APPENDIX 10.9

HYDROLOGY STUDY

South Coast Water District -

Doheny Desalination Project
Local Hazard Conditions and Drainage Study

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1. Introduction

1.1 Introduction

Water supply at southern Orange County is mostly sourced from import water from northern California and Colorado River. With the continuous growth in the area and in California in general, the availability and the reliability of water supply from import sources increasingly become a water management concern in the area. In response to the water supply challenge, South Coast Water District (SCWD) is in the planning stages to develop an ocean desalination facility in southern California. The proposed Doheny Desalination Plant, located in Dana Point along the east bank of San Juan Creek at the north of Highway 1, could supply up to 15 million gallons of local drinking water a day by desalinating Pacific Ocean using reverse osmosis membrane treatment.

As a part of the Doheny Desalination Plant project development, SCWD has completed a wide range of studies and analysis to support the evaluation, planning, permitting and design processes. This report documents a Local Hazard Conditions and drainage study to assess the existing and future coastal and fluvial flood vulnerability at the project site, and to propose flood improvement options to protect the project site.

The Local Hazard Conditions and Drainage Study for the Doheny Desalination Project includes the following components:

- Coastal Analysis: A Local Hazard Conditions assessment evaluated the potential coastal flooding under the projected sea level rise scenarios. The assessment was conducted pursuant to the California Coastal Commission Sea Level Rise Policy Guidance (August, 2015).
- Fluvial Analysis: This analysis evaluated the hydraulic capacity of San Juan Creek, specifically the water surface elevations and levee/floodwall overtopping potential during 25-year and 100-year events, under existing and projected sea level rise scenarios. A hydrology analysis is prepared to define watersheds and to estimate stormwater flow to the project site. A hydraulic analysis is prepared to evaluate the capacity of the major stormwater conveyance system at the project site and the upstream watersheds. The analysis also included floodplain modelling to map flood inundation extent and depth. Based on the existing condition evaluation findings, the study evaluated four alternatives to assess its feasibility and relative performance to protect the project site from a 100-year flood, while not to increase flooding at the upstream watershed.

Section 2 of this report, as well as Appendix A and Appendix B documents the coastal analysis. Section 3 of the report documents the fluvial analysis. The analysis articulated the existing and future flood vulnerability due to the projected sea level rise, San Juan Creek overtopping, stormwater flow from the upstream watersheds, and the storm drain system capacity limitation. Improvement alternatives are evaluated, with a recommended site improvement concept to address potential project site flooding.

1.2 Existing Condition at the Project Site

The project site is located in Dana Point, California. As shown in Figure 1.1, the project site is adjacent to the Pacific Ocean, along the east bank of San Juan Creek between Highway 1 and Stonehill Drive. This section of San Juan Creek has been modified by the Flood Division of the Orange County Public Works, with flood improvements including a combination of concrete banks, levees, and floodwall.

Currently the project site is mostly leased to various tenants for outdoor storages. The site is relatively flat, with mostly dirt and gravel surfaces. Federal Emergency Management Agency (FEMA) Flood Insurance Rated Map (FIRM) shows the project site is located in flood Zone AO, with an average of one foot inundation depth under a 1% annual chance (100-Year) flood.

There are two major underground storm drain systems conveying runoff from the upper watershed. Owned by Orange County Flood Control District, the L01S02 system is a double concrete box culvert, runs in parallel along the landside of the San Juan Creek levee and outfalls to San Juan Creek. The L01S02 system collects stormwater runoff from the upper watersheds, bounded by Stonehill Drive at the north and Highway 1 at the south, and extends to the east of Highway 5.

At the southern end of the project site, a 54-inch diameter (54”) Reinforced Concrete Pipe (RCP) runs in parallel with Highway 1. The pipe collects runoff from the project site and the watersheds at the east of the railroad tracks, and discharges to San Juan Creek. Local stormwater drainage within the project site is limited. A vegetated swale (South Drainage Swale) located at the northern end of the project site collects local runoff and discharges to the L01S02 system. The South Drainage Swale and the 54” RCP have gates to prevent backflow from San Juan Creek.

As a part of the study, a field survey was completed to verify the dimensions, locations, and elevations of the major storm drain system. The survey information is incorporated in the analysis to model the system hydraulic performance.

1.3 Study Assumptions

This study and the analysis is based on a range of assumptions to estimate the coastal and fluvial impacts to the project site. The following is a summary of the key assumptions in this study.

- The Doheny Desalination Plant will be in service in Year 2020. Hence, Year 2020 is defined as the existing condition in this study.
- The expected project life of this project, for the purpose of the projected sea level rise estimate, is 50 years. Hence, the projected sea level rise planning horizon is in Year 2070.
- The flood protection design target for the plant is to provide 100-year (1% annual chance) flood protection.
- Under the no project condition, the levee and floodwall system along San Juan Creek, between Stonehill Drive and Highway 1, will not be improved in Year 2020.
- The watershed landuse will not have significant change over the project life.

Section 2 and Section 3 discuss these and other study assumptions in detail.

2. Coastal Analysis

2.1 Introduction

This study was conducted pursuant to the California Coastal Commission Sea Level Rise Policy Guidance (August, 2015) and the following is a summary of the Local Hazard Conditions assessment based on the process outlined in Appendix B of the Sea Level Rise Policy Guidance.

The study was completed by Michael Baker International and presented in the February 2017 report titled Coastal Hazards Analysis for the Doheny Desalination Project by Scott A. Jenkins (Appendix B). Additional supplementary information on the historical shoreline evolution is provided in the GHD May 1, 2017 memo Doheny Desalination Plant Historical Shoreline Assessment (Appendix A). The mapping of flood impacts due to the modelled total water levels and tsunami events are provided in the summary below. Additional flood modelling available as part of The Coastal Storm Modeling System (CoSMoS) study prepared by the United States Geological Survey (USGS) is also presented for comparison. This includes predicted shoreline erosion due to sea level rise.

For this study, the project life expectancy was defined as 50 years. This was determined in consultation with the South Coast Water District (SCWD). The project life expectancy is an important design criterion to estimate the anticipated range of sea level rise. The guidance for project life expectancy ranges from 25 years for amenity structures, to 100 years for critical infrastructure. An additional prediction of water levels for a critical infrastructure planning horizon was also determined.

Assuming the project will be constructed in Year 2020, the 50 year project life expectancy sets the project planning horizon to Year 2070. A critical infrastructure planning horizon of Year 2100 was used as the National Research Council (NRC) sea level projections (NRC, 2012), which form the basis of the sea level projections along the California Shoreline, do not extend beyond Year 2100. Significant uncertainties and variables render reliable sea level projections beyond Year 2100 difficult.

Section 2.2 provides a brief summary of the Local Hazard Conditions assessment findings. For additional detail refer to GHD May 1, 2017 memo Doheny Desalination Plant Historical Shoreline Assessment (Appendix A), and, Michael Baker International (2017) Coastal Hazards Analysis for the Doheny Desalination Project by Scott A. Jenkins (Appendix B).

2.2 Summary of Findings

2.2.1 Sea Level Rise Projection

Sea level rise projections were based on the California Coastal Commission best fit equations (CCC, 2015 Appendix B) for a 50 year planning horizon at Year 2070, and critical infrastructure planning horizon at Year 2100. Sea level rise projections for the lower and upper ranges are provided in Table 2.1.

Table 2.1 Sea Level Rise Projections

Planning Time Period (Year)		Best Fit Equation	
		Lower Range (feet)	Upper Range (feet)
50 year planning horizon	2070	0.7	3.2
Critical Infrastructure Planning Horizon	2100	1.4	5.5

2.2.2 Tidal Range and Future Inundation

Tidal datums were based on water level measurements from the Scripps Pier tide gage station, NOAA #9410230 for the 1983 – 2001 tidal epoch. Tidal datums and future datums based on lower and upper sea level rise projections are provided in Table 2.2.

Table 2.2 Tidal Datums at Scripps Pier NOAA Tide Gage Station 1983-2001 with Projected Sea Level Rise

Datum	Elevation (ft NAVD)	SLR 2070 lower range (ft NAVD)	SLR 2070 upper range (ft NAVD)	SLR 2100 lower range (ft NAVD)	SLR 2100 upper range (ft NAVD)
Highest Observed	7.47	8.17	10.68	8.84	12.97
MHHW (Mean Higher High Water)	5.13	5.83	8.34	6.5	10.63
MHW (Mean High Water)	4.41	5.11	7.62	5.78	9.91
MTL (Mean Tide Level)	2.56	3.26	5.77	3.93	8.06
MSL (Mean Sea Level)	2.54	3.24	5.75	3.91	8.04
MLW (Mean Low Water)	0.71	1.41	3.92	2.08	6.21
MLLW (Mean Lower Low Water)	-0.19	0.51	3.02	1.18	5.31
Lowest Observed	-3.06	-2.36	0.15	-1.69	2.44

2.2.3 Potential Still Water Level Changes from Surge, El Nino and Pacific Decadal Oscillation

Determining the Local Hazard Conditions following the California Coastal Commission Sea Level Rise Policy Guidance (2015) document includes determining potential still water level changes due to other processes in addition to sea level rise. These processes include surge, El Niño events, and the Pacific Decadal Oscillation (PDO).

Water level recurrence statistics were derived from the record of ocean water levels at the NOAA Scripps Pier tide gage. A stage frequency curve or hydroperiod function was developed from the 1924 to 2016 time series. The hydroperiod function presented in Appendix B includes the effects of El Nino, the Pacific Decadal Oscillation (PDO), and surge on the still water level.

2.2.4 Beach Erosion

An assessment of long term beach erosion was conducted using historical aerial photographs (Appendix A) and historical beach surveys (Appendix B) at Doheny State beach. Overall, Doheny State Beach appears to be in an equilibrium condition based on the historical aerial photographs used for the assessment. Sediment input from the San Juan Creek and potential beach nourishment projects in the past have maintained a relatively stable shoreline. A historical erosion rate could not be determined to apply to future adjustments due to sea level rise.

Michael Baker International (2017) utilized Bruun's Rule (as described in Appendix B) to account for erosion impacts of sea level rise. The Coastal Evolution Model included retreat of the shoreline based on Bruun's Rule. This assumes retreat of the shoreline with no change to the shape of the profile. Using this approach, the shoreline retreat will have no impact on the total runup elevations.

Unlike shoreline retreat due to Bruun's Rule, seasonal fluctuations in beach morphology do impact the total water level computation. All total water level estimates were based on both a typical accreted profile and typical eroded profile to account for the impacts to wave setup and wave runup (Appendix B).

Preliminary estimates of shoreline erosion due to sea level rise were available from The Coastal Storm Modeling System (CoSMoS) for the study area (Erikson et al., 2017). CoSMoS is a dynamic modeling approach that has been developed by the United States Geological Survey (USGS) to allow for more detailed predictions of coastal flooding due to both future sea level rise and storms integrated with long-term coastal evolution over large geographic areas. CoSMoS uses wind and pressure from global climate models to project coastal storms under changing climatic conditions during the 21st century.

CoSMoS 3.0 preliminary results were available for sea level rise scenarios of 0 m to 2 m at increments of 0.50 m and for 5 m for Southern California. Shoreline retreat for these scenarios is shown in Figure 2.1. Note that the shoreline was defined in CoSMoS 3.0 as the mean high water (MHW) which will occur seaward of the beach berm. It is also possible that the shoreline was defined during conditions when the beach was much wider. It is clear that the proposed plant is located far inland from the predicted eroded shoreline.

2.2.5 Waves, Wave Runup and Flooding Conditions

Future flooding levels were determined by Michael Baker International (2017) as outlined in Appendix B of CCC (2015). The Federal Emergency Management Agency (FEMA) standards for flooding frequency were followed by determining flood levels for the 100 year event. Extremal total water levels (TWL's) were based on the occurrence of extreme waves concurrent with extreme ocean water levels. Extremal total water levels are the sum of the total runup and the still water level. The total runup consists of wave setup, dynamic wave setup and wave runup. A joint probability analysis was used to determine the occurrence of extremal wave heights concurrent with extreme ocean water levels.

Extremal total water levels were determined for the low and high range sea level rise projections for 2070 and 2100. Each wave and sea level scenario was modelled for both accreted and eroded beach profiles based on seasonal fluctuations at the site. Total water levels were always higher for

the accreted beaches since they were steeper and caused greater wave setup and runup. The potential future flooding extent for the different scenarios are shown in Figures 2.2 and 2.3 based on topography generated from USACE 2014 LiDAR data. Note that the flood extent based on the extremal total water levels is a worst case approach since it includes wave runup. Wave runup is a short term process and therefore may not result in flooding to the full extent of the runup elevation. It can be seen that only the extremal total water level for the high range sea level rise for 2100 may reach a very small portion at the very seaward tip of the project site where there is no proposed infrastructure. The potential for flooded well heads and overtopping rates for each scenario are summarized in Table ES-1 in Appendix B.

Alternative flood extent predictions with sea level rise were available from CoSMoS 3.0 (Erikson et al., 2017). Flooding extents at the study site for the 0.5 m and 2 m sea level rise scenarios for a 100 year storm event are presented in Figure 2.4. The flooding extents were much less than those based on the estimates of extremal total water levels provided by Michael Baker International (2017) because the CoSMoS flood elevations do not include wave runup.

2.2.6 Extreme Flooding Events Due to Tsunami

Tsunamic induced erosion, runup, and inundation were analyzed for the Doheny State Beach profiles for present and future sea levels, with low and high range sea level rise projections (Appendix B). The tsunami event scenario was based on a 2 m high solitary wave that could be anticipated for a catastrophic tsunami event from a major landside on the east side of San Clemente Island. The tsunami reaches 6 m in height due to shoaling, before breaking on the shoreline. The potential for flooded well heads and overtopping rates for each scenario are summarized in Table ES-2 in Appendix B. Flooding extents of the low and high range 2070 and 2100 sea level projection scenarios are illustrated in Figure 2.5 and Figure 2.6. It can be seen that only the tsunami for the 2100 high range sea level rise scenario may reach a very small portion at the seaward tip of the property where there is no proposed infrastructure.

Additional tsunamic inundation predictions published on June 1, 2009 were created through a joint effort by the State of California Office of Emergency Services (Cal OES), the California Geologic Survey, the University of Southern California Tsunami Research Center, and NOAA. The map of the tsunami prediction for the study area is provided in Figure 2.7. While the mapping resolution is low, the flooding extent appears to be similar to those provided in Figure 2.6.

The assessment shows that the projected sea level rise scenarios considered in this study does not pose significant flood risk to the project site.

3. Fluvial Analysis

3.1 Introduction

To quantify drainage and flooding conditions at the project site, a detail hydrologic and hydraulic analysis was performed. This analysis includes four primary components, as listed:

- San Juan Creek Hydraulic Analysis: This analysis was performed in order to estimate the water surface elevations in San Juan Creek during the design events, and to determine if the creek flow may overtop the levees;
- Watershed Analysis: This analysis was performed in order to quantify the flows entering the existing stormwater systems at the project site, from watersheds adjacent to and upstream from the project site;
- Floodplain Analysis: This analysis was performed to determine the available capacity of the existing stormwater system at the project site, and to determine floodplain inundation at and upstream of the project site under the existing condition; and
- Drainage Improvements: This analysis was performed to assess potential drainage improvement options for the project site.

Each of these analyses are discussed in further detail in this section. It should be noted that unless otherwise stated, all elevation data is reported in North American Vertical Datum of 1988 (NAVD 88).

3.2 San Juan Creek Hydraulic Analysis

3.2.1 Data Source

To analyze the hydraulics of San Juan Creek, GHD obtained a one-dimensional, steady-state hydraulic model of the creek, built in HEC-RAS hydraulic modelling software from Orange County. The model was originally constructed by PACE Engineering in the study Baseline Floodplain Hydraulics for San Juan Creek (PACE 2010). All elevations referenced in the model are in National Geodetic Vertical Datum of 1929 (NGVD 29).

In addition to the hydraulic model noted above, GHD also obtained San Juan Creek hydrograph information that was generated from the San Juan Creek Watershed Hydrology Study (PACE 2008). As will be noted in further detail in subsequent sections, the hydrographs obtained from the PACE analysis was for the 25-year and 100-year Expected Value storm events.

3.2.2 Boundary Condition at Pacific Ocean

As noted above, the tidal boundary condition at the Pacific Ocean could have backwater effect on the upstream water surface elevations along San Juan Creek. For this analysis, three separate tidal elevation scenarios were used to determine water surface profiles and its impacts due to the project sea level rise. The first tidal elevation scenario is the Mean Higher High Water (MHHW) elevation, which is typically applied in FEMA floodplain analysis. The other two tidal elevation scenarios are the Year 2070 projected low and high sea level rise elevations. All three tidal elevation scenarios,

based on the coastal analysis documented in Section 2 and Appendix B, are summarized in Table 3.1.

Table 3.1 Tidal Boundary Elevations Used in HEC-RAS Model

Pacific Ocean Tidal Boundary Condition	Tidal Elevation (NGVD 29)	Tidal Elevation (NAVD 88)
Mean Higher High Water (MHHW)	1.87	4.13
2070 Low Sea Level Rise Projection under MHHW	3.58	5.84
2070 High Sea Level Rise Projection under MHHW	6.08	8.34

It should be noted that to be used in the HEC-RAS model, these elevations had to be converted from NAVD 88 to NGVD 29 vertical datum, which has the elevation difference of -2.26 ft.

3.2.3 Analysis Methods and Assumptions

To compute water surface profiles, and ultimately create a stage-flow time series for each of the existing outfalls along San Juan Creek, the steady state HEC-RAS model was run at increments of approximately every 3 hours using the hydrographs referenced in Section 3.1.1. Estimated creek flow in each time increment is input to the model, to estimate the creek water surface profiles under the three tidal elevation scenarios listed in Table 3.1.

The HEC-RAS model provided by Orange County covers approximately 5 miles of San Juan Creek upstream from the Pacific Ocean. To simplify the hydrograph inputs and the modelling analysis, the model was truncated at approximately 3,500 feet upstream from Stonehill Drive (Creek Station 8610). At this location, the hydrograph flows estimated at Node 146 in the San Juan Creek Watershed Hydrology Study (PACE 2008) were applied as flow input to the HEC-RAS model. A second flow input was applied further downstream at creek station 3955, which corresponds to Node 147 of the hydrology study (PACE 2008). The 25-year and the 100-year design flow are 26,116 cfs and 45,847 cfs, respectively. As a comparison, the 100-year design flow in FEMA Flood Insurance Study (FIS) is 42,000 cfs (FEMA 2009).

3.2.4 Analysis Results

The HEC-RAS model was run using the 25-year and 100-year Expected Value hydrographs. The creek hydraulic analysis based on the three tidal elevation scenarios shows that the variations in downstream tidal elevation under the projected sea level rise conditions do not have an effect on the upstream creek water surface elevations. This appears to be due to the fact that the Highway 1 northbound and southbound bridges act as a significant creek flow constriction at the mouth of San Juan Creek. The backwater effects due to the bridges governs San Juan Creek hydraulics. In addition, in the HEC-RAS model, the channel flowline elevation at the mouth is 4.85 ft NGVD 29, which is above both the MHHW and Year 2070 low sea level rise tidal elevation projection. Therefore, it was determined that the various tidal elevations had no effect on the upstream creek water surface profile, and the subsequent results presented are valid under all three tidal elevation scenarios.

The maximum water surface profile for the 25-year storm event is shown in Figure 3.1, which occurs at approximately Hour 17.25 of the storm event. The peak of the 25-year flow is conveyed in the channel without overtopping the creek on either the left or right banks. The hydrograph used for the 25-year flow in San Juan Creek is shown in Figure 3.2. To compare the peak flow timing between the creek and the project site watershed, the hydrograph that was developed for the outfall of the Capistrano Beach Storm Drain (LO1SO2) is also shown in the plot. The development of the LO1SO2 hydrographs is discussed in further detail in Section 3.2.

The peak of the existing stormwater system adjacent to the project site occurs at approximately Hour 16.4 of the 24-hour design storm event. The peak flow in San Juan Creek occurs at Hour 17.25 of the 24-hour storm event, approximately 1-hour later.

The peak water surface profile for the 100-year storm event is shown in Figure 3.3. During the 100-year storm event, the peak creek flow overtops both channel banks for approximately 350 feet north of the Highway 1 northbound bridge, at the southern end of the project site. The hydrograph for the 100-year flow in San Juan Creek is shown in Figure 3.4. Similar to the 25-year storm event, the peak flow in San Juan Creek occurs approximately 1-hour after the peak of the Capistrano Beach Storm Drain.

3.2.5 Findings

The hydraulic analysis of the San Juan Creek Channel resulted in two key findings:

- The downstream tidal elevations modelled in this analysis do not have an effect on the upstream water surface profile.
- Based on the OCFCD HEC-RAS model, San Juan Creek overtops both the east and west banks during the 100-year expected value storm events. Adjacent to the project site, San Juan Creek overtops the east bank floodwall at the upstream of Highway 1. The overtopping contributes to potential flooding at the project site.

