 feet wide moment slab will be mounted with a modified Type 80SW or ST-40 barrier. As an alternate, the top of the wall can be moved about 10 feet south of sidewalk edge (to eliminate the moment slab), topped with a standard wall mount safety cable railing, and the approach traffic metal beam guardrail can be extended across the entire wall length for additional vehicle safety.

A tentative construction sequence for this option is as follows:

- Remove and grade Palm-Tree Island to a flat surface (of approximate elevation 75 feet) for CIDH installations in the “graben” area.
- Drill 36” diameter CIDH piles (also referred to as “shear-pins”) in a grid patterns (20 feet x 20 feet) with pile tips extending to -0 feet elevation.
- Drill 48” diameter CIDH front row piles (buttressed piles) at the proposed base of the MSE wall with pile tips extending to -10 feet elevation.
- Build a cast-in-place concrete grade beam connecting the top of the front row buttressed piles.
- Install and tension ground anchors at 20 feet spacing at the grade beam anchoring to a solid foundation layer below the slip plane.
- Build a MSE retaining wall behind the grade beam in a sloping or stepped pattern (due to its height in the range of 30 to 50 feet). Compact backfill using imported fill. Provide for the drainage of exposed scarp face within the MSE embankment.
- Provide the specified shoreline protection at the toe.
- Construct railings, sidewalks, curb, gutters, etc.

This option will also require the stabilization of the entire west side. It should be noted that the stability of the banks is not as critical as compared to the bridge option since a substantial section of proposed engineered fill will remain intact should a new landslide in the adjacent bank occur.
However, stabilization of entire west bank is essential to permanently restoring this segment of
the roadway and not requiring any major rework in the future.

The grading work will require the re-shaping of graben area and removal of Palm Tree Island.
This grading is necessary to provide a flat work surface for installation of CIDH shear pins
system, anchored CIDH at the front row, and MSE wall panels.

4.2 SITE CONSTRAINTS
This option is designed with the following site constraints:

- The roadway width will be restored to 64’-0” width which includes starting from south
  (Ocean) side: a 12’-0” sidewalk, 8’-0” parking lane / shoulder, 6’-0” bicycle lane, 11’-0”
  travel lane, 4’-0” median, 11’-0” travel lane, 6’-0” bicycle lane, and 6’-0” wide shoulder
  on north (head scarp) side. The wall section will be transitioned to match the existing
  roadway section on each end.

- No significant change to the current roadway centerline horizontal alignment or vertical
  profile grade will be made. Vertical and cross gradient will be provided for roadway
  surface drainage during the final design phase.

- Re-constructed Paseo Del Mar will be confined to the existing City’s right-of-way width
  of 70’-0”.

- CIDH piles, grade beam, MSE walls near east side will be located as such that they will
  not interfere with the recently installed drainage and slope stabilization anchor systems.

- Ends of the wall system are susceptible to damages and local roadway connection can be
  interrupted again during a potential new landslide movement in either bank. Stabilize
  west bank to prevent such damages.

- Future landslide under the wall footprint will be resisted by the proposed shear pins and
  anchored CIDH system.

4.3 DESIGN CRITERIA

4.3.1 Design
In general, the design of the wall will follow the AASHTO LRFD Design Specification, 6th
Abutments, Piers, and Walls. In addition, Caltrans Memos to Designers (MTD) Chapter 5-5
for Standard Earth Retaining Systems (ERS) and Caltrans Geotechnical Manual for
Mechanically Stabilized Earth (MSE) and Reinforced Soil Slopes (RSS) will also govern its
design. The wall is likely to be classified as “Non-Standard Earth Retaining Wall” and may
require the development of project-specific design criteria to address its performance
requirements during seismic and landslide events.
4.4 GEOTECHNICAL CONSIDERATIONS

4.4.1 Subsurface Conditions
See the Geotechnical Memo provided in Appendix D.

4.4.2 Geotechnical Considerations
See the Geotechnical Memo provided in Appendix D.

4.4.3 Foundation
MSE wall panels will be located on standard concrete leveling pads on graded or engineered fill. Additional bank excavation may be required if there is not enough room for laying out the soil reinforcement layer. No other special foundation requirements for MSE wall are anticipated at this time.

The foundation for the MSE wall system can be described as the shear pins and anchored front row of CIDH with grade beam and ground anchors. For shear pins, about 84 Cast-in-Drilled-Hole (CIDH) 36 inch diameter concrete piles with tip elevations of – 0 feet are anticipated.

For anchored front row piles, 24 CIDH 48 inch diameter concrete piles with tip elevation of -10 feet and topped with a 5 feet by 5 feet grade beam and ground anchors spaced at 20 feet are expected. The number, size and depth of CIDH piles and anchors will be adjusted in the final design phase based on the results of detailed analysis.

Other foundation options such as spread footings, precast and steel piles, and smaller and shorter CIDH piles are not considered feasible due to the new landslide potential at this location. Long pile lengths required to penetrate the landslide zone and to address the overall wall stability concerns to counteract excessive foundation movement.

4.5 CONSTRUCTION CONSIDERATIONS
Refer to Section 3.5 for general construction considerations.

4.6 UTILITY CONSIDERATIONS
In general, all existing and new utilities including the large diameter storm sewer can be easily accommodated in the embankment fill with minor adjustments to its reinforcement design. Refer to Section 2.6 (bridge option) for a list of the existing utilities and other considerations.
4.7 AESTHETIC CONSIDERATIONS

General aesthetics considerations are as follows. The final aesthetic features will be developed in conjunction with the City and project Landscape Architect at the start of the final design phase.

- This stretch of Paseo Del Mar is a designated scenic highway; therefore, careful considerations for its overall aesthetics as experienced by the traveling public will be necessary.
- The wall is proposed to be stepped or made with plantable units. The concrete wall panels can have architectural treatment such as concrete patterning or staining.
- This option will provide a great vista of the open Pacific Ocean, surface treatment of ocean facing south 12 feet sidewalks along with an architectural railing, bridge end pilasters as an entrance, built-in architectural lightings on barriers to enhance the sidewalks and railings, etc. may be desirable to enhance the experience of the travelling public and pedestrians.

4.8 MISCELLANEOUS CONSIDERATIONS

The miscellaneous considerations associated with this alternate are as follows:

- Surface drainage – No special provisions for roadway drainage are required. In general, all surface drainage should be arrested and directed away from the engineered fill limits.
- Subsurface drainage – Special provisions for draining the subterranean water at the interface of head scarp and new fill should be provided to improve the stability of the MSE wall. Measures may include providing perforated pipes embedded in pervious fill material.
- Lighting – there are no existing street lights in this reach of Paseo Del Mar, therefore, no street illumination provisions are provided at this time. Other lighting such as sidewalk illumination will be determined during the final design phase.
- Instrumentation and Monitoring – since this is an active landslide zone, it is recommended that the wall system, particularly the CIDH piles, anchors, and grade beams, be monitored for potential movement so timely mitigation measures, if any, can be undertaken.

4.9 ENVIRONMENTAL CONSIDERATIONS

The known environmental constraints that may impact implementation of the proposed option are as follows. Additional considerations, if any, will be included in the final design process.
4.9.1 Caltrans Environmental Area
Refer to Section 2.9.1 (Bridge Option) for Caltrans environmental area.

4.9.2 Soil Corrosion
Refer to Section 2.9.2 (Bridge Option) for Soil Corrosion.

4.9.3 Hazardous Material
Refer to Section 2.9.3 (Bridge Option) for Hazardous Material.

4.9.4 Work in Coast Line
Installation of the CIDH piles is not expected to disturb the shoreline and any temporary disturbance can be restored to its current (post landslide) conditions. Rock armor (rip-rap) is recommended but the time period required to install this feature can be deferred. The (future) placement and maintenance of any rock armor (or sand toe) in the shoreline area will require special environmental considerations.

4.10 EROSION CONSIDERATIONS
The structural support system is placed a significant distance away from the existing shoreline. Therefore it will be a significantly long time before erosion of the existing shoreline will impact the piles and shear pins. A lower level of shoreline protection using rock armor (rip-rap) is proposed for this option. The reason is that the proposed wall is supported by shear pins and anchored (CIDH piles) within the limits of the current slide and depends on surrounding foundation soil for its stability. In essence, the current slope is too steep and the stability of the proposed wall can be adversely impacted by continual wave erosion of the shoreline. The proposed shoreline protection involves placement of an 8’ to 10’ thick 5 ton rock armor to an elevation of 15’. Alternate measures such as the placement / maintenance of a new sand toe at the shoreline can also be investigated in the final design phase.

4.11 PERMITS AND AGREEMENTS
Refer to Section 2.11 (bridge option) for anticipated permits.

4.12 RIGHT OF WAYS AND TEMPORARY CONSTRUCTION EASEMENTS
Refer to Section 2.12 (bridge option) for TCE’s.
4.13 CONSTRUCTION COST

The total estimated construction cost for the Embankment Wall Option with Shear Pins and MSE Wall is $38.9 Million (about $16.9 Million for the structural wall system alone). The total cost estimate (see pre-design report) includes the anticipated construction cost with 20% mobilization, 30% contingencies appropriate for this level of conceptual design (Class C) and a 15% allowance for BOE construction administration. A more detailed cost breakdown of structural system construction costs can be found in Appendix A.

4.14 OPTION SUMMARY

A summary of key features for this option are summarized in the Table 3 below:

<table>
<thead>
<tr>
<th>Structure Name</th>
<th>White Point Landslide Wall.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structure Type</td>
<td>Shear Pins and MSE Wall.</td>
</tr>
<tr>
<td>Spans / Length</td>
<td>510’-0” (Fill length).</td>
</tr>
<tr>
<td>Structure Depth / Height</td>
<td>Varies (60 feet maximum height).</td>
</tr>
<tr>
<td>Abutments / End Treatment</td>
<td>Special fill or wall treatment at east end (steep slope).</td>
</tr>
<tr>
<td>Vertical Clearance</td>
<td>Not applicable.</td>
</tr>
<tr>
<td>Temporary Minimum Vertical Clearance</td>
<td>Not applicable.</td>
</tr>
<tr>
<td>Barriers / Railings</td>
<td>Type 80SW or ST-40 sidewalk railing (modified per City of Los Angeles BOE standards).</td>
</tr>
<tr>
<td>Slope Paving / Landscaping</td>
<td>No slope paving / Landscaping will be determined later.</td>
</tr>
<tr>
<td>Structure Approach</td>
<td>Not Applicable.</td>
</tr>
<tr>
<td>Utilities</td>
<td>Utilities can be buried under the roadway as needed.</td>
</tr>
<tr>
<td>Bank Stabilization</td>
<td>Required.</td>
</tr>
<tr>
<td>Slope Protection</td>
<td>Recommended but can be deferred. Provide a rock toe using 5 ton rock armor, 8’ to 10’ thick, to elevation of 15’ at shoreline.</td>
</tr>
<tr>
<td>Subsurface Drainage</td>
<td>Required. Provide a drainage system containing filter blankets and perforated pipes at the interface between the head scarp and new fill.</td>
</tr>
</tbody>
</table>
5. COMPARISON OF OPTIONS

A summary of pros and cons for each of the three options is presented below.