3.3 Watershed Analysis

A watershed analysis was performed to estimate the flows entering the existing storm drain infrastructure at the project site. The purpose of this analysis is to determine if there is available capacity in the existing stormwater system, and to evaluate potential impacts to flooding as a result of stormwater runoff generated from the upstream watershed. The following sections describe the existing conditions around the project site, and the watershed analysis.

3.3.1 Existing Stormwater Infrastructure

In general, drainage in the project area and surrounding watersheds is conveyed from east to west, towards San Juan Creek. To provide stormwater relief for urban developments, railroad, and roadways located in the watershed, a system of large stormwater drainage conduits has been constructed as the primary drainage route for stormwater generated in the watershed. These drainage features are shown in Figure 3.5. Each of these components is discussed in further detail below. It should be noted that each of these main features described below is supported by a network of smaller stormwater pipes and drainage inlets to collect local runoff. However, this analysis only

includes the major drainage features as listed to evaluate stormwater routing at the watershed scale, as well as drainage performance at the project site.

- **LO1**

  The San Juan Creek Channel. This creek acts as the primary receiving waters for runoff generated within the San Juan Creek watershed, including all stormwater generated at the project site.

- **LO1SO2**

  The Capistrano Beach Storm Drainage. This facility primarily collects stormwater generated east of the San Diego Freeway (I-5) and conveys flows west towards San Juan Creek. This facility also collects stormwater conveyed from facility LO1SO3, and both the northern and southern drainage swales (described below). Moving from east to west, this facility is composed of an 8-foot diameter (8') RCP pipe which connects to an 8' x 8' concrete box, a 7.5' x 12' railroad crossing, open concrete rectangular channel, then a double 11' x 11.5' box culvert and outfalls to San Juan Creek. The San Juan Creek outfall has no backflow prevention.

- **LO1SO3**

  LO1SO3 collects stormwater generated from the northern portion of the local watershed located within Stonehill Drive and the San Diego Freeway (I-5). It also collects runoff generated from the east side of the San Diego freeway though a box culvert, located at the upstream of the pipe terminus as shown in Figure 3.5. Moving from north to south, this facility is composed of a 6.5' RCP pipe, an 8' RCP pipe, a 6.5' x 13' concrete box railroad crossing, and an 8' RCP pipe that discharge to LO1SO2.

- **North Drainage Swale**

  This swale is located directly north of facility LO1SO2, and serves to drain the surrounding properties in the area. The outlet of the swale has a 3' RCP culvert that connects to facility LO1SO2, with a flap gate to prevent backwater flow from San Juan Creek.

- **South Drainage Swale**

  This swale is located at the northern end of the project site, and drains runoff generated in areas both north and south of the swale. The eastern portion of the swale shown on Figure 3.5 is connected to a storm drain that extends eastwards toward the developments on the east side of the railroad track. The outlet of the swale is connected to a 4' RCP pipe which connects with facility LO1SO2. The connection has a flap gate.

- **54" RCP Pipe**

  This pipe collects stormwater generated in the southern portion of the watershed bounded by the San Diego Freeway (I-5) and Highway 1. The pipe discharges to San Juan Creek upstream of Highway 1 bridges. The outfall has a flap gate.

3.3.2 Design Storm and Analysis Method

The following references establish the watershed analysis and design criteria, including design storms, for the project site:

- Orange County Hydrology Manual (1986);
- Orange County Hydrology Manual, Addendum No. 1 (1996); and
- Orange County Local Drainage Manual (1996).

From these documents, the following design storms criteria have been established, as shown in Table 3.2.

Table 3.2 Summary of Design Storms and Method

Criterion	Value	Source
Hydrology Method	Unit Hydrograph	Orange County Hydrology Manual
Design Storm for Pipes with Sumps or no Parallel Drainage Ways	25-year Expected Value	Orange County Local Drainage Manual
Design Storm for Habitable Structures	100-year Expected Value	Orange County Local Drainage Manual

Based on the Hydrology Manual (the manual), Addendum No. 1, the Expected Value hydrology was used in this analysis, as the focus of this analysis is to determine floodplain impacts and flood protection level of services.

The Orange County Hydrology Manual Unit Hydrograph method was used for contributing watershed hydrology analysis, since the total area contributing runoff to the local storm drain infrastructure is above 640 acres. The Unit Hydrograph method is based on developing an effective rainfall hyetograph, which accounts for initial and infiltration losses using the Curve Number method. The effective rainfall hyetograph is then transformed to a runoff hydrograph using the watershed Time of Concentration and S-Curves for the Orange County watersheds.

In addition to the Orange County Hydrology Method, the hydrologic model within PCSWMM, the hydraulic model used for this analysis, was used to develop runoff hydrographs. This was done to provide a second source of hydrologic analysis. To remain consistent with the Orange County Method, the hydrologic loss model used within PCWMM was the curve number method and the parameters developed for the Orange County Method were used in the PCSWMM model.

3.3.3 Watershed Descriptions

The initial step in the analysis was to estimate the runoff reaching the primary stormwater infrastructure. Watersheds were delineated using an iterative approach, which included checking surface elevation data from NOAA Coastal LiDAR, storm drain infrastructure maps from the Orange

County Flood Control District (OCFCD), and the City of Dana Point Master Plan of Drainage (1998). Working east from the project site, a total of 18 watersheds were identified, which are shown in Figure 3.6. Watersheds 1A and 1B represent the project site watersheds.

3.3.4 Topography

Watersheds located within the area bound by Stonehill Drive, the San Diego Freeway, and Highway 1 (Watersheds 1A to 13) have typically mild average slopes, generally on the order of 5% to 40%. The main source of topographic relief in these watersheds is the fill prism or natural grades leading to the surrounding roadways. Elevations in this area range between 20' and 150'. Watersheds located south and east of this area (Watersheds 14 to 17) have higher relief, with average slopes in these watersheds ranging from 50% to 90%. Elevations in these watersheds range between 150' and 900'.

3.3.5 Soil Types

Soils information was collected from the NRCS Web Soil Server. Hydrologic Soils Groups in the watersheds range between groups B and D. All of the group B soils are located in the watersheds nearest San Juan Creek. At the east of the project site, the remainder of the watersheds are composed of group C and D soils, exhibit poor infiltration potential.

3.3.6 Existing Land Use

The three primary land use types in the watersheds shown in Figure 3.6 are residential, commercial, and open space. Watersheds 1A to 13 are primarily composed of residential and commercial use types. Watersheds 14 to 17 are primarily residential and open space use types.

3.3.7 Existing Condition Watersheds Summary

The physical characteristics of each watershed shown in Figure 3.6 are summarized in Table 3.3 below:

Table 3.3 Summary of Watershed

Watershed	Area (acres)	Average Slope (%)	Land Use Type(s)	Soil Type(s)
1A	9.7	11%	Commercial	B, C
1B	3.4	8%	Commercial	B, C
2	6.9	7%	Commercial	B, C
3	6.1	17%	Commercial	B, C
4	7.3	22%	Commercial	B, C
5	5.6	45%	Commercial	B, C

Watershed	Area (acres)	Average Slope (%)	Land Use Type(s)	Soil Type(s)
6	12.2	16%	Commercial	C
7	16	5%	Commercial	C
8	2	6%	Commercial	C
9	46.4	39%	Commercial/Open Space	C, D
10	16.5	26%	Commercial/Residential	C, D
11	30.3	11%	Commercial/Residential	C, D
12	13.1	6%	Commercial/Residential	C
13	16.2	54%	Commercial/Residential	C, D
14	29.9	56%	Residential/Open Space	C, D
15	765.5	90%	Residential/Open Space	C, D
16	108.9	57%	Residential/Open Space	C, D
17	46.6	88%	Residential/Open Space	C, D

3.3.8 Orange County Method Hydrologic Analysis

After the watersheds were delineated the Orange County Hydrology Method was used to develop the hydrologic parameters necessary to construct outflow hydrographs. The derivation of these parameters is presented below.

There are three primary steps in the Orange County Method: 1) Develop synthetic unit hydrograph; 2) Develop recurrence interval storm pattern; and 3) Develop runoff hydrograph. To ultimately produce the runoff hydrograph, the following parameters were derived. It should be noted that there are several steps in the manual where these parameters are used to develop intermediate information which leads to producing a runoff hydrograph. The following discussion is limited to the derivation of the base parameters, which are the components unique to this analysis.

3.3.9 Design Storm

To develop the 10-, 25-, and 100-year design storms, the Orange County Point Precipitation Data from Table B.2 of the hydrology manual was used to create logarithmic regression equations. Using these equations, precipitation depths were calculated for unit time periods of 5-minutes. These depths

were then rearranged using the design storm distributions shown in Figures B-5a, b, and c of the hydrology manual. Due to the generally small size of the watersheds in this analysis, no depth area adjustments were performed to modify the design storms in this analysis.

3.3.10 Percent Impervious

Percent impervious was estimated by measuring the area of individual land use type in each watershed and multiplying each use area by a factor from the hydrology manual (Figure C-4). For this analysis, the recommended values for average conditions were assumed for each use type. Residential areas were assumed to have a density of 5 to 7 dwellings/acre, and open spaces were assumed to have no impervious cover. The resulting percent impervious cover for each watershed can be found in Table 3.4.

3.3.11 Curve Number

Composite curve numbers for antecedent moisture condition (AMC) II were generated for each watershed. This was done by using the total watershed area, and separating out the pervious from impervious area using the percent impervious values. As per the manual's recommendation, the impervious surfaces were assigned a curve number of 98. The remaining pervious area within a watershed was assigned a curve number based on the hydrologic soil group and the ground cover type. All open space areas in the watersheds was assigned values based on Open Brush cover, all residential areas were assigned values based on Turf cover, and all commercial areas were assigned values based on Commercial Landscaping cover. As per the manual recommendation, the quality of the cover was assumed to be 'good'. The resulting AMC II composite curve numbers for each watershed can be found in Table 3.4.

As per the manual, AMC II curve numbers are sufficient for watershed loss analysis for the 25-year storm event, however, AMC III curve number values are required for the 100-year event. To derive these values, the AMC II curve numbers were adjusted per the relationship presented in Table C.1 of the hydrology manual. The AMC II composite curve numbers can be found in Table 3.4.

3.3.12 Time of Concentration

As per the manual, the total time of concentration was calculated in two components. The first component is the sheet flow time, which was estimated using the nomograph from page D-4 of the manual. Flow lengths were measured in GIS from the most distant point in the watershed to the beginning of channelized flow, typically either gutter or pipe flow. The sheet flow lengths is limited to 1,000 feet. Elevation differences within each watershed were determined using contour surfaces generated using either NOAA Coastal LiDAR, or USGS elevation data.

The second component is the channelized flow time, which was calculated along the remaining length beginning at the end of sheet flow and ending at the watershed outlet. To be conservative, pipes and gutters for these components were assumed to be flowing full. The resulting total time of concentration for each watershed can be found in Table 3.4.

3.3.13 Lag Time

The lag time for each watershed was determined using the relationship from the manual, which states that the lag time should be equal to 80% of the watershed time of concentration. The lag time values for each watershed can be found in Table 3.4.

3.3.14 S-Curves

The watershed lag time was used to develop individual unit hydrographs based on locally produced S-Curves. These curves can be found in the hydrology manual, Figures E-3a, b, c, and d. For this analysis, a unit time period of 5-minutes was used to match the design storm patterns, and the Valley Developed S-curve (Figure E-3a) was used.

3.3.15 Orange County Watershed Hydrology Summary

The hydrologic parameters presented in the sections above are summarized in Table 3.4 below.

Table 3.4 Orange County Method Watershed Parameters

Watershed	Time of Concentration (min)	Lag Time (min)	Percent Impervious	Curve Number (AMC II)	Curve Number (AMC III)
1A	11.7	9.4	48%	81.3	94.5
1B	6.5	5.2	72%	89.7	97.9
2	7	5.6	64%	86.3	96.7
3	8	6.4	41%	78.4	93.0
4	24.3	19.5	32%	77.2	92.3
5	17.1	13.6	35%	79.0	93.3
6	10.2	8.2	85%	93.9	99.0
7	11.2	8.9	90%	95.1	99.2
8	25.6	20.4	0%	75.0	90.9
9	17.6	14.1	65%	90.1	98.0
10	19.2	15.4	69%	90.5	98.2
11	8	6.4	90%	95.2	99.2
12	12.9	10.3	90%	95.1	99.2
13	9.5	7.6	52%	88.8	97.6
14	10.7	8.6	38%	85.9	96.6
15	30.5	24.4	26%	84.8	96.2
16	22	17.6	9%	82.4	95.1

Watershed	Time of Concentration (min)	Lag Time (min)	Percent Impervious	Curve Number (AMC II)	Curve Number (AMC III)
17	14.1	11.3	58%	90.7	98.2

3.3.16 Proposed Development Watersheds

The hydrologic parameters for the developed conditions of Watersheds 1A and 1B were estimated using the same methodology as performed for the existing conditions watersheds above. To estimate these parameters, it was assumed that the entire Watersheds 1A and 1B would be converted to commercial use. The resulting parameters for the developed conditions of watersheds 1A and 1B are shown in Table 3.5.

Table 3.5 Hydrologic Parameters for Developed Project Site Watersheds

Watershed	Time of Concentration (min)	Lag Time (min)	Percent Impervious	Curve Number (AMC II)	Curve Number (AMC III)
1A – Developed	10.7	8.6	90%	95.5	99.3
1B - Developed	6.0	4.8	90%	96.3	99.4

3.3.17 Runoff Hydrograph Summary

The runoff hydrographs for each watershed were produced using the Orange County Hydrology Manual Method and the PCSWMM curve number hydrology method as noted above. Table 3.6 summarized the peak flow under 25-year and 100-year 24-hour storm events.

Table 3.6 Peak Flow Summary for Project Watersheds

Watershed	Peak Flow (cfs)	
	25-year Expected Value	100-year Expected Value
1A	12.2	16.4
1B	8.2	10.3
2	14.4	18.4
3	8.6	11.9
4	5.0	7.0
5	5.0	7.0
6	24.5	30.0
7	33.5	40.7
8	1.2	1.7
9	63.2	79.8

Watershed	Peak Flow (cfs)	
	25-year Expected Value	100-year Expected Value
10	22.6	28.4
11	79.9	96.6
12	25.9	31.4
13	28.4	36.2
14	42.4	55.5
15	634.8	834.8
16	96.8	129.1
17	66.0	84.8
1A Developed	20.4	24.7
1B Developed	10.4	12.5

3.4 Floodplain Analysis

To perform the floodplain analysis, a coupled 1D/2D routing model was constructed in PCSWMM. The following sections describe how the model was constructed, and present the results of the existing conditions model results.

The hydraulic routing for the runoff generated in each watershed was modelled using PCSWMM software by CHI. PCSWMM is a coupled hydrologic/hydraulic model that uses the base EPA SWMM program as the hydrology and one-dimensional flow routing platform. In addition to the base SWMM code, PCSWMM includes a proprietary two-dimensional flow engine, which can simulate unsteady-state flow routing across a surface.

3.4.1 Existing Conditions Model Geometry

The geometric information, including conduit inverts, dimensions, and swale cross-sectional geometries, was based on the project topographic survey prepared by MNS Engineers and as-built information for the major drainage structures provided by Orange County. The survey data covered areas bound between Stonehill Drive to the north, the existing railroad to the east, San Juan Creek to the west, and Highway 1 to the south. The survey data collected did not include information which would require confined space entry into the storm drain infrastructure. For areas outside of the survey limit, including portions of the 54" RCP pipe, LO1SO2, and LO1SO3 located east of the railroad track, existing as built information for each component was used to obtain geometric information.

It should be noted that the vertical datum varied between the project survey and the as-built information. All elevation data was converted to NAVD 88 datum.

3.4.2 **Manning's Roughness**

Table 3.7 listed the Manning's roughness coefficients used in the hydraulics model.

Table 3.7 **Manning's Roughness Coefficients**

Material	Value
Concrete Pipe	0.013
HDPE Pipe	0.011
Grassed Swales	0.035
Overland Flow Roughness[1]	0.05

[1]Used for 2D modelling surface.

3.4.3 Hydrologic Loading

The hydrographs generated using the Orange County Hydrology Manual Method and PCWMM were applied to receiving nodes in the hydraulic model. Table 3.8 below lists each watershed, which model junction the flows were applied to, and the receiving stormwater structure.

Table 3.8 Hydrograph Loading in PCSWMM Routing Model

Watershed	Model Node Applied	Receiving Infrastructure
1A	J13	54" RCP
1B	J20	South Drainage Swale
2	J19	South Drainage Swale
3	J24	North Drainage Swale
4	OF-4	San Juan Creek
5	OF-3	San Juan Creek
6	J15	54" RCP
7	J21	South Drainage Swale
8	J21	South Drainage Swale
9	J9	LO1SO2
10	J33	LO1SO3
11	J15	54" RCP
12	J10	LO1SO2
13	J11	LO1SO2
14	J15	54" RCP
15	J12	LO1SO2
16	J34	LO1SO3
17	J34	LO1SO3

3.4.4 Two-Dimensional Surface

A NOAA Coastal LiDAR digital elevation model was used as the base topographic surface data for two-dimensional modelling. The extents of the surface covered the San Juan Creek levee to the west, the San Diego Freeway to the east, Highway 1 to the south, and Stonehill Drive to the north. It should be noted that for this analysis, the model was constructed using LiDAR data, which does not include features such as buildings or other structures that would resist inundation.

3.4.5 Outfall Boundary Conditions

Outfall boundary conditions were determined using the San Juan Creek HEC-RAS analysis outlined in Section 3.1. Boundary conditions were produced for both the 25-year and 100-year storm events at five individual outfall locations. Table 3.8 below shows the time series that were developed for each model outfall, and the corresponding cross section from the HEC-RAS model that was used to determine the values.

Table 3.9 Model Outfall Boundary Conditions at San Juan Creek

HEC-RAS Model STA	1855	1502	4555	3955	3355	1855	1502	4555	3955	3355
Outfall	OF1	OF2	OF3	OF4	OF5	OF1	OF2	OF3	OF4	OF5
Hour	25-year Storm Outfall Water Surface Elevations (ft, NAVD 88)					100-year Storm Outfall Water Surface Elevations (ft, NAVD 88)				
0	9.6	8.4	19.4	16.3	14.3	13.1	12.4	23.5	21.3	19.0
1	10.7	9.8	20.8	17.9	15.8	13.1	12.4	23.5	21.2	18.9
3	12.4	11.7	22.9	20.3	18.1	14.1	13.4	24.6	22.6	20.2
6	13.6	12.8	24.0	21.8	19.5	15.0	14.2	25.6	23.6	21.2
9	13.9	13.2	24.4	22.3	19.9	15.5	14.6	26.1	24.2	21.8
12	14.5	13.7	25.0	23.0	20.6	16.3	15.3	26.9	25.1	22.6
15	17.0	15.9	27.6	26.0	23.4	20.4	19.3	31.0	29.7	27.1
17.25	19.0	18.0	29.6	28.2	25.6	26.1	27.2	33.2	32.2	29.4
18	19.0	17.9	29.6	28.2	25.6	25.9	27.0	32.9	31.9	29.2
21	16.6	15.6	27.2	25.5	23.0	18.7	17.7	29.3	27.9	25.3
24	14.1	13.3	24.6	22.5	20.1	15.5	14.6	26.0	24.2	21.8
27	12.4	11.6	22.7	20.2	18.1	12.8	12.1	23.1	20.8	18.6
30	10.4	9.3	20.3	17.3	15.2	10.7	9.6	20.6	17.7	15.6
33	10.0	8.8	19.7	16.8	14.6	10.5	9.4	20.4	17.4	15.3
36	9.9	8.7	19.6	16.7	14.5	10.1	8.9	19.8	16.9	14.7
39	9.9	8.7	19.6	16.6	14.5	9.9	8.8	19.7	16.7	14.5
42	9.8	8.7	19.6	16.6	14.5	9.9	8.7	19.6	16.6	14.5
45	9.8	8.6	19.6	16.6	14.4	9.8	8.7	19.6	16.6	14.5

HEC-RAS Model STA	1855	1502	4555	3955	3355	1855	1502	4555	3955	3355
Outfall	OF1	OF2	OF3	OF4	OF5	OF1	OF2	OF3	OF4	OF5
Hour	25-year Storm Outfall Water Surface Elevations (ft, NAVD 88)					100-year Storm Outfall Water Surface Elevations (ft, NAVD 88)				
48	9.8	8.6	19.5	16.5	14.4	9.8	8.6	19.6	16.6	14.4

3.4.6 Creek Overtopping

To model creek overtopping in the analysis, a boundary outfall condition was assigned in PCSWMM. This outfall condition was applied along the length of the floodwall that the hydraulic analysis in Section 3.1 indicated would overtop, which is about the final 350 feet of wall along the creek east bank, directly upstream of the Highway 1 northbound bridge. The outfall was modelled using the creek boundary condition developed for the 54" RCP pipe (model outfall 2), as most of the wall showing overtopping is located in this area.