Table 4: Comparison of Options

<table>
<thead>
<tr>
<th>Bridge Option (#1)</th>
<th>Cost = $26.5 Million</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>PROS</strong></td>
<td><strong>CONS</strong></td>
</tr>
<tr>
<td>Lowest Cost</td>
<td>Stabilization of west bank is critical to structural integrity; bridge can be irreparable under foundation movement due to a landslide</td>
</tr>
<tr>
<td></td>
<td>Homeless encampment concerns</td>
</tr>
<tr>
<td></td>
<td>Typical bridge annual inspection and maintenance costs; Graffiti removal</td>
</tr>
<tr>
<td></td>
<td>Special design considerations for span longer than 300 feet and supporting large storm drains on the structure</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Fill / Embankment Option (#2) - Anchored CIDH with Buttress</th>
<th>Cost = $45.3 Million</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>PROS</strong></td>
<td><strong>CONS</strong></td>
</tr>
<tr>
<td>Embankment tolerable to settlement</td>
<td>Highest Cost</td>
</tr>
<tr>
<td>A portion of embankment will remain intact during future landslide occurrence on either bank. Bank stabilization is desirable to maintain the integrity of roadway segment but not critical as bridge option.</td>
<td>Pile proximity to the shoreline closer than other options and may require rock armoring repair sooner than other options.</td>
</tr>
<tr>
<td>No large wall face or bridge element although erosion control such as netting may be required on the reinforced fill slope area.</td>
<td></td>
</tr>
</tbody>
</table>
### Wall / Embankment Option (#3)- Shear Pins with MSE Wall
Cost = $38.9 Million

<table>
<thead>
<tr>
<th>PROS</th>
<th>CONS</th>
</tr>
</thead>
<tbody>
<tr>
<td>MSE walls can take minor settlement and segmental units can be repaired.</td>
<td>Wall is massive and non-standard in City.</td>
</tr>
<tr>
<td>A portion of wall will remain intact during future landslide occurrence on either bank. Bank stabilization is desirable to maintain the integrity of roadway segment but not critical as bridge option.</td>
<td></td>
</tr>
<tr>
<td>Pile support set-back large so erosion will not affect system for a long time.</td>
<td></td>
</tr>
</tbody>
</table>
APPENDIX A -
PRELIMINARY STRUCTURES CONSTRUCTION COST ESTIMATES
GENERAL PLAN ESTIMATE

ADVANCE PLANNING ESTIMATE - DRAFT

Revised: March 5, 2015

RCVD BY: ____________________________ IN EST: ____________________________
OUT EST: ____________________________

BRIDGE: Paseo Del Mar at White Point Slide
BR. No.: N/A
DISTRICT: 07

TYPE: BRIDGE OPTION - CIP/PS CONCRETE BOX GIRDER - SINGLE SPAN
RTE: Paseo Del Mar
CO: LA

CU: 00-008
EA: NA

LENGTH: 380.00 WIDTH: 55.00 AREA (SF) = 20,900

DESIGN SECTION:

# OF STRUCTURES IN PROJECT: 1

PRICES CHECKED BY: RKB DATE: 11/30/2015

QUANTITIES BY: GE DATE: 11/16/2015

<table>
<thead>
<tr>
<th>ITEM No.</th>
<th>CONTRACT ITEMS</th>
<th>TYPE</th>
<th>UNIT</th>
<th>QUANTITY</th>
<th>PRICE</th>
<th>AMOUNT</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>STRUCTURE EXCAVATION (BRIDGE)</td>
<td>CY</td>
<td>900</td>
<td>$105</td>
<td></td>
<td>$94,500</td>
</tr>
<tr>
<td>2</td>
<td>STRUCTURE BACKFILL (BRIDGE)</td>
<td>CY</td>
<td>180</td>
<td>$75</td>
<td></td>
<td>$13,500</td>
</tr>
<tr>
<td>3</td>
<td>STRUCTURAL CONCRETE, BRIDGE</td>
<td>CY</td>
<td>3,350</td>
<td>$1,125</td>
<td></td>
<td>$3,768,750</td>
</tr>
<tr>
<td>4</td>
<td>STRUCTURAL CONCRETE, BRIDGE FOOTING</td>
<td>CY</td>
<td>525</td>
<td>$280</td>
<td></td>
<td>$147,000</td>
</tr>
<tr>
<td>5</td>
<td>PRESTRESSING STEEL</td>
<td>LB</td>
<td>271,700</td>
<td>$2.75</td>
<td></td>
<td>$747,175</td>
</tr>
<tr>
<td>6</td>
<td>BAR REINFORCING STEEL (BRIDGE)</td>
<td>LB</td>
<td>1,155,760</td>
<td>$1.14</td>
<td></td>
<td>$1,317,566</td>
</tr>
<tr>
<td>7</td>
<td>CONCRETE BARRIER</td>
<td>80SW</td>
<td>LF</td>
<td>880</td>
<td>$180</td>
<td>$158,400</td>
</tr>
<tr>
<td>8</td>
<td>CHAIN LINK RAILING - Pedestrian</td>
<td>7</td>
<td>LF</td>
<td>880</td>
<td>$75.40</td>
<td>$66,352</td>
</tr>
<tr>
<td>9</td>
<td>STRUCTURAL CONCRETE, APPROACH SLAB (TYPE N)</td>
<td>CY</td>
<td>123</td>
<td>$700</td>
<td></td>
<td>$86,100</td>
</tr>
<tr>
<td>10</td>
<td>JOINT SEAL</td>
<td>2&quot; MR</td>
<td>LF</td>
<td>110</td>
<td>$83</td>
<td>$9,130</td>
</tr>
<tr>
<td>11</td>
<td>PILES</td>
<td>144&quot; CIDH</td>
<td>LF</td>
<td>488</td>
<td>$4,500.00</td>
<td>$2,196,000</td>
</tr>
<tr>
<td>12</td>
<td>UNCLASSIFIED EXCAVATION - north scar p face</td>
<td>CY</td>
<td>10,000</td>
<td>15.00</td>
<td></td>
<td>$150,000</td>
</tr>
<tr>
<td>13</td>
<td>RETAINING WALLS - bridge south face near approaches</td>
<td>SF</td>
<td>3,900</td>
<td>$125.00</td>
<td></td>
<td>$487,500</td>
</tr>
<tr>
<td>14</td>
<td>BRIDGE BEARINGS</td>
<td>EA</td>
<td>8</td>
<td>$10,000.00</td>
<td></td>
<td>$80,000</td>
</tr>
<tr>
<td>15</td>
<td>UNCLASSIFIED EXCAVATION &amp; GRADING - Island</td>
<td>LS</td>
<td>1</td>
<td>$563,139</td>
<td></td>
<td>$563,139</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>ITEM No.</th>
<th>NOTES, REFERENCES &amp; ASSUMPTIONS</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>None</td>
</tr>
<tr>
<td>2</td>
<td>None</td>
</tr>
<tr>
<td>3</td>
<td>Use 25% cost premium over higher end unit cost for f’c = 6500 psi and unusual long span for CIP/PS Box</td>
</tr>
<tr>
<td>4</td>
<td>None</td>
</tr>
<tr>
<td>5</td>
<td>Use 25% cost premium over higher end unit cost for unusual long span for CIP/PS Box</td>
</tr>
<tr>
<td>6</td>
<td>None</td>
</tr>
<tr>
<td>7</td>
<td>None</td>
</tr>
<tr>
<td>8</td>
<td>None</td>
</tr>
<tr>
<td>9</td>
<td>Substitute other mounted Pedestrian style steel railing over the concrete barrier</td>
</tr>
<tr>
<td>10</td>
<td>None</td>
</tr>
<tr>
<td>11</td>
<td>None</td>
</tr>
<tr>
<td>12</td>
<td>122’ long piles (pile tip to -20’), 25% increase over Diemer Treatment Plant cost by Malcom bid of $8M for 25-12’ dia x 90’ caisson</td>
</tr>
<tr>
<td>13</td>
<td>None</td>
</tr>
<tr>
<td>14</td>
<td>Excavation to trim existing vertical Scrap face build bridge embankment - excavated and compacted at site</td>
</tr>
<tr>
<td>15</td>
<td>Assume Soldier Pile Wall or similar type due to steep/difficult terrain</td>
</tr>
<tr>
<td>16</td>
<td>Special requirements due to high loads/movement</td>
</tr>
<tr>
<td>17</td>
<td>Build a working platform by removing the Island and grading the graben to approx. elevation 75’ - see &quot;grading only&quot; worksheet for cost</td>
</tr>
<tr>
<td>18</td>
<td>Used for exporting to main cost worksheet</td>
</tr>
<tr>
<td>19</td>
<td>INCLUDED SEPARATELY</td>
</tr>
<tr>
<td>20</td>
<td>INCLUDED SEPARATELY</td>
</tr>
<tr>
<td>21</td>
<td>INCLUDED SEPARATELY</td>
</tr>
<tr>
<td>22</td>
<td>INCLUDED SEPARATELY</td>
</tr>
<tr>
<td>23</td>
<td>INCLUDED SEPARATELY</td>
</tr>
<tr>
<td>24</td>
<td>INCLUDED SEPARATELY</td>
</tr>
<tr>
<td>25</td>
<td>INCLUDED SEPARATELY</td>
</tr>
</tbody>
</table>

SUBTOTAL $9,885,112
TIME RELATED OVERHEAD (included separately) $472,972
MOBILIZATION (included separately) $150,000
SUBTOTAL BRIDGE ITEMS $9,885,112
CONTINGENCIES (included separately) $2,196,000
BRIDGE TOTAL COST $9,885,112
COST PER SQ. FOOT $472.97
SLOPE STABILIZATION OF WEST BANK (included separately) $487,500
GRAND TOTAL $9,885,112

BUDGET ESTIMATE AS OF 3/24/16 $9,900,000
### Bridge Information
- **Bridge:** Paseo Del Mar at White Point Slide
- **Bridge Number:** N/A
- **Location:** LA
- **Type of Structure:** Fill Option - Anchored CIDH w/ Reinforced Earth Buttress Fill on Top
- **Control Number:** 00-008
- **PM:** N/A
- **District:** 07
- **Benefit:** N/A
- **Length:** 530.00
- **Width:** 58.00
- **Area (SF):** 30,740

### Design Section
- **Number of Structures in Project:** 1
- **Estimated Number:** 1

### Prices
- **Prices by:** GE
- **Prices Checked by:** RKB
- **Date:** 11/24/2015
- **Cost Index:**
- **Quantity by:** GE
- **Date:** 11/16/2015

### Contract Items Table

<table>
<thead>
<tr>
<th>Contract Items</th>
<th>Type</th>
<th>Unit</th>
<th>Quantity</th>
<th>Price</th>
<th>Amount</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>UNCLASSIFIED EXCAVATION - Native Soil</td>
<td>CY</td>
<td>21,338</td>
<td>$6.00</td>
<td>$128,026</td>
</tr>
<tr>
<td>2</td>
<td>UNCLASSIFIED FILL - w/ Suitable Excavated Native Soil</td>
<td>CY</td>
<td>987</td>
<td>$900</td>
<td>$888,480</td>
</tr>
<tr>
<td>3</td>
<td>STRUCTURAL CONCRETE</td>
<td>EA</td>
<td>43</td>
<td>$33,000</td>
<td>$1,419,000</td>
</tr>
<tr>
<td>4</td>
<td>GROUND ANCHOR (SUBHORIZONTAL)</td>
<td>CY</td>
<td>360</td>
<td>$250.00</td>
<td>$90,000</td>
</tr>
<tr>
<td>5</td>
<td>GUTTER CONCRETE</td>
<td>CY</td>
<td>360</td>
<td>$250.00</td>
<td>$90,000</td>
</tr>
<tr>
<td>6</td>
<td>BAR REINFORCING STEEL</td>
<td>LB</td>
<td>231,740</td>
<td>$1.14</td>
<td>$264,184</td>
</tr>
<tr>
<td>7</td>
<td>CABLE RAILING</td>
<td>LF</td>
<td>510</td>
<td>$30.00</td>
<td>$15,300</td>
</tr>
<tr>
<td>8</td>
<td>UNCLASSIFIED FILL - add / trim for flatter slopes of rock armor</td>
<td>CY</td>
<td>7,407</td>
<td>$18</td>
<td>$133,333</td>
</tr>
<tr>
<td>9</td>
<td>PILES - under Grade Beam with anchors</td>
<td>96' CIDH</td>
<td>4,724</td>
<td>$1,800.00</td>
<td>$8,503,200</td>
</tr>
<tr>
<td>10</td>
<td>REMOVE UNSUITABLE EXCAVATION - Haul &amp; Dispose off-site</td>
<td>CY</td>
<td>26,672</td>
<td>$25.00</td>
<td>$666,804</td>
</tr>
<tr>
<td>11</td>
<td>REINFORCED EARTH FILL - w/ Imported fill</td>
<td>CY</td>
<td>61,516</td>
<td>$76.56</td>
<td>$4,709,790</td>
</tr>
<tr>
<td>12</td>
<td>PERMANENT EROSION CONTROL - front slope</td>
<td>LS</td>
<td>1</td>
<td>$250,000.00</td>
<td>$250,000</td>
</tr>
<tr>
<td>13</td>
<td>SUBDRAIN SYSTEM</td>
<td>LS</td>
<td>1</td>
<td>$1,320,000.00</td>
<td>$1,320,000</td>
</tr>
<tr>
<td>14</td>
<td>TEMPORARY SHORING / DEWATERING</td>
<td>SF</td>
<td>8,000</td>
<td>$100.00</td>
<td>$800,000</td>
</tr>
</tbody>
</table>