To verify the creek overtopping flow rate at the floodwall that is estimated in the PCSWMM 2D model, we prepared a lateral weir model run at the floodwall in the San Juan Creek HEC-RAS model. When comparing the two analysis methods between PCSWMM and HEC-RAS, the estimated peak overtopping flows were within approximately 7% of one another at around 2,500 cfs.

3.4.7 Existing Conditions Results

Three existing condition model scenarios were simulated, the 25-year storm, the 100-year storm, and the 100-year storm without overtopping of San Juan Creek. As noted in Section 3.1, the hydraulic analysis of San Juan Creek determined that the creek does overtop near the southern area of the project site. However, to gain an understanding of how the floodplain near the site may change due to improvements of this floodwall, the 100-year storm was simulated assuming that the creek does not overtop.

Figure 3.7 shows the flood inundation for the existing condition 25-year storm event. During the 25-year event, flooding occurs at the eastern most model extent along the 54" RCP pipe, at the eastern side of the railroad tracks. This area represents a low point in the system, and can be seen in aerial imagery and from the LiDAR surface as a localized ponding area. Flooding in this case is caused primarily due to the boundary condition in San Juan Creek, which during high creek flow creates backwater to the pipe system. The estimated flood depth ranges up to one feet. During the 25-year event, no surcharging occurs in the South Swale or any features associated with LO1SO2.

Figure 3.8 shows the flood inundation for the existing condition 25-year storm event. During the 100-year event, the entire project site and much of the surrounding area is subject to flooding. As the storm progresses there are two primary contributing factors to flooding in the area. As the level of San Juan Creek begins to rise, the incoming flow from upstream watersheds begins to surcharge LO1SO2, the 54" RCP, and the South Swale. In addition the upstream flow, backflow from San Juan Creek through LO1SO2 begins to surcharge at the open channel sections and inundate the area. As the water level in San Juan Creek approaches its peak, the east bank floodwall at the north of Highway 1 overtops, which inundates additional areas in the project site and increase flood depths.

Maximum flood depths at the project site during this storm range from one to six feet, and areas surrounding the project site are inundated around one to four feet.

Figure 3.9 shows the flood inundation for the existing conditions 100-year storm event, assuming no overtopping from San Juan Creek along the east bank floodwall. Under this scenario, flooding in the project site is due to a product of upstream stormwater surcharging and backflow from San Juan Creek through LO1SO2. When the creek does not overtop the floodwall, there is still flooding that occurs at the project site, but in less extent and depth. The depths of flooding range from approximately one feet to four feet.

3.4.8 Comparison to Previous Work

In addition to this study, there are a number of previous studies on flooding in the vicinity of the project area.

The FEMA FIS for the City of San Juan Capistrano shows similar floodplain boundaries to those shown in Figures 3.8 and 3.9. It should be noted that in the FEMA study, the flood zone was designated AO, overtopping from San Juan Creek was not a contributing factor. The FEMA floodplain delineation also references a portion of the flood zone which occurs due to surcharging at a stormwater manhole located at Sulpevida and Camino Capistrano. This result is not reproduced in this analysis, as this manhole is located outside the area of detailed study in the PCSWMM model.

Tetra Tech, Inc. completed a Floodway Compliance Technical Memo (2015) for the property known as Parking Lot B, which is located adjacent to the open channel portion of LO1SO2 on the north side. Using a one-dimensional WSPG model, the analysis estimated water surface elevation of 30.2’ at the open channel portion of LO1SO2, which when extrapolated across the property, it results in a flood depth range of 1.9 to 4.6 feet for the proposed lot grading. This analysis indicated a maximum water surface elevation of approximately 26 feet.

Tetra Tech 2016 assessed flooding at the South Orange County Wastewater Authority JB Latham Treatment Plant, which is located across San Juan Creek from the project site. The study also indicated backwater effect in San Juan Creek caused by the Highway 1 Bridge.

PACE completed a capacity analysis of San Juan Creek (PACE, 2010). This analysis was completed using the same HEC-RAS model that was used for the analysis presented in Section 3.1 of this report. The analysis demonstrated that the channel in the area of the project site has a bank full capacity of 19,000, indicating that flows higher than that would overtop the levee and flood the adjacent areas.

3.5 Drainage Improvements

To mitigate flooding at the project site, a set of site improvement options has been proposed that could be incorporated into the project design. The options are shown in Figure 3.10.

As shown in the figure, there are six main improvement options that have been developed for this site, they are:

- Regrade and raise the project site elevation;
- Add a stormwater detention basin to mitigate for post project runoff conditions;
- Abandon and relocate the existing drainage inlet (DI) near the outfall of the 54" RCP;
- Replace floodwall along San Juan Creek flood in section showing creek overtopping;
- Add a flap gate to prevent backflow from San Juan Creek entering LO1SO2; and
- Add a pump station to mitigate flooding originating from upstream watersheds.

Using these six improvement options, four separate alternatives have been developed which includes a subset of these improvement options. Each of the four alternatives was modelled in PCSWMM, and the results are discussed in further detail below. It should be noted that in each alternative section below, residual flooding that occurs after the implementation of the alternative is compared to a base case flooding scenario. The base case scenario is the 100-year flooding that is shown in Figure 3.8, which includes overtopping flows from San Juan Creek near the Highway 1 Bridge.

3.5.1 Alternative 1

Alternative 1 was formulated to be a standalone option that would mitigate flooding at the project site regardless of future flood improvements along San Juan Creek. This alternative includes re-grading the project site to 28.2 feet elevation, which is approximately 1 foot higher than the 100-year flood elevation with creek overtopping. Figure 3.10 shows the limits of grading for this alternative. This alternative also includes a detention basin to mitigate additional runoff generated from development of the project site. The detention basin was sized to be 0.5 acres and 3 feet deep. Lastly, this alternative includes the capping of the existing DI adjacent to the 54" RCP pipe outfall at San Juan Creek, and relocating the DI eastward in the project site. Otherwise, overtopping flow from the creek could enter the existing DI, backflows up the pipe and inundates areas east of the project site. See Figure 3.11 for the flood inundation map for this alternative, and Figure 3.12 for the change in flood elevations as compared to the base scenario.

Raising the ground elevation reduces both the extents and depth of flooding. As shown in Figure 3.12, the average flooding depth reduction in areas surrounding the project site is approximately one to four feet. Raising the ground elevation essentially acts as a flood barrier from the creek, which blocks overtopping flows from the creek to reach the project site and upstream watershed. Note that the proposed site regrade needs to include the area between the southern boundary of the project site property and the Highway 1 northbound bridge. Otherwise, the area between the southern edge of the project site and Highway 1 provides a flood path for overtopping creek flow to inundate upstream watershed area at the east of the project site.

3.5.2 Alternative 2

Alternative 2 includes floodwall improvements to eliminate San Juan Creek overtopping during a 100-year event. The proposed improvement for the floodwall includes approximately 500 feet of new concrete reinforced retaining wall set 1 foot above the 100-year flood elevation. This alternative also includes re-grading the project site to 26 feet elevation, which as discussed previously is approximately 1 foot higher than the 100-year flood elevation, without creek overtopping. The limits of grading for this alternative are the same as alternative 1. See Figure 3.13 for the flood inundation

map for this alternative, and Figure 3.14 for the change in flood elevations as compared to the base scenario.

As shown, raising the ground elevation and improving the floodwall in this alternative reduces both the extents and depth of the flooding, similar to Alternative 1. As shown in Figure 3.14, the average flooding depth reduction in areas surrounding the project site is approximately one to four feet.

3.5.3 Alternative 3

Alternative 3 is the same as Alternative 2 except the addition of a new detention basin at the southern end of the project site. See Figure 3.15 for the flood inundation map for this alternative, and Figure 3.16 for the change in flood elevations as compared to the base scenario.

Raising the ground elevation and improving the levee for this alternative reduces both the extents and depth of the flooding, similar to Alternative 2. As shown in Figure 3.16, the average flooding depth reduction in areas surrounding the project site is approximately one to four feet. Adding the detention basin in this alternative results in a negligible reduction in flooding depths.

3.5.4 Alternative 4

Alternative 4 was formulated to work as an option that requires minimal site grading to mitigate flooding at the project site. This was achieved through a combination of alternatives to mitigate flooding from creek overtopping, creek backflow through LO1SO2, and upstream storm drain surcharging. This alternative includes installing a flap gate at LO1SO2 to cease backflow from San Juan Creek, floodwall improvements as presented in Alternatives 2 & 3, raising the site to a minimum elevation of 23.5 feet, and installing a pump station to mitigate flooding at the South Swale and LO1SO2. See Figure 3.17 for the flood inundation map for this alternative, and Figure 3.18 for the change in flood elevations as compared to the base scenario.

As shown, this alternative results in the greatest reduction in flooding over the project and urban area. Essentially, to make this alternative work to mitigate flooding originating from the South Swale and LO1SO2, the pump station must work to evacuate the peak three hours of the storm from these features. Some site grading in this area is still required, however, due to the flooding that originates from the 54” RCP, which as it surcharges flows north along the railroad tracks it would eventually inundate some of the area at the northeast corner of the project site.

3.5.5 Alternatives Summary and Planning Level Cost Estimates

Four flood improvement alternatives were formulated for the project site with the intent of eliminating flooding on the property during the 100-year expected value storm event. The work included in each alternative is summarized in Table 3.10. For this summary, preliminary earthwork fill quantities were generated from the model’s coastal LiDAR surface. These quantities are meant to be approximate and convey the relative magnitude of fill required for each alternative.

Table 3.10 Summary of Work for Each Improvement Alternative

Alternative	Improvements
1	• Raise project site to 28.2 feet (appx 67,000 CY fill volume) • Demolish existing DI and cap, install new DI • Install detention basin (0.5 acres, 3 feet deep)
2	• Raise project site to 26 feet (appx 27,000 CY fill volume) • Install 500 feet of new sheet pile floodwall along San Juan Creek levee
3	• Raise project site to 26 feet (appx 27,000 CY fill volume) • Install 500 feet of new sheet pile floodwall along San Juan Creek levee • Install detention basin (0.5 acres, 3 feet deep)
4	• Raise project site to 23.5 feet (appx 6,500 CY fill volume) • Install 500 feet of new sheet pile floodwall along San Juan Creek levee • Install detention basin (0.5 acres, 3 feet deep) • Install flap gate on LO1SO2 • Install a pump station and high flow bypass lines for LO1SO2 and South Swale

Planning level construction cost estimates were developed for each of the four alternative scenarios, as shown in Table 3.11. Detail cost estimate breakdowns for Alternatives 1, 2 and 3 are included in Appendix C.

Table 3.11 Summary of Work for Each Improvement Alternative

Alternative	Planning Level Cost Estimate
1	$2,312,000
2	$3,778,000
3	$3,961,000
4	$10,000,000+

The construction cost estimate for Alternative 4 is only an order-of-magnitude estimate, to illustrate the significant cost difference between Alternative 4 and the other alternatives. Should Alternative 4 be ultimately selected as the preferred flood improvement alternative for the project site, additional construction cost assessment is needed to fine tune the estimate.

Note that the cost estimates for Alternatives 2 and 3 included a construction cost estimate to construct 500 feet of sheet pile floodwall along San Juan Creek at the upstream of the Highway 1 bridges. It is deemed the minimum improvements necessary to alleviate the project site from 100-year flood. However, any floodwall improvements will need to be coordinated with Orange County, the owner of the floodwall and levee facilities. Orange County is working on a long range plan to potentially construct new sheet pile floodwall between Highway 1 and Stonehill Drive as a part of a regional flood

improvement project. At 3,200 linear feet in length and a unit cost of $4,000 per linear foot for a new sheet pile wall, the total construction cost estimate would be around $12,800,000. The floodwall improvement estimate as included in Alternatives 2 and 3 may be considered as a starting point to assess a potential cost sharing for the regional flood improvement project, should the SCWD select either Alternatives 2 or 3 as the preferred alternative, and consider flood improvement partnership with Orange County.

3.6 Recommendations

It is recommended that Alternative 1 as the preferred flood improvement alternative for the project site. The following listed the advantages of Alternative 1.

- It provides the same level of flood protection as Alternatives 2 & 3, yet lower residual flood risk since the flood protection mechanism is by elevation, not by floodwall;
- It reduces flooding in adjacent areas by blocking overtopping flows from San Juan Creek;
- It is the least costly alternative;
- It is a standalone option that works independent of the need to improve San Juan Creek, hence reduce the level of inter-agencies dependence and streamline inter-agencies collaboration;
- It has minimal long term maintenance, especially when compared to Alternative 4 which requires a major pump station facilities; and
- It is the simplest alternative to design and construct.

Since the premise of Alternative 1 is to build up the project site to be above the flood level, this option has the highest reliability, whereas Alternatives 2 and 3 depends on floodwall, and Alternative 4 depends on pump station, which all of these options have highest risk of failure. In addition, since Alternative 1 does not depend on San Juan Creek improvements, its project schedule does not need to be held depending upon the regional flood improvement project from Orange County. On the other hand, the proposed improvements in Alternative 1 will not preclude future improvement works on San Juan Creek.

4. References

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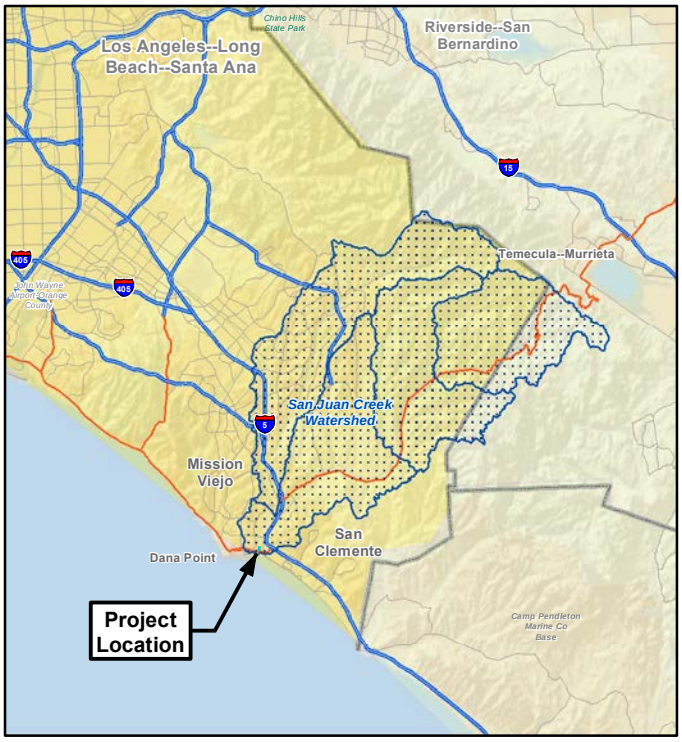
Figures



Project Site
San Juan Creek Watershed
Freeway
Highway
Major Road
Local Road
Parks/Open Space

Paper Size 8.5" x 11" (ANSI A)
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Feet
Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane California III FIPS 0403 Feet

N

South Coast Water District
Doheny Desalination Project - EIR

Job Number 11125157
Revision A
Date 08 May 2017

Vicinity and Project Location

Figure 1.1

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Paper Size ANSI A

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Feet

Map Projection: Transverse Mercator
Horizontal Datum: North American 1983
Grid: NAD 1983 UTM Zone 11N

N

LEGEND

- Site Location
- 2100 Shoreline - 0 m Sea Level Rise
- 2100 Shoreline - 0.5 m Sea Level Rise
- 2100 Shoreline - 1 m Sea Level Rise
- 2100 Shoreline - 1.5 m Sea Level Rise
- 2100 Shoreline - 2 m Sea Level Rise
- 2100 Shoreline - 5 m Sea Level Rise

Shoreline Data: U.S. Geological Survey, Pacific Coastal and Marine Science Center, 2015.

Doheny Desalination Project
Coastal Hazard Assessment

Job Number 11140040
Revision A
Date 04 May 2017

CosMos 3.0 Shoreline Retreat

Figure 2.1

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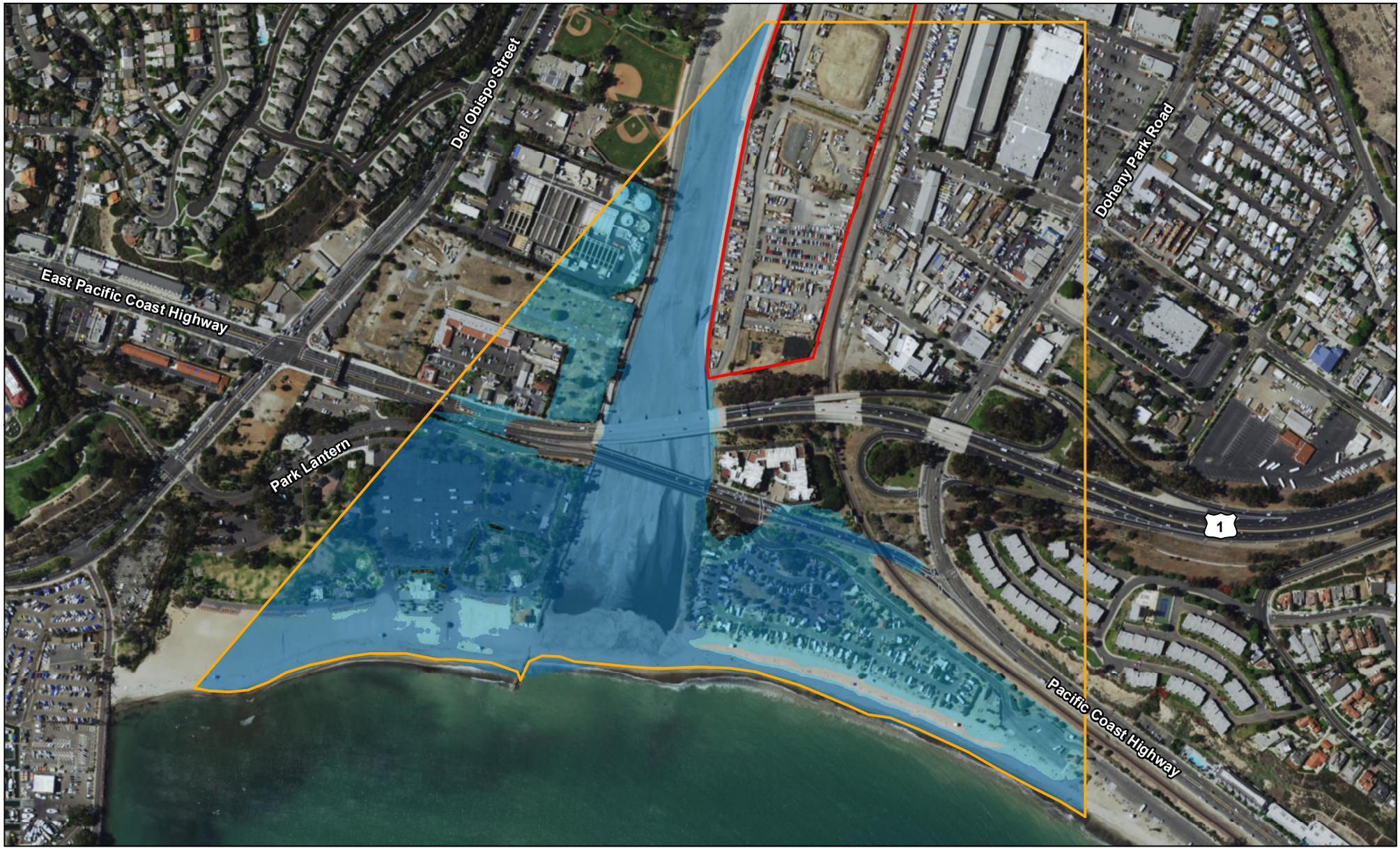
Data Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community; Site Location: GHD, 2017; Shoreline Data: U.S. Geological Survey, Pacific Coastal and Marine Science Center, 2015.. Created by:rgalos



Paper Size ANSI A

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Feet

Map Projection: Transverse Mercator
Horizontal Datum: North American 1983
Grid: NAD 1983 UTM Zone 11N

N

LEGEND

Site Location

Extent of Assessment

Flooded Areas for Extremal Total Water Level

2070 Low Range Sea Level Rise (15.5 ft*)

2070 High Range Sea Level Rise (18 ft*)

*Source: Michael Baker International, 2017. Coastal Hazards Analysis for the Doheny Desalination Project. February 2017

Doheny Desalination Project
Coastal Hazard Assessment

Job Number 11140040
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Date 04 May 2017

Future Flooding 2070

Figure 2.2

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Data Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community; Site Location, and Extent of Assessment: GHD, 2017; Flooded Areas: GHD, 2017, and U.S. Army Corps of Engineers (USACE), 2016. Created by:rgalos

Paper Size ANSI A

0 100 200 300 400 500

Feet

Map Projection: Transverse Mercator
Horizontal Datum: North American 1983
Grid: NAD 1983 UTM Zone 11N

N

LEGEND

Site Location

Extent of Assessment

Flooded Areas for Extremal Total Water Level

2100 Low Range Sea Level Rise (16.1 ft*)

2100 High Range Sea Level Rise (20.3 ft*)

*Source: Michael Baker International, 2017. Coastal Hazards Analysis for the Doheny Desalination Project. February 2017

Doheny Desalination Project
Coastal Hazard Assessment

Future Flooding 2100

Job Number	11140040
Revision	A
Date	04 May 2017

Figure 2.3

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Data Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community; Site Location, and Extent of Assessment: GHD, 2017; Flooded Areas: GHD, 2017, and U.S. Army Corps of Engineers (USACE), 2016. Created by:rgalos

Paper Size ANSI A

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Feet

Map Projection: Transverse Mercator
Horizontal Datum: North American 1983
Grid: NAD 1983 UTM Zone 11N

N

LEGEND

- Site Location
- CosMos Flood Limits - 0.5 m Sea Level Rise
- CosMos Flood Limits - 2 m Sea Level Rise

Flood Data: U.S. Geological Survey, Pacific Coastal and Marine Science Center, 2015.