#### Subtotal
- $19,188,118

#### Time Related Overhead
- Included separately

#### Mobilization
- Included separately

#### Subtotal Bridge Items
- $19,188,118

#### Contingencies
- Included separately

#### Bridge Total Cost
- $19,188,118

#### Cost Per Sq. Foot
- Incluated separately

#### Slope Stabilization of West Bank
- Included separately

#### Subtotal
- $19,188,118

#### Grand Total
- $19,200,000

---

**Item No. Notes, References & Assumptions**

1. Soft rock excavation with steep slopes - assume stock pile near the site - use normal excavation unit cost
2. Assume 0% of excavated native soil is used for Reinforced Earth Fill
3. Assume Grade Beam over 8’ dia CIDH - 8’ wide x 4’ deep with 2’ thick x 4’ high Retaining Wall + 8’x2’ anchor block at 12’ o.c. - Ratio 10’x10’ to 8’x8’ size
4. Ground Anchor not designed - assume 10’ dia, 200’ long, 500 kips, at 12’ OC., Use $18,000 for 100’ plus $1500 per 10’ increase
5. None
6. Ratio 10’x10’ reinforcing to 8’x8’ size
7. None
8. None
9. Cable railing over 10’ high RW at fill toe grade beam
10. None
11. Assume 25’H x 10’W x 800’L soil wedge brought in for building a flatter rock armor slope, add 100% cost premium for work at shoreline
12. Assume 90’ long piles (to elev. -20’), 0% increase over Diemer Treatment Plant cost by Malcolm bid of $8M for 25-12’ dia x 90’ caisson and 50% decrease for 12’ to 8’ dia
13. Assume 100% of excavated material will unsuitable for Reinforced Earth and can not be used as backfill - haul and dispose off the site. 25% volume increase for loose soils
14. Assume 100% of fill to be imported and compacted with reinforced layers at steeper slopes
15. Assume seeding, fertilizing, top soil, irrigation or other means (concrete face/geofabric) to stabilize the front RE slope - $10/Sq. Ft. over 500’L x 50’ W slope
16. Assume 8’ perforated pipes embedded in permeable materials at back slope and concrete V-channels on front slope - computed as $24K/ft. of embankment height
17. Assume temporary shoring/coffer dam / dewatering required at toe excavation and fill - 10 high exposed area x 800’ long - use high end cost, work in rocky beach
18. Used for exporting to main cost worksheet
19. Included separately
20. Included separately
21. None
22. Included separately
23. None
24. None
25. Included separately
**General Plan Estimate**

**Advance Planning Estimate - Draft**

**Revised**: March 5, 2015

**RCVD BY:**

**IN EST:**

**OUT EST:**

**BRIDGE**: Paseo Del Mar at White Point Slide

**BR. No.**: N/A

**DISTRICT**: 07

**TYPE**: WALL OPTION - SHEAR PIN WITH MSE BUTTRESS FILL ON TOP

**RTE**: Paseo Del Mar

**CU**: 00-008

**EA**: NA

**LENGTH**: 530.00

**WIDTH**: 58.00

**AREA (SF)**: 30,740

---

**Design Section**

**# of Structures in Project**: 1

**Est. No.**: 1

**Prices By**: GE

**Prices Checked By**: RKB

**Date**: 11/24/2015

**Quantities By**: GE

**Date**: 11/23/2015

---

**Contract Items**

<table>
<thead>
<tr>
<th>Item No.</th>
<th>Category</th>
<th>Type</th>
<th>Unit</th>
<th>Quantity</th>
<th>Price</th>
<th>Amount</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Unclassified Excavation - Native Soil</td>
<td>CY</td>
<td>21,556</td>
<td>$6.00</td>
<td>$129,333</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Unclassified Fill - w/ Suitable Excavated Native Soil</td>
<td>CY</td>
<td>9.00</td>
<td>$900</td>
<td>$257,400</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Structural Concrete</td>
<td>CY</td>
<td>286</td>
<td>$22,000</td>
<td>$1,000,000</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Ground Anchor (Subhorizontal)</td>
<td>EA</td>
<td>50</td>
<td>$22,000</td>
<td>$1,000,000</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Gutter Concrete</td>
<td>CY</td>
<td>340</td>
<td>$250.00</td>
<td>$85,000</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>Bar Reinforcing Steel</td>
<td>LB</td>
<td>74,470</td>
<td>$1.14</td>
<td>$84,896</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>Earth Retaining Structure (MSE Wall)</td>
<td>SQFT</td>
<td>22,560</td>
<td>$80</td>
<td>$1,804,800</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>Structural Concrete, Footing (Leveling Pad)</td>
<td>CY</td>
<td>9</td>
<td>$280</td>
<td>$2,520</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>Chain Link Railing - Pedestrian</td>
<td>LF</td>
<td>530</td>
<td>$75.40</td>
<td>$39,962</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>Structural Concrete, Moment Slab</td>
<td>Appr Type N</td>
<td>CY</td>
<td>262</td>
<td>$1,000</td>
<td>$262,000</td>
</tr>
<tr>
<td>11</td>
<td>Concrete Barrier</td>
<td>LF</td>
<td>530</td>
<td>$180</td>
<td>$95,400</td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>Piles - As Shear Pins</td>
<td>36&quot; CIDH</td>
<td>LF</td>
<td>6,216</td>
<td>$800.00</td>
<td>$4,972,800</td>
</tr>
<tr>
<td>13</td>
<td>Remove Unsuitable Excavation - Haul &amp; Dispose off-site</td>
<td>CY</td>
<td>26,944</td>
<td>$25.00</td>
<td>$673,611</td>
<td></td>
</tr>
<tr>
<td>14</td>
<td>MSE Fill - w/ cribbing using Imported fill</td>
<td>CY</td>
<td>46,557</td>
<td>$103.71</td>
<td>$4,828,214</td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>Piles - Under Grade Beam with Anchor</td>
<td>48&quot; CIDH</td>
<td>LF</td>
<td>1,932</td>
<td>$1,000.00</td>
<td>$1,932,000</td>
</tr>
<tr>
<td>16</td>
<td>Subdrain System</td>
<td>LS</td>
<td>1</td>
<td>$660,000.00</td>
<td>$660,000</td>
<td></td>
</tr>
<tr>
<td>17</td>
<td>Subtotal</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$16,927,936</td>
</tr>
<tr>
<td>18</td>
<td>Time Related Overhead (included separately)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$16,927,936</td>
</tr>
<tr>
<td>19</td>
<td>Mobilization (included separately)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$16,927,936</td>
</tr>
<tr>
<td>20</td>
<td>Subtotal Bridge Items</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$16,927,936</td>
</tr>
<tr>
<td>21</td>
<td>Contingencies (included separately)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$16,927,936</td>
</tr>
<tr>
<td>22</td>
<td>Bridge Total Cost</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$16,927,936</td>
</tr>
<tr>
<td>23</td>
<td>Non-Imported Pile Diamond per foot</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$16,927,936</td>
</tr>
<tr>
<td>24</td>
<td>Slope Stabilization of West Bank (included separately)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$16,927,936</td>
</tr>
<tr>
<td>25</td>
<td>Grand Total</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$16,927,936</td>
</tr>
</tbody>
</table>

**Budget Estimate as of**: 3/24/16

**Total Cost**: $16,927,936

---

**Item No. Notes, References & Assumptions**

1. Soft rock excavation with steep slopes - assume stock pile near the site - use normal excavation unit cost
2. Assume 0% of excavated native soil is used for Reinforced Earth Fill
3. Assume Grade Beam over 3' dia CIDH - 5' wide x 2' deep with 2' thick x 3' high Retaining Wall + 5'x5'x1' anchor block at 20' o.c.
4. Ground Anchor not designed - assume 8" dia, 200' long, 300 kips, at 10' OC., Use $12,000 for 100' plus $1000 per 10' increase
5. None
6. None
7. Assume MSE panel with reinforcement and staggered to achieve about 1/4H : 1V slope; use higher end of unit price
8. Mount pedestrian railing to top of concrete barrier
9. None
10. None
11. None
12. 74' long 36" dia CIDH piles (pile tip to 0')- about 84 count, use 100% of high Caltrans bid tab cost due to unknown subsurface condition an dlong lengths
13. Assume 100% of excavated material will unsuitable for Reinforced Earth and can not be used as backfill - haul and dispose off the site. 25% volume increase for loo
14. Assume small lifts due to constraint work site. 350 CY per day at $9500 per day rate for preparing cribbing for MSE wall fill
15. 84' long 48" dia CIDH piles (pile tip to -10') - about 23 count, use 100% of high Caltrans bid tab cost due to unknown subsurface condition and long lengths
16. Assume 8" perforated pipes embeded in permeable materials at back slope - computed as $12K/ft. of embankment height
17. None
18. Used for exporting to main cost worksheet
19. Included separately
20. Included separately
21. None
22. Included separately
23. None
24. None
25. Included separately
APPENDIX B -
PRELIMINARY REPORT SUBMITTAL CHECKLIST
Consultant Prepared Advance Planning Study (APS) Checklist

Sheet 1 of 2

Date:  Consultant Firm (for structures):  Phone No:
Jan 2016  AECOM  (714) 689-7246

Designed by:  Phone No:
Raj Bharil, PE, SE  (714) 554-0440

EA:  County:  Rte:  KP(PM)
N/A  Los Angeles  Paseo Del Mar  N/A

Project Description:
Reconstruct a portion of Paseo Del Mar Roadway that was damaged during a massive landslide in 2011 in San Pedro neighborhood of the City of Los Angeles. Options considered in this study includes: bridge, wall, and fill. This study will compare these options and document the planning processes and study findings.

Bridge No(s):  Bridge Name(s):
N/A  Paseo Del Mar over White Point Landslide

Total number of bridges in project: 1  APS Alternative Letter or Number (if more than one): N.A

Purpose of this APS:  Initial APS Cost & Feasibility  Revised scope  Update cost

Part A  Items to collect and considerations prior to beginning the APS

All items listed in Part A are to be made available and submitted if requested by the Liaison Engineer. (Mark N/A if not applicable)

- Preliminary profile grade of proposed structure.
- Typical section of the proposed structure. (Including barrier type, sidewalks, cross slope %, etc.)
- Grades or spot elevations of roadway below the structure.
- Typical section of roadway below the structure. (Including shoulders, gutters, embankment slope.) N/A
- Site map: including horizontal alignment of new structure and the roadway below, topo, contours, etc.
- Stage construction or detour plan for traffic on the structure, N/A
  (number of lanes to remain open, Temp Railing, etc.)
- Stage construction or detour plan for the roadway below the structure, N/A
  (falsework openings for each stage and any restrictions.)
- "As Built" plans for existing structures. N/A
- Future widening plans of upper and lower roadway (verify with Route Concept Report). N/A
- Site aerial photograph (at the proposed structure).
- Environmental and/or permit requirements (areas of potential impact, construction windows, etc.)
- Overhead and underground utility plans (N/A – only available as-built plan of adjacent area)
- Any other information that you feel is necessary to complete the study. (Other concerns that may affect the APS: local agency requirements such as aesthetics, improvements in vicinity of structure, airspace usage, other obstructions, etc.)
Consultant Prepared Advance Planning Study (APS) Checklist
Sheet 2 of 2

Part B  Considerations during the APS design and cost estimate preparation

1. Has this project been discussed with:  
   - the OSFP Liaison Engineer?  
   - the Caltrans District Project Manager?  
   - the roadway consultant?  
   Yes  No

2. Have the Caltrans Structures Maintenance records been reviewed?  
   If the records recommend any work for the structure, is it included in the APS?  
   Yes  No

3. Are there special aesthetic considerations?  
   Yes  No

4. (Widenings and Modifications) N/A  
   Has this project been reviewed for seismic retrofit requirements?  
   Are seismic retrofit requirements included in the APS?  
   Yes  No

5. Any special Railroad requirements?  
   Shoofly required?  
   Cost of shoofly included as a separate item in the project cost estimate?  
   Yes  No

6. Any special foundation requirements, including scour critical work, special excavation such as Type A, Type D, and/or hazardous or contaminated material?  
   Yes  No

7. Any special construction requirements, including limited site accessibility or seasonal work?  
   Yes  No

8. Other items to be included in the cost such as slope paving, approach slabs, and/or adjacent retaining walls?  
   Yes  No

9. Remove existing bridge?  
   Total Deck Area: 20,900 sq. ft. (New Bridge)  
   Yes  No

10. Any other unusual or special requirements?  
    Yes  No

11. Provide and attach a consultant prepared Design Memo to summarize and document any important assumptions, discussions, decisions, unusual items, local agency requirements such as aesthetics, improvements in vicinity of the structure, airspace usage, other obstructions, or any items noted above.  
    Summary attached?  
    Yes  No

Note: This checklist is prepared as part of a study for the permanent restoration options of Paseo Del Mar.

Designer:  (Printed Name)  
Raj Bharil, PE, SE  
Designer’s Signature:  
Date: 12/29/15
APPENDIX C - PRELIMINARY STRUCTURES PLANS
APPENDIX D -
SELECTED AS-BUILT PLANS
Removal:

Conc 4" [2(5X8)+(5X5)] SF = 105 SF

Chain Link Fence 87 LF

TYPE C intergal C&G 44 LF

AC 10" (1540 + 440 + 3150) SF + (163 LF* 2.5 FT)= 5,537.5 SF

Unclassified Excavation (5537.5+106.74+65.34+105) SF X .67=3,781.64 SF/27=144 CY

AC Cold Plane 2" min 1360 SF

Construction:

0" to 6" AC Curb 16 LF
6" curb 130 LF
8" curb 40 LF
12' transition to 8" curb 23 LF
12" curb 151 LF

AC Overlay 2" min (1540 + 3150 + 1360) SF x 0.0141 TON/SF = 86 TON

AC 6" (1540 SF + 3150 SF) x 0.0391 TON/SF = 185 TON

Conc 8" Cross Gutter 437 SF

Conc 8" DWY 66 SF

CMB 10" 1540 SF + 437 SF +3150 SF = 5127 SF
130 LF * 2.5 FT= 325 SF
5127 SF+325 SF = 5452 SF X 0.83 FT = 4525.161 CF /27 CY/CF =167.60 CY

CMB 4" 66 SF+ 50 SF + 109 SF = 225 SF X 0.33 FT=74.22 CF /27 =2.75 CY
Appendix F.
Wave Run-Up Study
CITY OF LOS ANGELES

Paseo Del Mar (White Point) Permanent Restoration – Pre-Design Phase

Final Wave Run-up Study

Submitted by:

AECOM
310 Golden Shore, Suite 100
Long Beach, CA.  90802
www.aecom.com

Date:
May 9, 2016
# Table of Contents

1. Introduction and Objective ................................................................. 1
   1.1 Introduction ..................................................................................... 1
   1.2 Objective ....................................................................................... 2

2. Water Levels ...................................................................................... 2
   2.1 Tides ................................................................................................. 2
   2.2 Long-term sea level rise ................................................................. 4
   2.3 Stillwater elevations under storm conditions ......................... 5

3. Waves ................................................................................................ 6
   3.1 Bathymetry and model mesh ......................................................... 6
   3.2 Offshore boundary conditions ...................................................... 8
   3.3 Wind ................................................................................................. 10
   3.4 Simulation of Offshore and Nearshore Waves ......................... 12

4. Wave Run-Up Analysis ....................................................................... 20
   4.1 Restoration concepts ................................................................. 22
   4.2 Wave run-up analysis with empirical TAW methodology .......... 26
   4.3 Wave run-up analysis with empirical CEM equations ............... 27
   4.4 Wave run-up simulation with Boussinesq wave model .......... 28

5. Permits and Regulatory Issues ............................................................ 35

6. Summary of Findings, Discussions, and Recommendations ............. 37
   6.1 Summary of Findings ................................................................. 37
   6.2 Discussions .................................................................................... 37
   6.3 Recommendations ....................................................................... 37

7. References .......................................................................................... 41
LIST OF FIGURES

Figure 1. Paseo Del Mar Landslide Location.................................................................1
Figure 2. Existing Tidal Station Locations.................................................................3
Figure 3: Overview of Mike 21 SW Wave Model .........................................................7
Figure 4: Close-up View of Model Domain in Project Area .........................................7
Figure 5: Wave Rose Chart for CDIP Station# 092 ....................................................8
Figure 6: Wave Rose Chart for WIS Station# 83101 (After USACE) .........................8
Figure 7: Extreme Analysis Plot of Significant Wave Height vs Return Period at WIS
       Station#83101 (After USACE) ...........................................................................9
Figure 8: Wind Rose Chart for Long Beach Airport (inland) .......................................10
Figure 9: Wind Rose Chart for NOAA CO-OPS Angels Gate (nearshore) ..................10
Figure 10: Wind Rose Chart for Offshore WIS Station# 83101 (After USACE) ..........11
Figure 11: Model Monitor Stations for Wave Run-Up Study & Scour Protection ..........12
Figure 12: Screenshot of Wave Pattern during the Extreme Storm Condition for 100-year Storm
           (West Wind Direction) ..................................................................................13
Figure 13: Close-up View of Wave Pattern for 100-year Storm Event (West Wind Direction).....14
Figure 14: Screenshot of Wave Pattern during the Extreme Storm Condition for 100-year Storm
           (Southwest Wind Direction) .........................................................................15
Figure 15: Close-up View of Wave Pattern for 100-year Storm Event (Southwest Wind Direction)
           ..............................................................................................................16
Figure 16: Screenshot of Wave Pattern during the Extreme Storm Condition for 100-year Storm
           (South Wind Direction) ..................................................................................17
Figure 17: Close-up View of Wave Pattern for 100-year Storm Event (South Wind Direction) .18
Figure 18: Selected Transect for Wave Run-up Study..................................................20
Figure 19: Pre- and Post-slide Elevation Profiles for Selected Transect .......................21
Figure 20: Restoration Concept 1 - Existing Situation ...............................................23
Figure 21: Restoration Concept 2 - Existing Situation with Scour Protection ..............24
Figure 22: Restoration Concept 3 – Excavation and setback Toe about 60 feet to the onshore 25
Figure 23: Definition Sketch of Wave Run-up (After FEMA 2011) ...............................26
Figure 24: Snapshot of Wave Propagation and Run-up for Concept 1 at one time step in the
           100-year Design Storm Event ......................................................................29
Figure 25: Snapshot of Wave Propagation and Run-up for Concept 2 at one time step in the
           100-year Design Storm Event ......................................................................30
Figure 26: Snapshot of Wave Propagation and Run-up for Concept 3 at one time step in the
           100-year Design Storm Event ......................................................................30
Figure 27: Wave Run-up and Peak Values for Concept 1 for 100-year Storm Event ........31
Figure 28: Wave Run-up and Peak Values for Concept 2 for 100-year Storm Event ..........32
Figure 29: Wave Run-up and Peak Values for Concept 3 for 100-year Storm Event ..........33
Figure 30: Typical sand Dune Design ..............................................................................38
Figure 31: Revetment Toe Protection ................................................................................39
Figure 32: Reduced revetment Toe Protection ..................................................................39

LIST OF TABLES

Table 1: Tidal Information ..................................................................................................3
Table 2: Sea Level Rise Estimates for Los Angeles Relative to the Year 2000 ......................4
Table 3: Water Level at NOAA Tidal Stations for Different Return Period .......................5
Table 4: Stillwater Elevations for 75-Year Structure Design Life (Sea Level Rise Incorporated)........5
Table 5: Extreme Waves Conditions at Offshore Boundary ...............................................9
Table 6: Extreme Wind Speed at NOAA CO-OPS Angels Gate Station ............................11
Table 7: Extreme Wind Speed at WIS Station #83101 ......................................................12
Table 8: Model Output Stations for Wave Run-Up Study ..................................................13
Table 9: Extreme Waves Conditions in the Front of Toe for Different Return Periods .......19
Table 10: Wave Run-up Estimate with TAW Methodology under 100-year Design Storm Event .................................................................27
Table 11: Wave Run-up Estimate with CEM Equations under 100-year Design Storm Event...28
Table 12: Wave Run-up Simulation with Boussinesq Wave Model under 100-year Design Storm Event ......................................................................................34
Table 13: Regulatory Requirements ..................................................................................35
1. Introduction and Objective

1.1 Introduction

The White Point Landslide occurred on November 20, 2011 in the Palos Verdes area of the City of Los Angeles (Figure 1). The landslide damaged the roadway along the top of the bluff and the City of Los Angeles is currently evaluating options to stabilize the bluff and reconstruct the road. AECOM was contracted to provide pre-design and design support services for the project.

To develop effective engineering alternatives for permanent solutions for this project, understanding future flood risk caused by high tides, storm surge, sea level rise, and wave run-up is a key requirement. The wave run-up is one of the most important factors affecting the design of the coastal shore protection because it determines the design crest level of the coastal defense structure in cases where no (or only marginal) overtopping is acceptable. For coastal bluffs, sea-level rise will lead to an increase in bluff erosion and bluff retreat because waves will break closer to the coastline and will reach the base of the cliff or bluff more frequently. For Paseo del Mar, the current best available science is presented in the National Research Council’s 2012 report on sea level rise for the Pacific coast. The California Coastal Commission endorses this report within its recent sea level rise policy guidance and this was used for the study.

Figure 1. Paseo Del Mar Landslide Location.
Wave run-up is defined as the maximum water-surface elevation measured vertically from the stillwater level. Wave run-up is one type of wave-structure interaction (e.g., where the structure could include a beach, coastal bluff, etc.). Wave run-up is a function of nearshore wave transformation and wave breaking across the surf zone, and site-specific characteristics of the beach and structure. Evaluation of wave conditions at the project site is one of the most important work components required for the wave run-up study.

For the wave run-up study the following factors are considered.

- Type, height, peak period, steepness, direction, distribution nature of incident waves;
- Bathymetry, beach slope, and topography;
- Design stillwater levels (including tides and storm surge);
- Sea level rise scenario(s);
- Coastal protection structures;
- Coastal line characteristics (slope, roughness, and the permeability and porosity of the slope);

For this wave run-up study, the methodology, data sources, results, alternative analysis, and recommendations are summarized in the report.

1.2 Objective

The primary objectives of this wave run-up study are:

- Evaluate the elevations of wave run-up under design storm condition;
- Provide design wave conditions at project site in case the design of slope protection is needed.
- Provide protection recommendations and construction feasibility evaluations for the alternatives being considered for re-construction of the roadway.

2. Water Levels

2.1 Tides

There is no tidal station available directly at the project site. The closest tidal station is the NOAA Port of Los Angeles (POLA) Tidal Station #9410660 (33°43.2' N, 118°16.3' W), which is approximately 3 miles from the project site within the harbor. The next closest tidal station is the NOAA Santa Monica Tidal Station #9410840 (34°0.5' N, 118°30.0' W), which is approximately 25 miles from project site in the open coast. Both stations are shown in Figure 2. The tidal information at these two tidal stations is summarized in Table 1.
Table 1: Tidal Information

<table>
<thead>
<tr>
<th>Tidal Datum</th>
<th>Elevation at POLA ¹(ft)</th>
<th>Elevation at Santa Monica ¹(ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Highest Observed Water Level</td>
<td>7.72 (01/10/2005)</td>
<td>8.31 (11/30/1982)</td>
</tr>
<tr>
<td>Highest High Tide Line ²</td>
<td>5.30</td>
<td>5.30</td>
</tr>
<tr>
<td>Mean Higher High Water</td>
<td>5.29</td>
<td>5.23</td>
</tr>
<tr>
<td>Mean High Water</td>
<td>4.55</td>
<td>4.50</td>
</tr>
<tr>
<td>Mean Tide Level</td>
<td>2.65</td>
<td>2.62</td>
</tr>
<tr>
<td>Mean Sea Level</td>
<td>2.62</td>
<td>2.60</td>
</tr>
<tr>
<td>Mean Low Water</td>
<td>0.74</td>
<td>0.74</td>
</tr>
<tr>
<td>NAVD88</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>Mean Lower Low Water</td>
<td>-0.20</td>
<td>-0.19</td>
</tr>
<tr>
<td>Lowest Observed Water Level</td>
<td>-2.93 (12/17/1933)</td>
<td>-3.03 (12/17/1933)</td>
</tr>
</tbody>
</table>

1. Elevation is measured relative to NAVD88.
2. USACE 404 Regulatory Jurisdiction Limit

From Table 1, it can be seen that the tidal difference between the two tidal stations is minor and the tidal information at project site is close to the tidal information at NOAA POLA Tidal Station.
2.2 Long-term sea level rise

The science associated with sea level rise is continually being updated, revised, and strengthened. Although there is no doubt that sea levels have risen and will continue to rise at an accelerated rate over the coming century, it is difficult to predict with certainty what amount of sea level rise will occur at any given time in the future. The uncertainty increases over time (e.g. the uncertainties associated with 2100 projections are greater than with 2050 projections) because of uncertainties in future greenhouse gas (GHG) emissions trends, the sensitivity of climate conditions to GHG concentrations, and the overall skill of climate models. Given these uncertainties, the sea level rise projections presented in this document for use in the Paseo del Mar roadway designs draw on the best available science on the potential effects of sea level rise in California as of January 2014.