Doheny Desalination Project
Coastal Hazard Assessment

Job Number | 11140040
Revision | A
Date | 04 May 2017

CosMos 3.0 Future Flooding

Figure 2.4

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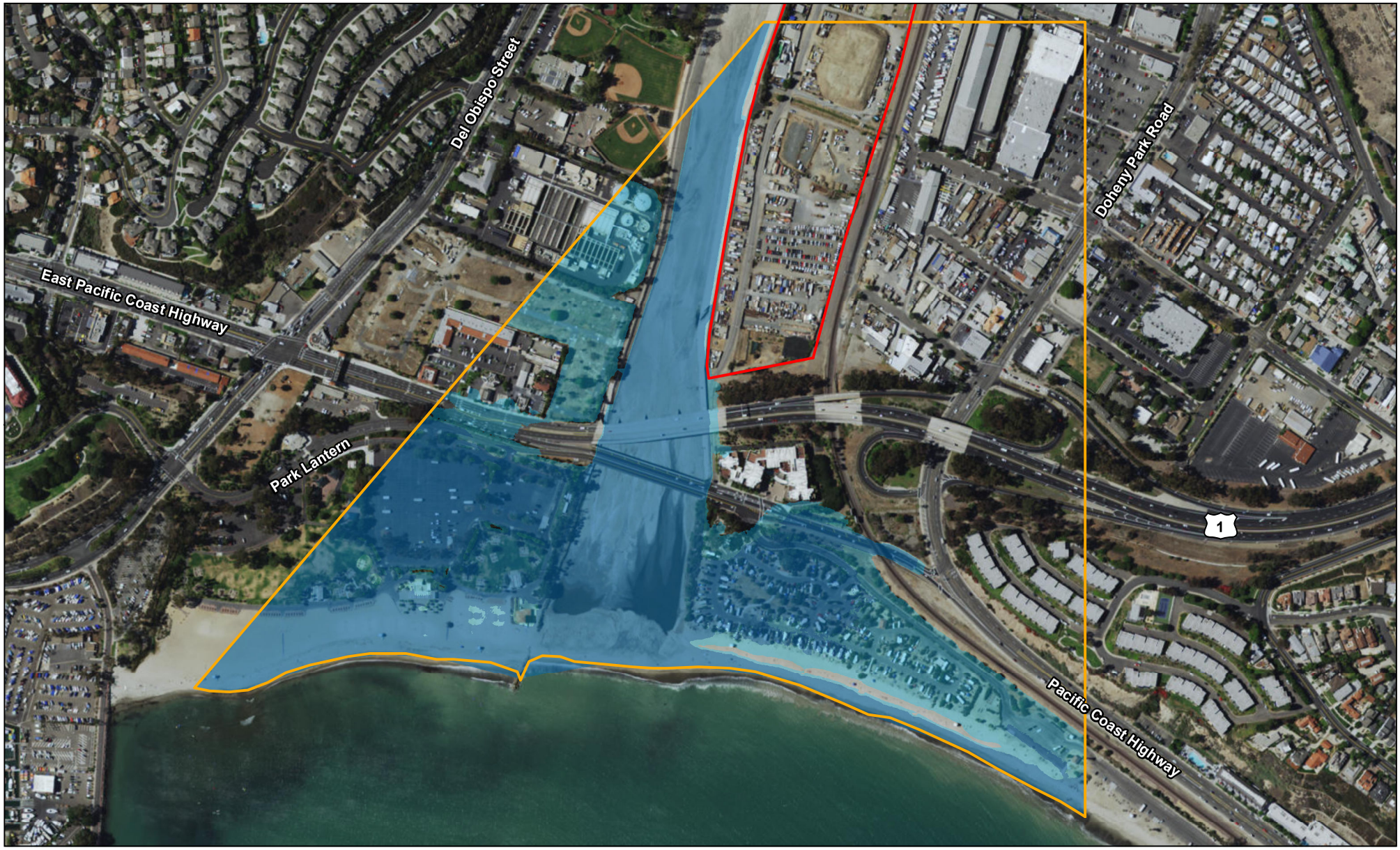
Data Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community; Site Location: GHD, 2017; Flood Data: U.S. Geological Survey, Pacific Coastal and Marine Science Center, 2015. Created by:rgalos



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Feet

Map Projection: Transverse Mercator
Horizontal Datum: North American 1983
Grid: NAD 1983 UTM Zone 11N

N

LEGEND

Site Location

Extent of Assessment

Flooded Areas for Extremal Total Water Level

2070 Low Range Sea Level Rise (15.92 ft*)

2070 High Range Sea Level Rise (18.43 ft*)

*Source: Michael Baker International, 2017. Coastal Hazards Analysis for the Doheny Desalination Project. February 2017

Doheny Desalination Project
Coastal Hazard Assessment

Job Number 11140040
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Tsunami Flooding 2070

Figure 2.5

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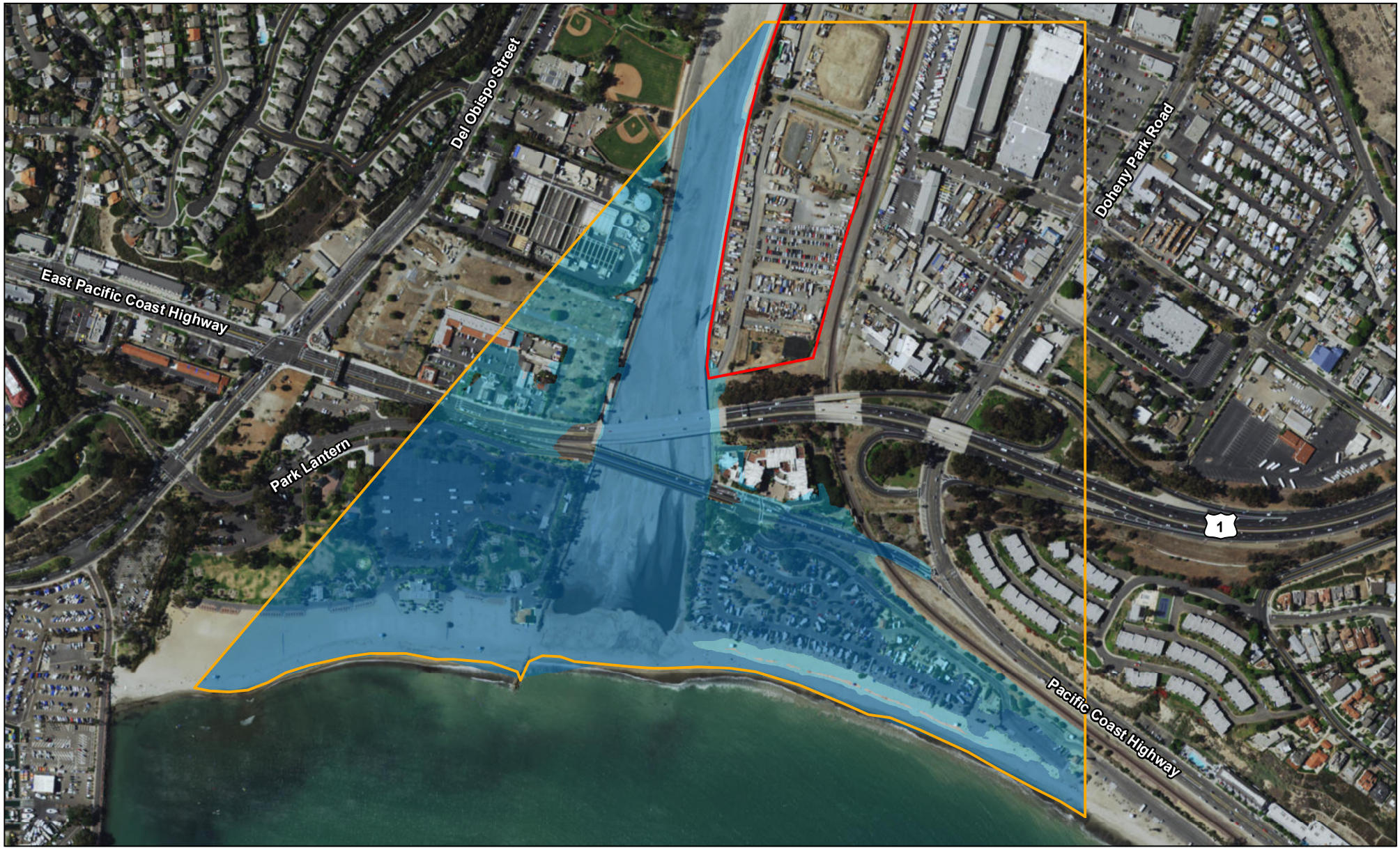
Data Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community; Site Location, and Extent of Assessment: GHD, 2017; Flooded Areas: GHD, 2017, and U.S. Army Corps of Engineers (USACE), 2016. Created by:rgalos



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Feet

Map Projection: Transverse Mercator
Horizontal Datum: North American 1983
Grid: NAD 1983 UTM Zone 11N

N

LEGEND

Site Location

Extent of Assessment

Flooded Areas for Extremal Total Water Level

2100 Low Range Sea Level Rise (16.59 ft*)

2100 High Range Sea Level Rise (20.71 ft*)

*Source: Michael Baker International, 2017. Coastal Hazards Analysis for the Doheny Desalination Project. February 2017

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Coastal Hazard Assessment

Job Number 11140040
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Tsunami Flooding 2100

Figure 2.6

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Data Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community; Site Location, and Extent of Assessment: GHD, 2017; Flooded Areas: GHD, 2017, and U.S. Army Corps of Engineers (USACE), 2016. Created by:rgalos

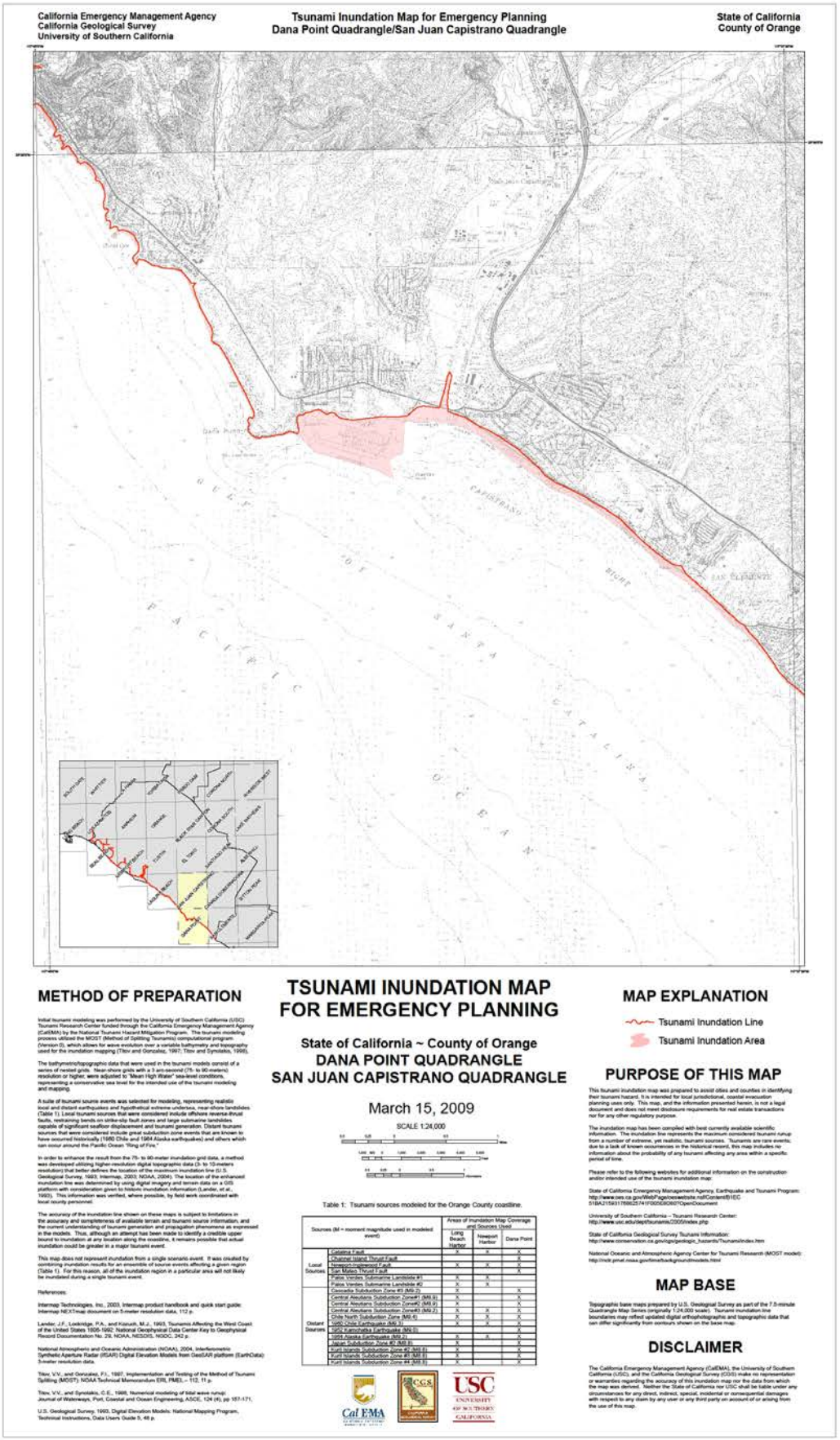


Figure 2.7 – Tsunami Inundation Map for Emergency Planning

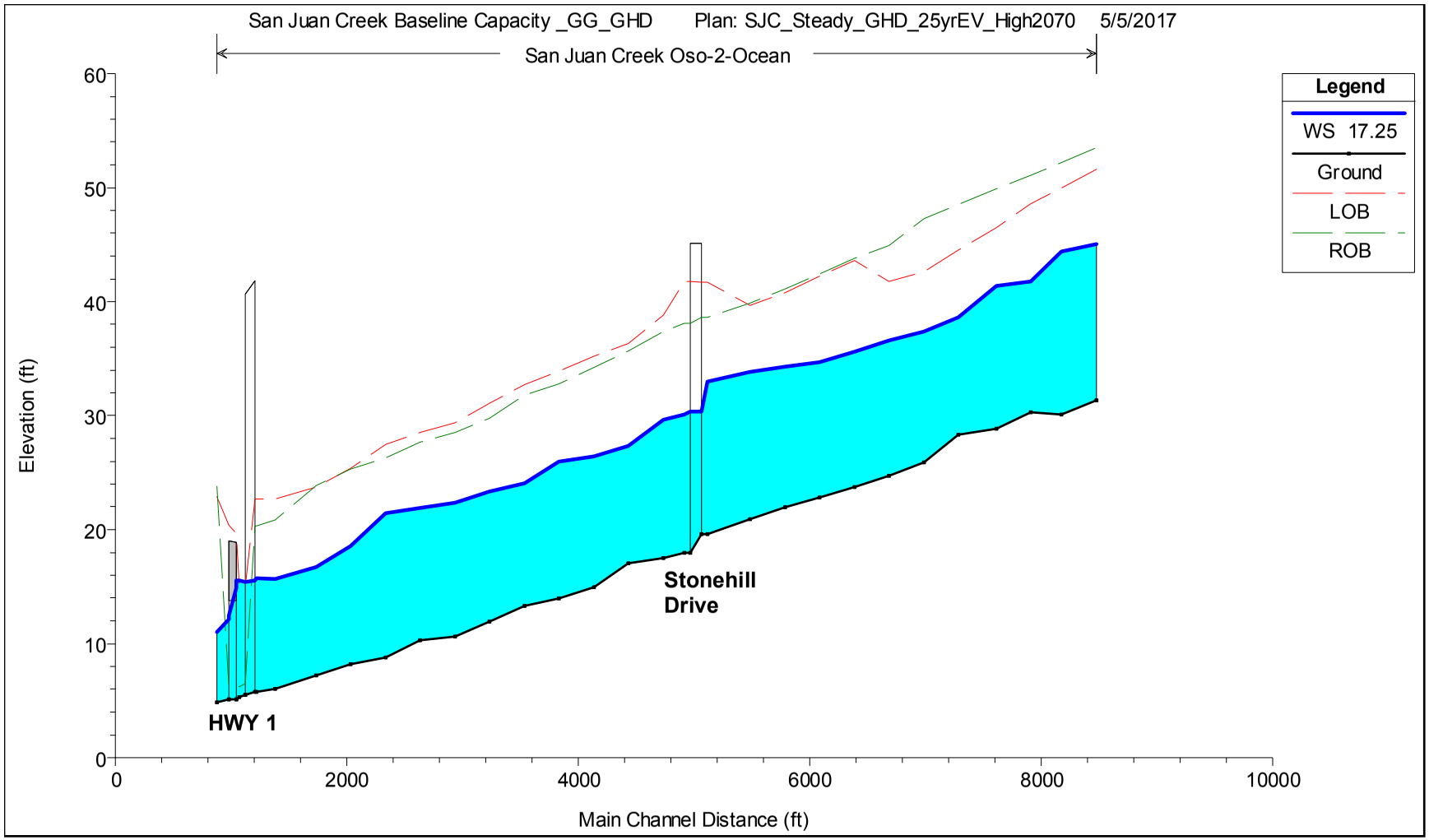
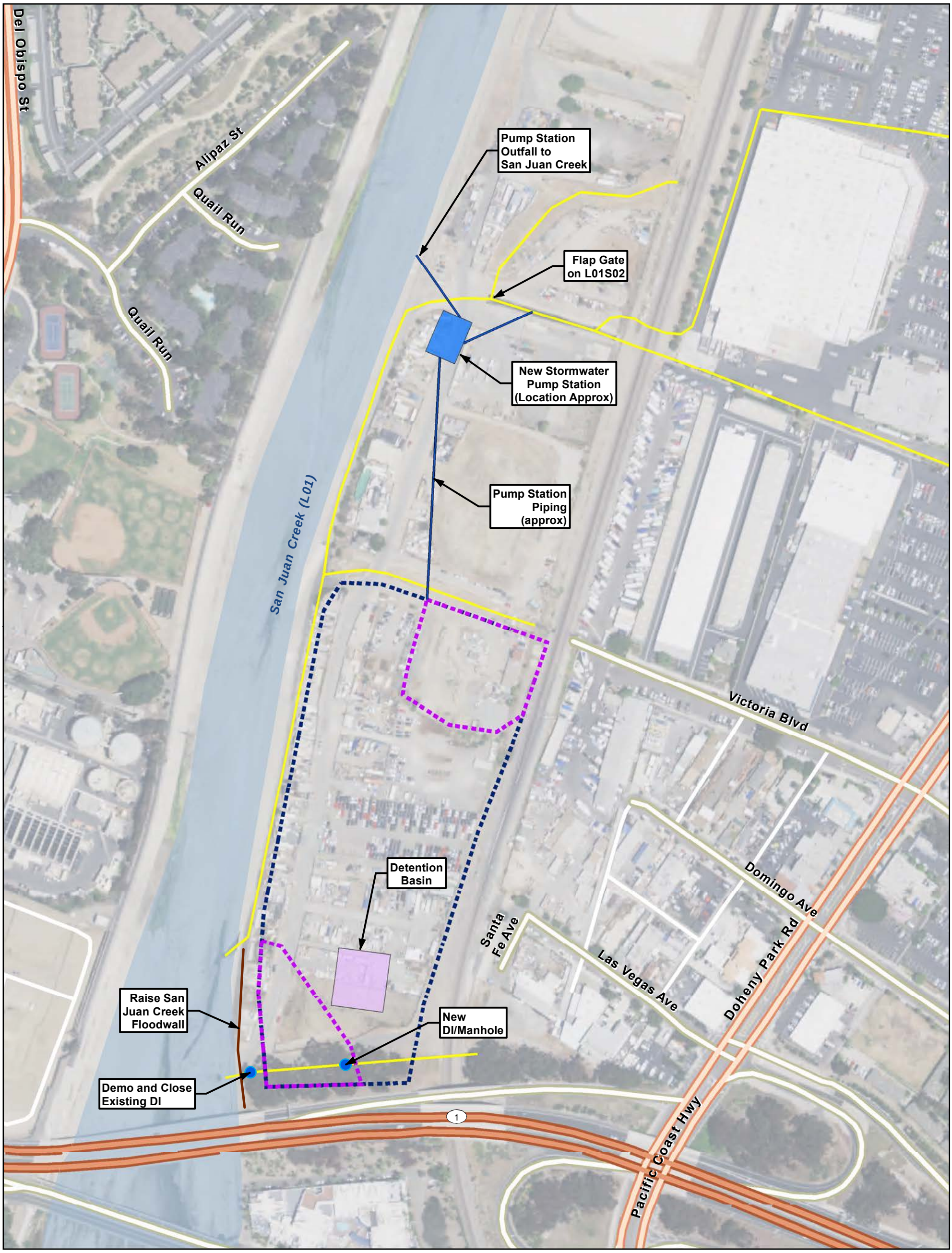


Figure 3.1 – San Juan Creek maximum water surface profile, 25-year event.