For Paseo del Mar, the current best available science is presented in the National Research Council’s 2012 report on sea level rise for the Pacific coast. The California Coastal Commission endorses this report within its recent sea level rise policy guidance.

Table 2 presents the sea level rise estimates for the Los Angeles area. The table presents the local projections ± one standard deviation. These projections (for example, 36 ± 10 inches in 2100) represent the likely sea level rise values based on a moderate level of greenhouse gas emissions and extrapolation of continued accelerating land ice melt patterns, plus or minus 1 standard deviation. The extreme limits of the ranges (for example, 17 and 66 inches for 2100) represent unlikely but possible levels of sea level rise using both very low and very high emissions scenarios and, at the high end, including significant land ice melt that is currently not anticipated but could occur.

Table 2: Sea Level Rise Estimates for Los Angeles Relative to the Year 2000

<table>
<thead>
<tr>
<th>Year</th>
<th>Projections</th>
<th>Ranges</th>
</tr>
</thead>
<tbody>
<tr>
<td>2030</td>
<td>6 ± 2 in</td>
<td>2 to 12 in</td>
</tr>
<tr>
<td>2050</td>
<td>11 ± 4 in</td>
<td>5 to 24 in</td>
</tr>
<tr>
<td>2100</td>
<td>36 ± 10 in</td>
<td>17 to 66 in</td>
</tr>
</tbody>
</table>

The lower range estimates (for example, 17 inches by 2100) are not appropriate for use in project planning or design. Using the precautionary principle and considering an appropriate level of uncertainty in future GHG emissions, responsible planning should consider only the likely and upper range sea level rise projections (for example, 36 and 66 inches by 2100, respectively).

Projects that can be adapted in the future to accommodate higher rates of sea level rise can be constructed using the likely projections, as long as the project is constructed in a way to accommodate future adaptation. For example, a levee can be constructed today to accommodate 36 inches of sea level rise, and include an over-sized foundation that can more easily accommodate future increases in levee height if sea level rise tracks toward the upper range estimate of 66 inches. However, if a project is not easily adaptable, the upper range estimate should be used for planning and design.

For this project, the design life is 75-year. If construction can be complete by 2020, the sea level estimate for 2100 can be used. Considering the uncertainty of sea level rise projections and potential longer lifespan than the design life, 66 inch of sea level rise is used in this wave run-up study.
2.3  **Stillwater elevations under storm conditions**

Extreme water elevation analysis was performed at the NOAA POLA Tidal Station and Santa Monica Tidal Station to derive stillwater elevations for different return periods, the analysis results are summarized in Table 3.

**Table 3: Water Level at NOAA Tidal Stations for Different Return Period**

<table>
<thead>
<tr>
<th>Return Period of Storm Event (year)</th>
<th>Elevation at POLA$^1$ (ft)</th>
<th>Elevation at Santa Monica$^1$ (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>6.8</td>
<td>6.9</td>
</tr>
<tr>
<td>10</td>
<td>7.3</td>
<td>7.4</td>
</tr>
<tr>
<td>50</td>
<td>7.6</td>
<td>7.8</td>
</tr>
<tr>
<td>100</td>
<td>7.7</td>
<td>7.9</td>
</tr>
<tr>
<td>500</td>
<td>7.9</td>
<td>8.3</td>
</tr>
</tbody>
</table>

1. Elevation is measured relative to NAVD88.

For this study, extreme water elevations derived from NOAA POLA Tidal Station were employed.

For a 75-year design life of restoration countermeasure and on the basis of an assumed project completion date on or before 2020, the calculated stillwater elevations which include sea level rise are summarized in Table 4.

**Table 4: Stillwater Elevations for 75-Year Structure Design Life (Sea Level Rise Incorporated)**

<table>
<thead>
<tr>
<th>Return Period of Storm Event (year)</th>
<th>Elevation (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>12.3</td>
</tr>
<tr>
<td>10</td>
<td>12.8</td>
</tr>
<tr>
<td>50</td>
<td>13.1</td>
</tr>
<tr>
<td>100</td>
<td>13.2</td>
</tr>
<tr>
<td>500</td>
<td>13.4</td>
</tr>
</tbody>
</table>

1. Elevation is measured relative to NAVD88.
3. Waves

To perform the wave run-up study, the wave modeling is an essential tool for developing a solid understanding of the complex coastal process at the project site.

AECOM developed a MIKE 21 SW offshore and nearshore Spectral Wave Model for this region through our work undertaken for previous projects. For this project the model mesh was refined near the project site for this wave run-up study.

MIKE 21 SW Spectral Wave Model was used in this study to provide wave conditions at the project site. The primary purposes of this MIKE 21 SW Wave Model are:

- Provide ocean boundary conditions for a MIKE 21 Boussinesq local wave model for the wave run-up study; and
- Provide design wave conditions for the wave run-up study and toe/slope scour protection if needed.

This MIKE 21 SW offshore and nearshore spectral wave model is a new 3rd generation spectral wind-wave model with an unstructured mesh that simulates the growth, decay and transformation of wind-generated waves and swells in offshore and nearshore coastal areas. The unstructured mesh approach provides the model the maximum degree of flexibility. The model includes the following physical phenomena:

- wave growth by action of wind;
- non-linear wave-wave interaction;
- dissipation by white-capping;
- dissipation due to bottom friction;
- dissipation due to depth-induced wave breaking;
- refraction and shoaling due to depth variations;
- wave-current interaction; and
- effect of time-varying water depth and flooding and drying.

3.1 Bathymetry and model mesh

The bathymetry source data used to develop the model mesh has been obtained from several sources:

- GEODAS, 3 second bathymetry data, covering San Pedro Bay;
- Navigation Chart; and
- Survey data around project site (Wagner Engineering & Survey, Inc, 2012).

The upper limit of the sea level rise estimate for year 2100 (66-inches) are included in the wave model for the extreme storm conditions. The overview and close-up view of the model domain are shown in Figures 3 and 4.
Figure 3: Overview of Mike 21 SW Wave Model

Figure 4: Close-up View of Model Domain in Project Area
3.2 Offshore boundary conditions

There are several measured time series of wave data available from year 2000 to 2015 at CDIP (Coastal Data Information Program) Buoy Station #028 (33° 51.27' N, 118° 37.97' W), Station #092 (33° 37.07' N 118° 19.00' W), and Station #096 (33° 27.51' N 117° 46.03' W). There is also hindcast wave data from WIS Station #83101 (33° 34.80' N, 118° 15.00' W) from 1981 to 2011.
From above Figures 5 and 6, it can be seen that the primary wave directions in the offshore are from West, Southwest, and South.

In this study, measured data from CDIP Stations was used to provide the offshore wave boundary conditions. The hindcast wave data from WIS Station #83101 was used for cross-checking of the data.

The extreme waves at the offshore boundary for a 100-year return period are shown in Table 5.

<table>
<thead>
<tr>
<th>Return Period (year)</th>
<th>Significant Wave Height (ft)</th>
<th>Peak Wave Period (sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>17.4</td>
<td>18.5</td>
</tr>
</tbody>
</table>

For comparison purpose, the extreme analysis plot of significant hindcast wave data from WIS Station #83101 is shown in Figure 7.

Figure 7: Extreme Analysis Plot of Significant Wave Height vs Return Period at WIS Station#83101 (After USACE)

It can be seen that, for extreme wave analysis at WIS Station #83101, the significant wave height for 100-year return period is 5.5 m (or 18.0 ft) which compares well with the 17.4 ft wave used to provide the offshore wave boundary conditions as derived from the CDIP Stations.
3.3 Wind

There are different measured wind stations available in support of the wave study. This includes long-term time series from the Long Beach Airport from 1943 to 2015, and NOAA CO-OPS Angels Gate wind data from 2005 to 2011 and 2013 to 2015. There is also hindcast wind data from WIS Station #83101 (33° 34.80’ N, 118° 15.00’ W) from 1981 to 2011.

Figure 8: Wind Rose Chart for Long Beach Airport (inland)

Figure 9: Wind Rose Chart for NOAA CO-OPS Angels Gate (nearshore)
Figure 10: Wind Rose Chart for Offshore WIS Station# 83101 (After USACE)

From above wind rose charts, it can be seen that the distributions/probabilities of wind speed and wind direction at inland, nearshore, and offshore are considerably different. For this wave model, it is more appropriate to use measured nearshore and offshore wind data.

Extreme wind analysis was performed for both NOAA CO-OPS Angels Gate wind data (measured) and WIS Station# 83101 (hindcast) wind data. The analysis result is shown in Table 6 and 7.

Table 6: Extreme Wind Speed at NOAA CO-OPS Angels Gate Station

<table>
<thead>
<tr>
<th>Return Period (year)</th>
<th>Hourly Wind Speed (mph)</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>45</td>
</tr>
</tbody>
</table>
Table 7: Extreme Wind Speed at WIS Station #83101

<table>
<thead>
<tr>
<th>Return Period (year)</th>
<th>Hourly Wind Speed (mph)</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>44</td>
</tr>
</tbody>
</table>

3.4 Simulation of Offshore and Nearshore Waves

For this wave study, wave simulations were performed for offshore waves and winds from West, Southwest, and South.

Model output locations were setup within the model domain for the extraction of model simulation results, as shown in Figure 11 and Table 8.

![Figure 11: Model Monitor Stations for Wave Run-Up Study & Scour Protection](image-url)
Table 8: Model Output Stations for Wave Run-Up Study

<table>
<thead>
<tr>
<th>Station Name</th>
<th>Longitude (deg, W)</th>
<th>Latitude (deg, N)</th>
<th>Elevation (ft, NAVD88)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pt1</td>
<td>-118.313</td>
<td>33.714</td>
<td>-1.6</td>
</tr>
<tr>
<td>Pt2</td>
<td>-118.317</td>
<td>33.708</td>
<td>-49.2</td>
</tr>
<tr>
<td>Pt3</td>
<td>-118.316</td>
<td>33.709</td>
<td>-62.3</td>
</tr>
<tr>
<td>Pt4</td>
<td>-118.323</td>
<td>33.700</td>
<td>-131.2</td>
</tr>
</tbody>
</table>

Screenshots of wave simulation for 100-year return periods for offshore waves and winds from three directions are shown in Figures 12 to 17.

Figure 12: Screenshot of Wave Pattern during the Extreme Storm Condition for 100-year Storm (West Wind Direction)
Figure 13: Close-up View of Wave Pattern for 100-year Storm Event (West Wind Direction)
Figure 14: Screenshot of Wave Pattern during the Extreme Storm Condition for 100-year Storm (Southwest Wind Direction)
Figure 15: Close-up View of Wave Pattern for 100-year Storm Event (Southwest Wind Direction)
Figure 16: Screenshot of Wave Pattern during the Extreme Storm Condition for 100-year Storm (South Wind Direction)
Figure 17: Close-up View of Wave Pattern for 100-year Storm Event (South Wind Direction)
Wave conditions in the front of toe of bluff (Monitor Station #1) under 100-year storm conditions have been extracted from the model results and are summarized in Table 9 below.

### Table 9: Extreme Waves Conditions in the Front of Toe for Different Return Periods

<table>
<thead>
<tr>
<th>Return Period (yr)</th>
<th>Offshore Waves and Wind Direction</th>
<th>Stillwater Elevation (ft, NAVD88)</th>
<th>Significant Wave Height (ft)</th>
<th>Peak Wave Period (sec)</th>
<th>Wave Direction (deg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>South</td>
<td>13.2</td>
<td>9.5</td>
<td>18.8</td>
<td>195</td>
</tr>
<tr>
<td></td>
<td>Southwest</td>
<td>13.2</td>
<td>9.5</td>
<td>19.0</td>
<td>210</td>
</tr>
<tr>
<td></td>
<td>West</td>
<td>13.2</td>
<td>8.6</td>
<td>18.6</td>
<td>220</td>
</tr>
</tbody>
</table>

Notes:  
1. The wave direction refers to True North (come from, clockwise).  
2. Long-term sea level rise is included in the Stillwater elevation.
4. Wave Run-Up Analysis

Wave run-up is the maximum vertical extent of wave uprush on a beach, bluff or structure above the stillwater level.

For this wave run-up study, one transect with maximum change between pre- and post-landslide section was selected and extracted to conduct the wave run-up study. This transect is shown in Figure 18 below. The pre and post landslide section is shown in figure 19.

Figure 18: Selected Transect for Wave Run-up Study
Figure 19: Pre- and Post-slide Elevation Profiles for Selected Transect
4.1 Restoration concepts

Three (3) restoration concepts were selected for analysis:

Concept 1 – Existing Situation
In this concept the objective would be to leave the post-landslide bluff intact and assess if this will maintain a stable face.

Concept 2 – Existing Situation with Scour protection
In this concept the expectation is the post-landslide bluff would generally remain intact, but long term stability would be achieved by providing a scour protection consisting of rock armor stones to protect in place the post-landslide bluff.

Concept 3 - Excavation and setback toe about 60 feet to the onshore
In this concept excavation and ground works would take place and the bluff would be reconstructed to an approximate pre-slide position, in the expectation that this pre-slide face would be able to maintain a stable face.

Schematics of the three concepts are shown in Figures 20 to 22.
Figure 20: Restoration Concept 1 - Existing Situation
Figure 21: Restoration Concept 2 - Existing Situation with Scour Protection
Figure 22: Restoration Concept 3 – Excavation and setback Toe about 60 feet to the onshore
In general, methods for the prediction of wave run-up include empirical equations, computer programs based on empirical equations, numerical models, and physical models.

In this study, based on our proposed scope of work, wave run-up was analyzed using the following methods:

- Empirical equations: TAW Run-up formula and CEM Run-up Formula;
- Numerical model: Boussinesq wave model.

4.2 Wave run-up analysis with empirical TAW methodology

The TAW wave run-up methodology is supported by FEMA for wave run-up studies along the open Pacific Coast. TAW can be used to analyze wave run-up on a structure for which the slope, roughness and other relevant characteristics are known. Application of the methodology requires that the significant wave height, peak wave period of the wave spectrum and water level are known at the toe of the slope (TOS), see Figure 23.

Figure 23: Definition Sketch of Wave Run-up (After FEMA 2011)

The TAW equations for the 2% wave run-up are (after FEMA 2011):

\[
R_{2\%} = H_m \begin{cases} 
1.75 \gamma_b \gamma_f \beta \xi_0 & 0.5 \leq \gamma_b \xi_0 < 1.8 \\
\gamma_f \beta \left( 4.3 - \frac{1.6}{\sqrt{\xi_0}} \right) & \gamma_b \xi_0 \geq 1.8 
\end{cases}
\]

Where:

\[\xi_0\] Iribarren Number defined as:
\[ \xi_0 = \tan \alpha / \sqrt{S_0} \]

\text{tana} \hspace{1cm} \text{Structure Slope}

\[ S_0 = H_{m0} / \left( g T_{m-1,0}^2 / 2\pi \right) \]

\text{H}_{m0} \hspace{1cm} \text{Spectral significant wave height at the toe of the structure, defined as:}

\[ H_{m0} = 4.0 \sqrt{m_0}, \text{ where } m_0 \text{ is the total wave energy (or equivalently the area under the spectrum,} \]

\text{\gamma_b} \hspace{1cm} \text{Reduction Factor for influence of a berm}

\text{\gamma_f} \hspace{1cm} \text{Reduction Factor for influence of surface roughness}

\text{\gamma_\beta} \hspace{1cm} \text{Reduction Factor for influence of angled wave attack}

\[ T_{m-1,0} \approx T_p / 1.1 \text{ and } T_p \text{ is the wave period associated with the peak of the wave spectrum.} \]

The results of the wave run-up estimate using the TAW methodology for the three (3) restoration concepts under 100-year design storm are summarized in Table 10.

<table>
<thead>
<tr>
<th>Restoration Concept</th>
<th>Toe Elevation (ft, NAVD88)</th>
<th>Stillwater Elevation (ft, NAVD88)</th>
<th>Sig. Wave Height at Toe, H_s (ft)</th>
<th>Peak Wave Period at Toe, T_p (sec)</th>
<th>Wave Run-up Height, R_{2%} (ft)</th>
<th>Elevation Limit of Wave Run-up (ft, NAVD88)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.5</td>
<td>13.2</td>
<td>9.5</td>
<td>19</td>
<td>35.4</td>
<td>48.8</td>
</tr>
<tr>
<td>2</td>
<td>2.5</td>
<td>13.2</td>
<td>9.5</td>
<td>19</td>
<td>19.2</td>
<td>32.6</td>
</tr>
<tr>
<td>3</td>
<td>12.0</td>
<td>13.2</td>
<td>7.5</td>
<td>20</td>
<td>27.6</td>
<td>40.8</td>
</tr>
</tbody>
</table>

4.3 Wave run-up analysis with empirical CEM equations

Wave run-up can also be approximated using the empirical CEM (Coastal Engineering Manual) equations. Wave run-up on impermeable slopes can be formulated in a general expression for irregular waves having the form (ref. Battjes 1974):

\[ \frac{R_{i\%}}{H_s} = (A\xi + C)\gamma_r\gamma_b\gamma_h\gamma_\beta \]

Where:

\[ R_{i\%} \] Run-up level exceeded by i percent if the incident waves

\[ \xi \] Surf-similarity parameter, \( \xi_{0m} \) or \( \xi_{0p} \)
Coefficients dependent on $\xi$ and $i$ but related to the reference case of a smooth, straight impermeable slope, long-crested head-on waves Rayleigh-distributed wave heights.

$\gamma_r$ Reduction Factor for influence of surface roughness ($\gamma_r = 1$ for smooth slopes)

$\gamma_b$ Reduction Factor for influence of a berm ($\gamma_b = 1$ for non-bermed profiles)

$\gamma_h$ Reduction Factor for influence of shallow-water conditions where the wave height distribution deviates from the Rayleigh-distributed ($\gamma_h = 1$ for Rayleigh-distributed waves)

$\gamma_\beta$ Factor for influence of angle of incidence of the waves $\gamma_\beta = 1$ for head-on long-crested waves, i.e. $\beta=0^\circ$). The influence of directional spreading in short-crested waves is included in $\gamma_\beta$ as well.

The wave run-up estimates using the empirical CEM equations for the three (3) restoration concepts under 100-year design storm are summarized in Table 11.

**Table 11: Wave Run-up Estimate with CEM Equations under 100-year Design Storm Event**

<table>
<thead>
<tr>
<th>Restoration Concept</th>
<th>Toe Elevation (ft, NAVD88)</th>
<th>Stillwater Elevation (ft, NAVD88)</th>
<th>Sig. Wave Height $H_s$ at Toe (ft)</th>
<th>Peak Wave Period $T_p$ at Toe (sec)</th>
<th>Wave Run-up Height $R_{2%}$ (ft)</th>
<th>Elevation Limit of Wave Run-up (ft, NAVD88)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.5</td>
<td>13.4</td>
<td>9.5</td>
<td>19</td>
<td>29.2</td>
<td>42.6</td>
</tr>
<tr>
<td>2</td>
<td>2.5</td>
<td>13.4</td>
<td>9.5</td>
<td>19</td>
<td>17.0</td>
<td>30.4</td>
</tr>
<tr>
<td>3</td>
<td>12.0</td>
<td>13.4</td>
<td>7.5</td>
<td>20</td>
<td>24.5</td>
<td>37.9</td>
</tr>
</tbody>
</table>

**4.4 Wave run-up simulation with Boussinesq wave model**

Boussinesq wave model is the state-of-the-art numerical model for the calculation and analysis of short- and long-period waves in coastal areas. The model is capable of reproducing the combined effects of all important wave phenomena of interest in coastal and marine environments. These include shoaling, refraction, diffraction, wave breaking, bottom friction, moving shoreline, partial reflection and transmission, non-linear wave-wave interaction, frequency spreading, and directional spreading. Thus, details like the generation and release of low-frequency oscillations due to primary wave transformation are well described in the model. This is of significant important for coastal processes. The wave model can also account for wave reflection and absorption by revetments and vertical walls.

The 1DH (one horizontal space co-ordinates) Mike 21 Boussinesq wave model was developed to simulate wave run-up under different conditions. The Boussinesq wave model solves the enhanced Boussinesq equations by a standard Galerkin finite element method with mixed interpolation for variables defined on an unstructured (or a structured) grid. Surf zone dynamics and swash zone oscillations (including wave run-up) can be simulated for any coastal profile in this model.
The model setup is described below.

- **Bathymetry and model domain:** The BW model uses a resolution of 1 meter and the model covers a total length of about 950 meter (3116 feet) along the transect profile. (Figure 24 depicts the limits of the model);
- **Type of equation:** The model employs the one dimensional Boussinesq wave module with deep water terms;
- **Simulation period:** The simulation period is 25 minutes;
- **Boundary:** The BW model uses open boundary at the seaward direction and moving shoreline boundary at the bluff slope;
- **Internal wave generation:** The MIKE 21 Wave Generation toolbox uses the output wave information from the spectral wave model as the internal wave for the BW model;
- **Bottom friction:** Manning’s number is applied in the model;
- **Filtering:** A filter layer is set up to filter the high frequency noise during wave runup;
- **Wave breaking:** Wave breaking is considered and Type 3 roller celerity is used;
- **Moving shoreline for wave run-up:** Moving shoreline is included in the BW model;
- **Porosity layer:** Porosity layer is applied when the reflection is important;
- **Sponge layer:** Two sets of sponge layers are employed in the BW model. One set layer at the open boundary to absorb the wave propagates out of the domain from the internal wave generator. The other set sponge layer is at the edge of the slope.

The model simulation results are shown in Figures 24 to 29. Several screen shots of wave surface elevation are shown in the following figures. The elevation in the figures refers to design still water level. Also notice the different horizontal and vertical scales for clarification of wave magnitude.

![Figure 24: Snapshot of Wave Propagation and Run-up for Concept 1 at one time step in the 100-year Design Storm Event](image-url)
Figure 25: Snapshot of Wave Propagation and Run-up for Concept 2 at one time step in the 100-year Design Storm Event.

Figure 26: Snapshot of Wave Propagation and Run-up for Concept 3 at one time step in the 100-year Design Storm Event.
Figure 27: Wave Run-up and Peak Values for Concept 1 for 100-year Storm Event.
Figure 28: Wave Run-up and Peak Values for Concept 2 for 100-year Storm Event
Figure 29: Wave Run-up and Peak Values for Concept 3 for 100-year Storm Event
The results of the wave run-up simulations with the Boussinesq Wave Model for the three (3) restoration concepts under the 100-year design storm are summarized in Table 12.