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Feet

Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane California VI FIPS 0406 Feet

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South Coast Water District
Doheny Desalination Project Drainage Analysis

Job Number 11125157
Revision A
Date 08 May 2017

Proposed Improvement Options Figure 3.10

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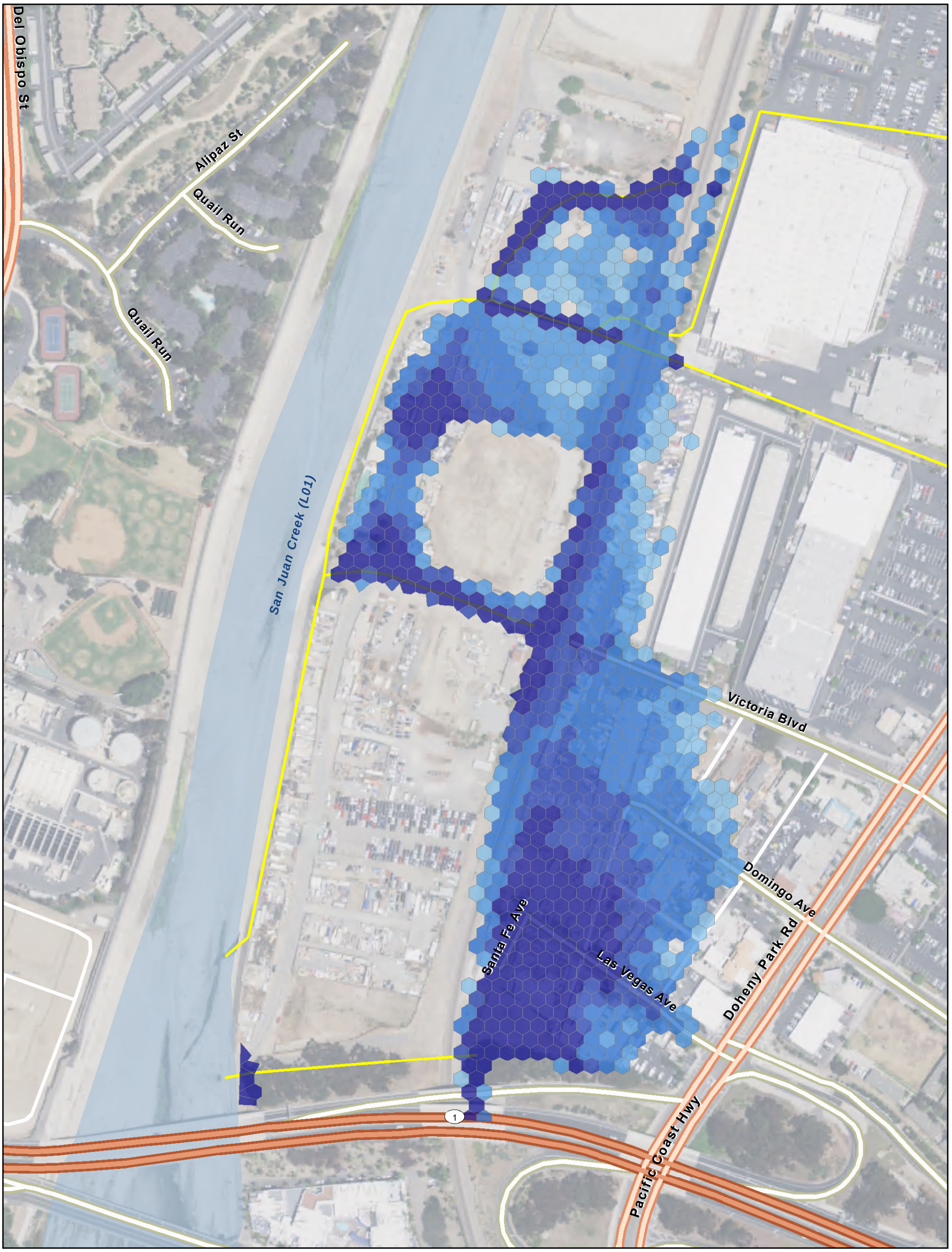
Data source: Aerial: NAIP, 2016; ESRI Street Map; GHD: elevation model. Created by:jclark2



Storm Water Conduits (in model)

Maximum Flooding Depth

2-6" 6-12" 1-2' 2-3' >3'

Paper Size ANSI B

0 100 200 300

Feet

Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane California VI FIPS 0406 Feet

N

South Coast Water District

Doheny Desalination Project Drainage Analysis

Flood Inundation Map, 100-Year Event, Future Condition, Alternative 1

Job Number 11125157
Revision A
Date 09 May 2017

Figure 3.11

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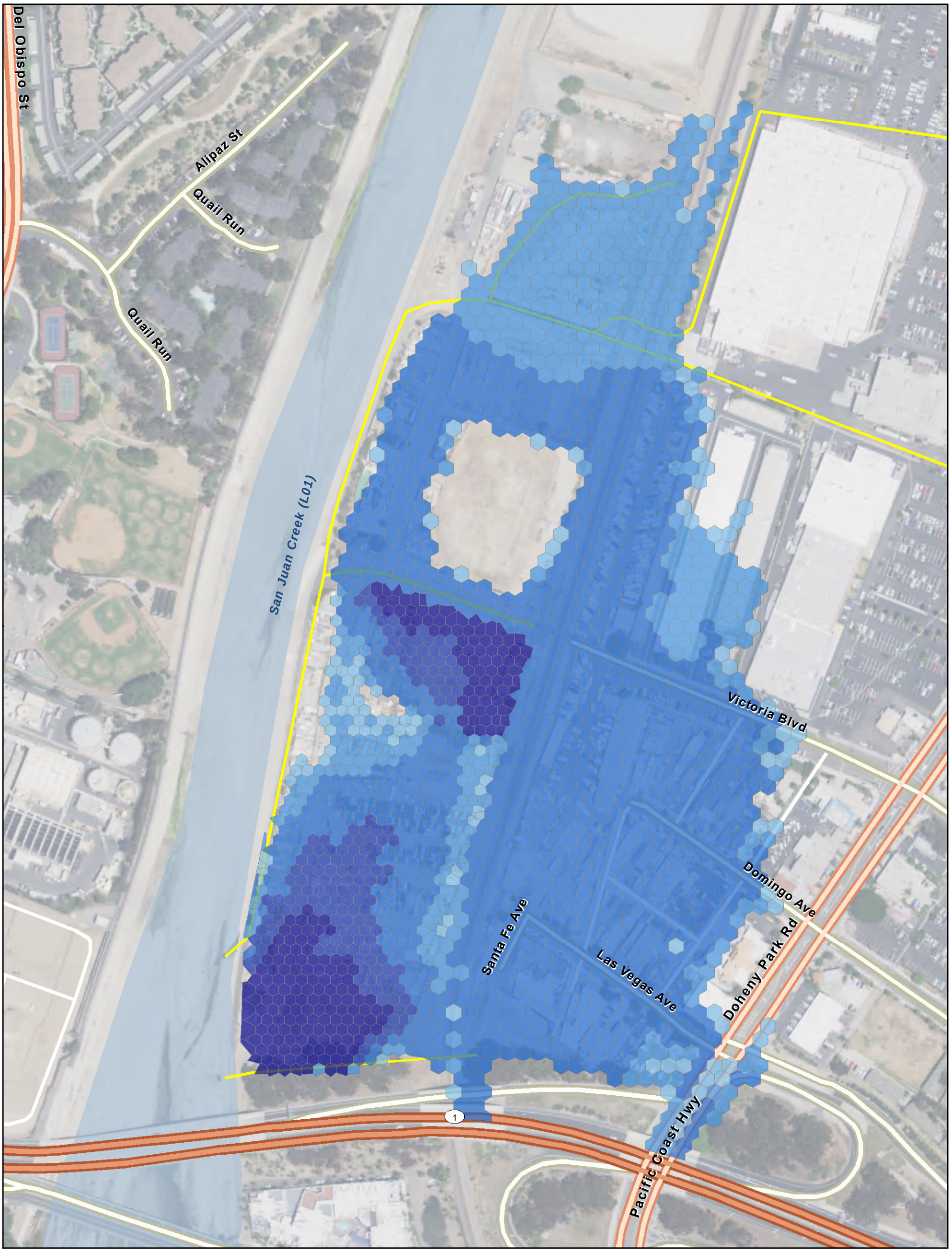
Data source: Aerial: NAIP, 2016; ESRI Street Map; GHD: elevation model. Created by:jclark2



Storm Water Conduits (in model)

Reduction in Maximum Flood Depth

2-6" 6-12" 1-2' 2-3' >3'

Paper Size ANSI B

0 100 200 300

Feet

Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane California VI FIPS 0406 Feet

N

South Coast Water District

Doheny Desalination Project Drainage Analysis

Change in Flood Inundation, Existing Condition vs Alternative 1

Job Number	11125157
Revision	A
Date	09 May 2017

Figure 3.12

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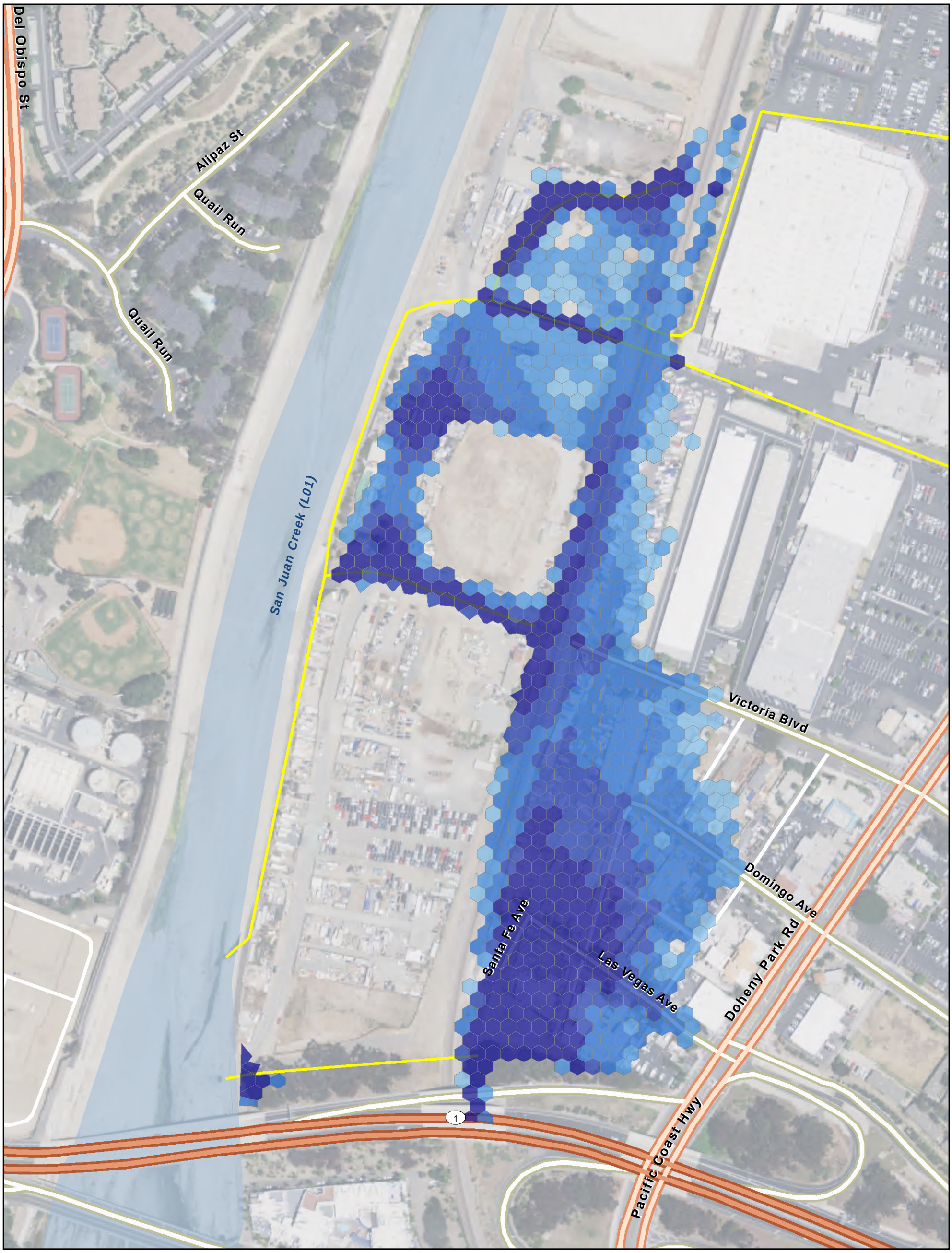
Data source: Aerial: NAIP, 2016; ESRI Street Map; GHD: elevation model. Created by:jclark2



Storm Water Conduits (in model)

Maximum Flooding Depth

2-6" 6-12" 1-2' 2-3' >3'

Paper Size ANSI B

0 100 200 300

Feet

Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane California VI FIPS 0406 Feet

N

South Coast Water District

Doheny Desalination Project Drainage Analysis

Flood Inundation Map, 100-Year Event, Future Condition, Alternative 2

Job Number 11125157
Revision A
Date 09 May 2017

Figure 3.13

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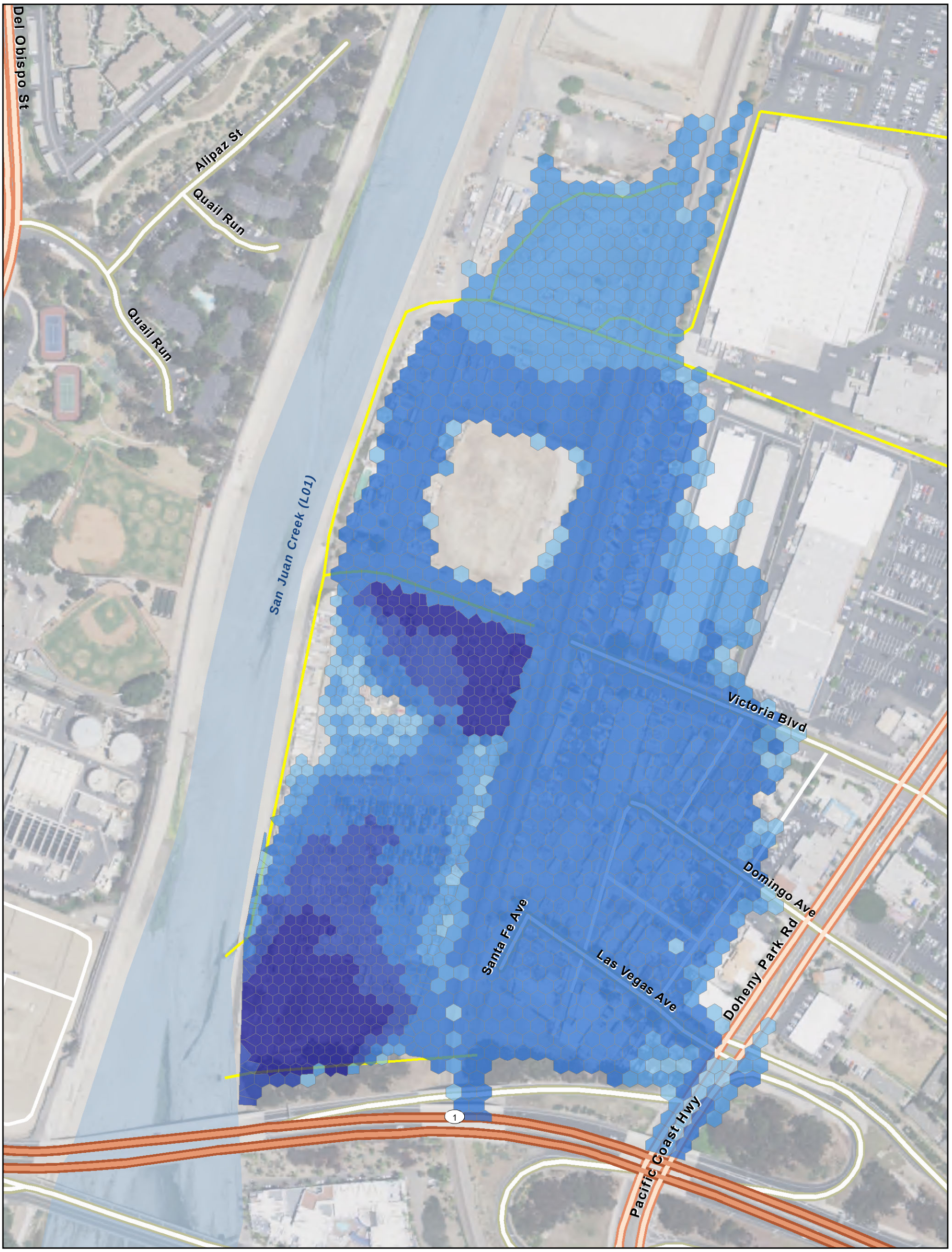
Data source: Aerial: NAIP, 2016; ESRI Street Map; GHD: elevation model. Created by:jclark2



Storm Water Conduits (in model)

Reduction in Maximum Flood Depth

2-6" 6-12" 1-2' 2-3' >3'

Paper Size ANSI B

0 100 200 300

Feet

Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane California VI FIPS 0406 Feet

N

South Coast Water District

Doheny Desalination Project Drainage Analysis

Change in Flood Inundation, Existing Condition vs Alternative 2

Job Number	11125157
Revision	A
Date	09 May 2017

Figure 3.14

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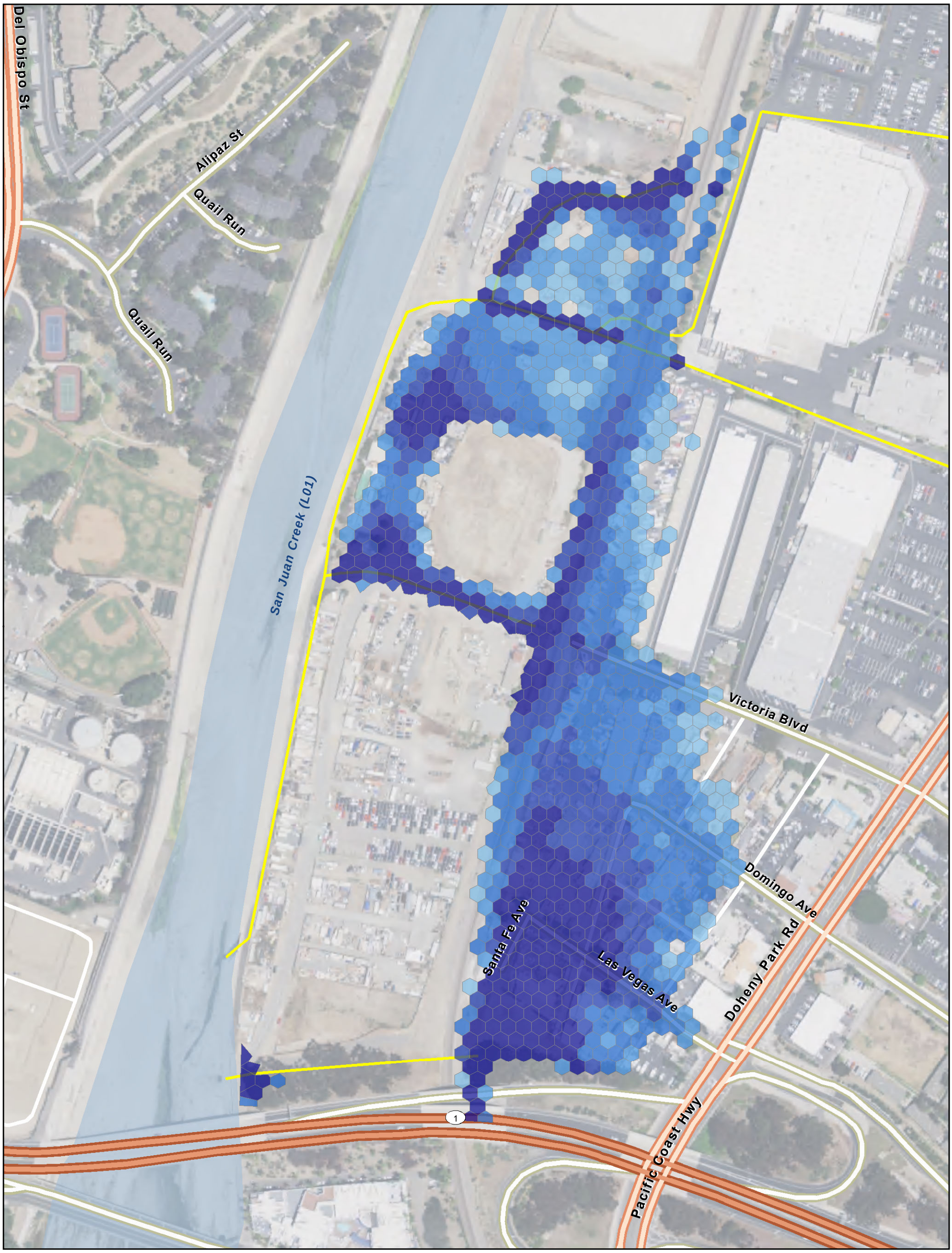
Data source: Aerial: NAIP, 2016; ESRI Street Map; GHD: elevation model. Created by:jclark2



Storm Water Conduits (in model)

Maximum Flooding Depth

2-6" 6-12" 1-2' 2-3' >3'

Paper Size ANSI B

0 100 200 300

Feet

Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane California VI FIPS 0406 Feet

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South Coast Water District

Doheny Desalination Project Drainage Analysis

Flood Inundation Map, 100-Year Event, Future Condition, Alternative 3

Job Number 11125157
Revision A
Date 09 May 2017

Figure 3.15

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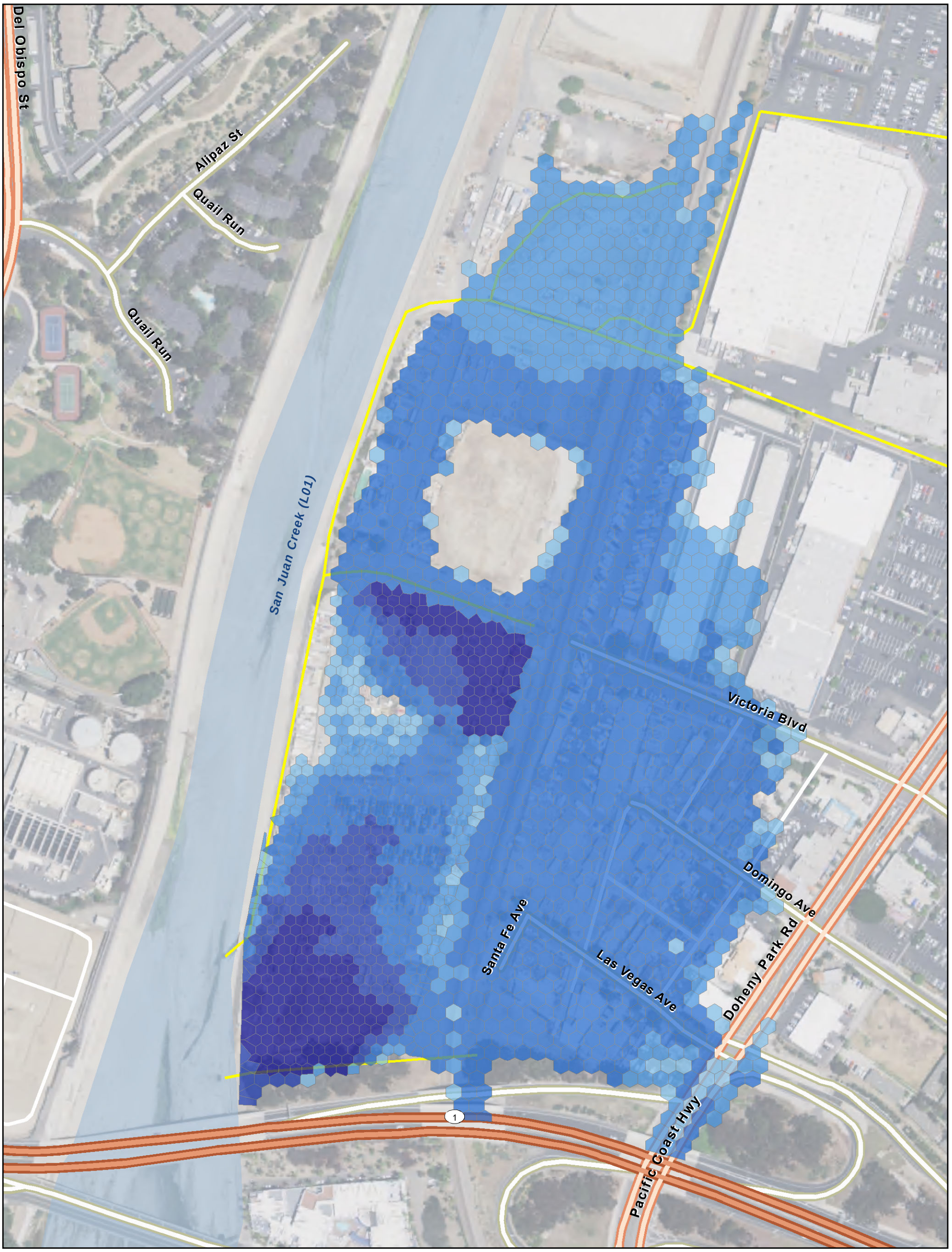

Storm Water Conduits (in model)

Reduction in Maximum Flood Depth

2-6" 6-12" 1-2' 2-3' >3'

Paper Size ANSI B

0 100 200 300

Feet

Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane California VI FIPS 0406 Feet

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South Coast Water District

Doheny Desalination Project Drainage Analysis

Change in Flood Inundation, Existing Condition vs Alternative 3

Job Number	11125157
Revision	A
Date	09 May 2017

Figure 3.16

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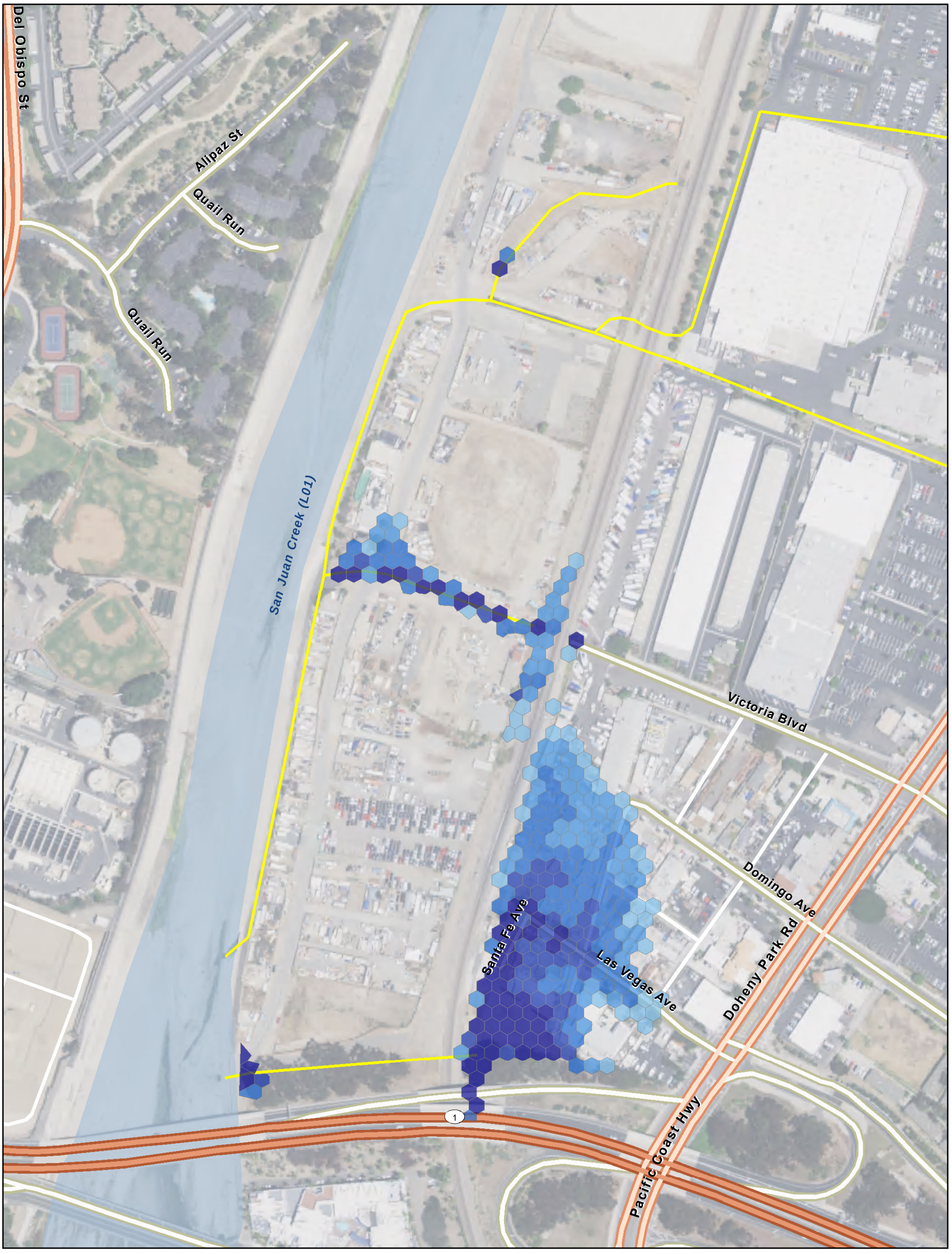

Storm Water Conduits (in model)

Maximum Flooding Depth

2-6" 6-12" 1-2' 2-3' >3'

Paper Size ANSI B

0 100 200 300

Feet

Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane California VI FIPS 0406 Feet

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South Coast Water District
Doheny Desalination Project Drainage Analysis

Flood Inundation Map, 100-Year Event, Future Condition, Alternative 4

Job Number 11125157
Revision A
Date 09 May 2017

Figure 3.17

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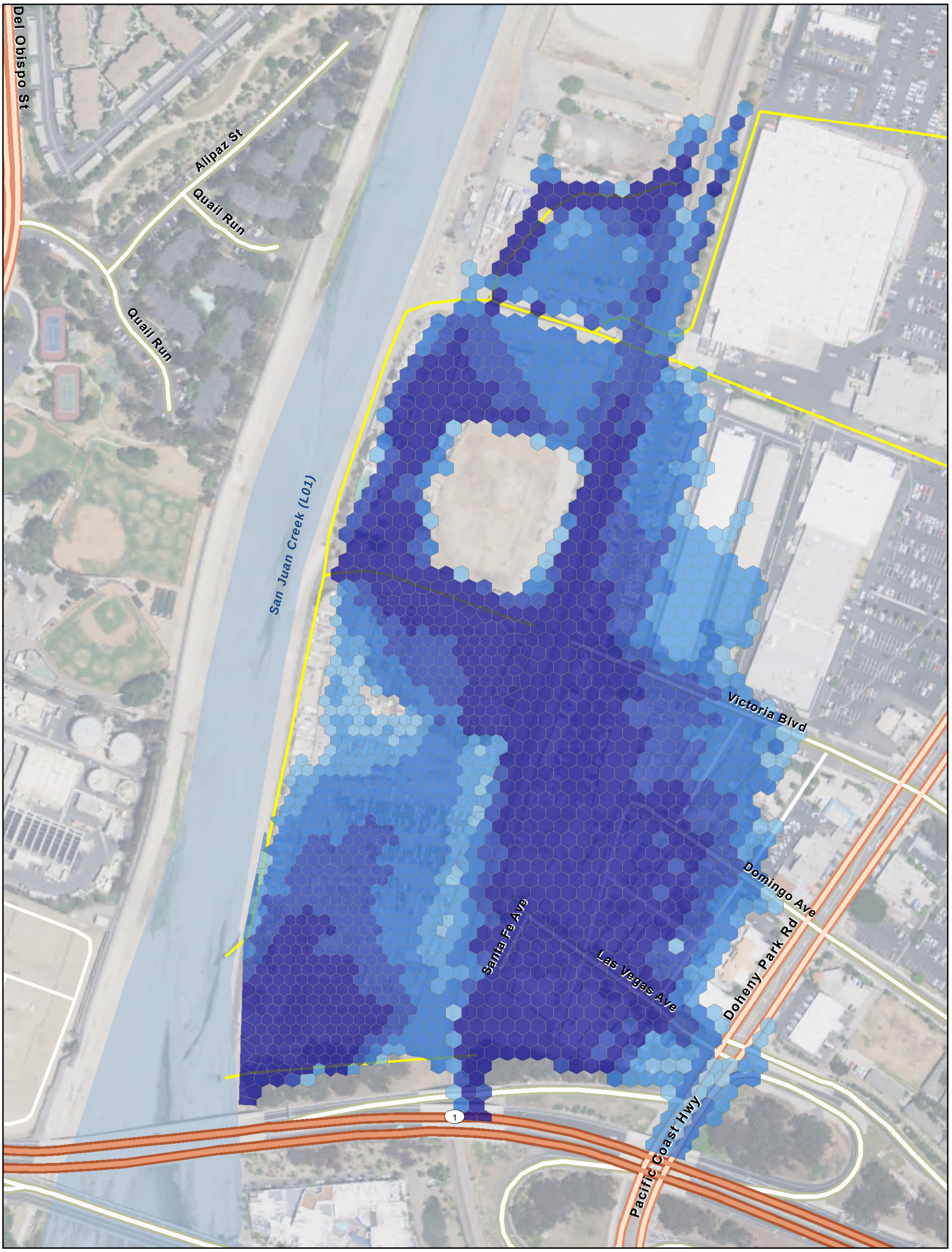

Storm Water Conduits (in model)

Reduction in Maximum Flood Depth

2-6" 6-12" 1-2' 2-3' >3'

Paper Size ANSI B

0 100 200 300

Feet

Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane California VI FIPS 0406 Feet

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South Coast Water District

Doheny Desalination Project Drainage Analysis

Change in Flood Inundation, Existing Condition vs Alternative 4

Job Number	11125157
Revision	A
Date	09 May 2017

Figure 3.18

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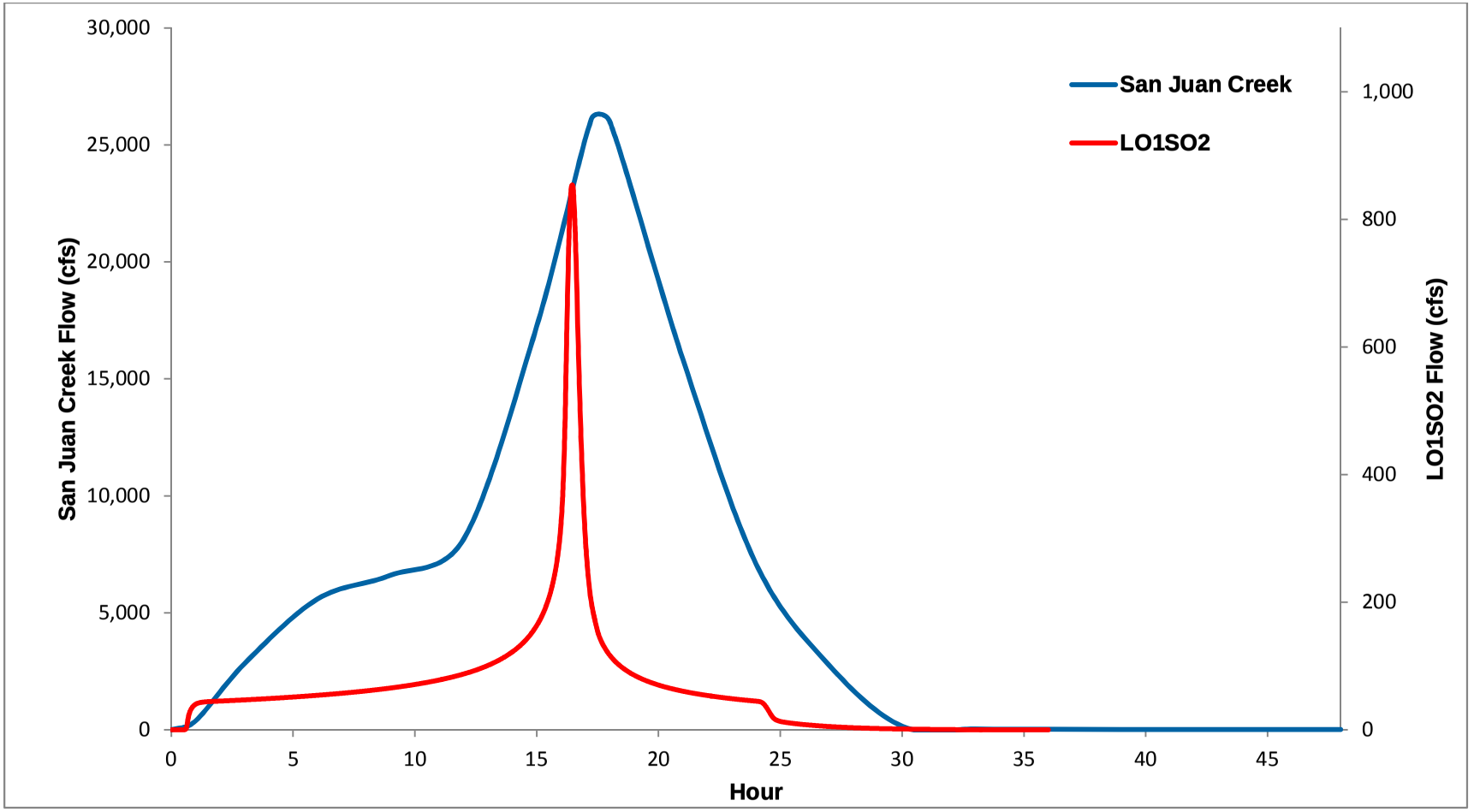


Figure 3.2 – San Juan Creek and LO1SO2 hydrograph comparison, 25-year event.

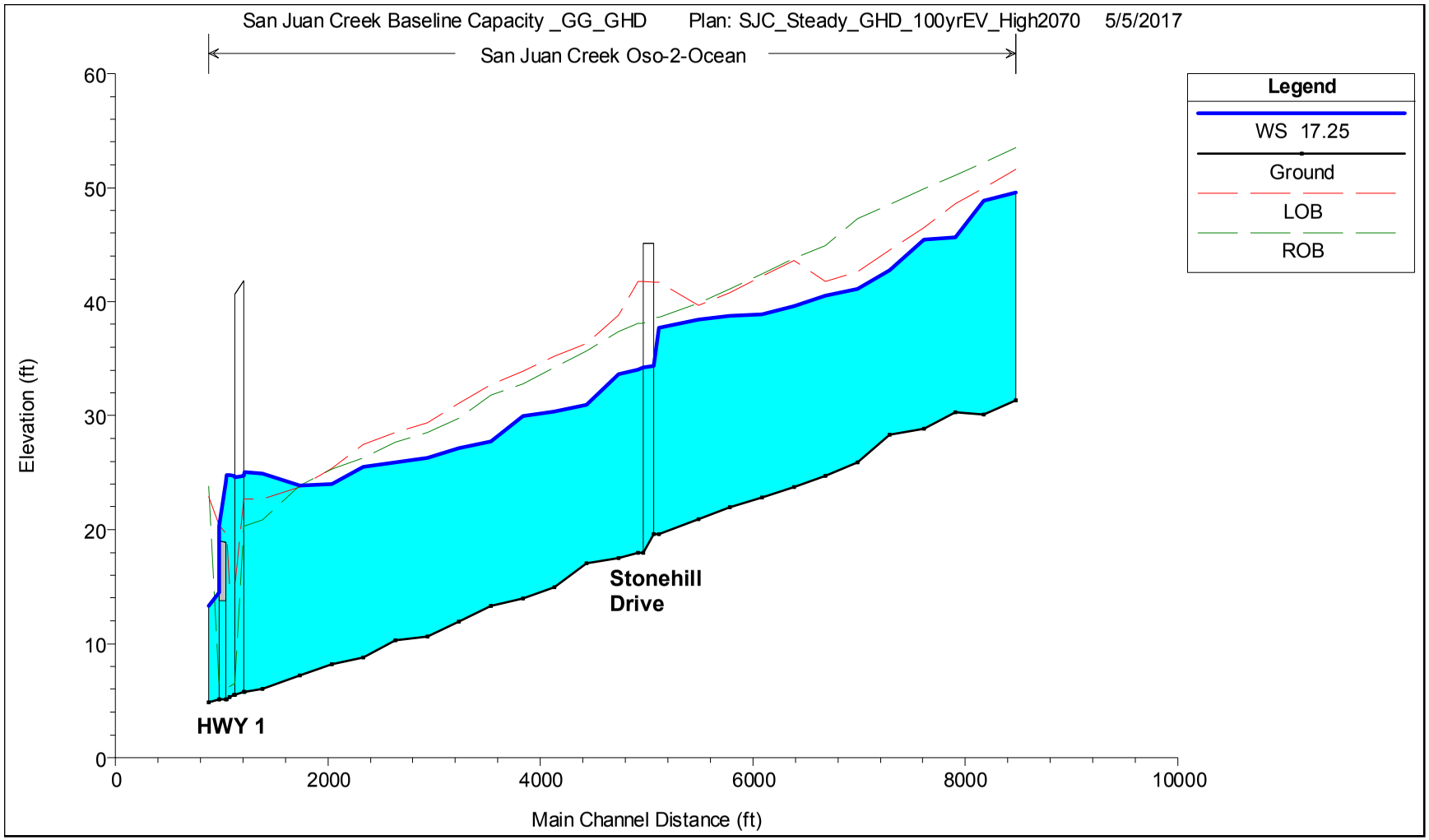


Figure 3.3 – San Juan Creek maximum water surface profile, 100-year event.