**Table 12: Wave Run-up Simulation with Boussinesq Wave Model under 100-year Design Storm Event**

<table>
<thead>
<tr>
<th>Restoration Concept</th>
<th>Toe Elevation (ft, NAVD88)</th>
<th>Stillwater Elevation (ft, NAVD88)</th>
<th>Offshore Sig. Wave Height $H_s$ (ft)</th>
<th>Offshore Peak Wave Period $T_p$ (sec)</th>
<th>Wave Run-up Height $R_{2%}$ (ft)</th>
<th>Elevation Limit of Wave Run-up (ft, NAVD88)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.5</td>
<td>13.4</td>
<td>15.8</td>
<td>19.0</td>
<td>26.6</td>
<td>40.0</td>
</tr>
<tr>
<td>2</td>
<td>2.5</td>
<td>13.4</td>
<td>15.8</td>
<td>19.0</td>
<td>11.8</td>
<td>25.2</td>
</tr>
<tr>
<td>3</td>
<td>12.0</td>
<td>13.4</td>
<td>15.8</td>
<td>19.0</td>
<td>17.8</td>
<td>31.1</td>
</tr>
</tbody>
</table>

Based on FEMA’s comparison and assessment, the main advantage of numerical runup models (MIKE 21 Boussinesq Model in our project) over simple procedures (empirical formulas, in our project TAW or CEM) is that with numerical models, arbitrary profile shapes can be studied in combination with widely varying water level and wave parameters. The utility of simple formulas is restricted by the empirical data and conditions that led to their development, and extrapolation to other geometries and conditions may be questionable.

Based on FEMA’s comparison and assessment, and our engineering judgment, the wave runup results from Mike 21 Boussinesq Model should be used for this project for the assessment of wave runup and these are adopted for further design.
5. Permits and Regulatory Issues

A list of agencies that will likely require permits or consultation for the works along the shoreline for all 3 concepts are listed in Table 13:

Table 13: Regulatory Requirements

<table>
<thead>
<tr>
<th>Permit</th>
<th>Contact information</th>
<th>Requirement</th>
<th>Option</th>
</tr>
</thead>
<tbody>
<tr>
<td>USACE Section 404/10 permit for construction(1)</td>
<td>Department of the Army Los Angeles District, U.S. Army Corps of Engineers 915 Wilshire Boulevard, Suite 930 ATTN: Regulatory Division, CESPL-RG Los Angeles, California 90017-3401 Pam Kostka - Regulatory Project Manager T 213-452-3420 F 213-452-4196 <a href="http://www.spl.usace.army.mil/Missions/Regulatory.aspx">http://www.spl.usace.army.mil/Missions/Regulatory.aspx</a></td>
<td>Permit / Consultation</td>
<td>Cons. (2) Permit Permit</td>
</tr>
<tr>
<td>Nationwide permit (14) - USACE</td>
<td>See above</td>
<td>Permit / Consultation</td>
<td>Permit NA(^{(3)}) NA(^{(3)})</td>
</tr>
<tr>
<td>California Coastal Commission</td>
<td>California Coastal Commission Al J. Padilla - Regulatory Permit Supervisor, 200 Oceangate, 10th Floor Long Beach, CA 90801 T 562 590-5071</td>
<td>Permit</td>
<td>Yes Yes Yes</td>
</tr>
<tr>
<td>NOAA Fisheries (formerly NMFS) Magnuson-Stevens Fishery Conservation and</td>
<td>NOAA Fisheries - Southern California Branch, Anthony Spina, P 562 980-4045 <a href="mailto:Anthony.Spina@noaa.gov">Anthony.Spina@noaa.gov</a></td>
<td>Consultation</td>
<td>Yes Yes Yes</td>
</tr>
</tbody>
</table>
### Management Act

<table>
<thead>
<tr>
<th>Agency</th>
<th>Contact Information</th>
<th>Permit Required</th>
<th>Consultation Required</th>
<th>Status</th>
</tr>
</thead>
<tbody>
<tr>
<td>NOAA Fisheries and US Fish and Wildlife Service Endangered Species Act (ESA) Section 10</td>
<td>See above</td>
<td>Consultation</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>CA Department of Fish and Game (CDFG) California Endangered Species Act (CESA)</td>
<td>South Coast Region (Region 5), Regional Manager: Ed Pert Main Office: 3883 Ruffin Road, San Diego, CA 92123, T 858 467-4201, F 858 467-4299 <a href="mailto:AskR5@wildlife.ca.gov">AskR5@wildlife.ca.gov</a></td>
<td>Consultation</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>NOAA Office for Coastal Management Coastal Zone Management Act (CZMA)</td>
<td>See Coastal Commission</td>
<td>Consultation</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>California State Lands Commission (CSLC) for Aquatic Lands Lease if located on state aquatic lands</td>
<td>100 Howe Avenue, Suite 100 South, Sacramento, CA 95825-8202, P 916 574-1900, F 916 574-1810</td>
<td>Permit</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>RWQCB Porter-Cologne Water Quality Control Act – State law equivalent of the 401 Water Quality Certification</td>
<td>320 W. Fourth Street, Suite 200 Los Angeles, CA 90013 P 213 576-6600 Eric Wu 213 576-6683</td>
<td>Permit</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>Los Angeles County “Land Use Permit”</td>
<td>P 213 974-6411, F 213 626-0434, <a href="mailto:zoningldcc@planning.lacounty.gov">zoningldcc@planning.lacounty.gov</a></td>
<td>More research needed to identify these details</td>
<td>TBA</td>
<td>TBA</td>
</tr>
<tr>
<td>Los Angeles County “Flood Plain/Flood Control”</td>
<td>Land Development Division - 3rd Floor, Alhambra, CA 91803, P 626 458-4921</td>
<td>More research needed to identify these details</td>
<td>TBA</td>
<td>TBA</td>
</tr>
</tbody>
</table>

1. USACE 404 Regulatory jurisdictional limit is the Highest High Tide line (~+5.3' NAVD88) and USACE Section 10 jurisdictional limit is the Mean High Water level line (~+4.5' NAVD88).

2. USACE permit generally not required if not working above the high water mark; however, it would still be prudent to have a consultation with the USACE at some point early in the design process to clarify this with them.

3. Works cannot be undertaken under USACE nationwide permit (14) for linear transportation projects since the impacts will be larger than 1/3 acre.
6. Summary of Findings, Discussions, and Recommendations

6.1 Summary of Findings

The findings of this wave run-up study for different restoration concepts are summarized as follows:

- For Concept 1 (existing slope), the estimated wave run-up under 100-year design storm event (future sea level rise incorporated) is about 40 ft above NAVD 88;
- For Concept 2 (existing slope with rock scour protection), the wave run-up under 100-year design storm event (future sea level rise incorporated) is about 25 ft above NAVD 88;
- For Concept 3 (excavation and setback toe about 60 feet to the onshore), the wave run-up under 100-year design storm event (future sea level rise incorporated) is about 30 ft above NAVD 88.

6.2 Discussions

- Shoreline changes and bluff retreat are complicated processes and prediction of resulting changes on these is beyond the scope of work for this study. Therefore, the effect of bluff retreat on the wave run-up is not considered here;
- The stability of the coastal bluff can be affected by contact with water. Water coming in contact with the surface of the bluff face via runoff or storm surge/waves can also cause wearing of the soil mass and gradual loss of soil at the face;
- Typically, external erosion cannot be prevented without a structural solution (either a soft structure or hard structure, or combined soft and hard structure). However, a structural solution will likely not prevent the internal erosion that is occurring from infiltration and the seepage of groundwater;
- To reduce the rate of bluff retreat, protecting the toe of bluff from storm surge and wave run-up is necessary. However, if a hard-structure is introduced on this segment of the shoreline for slope protection, it invariably has potential adverse effects on adjacent, natural segments without such protection. It is imperative that such effects should be analyzed and considered with any alternative during the design phase.
- With the implementation of a shoreline improvement the City takes on a significant obligation in regards to the cost for the structure, future maintenance of the structure, as well as any potential impacts that might result up and downstream of the structure. The Project Team will seek US Army Corps of Engineers participation for these works.

6.3 Recommendations

- It is recommended to perform a desktop study of shoreline change or bluff retreat around project area by analyzing historic aerial/satellite photos (geo-referenced) or related GIS data during the design phase;
The potential bluff protection engineering measures could be toe protection only or toe plus slope protection. Following alternative concepts will be assessed during the design phase for the project:

1) Sand Dune Protection Alternative:

In general, sand dunes are the first line of defense against coastal storms and beach erosion at many coastal locations. They attenuate the impact of storm surge and destructive waves and prevent or reduce damage/erosion to upland coastal shorelines and structures-properties. They can also act as sand reservoirs to supply sand to eroded beaches during storms.

A typical sand dune design may have dimensions on the order of 16-feet above Mean Sea Level (MSL), with a 30-foot dune crest width and one on five side slopes (1V:5H). For sand dunes to work efficiently, adequate berm width is required seaward of the sand dune; otherwise, substantial sections could disappear during severe storms. A typical section is shown in Figure 30.

![Figure 30: Typical sand Dune Design](image)

For this project site, it needs to be assessed if sand dune can survive for a reasonable duration or if it is worth to build a sand dune for bluff toe protection (sand dunes will require maintenance/nourishment).

2) Revetment Toe Protection Alternative:

A revetment could be used for this site to protect the toe of the bluff from erosion. Compared to a seawall, a revetment reflects less wave action and is considered to be an efficient wave energy absorber. By designing an embedded toe for the revetment, this adverse effect of the revetment on the shoreline could be reduced. A full section of the Revetment Toe protection is shown in Figure 31.
As a temporary measure, a reduced toe protection section that consists of large rock only can be considered as shown in Figure 32. This alternative would provide some stabilization of the toe, but will require significant more maintenance and could be considered for the alternatives were there is a significant offset between toe and structure.
The alternatives are constructed using regular applied construction techniques, and deemed feasible for the location. The Sand Dune alternative can be constructed either by trucking in the coarse material and placing this at the toe of the bluff using standard earthmoving equipment, or alternatively be delivered by dredge and pumped to shore. The construction of the Revetment Toe Protection Alternative would require a more comprehensive operation and will include earth moving equipment to cut the trench and grade the embankment, followed by geotextile installation, and backfill with the underlayer and rock armor protection layers. The top armor layer will require stones to be individually placed. Rock materials can be delivered to site by truck, or alternatively by barge. Most likely the supply will be from the rock quarry at Catalina Island, which will require barge transport regardless. However barge delivery to site is difficult because the water is very shallow, and the barge cannot get very close to shore, so a more likely scenario would be that materials are offloaded in the Port of Los Angeles and delivered by truck.

For each of the alternative improvements considered (bridge and embankment options) it is believed that there is considerable buffer/soil mass in front of the new structure to accommodate some erosion before it starts to impact/undermine the proposed improvements. As such it is deemed feasible to install the shore side improvements and implement a monitoring program, and as necessary implement shoreline improvements as the need arises.