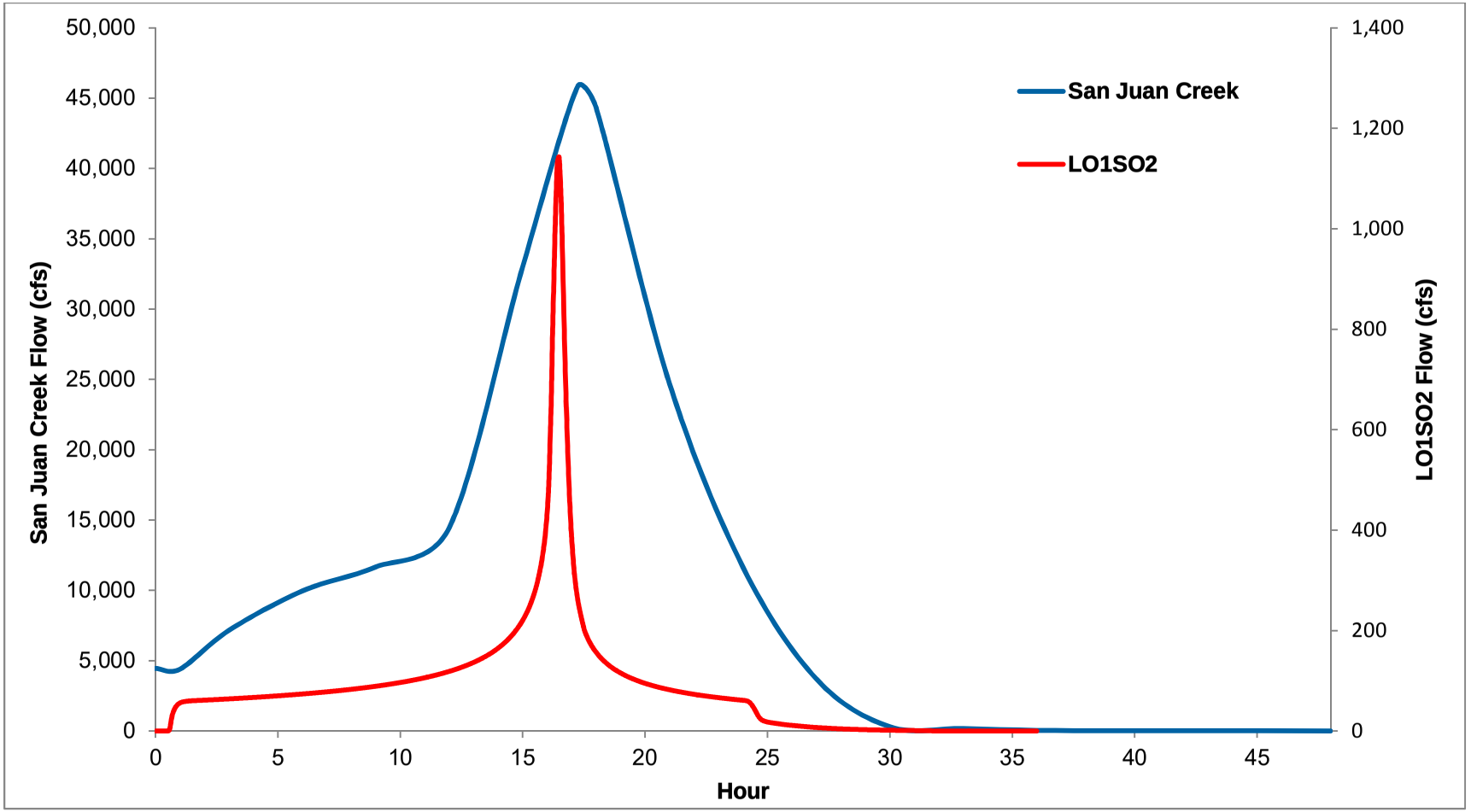
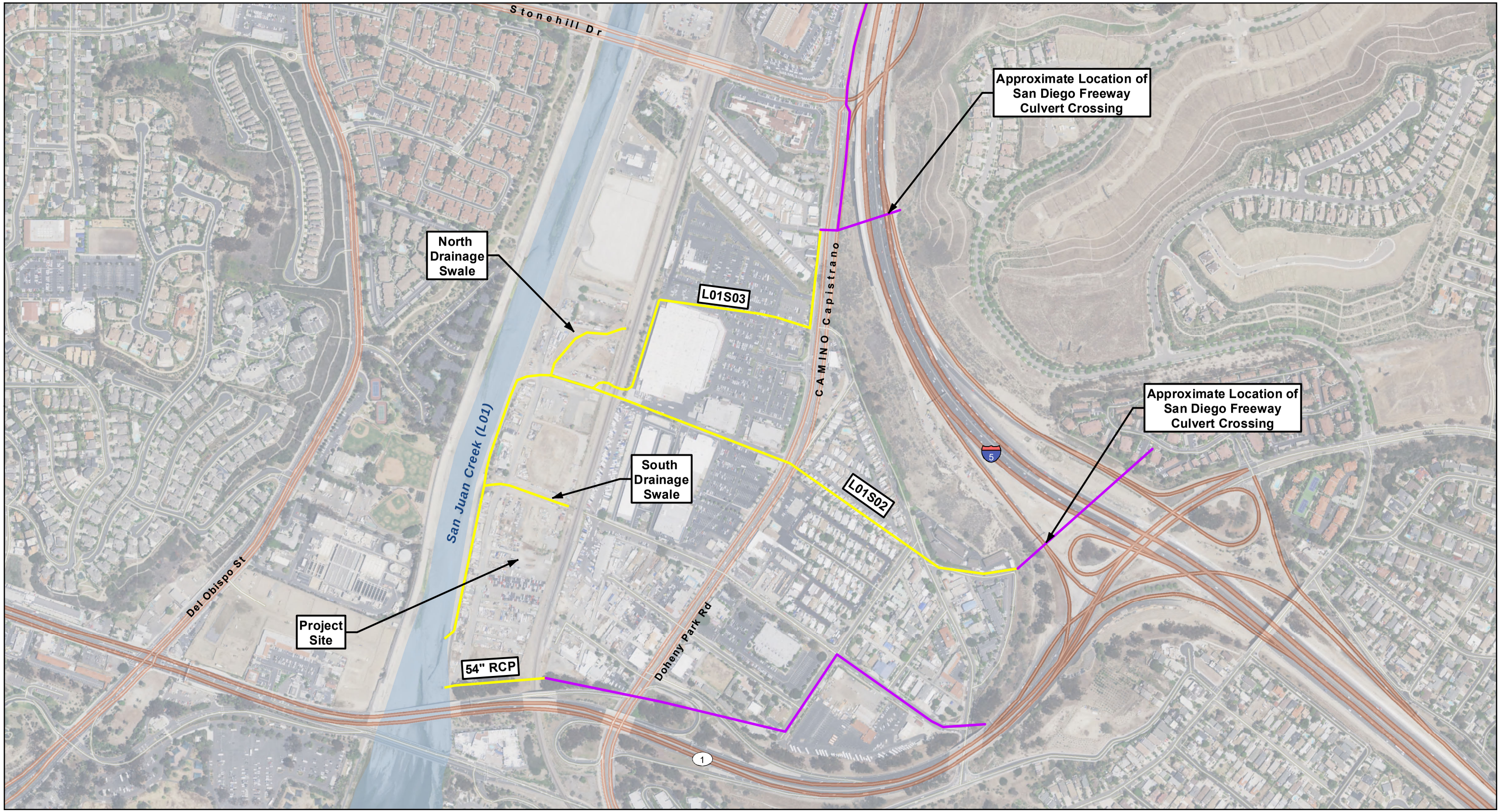


Figure 3.4 – San Juan Creek and LO1SO2 hydrograph comparison, 100-year event.



Paper Size ANSI B

0 100 200 300 400 500

Feet

Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane California VI FIPS 0406 Feet

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LEGEND

Storm Water Conduits (in model)

Storm Water Conduits (not in model)

South Coast Water District
Doheny Desalination Project Drainage Analysis

Job Number	11125157
Revision	A
Date	08 May 2017

Existing Infrastructure Map

Figure 3.5

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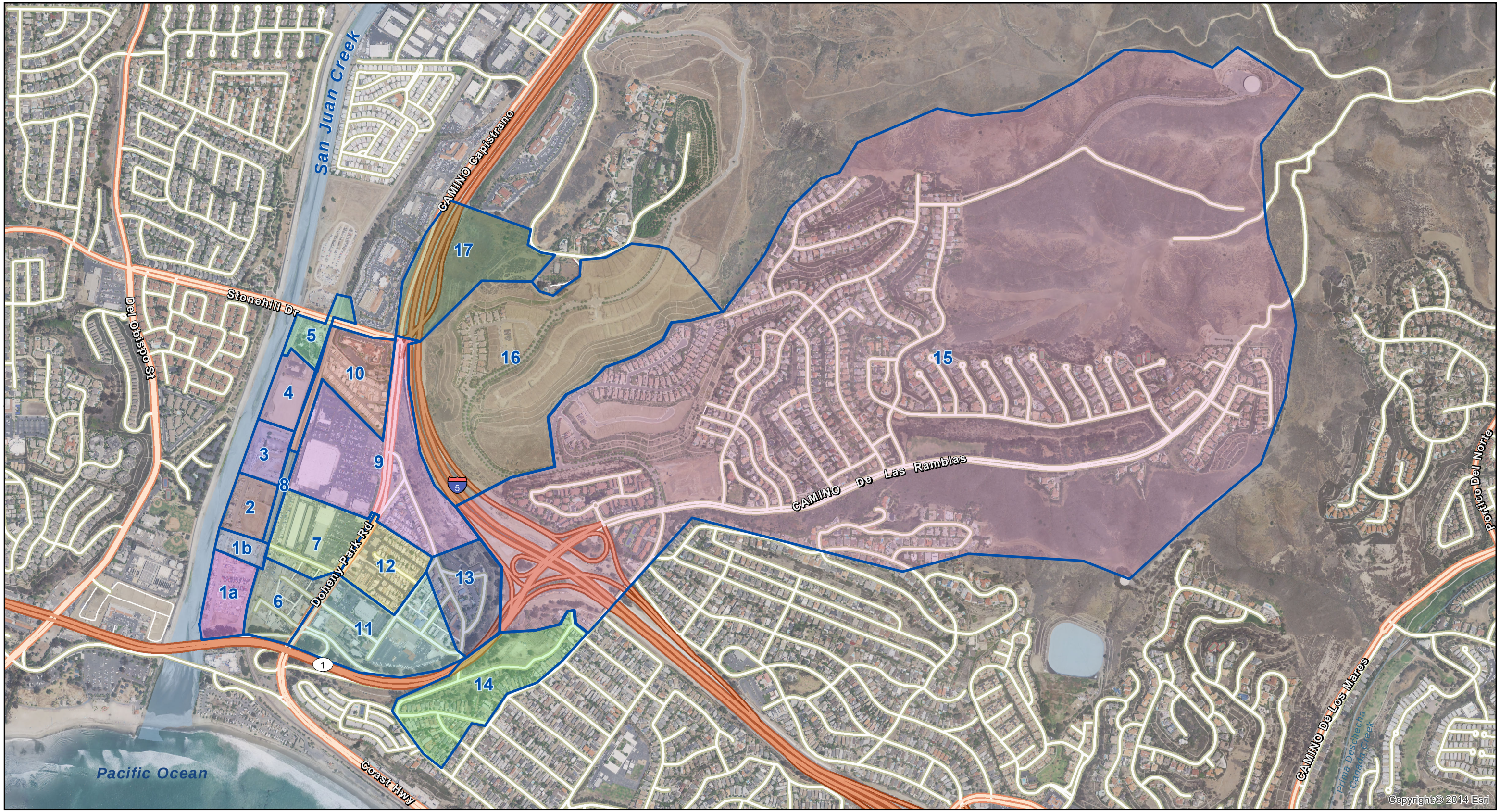

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Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane California VI FIPS 0406 Feet

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LEGEND

Doheny Watersheds

15 Watershed ID

South Coast Water District
Doheny Desalination Project Drainage Analysis

Job Number	11125157
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Doheny Watersheds Overview Map

Figure 3.6

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Storm Water Conduits (in model)

Maximum Flooding Depth

2-6" 6-12" 1-2' 2-3' >3'

Paper Size ANSI B

0 100 200 300

Feet

Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane California VI FIPS 0406 Feet

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South Coast Water District
Doheny Desalination Project Drainage Analysis

Flood Inundation Map, 25-Year Event, Existing Condition

Job Number	11125157
Revision	A
Date	09 May 2017

Figure 3.7

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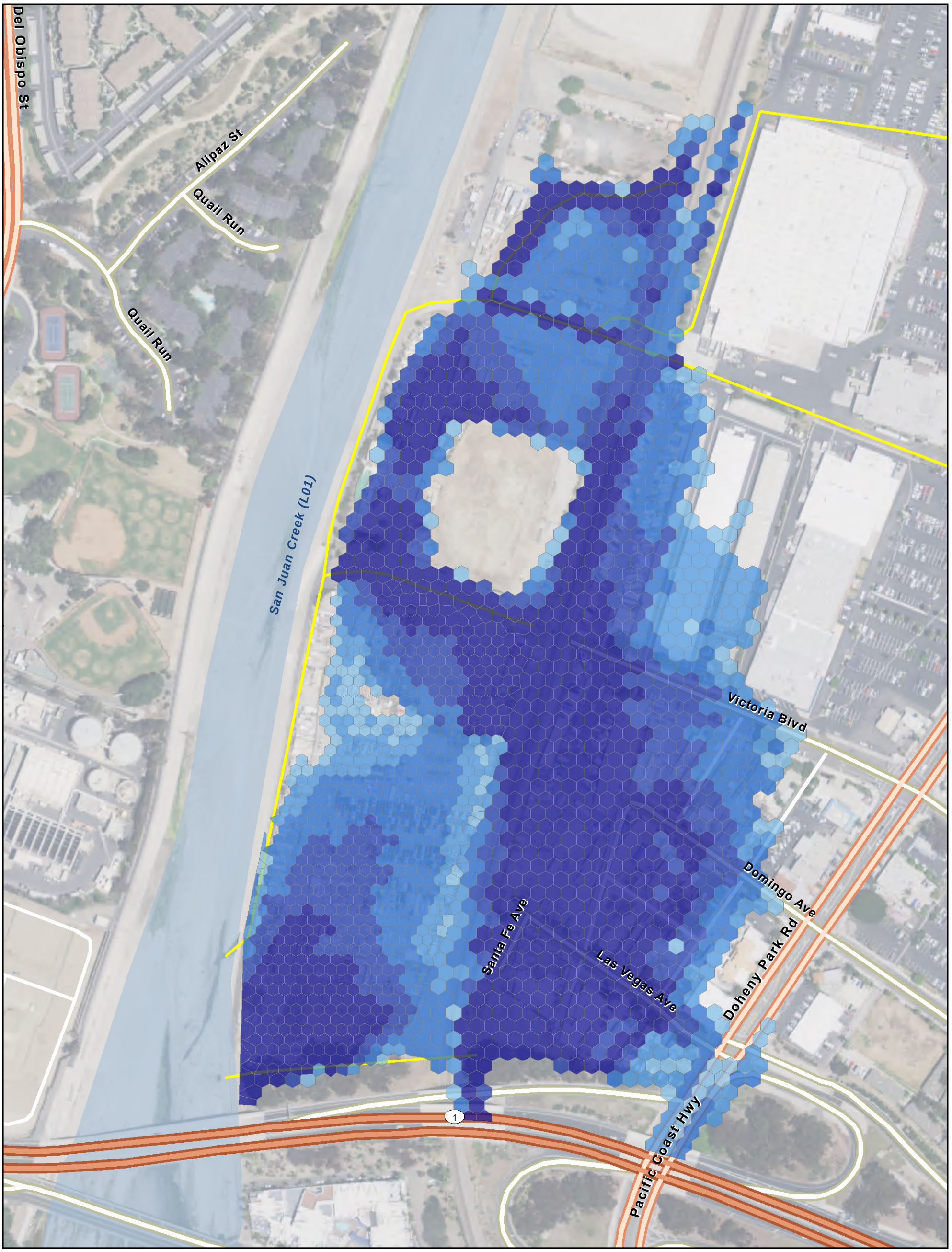

Storm Water Conduits (in model)

Maximum Flooding Depth

2-6" 6-12" 1-2' 2-3' >3'

Paper Size ANSI B

0 100 200 300

Feet

Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane California VI FIPS 0406 Feet

N

South Coast Water District

Doheny Desalination Project Drainage Analysis

Flood Inundation Map, 100-Year Event, Existing Condition

Job Number	11125157
Revision	A
Date	09 May 2017

Figure 3.8

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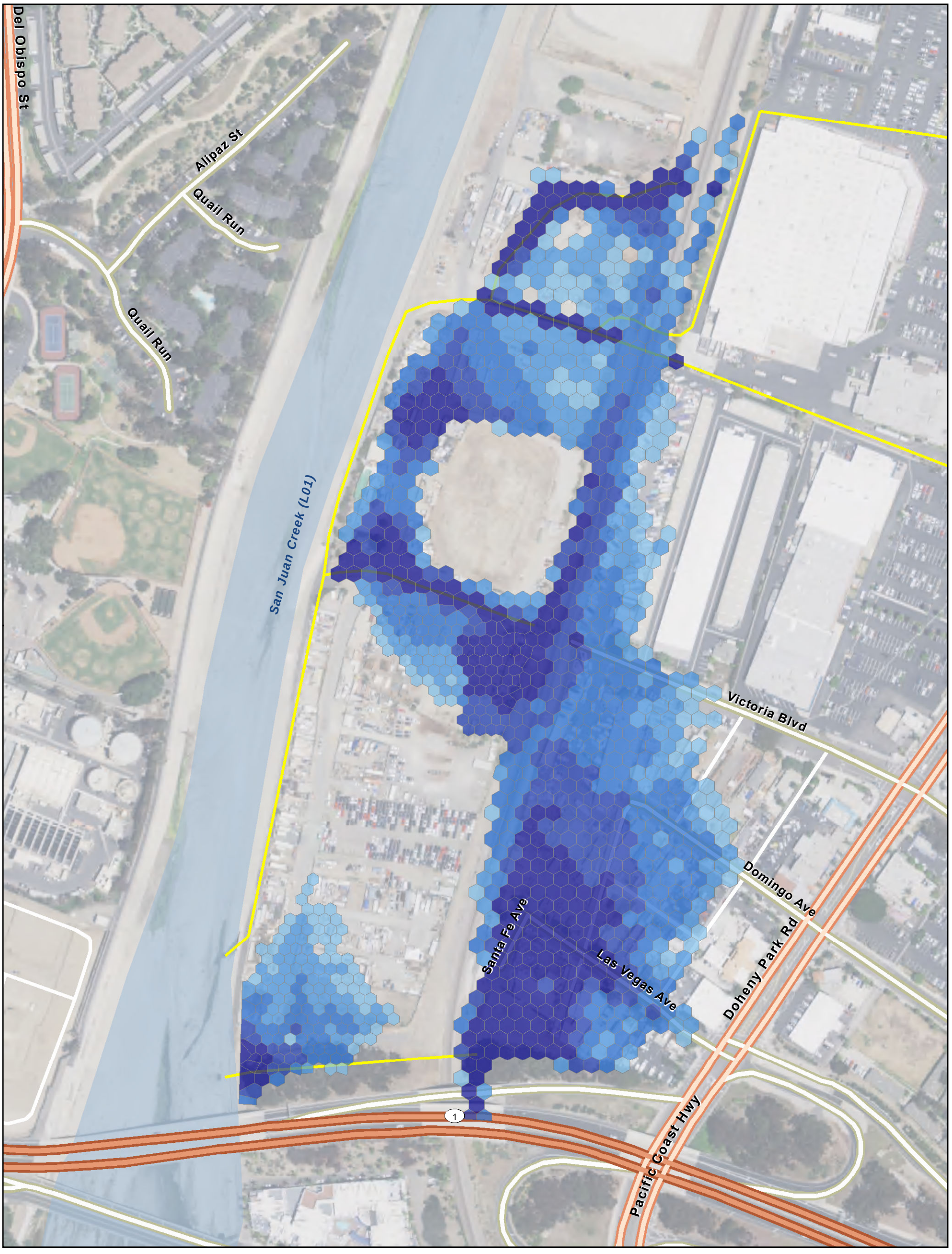

Storm Water Conduits (in model)

Maximum Flooding Depth

2-6" 6-12" 1-2' 2-3' >3'

Paper Size ANSI B

0 100 200 300

Feet

Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane California VI FIPS 0406 Feet

N

South Coast Water District
Doheny Desalination Project Drainage Analysis

Flood Inundation Map, 100-Year Event, Existing Condition without San Juan Creek overtopping

Job Number 11125157
Revision A
Date 09 May 2017

Figure 3.9

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Data source: Aerial: NAIP, 2016; ESRI Street Map; GHD: elevation model. Created by:jclark2

Appendix A:

Historical Shoreline Assessment

Memorandum

To:	Mark Donovan, PE	Ref. No.:	11140040
From:	Jeffrey Doucette, Ph.D., P.Geo.(limited)	Tel:	19058144355
cc:	Raymond Wong, PhD, PE, LEED AP, CPESC		
Subject:	**Doheny Desalinization Plant Historical Shoreline Assessment**		

An historical aerial photographic assessment was conducted to complement the beach erosion assessment provided in the *Coastal Hazards Analysis for the Doheny Desalination Project* by Jenkins (2017). Jenkins (2017) reviewed historical beach surveys at Doheny State Beach from 2001 to 2007. He concluded that the beach was stable due to the relatively small seasonal variation in beach widths. The site was located at San Juan Creek which was a significant sediment source providing approximately 51,000 metric tons of beach grade sand to the site annually (Jenkins, 2017). This large sediment source allows the beach to maintain equilibrium profile adjustments through high El Nino and large seasonal cycles. Jenkins (2017) concluded that there was adequate sand cover to assume that the profile would shift according to Bruun's Rule due to sea level rise. This means that the profile will shift landward, however it will maintain its existing equilibrium shape. Runup elevation will therefore not be impacted by changes to the shoreline due to sea level rise.