The project Team will engage the US Army Corps of Engineers for further study, consultation and ultimately ownership and funding of any required shoreline improvements.
7. References


4. EM 1110-2-1100 Coastal Engineering Manual (CEM), USACE.

Appendix G.
Conceptual Striping Plans
Appendix H.
Material and Dump Site Facilities
Map 109
Population Density within 500 Feet of Truck Routes (2010)

Legend
2010 Population Density (persons per square mile)
- No Population Reported
- Less than 500
- 500 to 5,000
- 5,000 to 10,000
- 10,000 to 20,000
- 20,000 to 30,000
- 30,000 to 50,000
- Greater than 50,000

Jurisdictional Boundaries
- Community Plan Areas

Transportation Systems
- Metro Rail Stations
- Metro Liner Stations
- Metrolink Stations
- Proposed Metro Rail Stations
- Truck Routes
  - Metro Rail Lines
  - Metro Liner Lines
  - Proposed Metro Rail Projects
  - Interstates and Highways
  - State Highways
  - Metrolink Rail Lines


Permission for use of these proprietary data is granted by the City of Los Angeles Department of City Planning. Copyright © 2013 City of Los Angeles. All Rights Reserved. Produced by Raimi + Associates for the City of Los Angeles and Los Angeles County. Made possible with funding from the Centers for Disease Control and Prevention through the Los Angeles County Department of Public Health and The California Endowment May 2013
Figure 4.13-1
Existing Roadway Designations

Source: Iteris, September 2011.
<table>
<thead>
<tr>
<th>Company Name &amp; Address</th>
<th>Phone Number</th>
<th>Materials Accepted</th>
<th>Recycling Rate</th>
<th>Services</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>BURRTEC SERVICES</strong></td>
<td>(866) 270-5370</td>
<td>C&amp;D debris: wood, metal, drywall, cardboard, rock, soil, gravel, asphalt, concrete</td>
<td>65%</td>
<td>Bin service</td>
</tr>
<tr>
<td><strong>REPUBLIC SERVICES</strong></td>
<td>(800) 299-4898</td>
<td>C&amp;D debris: wood, metal, drywall, cardboard, rock, soil, gravel, asphalt, concrete</td>
<td>65%</td>
<td>Bin service</td>
</tr>
<tr>
<td><strong>SOUTHERN CALIFORNIA DISPOSAL</strong></td>
<td>(310) 828-6444</td>
<td>C&amp;D debris: wood, metal, drywall, cardboard, rock, soil, gravel, asphalt, concrete</td>
<td>65%</td>
<td>Bin service or self-haul</td>
</tr>
<tr>
<td><strong>AMERICAN RECLAMATION</strong></td>
<td>(323) 245-0125</td>
<td>C&amp;D debris: wood, metal, drywall, cardboard, rock, soil, gravel, asphalt, concrete</td>
<td>65%</td>
<td>Bin service</td>
</tr>
<tr>
<td><strong>BRADLEY EAST TRANSFER STATION</strong></td>
<td>(818) 767-6180</td>
<td>Wood, tree trimmings, green waste</td>
<td>100%</td>
<td>Self-haul</td>
</tr>
<tr>
<td><strong>RECOLOGY LOS ANGELES formerly CROWN DISPOSAL</strong></td>
<td>(818) 767-6000</td>
<td>C&amp;D debris: wood, metal, drywall, cardboard, rock, soil, gravel, asphalt, concrete</td>
<td>89%</td>
<td>Bin service or self-haul</td>
</tr>
<tr>
<td><strong>EAST VALLEY DIVERSION - WM</strong></td>
<td>(818) 252-0019</td>
<td>C&amp;D debris: wood, metal, drywall, cardboard, rock, soil, gravel, asphalt, concrete</td>
<td>65%</td>
<td>Bin service or self-haul</td>
</tr>
<tr>
<td><strong>NORTH HILLS RECYCLING</strong></td>
<td>(818) 364-1278</td>
<td>Brush, logs, plants, tree trimmings, and clean lumber</td>
<td>100%</td>
<td>Bin service or self-haul</td>
</tr>
<tr>
<td><strong>RAMCO</strong></td>
<td>(818) 767-0700</td>
<td>Asphalt, concrete, gravel, rock, and sand. No dirt</td>
<td>100%</td>
<td>Self-haul or on-site crushing</td>
</tr>
<tr>
<td><strong>THE REUSE PEOPLE</strong></td>
<td>(818) 244-5635</td>
<td>Architectural salvage</td>
<td>100%</td>
<td>Self-haul</td>
</tr>
<tr>
<td><strong>VALLEY BASE MATERIALS</strong></td>
<td>(818) 767-3088</td>
<td>Asphalt, concrete, concrete block, gravel, rock</td>
<td>100%</td>
<td>Self-haul or on-site crushing</td>
</tr>
<tr>
<td><strong>VULCAN MATERIALS</strong></td>
<td>(818) 983-0146</td>
<td>Asphalt grindings, concrete</td>
<td>100%</td>
<td>Self-haul</td>
</tr>
</tbody>
</table>
## San Gabriel Valley and Southeast Los Angeles County

<table>
<thead>
<tr>
<th>Company Name &amp; Address</th>
<th>Phone Number</th>
<th>Business Hours</th>
<th>Materials Accepted</th>
<th>Recycling Rate</th>
<th>Services</th>
</tr>
</thead>
<tbody>
<tr>
<td>CONSTRUCTION &amp; DEMOLITION RECYCLING, INC.</td>
<td>(323) 357-6900</td>
<td>Mon - Fri, 4am - 4pm</td>
<td>C&amp;D debris: wood, metal, drywall, cardboard, rock, soil, gravel, asphalt, concrete</td>
<td>65%</td>
<td>Bin service or self-haul</td>
</tr>
<tr>
<td>9309 Rayo Avenue, South Gate</td>
<td>Sat, 4am - 4pm</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>DAN COPP CRUSHING</td>
<td>(800) DUMPSITE</td>
<td>Mon - Fri, 7am - 3:30pm</td>
<td>Asphalt, concrete, gravel, rock</td>
<td>100%</td>
<td>Self-haul or on-site crushing</td>
</tr>
<tr>
<td>12017 Greenstone Avenue, Santa Fe Springs</td>
<td>Sat, 7am - 1pm</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>GREENCYCLE</td>
<td>(562) 906-5223</td>
<td>Mon - Fri, 6:30am - 5pm</td>
<td>Wood, soil, and greenwaste including palm and yucca</td>
<td>100%</td>
<td>Self-haul</td>
</tr>
<tr>
<td>12815 East Imperial Highway, Santa Fe Springs</td>
<td>Sat, 7am - 5pm</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>LB CRUSHING</td>
<td>(951) 691-0625</td>
<td>Mon - Fri, 6:00am - 4pm</td>
<td>Asphalt, concrete</td>
<td>100%</td>
<td>Pick-up or portable crushing</td>
</tr>
<tr>
<td>13620 Live Oak Lane, Inverdale</td>
<td>Sat, 4am - 2:00pm</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>PASADENA ARCHITECTURAL SALVAGE</td>
<td>(626) 535-9655</td>
<td>Tue - Sat, 9am - 5pm</td>
<td>Architectural salvage</td>
<td>100%</td>
<td>Self-haul</td>
</tr>
<tr>
<td>2600 East Foothill Boulevard, Pasadena</td>
<td>Sun, 12pm - 5pm</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>PECK ROAD GRAVEL</td>
<td>(626) 574-1855</td>
<td>Mon - Fri, 6am - 4:30pm</td>
<td>Asphalt, concrete, brick</td>
<td>100%</td>
<td>Self-haul</td>
</tr>
<tr>
<td>128 East Live Oak Avenue, Monrovia</td>
<td>Sat, 6:00am-2:30pm</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SILVERLAKE ARCHITECTURAL SALVAGE</td>
<td>(626) 445-1092</td>
<td>Daily, 11am - 5pm</td>
<td>Architectural salvage</td>
<td>100%</td>
<td>Self-haul</td>
</tr>
<tr>
<td>169 Cook Street, Pasadena</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>RECYCLED WOOD PRODUCTS</td>
<td>(909) 868-6882</td>
<td>Mon - Fri, 7am - 5pm</td>
<td>Green waste, soil</td>
<td>100%</td>
<td>Bin service or self-haul</td>
</tr>
<tr>
<td>1313 East Phillips Boulevard, Pomona</td>
<td>Sat, 7am - 1pm</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

## City of Los Angeles and Central Los Angeles County

<table>
<thead>
<tr>
<th>Company Name &amp; Address</th>
<th>Phone Number</th>
<th>Business Hours</th>
<th>Materials Accepted</th>
<th>Recycling Rate</th>
<th>Services</th>
</tr>
</thead>
<tbody>
<tr>
<td>25TH STREET RECYCLING</td>
<td>(323) 583-7913</td>
<td>Mon - Fri, 6:30am - 11pm</td>
<td>Concrete, concrete block, asphalt</td>
<td>100%</td>
<td>Self-haul</td>
</tr>
<tr>
<td>2121 East 25th Street, Los Angeles</td>
<td>Sat, 6am - 6pm</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CALIFORNIA WASTE SERVICES</td>
<td>(800) 839-5550</td>
<td>Mon - Fri, 6am - 10pm</td>
<td>C&amp;D debris: wood, metal, drywall, cardboard, rock, soil, gravel, asphalt, concrete</td>
<td>65%</td>
<td>Bin service or self-haul</td>
</tr>
<tr>
<td>621 West 152nd Street, Gardena</td>
<td>Sat, 7am - 7pm</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CLEAN UP AMERICA</td>
<td>(323) 980-9930</td>
<td>Mon - Sat, 5:30am-4pm</td>
<td>C&amp;D debris: wood, metal, drywall, cardboard, rock, soil, gravel, asphalt, concrete</td>
<td>65%</td>
<td>Bin service or self-haul</td>
</tr>
<tr>
<td>2900 East Lugo Street, Los Angeles</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>DIRECT DISPOSAL</td>
<td>(323) 262-1604</td>
<td>Mon - Fri, 5am - 4pm</td>
<td>C&amp;D debris: wood, metal, drywall, cardboard, rock, soil, gravel, asphalt, concrete</td>
<td>65%</td>
<td>Bin service or self-haul</td>
</tr>
<tr>
<td>3720 Noakes Street, Los Angeles (East L. A.)</td>
<td>Sat, 6am - 11am</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>DOWNTOWN DIVERSION - WM</td>
<td>(213) 612-5005</td>
<td>Mon - Fri, 6am - 6pm</td>
<td>C&amp;D debris: wood, metal, drywall, cardboard, rock, soil, gravel, asphalt, concrete</td>
<td>65%</td>
<td>Bin service or self-haul</td>
</tr>
<tr>
<td>2424 East Olympic Boulevard, Bldg. 3, Los Angeles</td>
<td>Sat, 6am - 3pm</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Company Name &amp; Address</td>
<td>Phone Number</td>
<td>Business Hours</td>
<td>Materials Accepted</td>
<td>Recycling Rate</td>
<td>Services</td>
</tr>
<tr>
<td>------------------------</td>
<td>--------------</td>
<td>----------------</td>
<td>--------------------</td>
<td>---------------</td>
<td>----------</td>
</tr>
<tr>
<td>FREEWAY BUILDING MATERIALS</td>
<td>(323) 261-8904</td>
<td>Mon - Fri, 8am - 4pm, Sat, 8am - 3pm</td>
<td>Architectural salvage</td>
<td>100%</td>
<td>Self-haul</td>
</tr>
<tr>
<td>1124 South Boyle Avenue</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Los Angeles</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SOUTHERN CALIFORNIA ARCHITECTURAL SALVAGE</td>
<td>(213) 623-3119</td>
<td>Mon - Fri, 8am - 5pm, Sat - Sun, 9:30am - 3pm</td>
<td>Architectural salvage</td>
<td>100%</td>
<td>Self-haul</td>
</tr>
<tr>
<td>1600 South Santa Fe Avenue</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Los Angeles</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

| CITY OF LONG BEACH and SOUTHERN LOS ANGELES COUNTY |
| ALLIED WASTE FALCON | (562) 590-8531 | Mon - Fri, 8am - 5pm, Sat, 6am - 2pm | C&D debris: wood, metal, drywall, cardboard, rock, soil, gravel, asphalt, concrete | 65% | Bin service or self-haul |
| 3031 East "I" Street | | | | | |
| Wilmington | | | | | |
| HANSON AGGREGATES | (626) 856-6700 Option 1 | Mon - Fri, 7am - 3:30pm, Sat, 7am - 2pm | Asphalt, concrete | 100% | Self-haul |
| 6956 Cherry Avenue | | | | | |
| North Long Beach | | | | | |
| HANSON AGGREGATES | (626) 856-6700 Option 1 | Mon - Fri, 7am - 3:30pm | Asphalt, concrete | 100% | Self-haul |
| 2850 California Avenue | | | | | |
| South Long Beach | | | | | |

Updated August 25, 2015

The companies listed and the information presented are subject to change without notice and are based on the most readily available information. The companies listed are not endorsed or recommended by the County, nor is the list necessarily inclusive of all recycling companies in the region.

If you are a recycling company and you would like to be added to the database, or you have information to update the existing list, please call the Construction & Demolition Unit at (626) 458-3517.
BLANK SHEET
About AECOM

AECOM (NYSE: ACM) is a global provider of professional technical and management support services to a broad range of markets, including transportation, facilities, environmental, energy, water and government. With approximately 45,000 employees around the world, AECOM is a leader in all of the key markets that it serves. AECOM provides a blend of global reach, local knowledge, innovation, and collaborative technical excellence in delivering solutions that enhance and sustain the world’s built, natural, and social environments. A Fortune 500 company, AECOM serves clients in more than 100 countries and has annual revenue in excess of $6 billion.

More information on AECOM and its services can be found at www.aecom.com.