An historical aerial photographic assessment was conducted to confirm the observations that the beach width has been relatively stable by using older aerial photographs to determine if there has been any systematic shoreline change over time. The shoreline was traced in aerial photographs for 1975, 1980, 2004, and 2011 from the U.S. Geological Survey (USGS) and 2009, 2015, and 2016 from Google Earth Pro.

The shoreline was defined in each aerial photograph as the top of berm. Interpretation of the top of berm location within the aerial photographs was not exact given the difficulty in identifying this change in slope within the two-dimensional photographs. Generally changes in shading on the beach were used to determine the break in slope. Interpretation of the top of berm was spot checked using LIDAR data from NOAA from 2004 and 2014. All traces of the berm location were overlaid and shown in **Attachment A**.

Most of the traces of berm location fall within an approximately 60 m wide band over the approximate 180 m to 300 m wide subaerial beach. Notable exceptions were the location of the berm in 1980 and 2011. Two distinct berm locations were noted in 1980. One landward berm at a similar location to other years, and one berm much further seaward than any other years assessed. The seaward berm was likely due to a large flood event within San Juan Creek which discharged a large amount of sediment into the nearshore. This material was redistributed parallel to the shore by the incident waves. The landward berm was likely the location of the beach face before the large discharge event. A similar event was evident in 2011 where a

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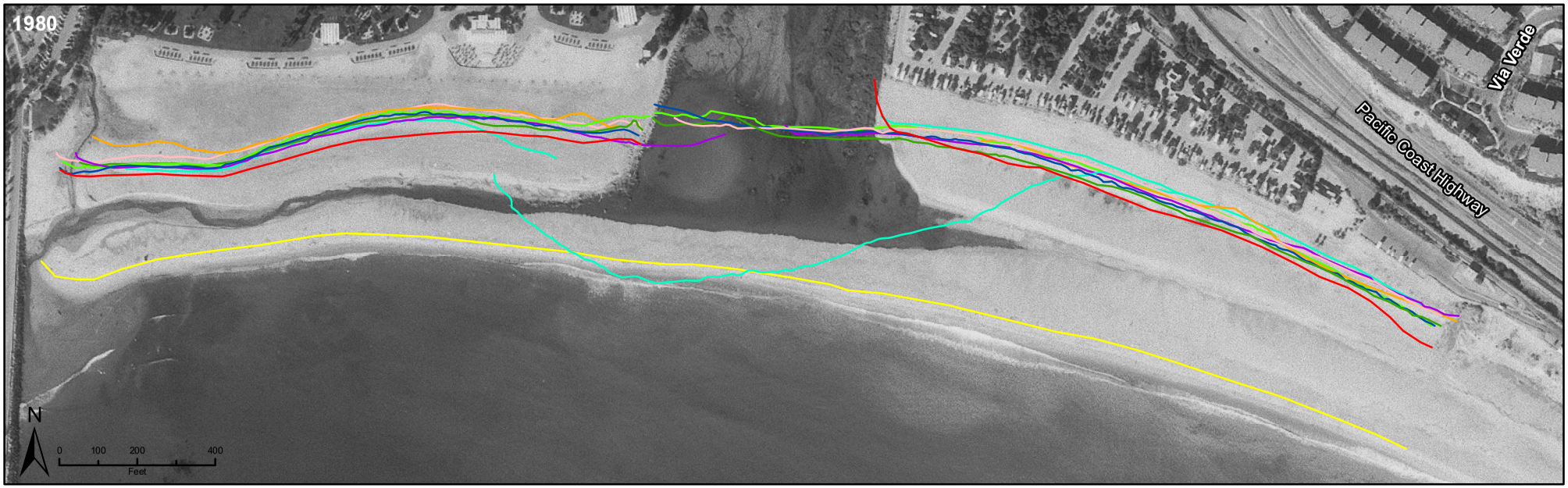
large bar/delta feature was protruding from the shoreline. This feature would be redistributed parallel to the shoreline over time by the incident waves.

If we discount the seaward 1980 berm, the 1975 image had the widest beach and more recent images had narrower beaches. However there was no systematic trend in beach narrowing as the position of the berm moved both seaward and landward from year to year. Additional seasonal variability would be expected based on the typical California beach evolution between accretional summer profiles and erosional winter profiles. Other impacts to shoreline variability could be due to beach nourishment projects.

Overall Doheny State Beach generally appeared to be in an equilibrium condition based on the historical aerial photographs used on the assessment. Sediment input from the San Juan Creek and potential beach nourishment projects in the past have maintained a relatively stable shoreline. A historical erosion rate could not be determined to apply to future adjustments due to sea level rise. Application of the Bruun Rule as described in Jenkins (2017) is recommended to account for erosion impacts of sea level rise.



Legend

Historical Berm Locations

- 1975
- 1980 (Fore Berm)
- 1980 (Back Berm)
- 2004
- 2009
- 2010
- 2011
- 2015
- 2016

Berm Locations: GHD, 2016; Imagery: USGS, 1980 & Google Earth Pro, 2016.

Historical Shoreline Assessment

Doheny State Park,
California

ATTACHMENT A	
DATE: MARCH 2017	
PROJECT: 11140040	
DRAWN BY: J.D., R.G.	

GHD

Appendix **B**:

Coastal Hazards Analysis

Coastal Hazards Analysis for the Doheny Desalination Project

By Scott A. Jenkins, Ph.D.

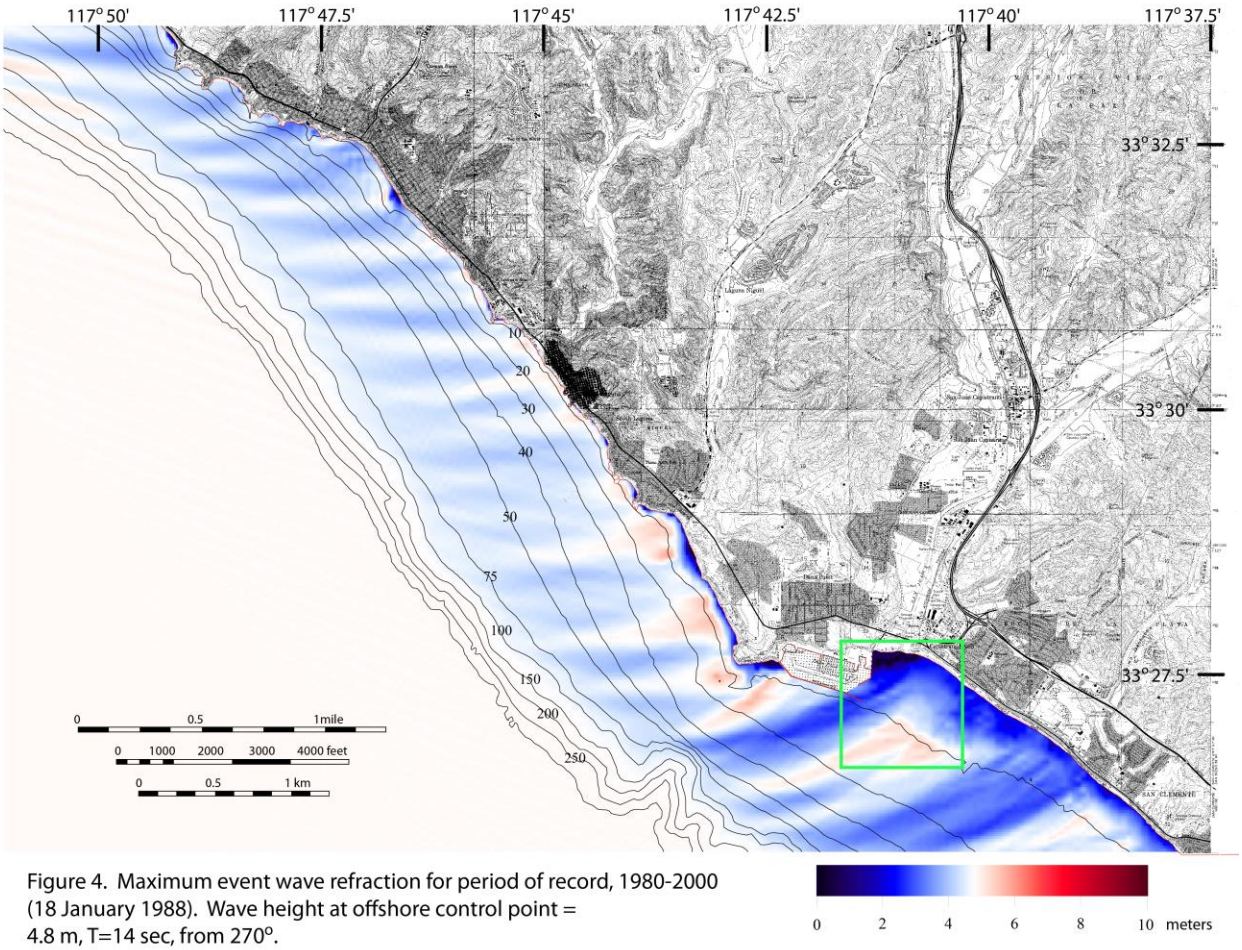


Figure 4. Maximum event wave refraction for period of record, 1980-2000 (18 January 1988). Wave height at offshore control point = 4.8 m, T=14 sec, from 270°.

Submitted by:
Scott A. Jenkins, Ph.D.
Technical Manager, Coastal Sciences & Engineering
Michael Baker International

Submitted to:
Mark Donovan, PE
Senior Process Engineer
GHD
175 Technology Drive, Suite 200
Irvine CA 92618 USA

Draft Final: 15 February 2017

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EXECUTIVE SUMMARY:

The *Coastal Evolution Model* (CEM) developed at the Scripps Institution of Oceanogrphy was used to evaluate Appendix-B requirements of the *California Coastal Commision Sea Level Rise Policy Guidance* document (CCC, 2015) for a sea level rise/coastal hazards analysis of the Doheny Desalination Project (DDP). The Coastal Evolution Model is public domain and available from the University of California Digital Library at: http://repositories.cdlib.org/sio/techreport/58/. The Coastal Evolution Model employs algorithms consistent with the U.S. Army Corps of Engineers *Coastal Engineering Manual*, (USACE, 2006), but employs the latest generation equilibrium beach profile algorithms from Jenkins and Inman (2006) that provide 3-dimensional predictive and mapping capability of the wave run-up field, beach erosion and shoreline recession under the effects of wave climate variability, climate cycles and sea level rise. The CEM input files were populated with National Ocean Survey digital bathymetry in the offshore domain; beach profiles sediment gran size measurements by the U.S. Army Corps of Engineers, Coastal Environments and Coastal Frontiers; long-term wave data from the Coastal Data Information Program; long-term ocean water level measurements by the National Atmospheric and Oceanic Administration; and stream flow and sediment flux for the San Juan Creek from the United States Geological Survey and the Federal Emergency Management Agency. Sea level rise projections used in this study were based on the *best fit equation* from Appendix-B of the *California Coastal Commision Sea Level Rise Policy Guidance* document for a 50 year project planning horizon (year 2070) and for a *critical infrastructure* planning horizon (year 2100). Critical project infrastructure subject to potential flooding by extreme event waves concurrent with extreme ocean water levels and sea level rise include: ***Wellhead 1***, elevation 17 ft.NAVD, at 33 27 43.99834 N, 117 41 13.25899 W; ***Wellhead 2***, elevation 17 ft. NAVD, at 33 27 44.16980 N, 117 41 06.01901 W; ***Wellhead 3***, elevation 18 ft. NAVD at 33 27 44.07064 N, 117 40 53.96544 W; ***Well Pump Electrical (North)***, at elevation 18 ft. NAVD at 33 27 48.54582 N, 117 41 02.09570 W; and **Well Pump Electrical (South)**, at elevation 21 ft. NAVD at 33 27 47.44890 N, 117 40 54.92019 W; (see Figure ES-1).

Appendix-B of the *California Coastal Commision Sea Level Rise Policy Guidance* document (CCC, 2015) provides no specific guidance on the redline frequency for flooding or inundation. In the absence of such guidance we have adopted Federal Emergency Management Agency (FEMA) standards for flooding frequency and set redline planning frequency at the 100 year event (1% probability of recurrence). The 100 year wave event was the two day storm of 17-18 January, 1988, which produced deep water significant wave heights off Doheny State Beach reaching 15.5 ft., approaching the beach from 270^0 with 15 second significant wave periods. An analysis of extremal total water levels, (TWL's), based on the occurrence of extreme waves concurrent with extreme ocean water levels, is summarized in Table ES-1. Inspection of Table ES-1 reveals that all the beach front facilities for the Doheny Desalination Project (Figure ES-1) are safe from flooding or inundation at present sea levels by extreme event waves concurrent with extreme ocean water levels, even for event return periods as long as 500 yr. In fact, all these facilities are safe from flooding by the 100-year event in environments of the low-range predictions for 2070 and 2100 sea levels. However, once we admit to the high-range sea level rise projections, a number of the beach front facilities for the Doheny Desalination Project will suffer some flooding and overtopping to varying degrees.

Figure ES-1: Critical shore-front infrastructure locations for the Doheny Desalination Project

For the high-range 2070 sea level projections, the 100-yr total water level events reach $TWL(100) = 18.0$ ft. NAVD for the accreted beach conditions and $TWL(100) = 16.3$ ft. NAVD for the eroded beach conditions. This indicates that well heads #1 and #2 are vulnerable to flooding by any event that exceeds a return period of 36 years; and could be flooded by as much as 1.0 ft of overtopping during accreted beach conditions, with overtopping rates of as much as $Q' = 0.04$ cfs/ft. Flooding and overtopping of well heads #1 and #2 under eroded beach conditions would not happen until events exceeding 225 year return periods when extreme waves happen concurrently with extreme ocean water levels under high-range 2070 sea level projections. Total water levels for eroded beach conditions are always less, because these beaches have flatter slopes and are more dissipative of wave set-up and run-up than the steeper accreted beaches.

For the high-range 2100 sea level projections, Table ES-1 indicates the 100 year total water level events reach $TWL(100) = 20.3$ ft. NAVD for the accreted beach conditions and $TWL(100) = 18.5$ ft. NAVD for the eroded beach conditions. Consequently all beach front facilities for the Doheny Desalination Project, except well pump electrical south, are vulnerable to flooding by the 100-year event. Well heads #1 and #2 would be flooded by even a 3-year event under accreted beach conditions and by a 16-year event under eroded beach conditions. Well heads #1 and #2 are vulnerable to flooding by as much as 3.3 ft of overtopping during a 100 year event, which results in an overtopping rate of $Q' = 0.26$ cfs/ft under conditions when extreme waves happen concurrently with extreme ocean water levels that are superimposed

Table ES-1: Extremal Total Water Level (*TWL) and Overtopping Rates (Q') Analysis

	Well Head #1 Elevation = 17 ft. NAVD	Well Head #2 Elevation = 17 ft. NAVD	Well Head #3 Elevation = 18 ft. NAVD	Well Pump Electrical South Elevation = 21 ft. NAVD	Well pump Electrical North Elevation = 18 ft. NAVD
*TWL(100) Present Sea Level (eroded/accreted)	13.1/14.8 ft. NAVD status = dry	13.1/14.8 ft. NAVD status = dry	13.1/14.8 ft. NAVD status = dry	13.1/14.8 ft. NAVD status = dry	13.1/14.8 ft. NAVD status = dry
*Q'(100) Present Sea Level (eroded/accreted)	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.
*TWL(100) 2070 Sea Level Low Range Projection (eroded/accreted)	13.8/15.5 ft. NAVD status = dry	13.8/15.5 ft. NAVD status = dry	13.8/15.5 ft. NAVD status = dry	13.8/15.5 ft. NAVD status = dry	13.8/15.5 ft. NAVD status = dry
*Q'(100) 2070 Sea Level Low Range Projection (eroded/accreted)	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.
*TWL(100) 2070 Sea Level High Range Projection (eroded/accreted)	16.3/18.0 ft. NAVD status = flooded	16.3/18.0 ft. NAVD status = flooded	16.3/18.0 ft. NAVD status = dry	16.3/18.0 ft. NAVD status = dry	16.3/18.0 ft. NAVD status = dry
*Q'(100) 2070 Sea Level High Range Projection (eroded/accreted)	0.0/0.04 cfs/ft.	0.0/0.04 cfs/ft.	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.
*TWL(100) 2100 Sea Level Low Range Projection (eroded/accreted)	14.5/16.1 ft. NAVD status = dry	14.5/16.1 ft. NAVD status = dry	14.5/16.1 ft. NAVD status = dry	14.5/16.1 ft. NAVD status = dry	14.5/16.1 ft. NAVD status = dry
*Q'(100) 2100 Sea Level Low Range Projection (eroded/accreted)	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.
*TWL(100) @ 2100 Sea Level High Range Projection (eroded/accreted)	18.5/20.3 ft. NAVD status = flooded	18.5/20.3 ft. NAVD status = flooded	18.5/20.3 ft. NAVD status = flooded	18.5/20.3 ft. NAVD status = dry	18.5/20.3 ft. NAVD status = flooded
*Q'(100) 2100 Sea Level High Range Projection (eroded/accreted)	0.08/0.26 cfs/ft.	0.08/0.26 cfs/ft.	0.01/0.15 cfs/ft.	0.0/0.0 cfs/ft.	0.01/0.15 cfs/ft.

*Evaluated for the 100-yr return period

on the high range of 2100 sea levels. Similarly well head #3 and well pump electrical north could be flooded by 2.3 ft of overtopping, resulting in an overtopping rate of $Q' = 0.15$ cfs/ft when extreme waves happen concurrently with extreme ocean water levels that are superimposed on the high range of 2100 sea levels. Well pump electrical south is safe from flooding by any event less than 265-year event in environments of the high-range predictions for 2100 sea levels. Because all of the shore facilities in Table ES-1 and Figure ES-1 are in close proximity to one another,

(clustered around the mouth of the San Juan Creek with nothing to support water surface gradients between each installation), there are insignificant differences in extremal total water levels occurring at each facility for any given event.

Tsunami induced erosion, runup, and inundation were analyzed for the Doheny State Beach bottom profiles and shore-side facilities associated with the Doheny Desalination Project for present and future sea levels according to low and high range sea level rise predictions. The tsunami event scenario is based on a 2m high solitary wave approaching Doheny Beach from 165 degrees true, as could be anticipated for a catastrophic tsunami event arising from a major landside on the east side of San Clemente Island. The local refraction/diffraction pattern from the solitary wave reveals the tsunami wave height begins to increase at 50 m of water depth due to shoaling, and reaches 6m of height before breaking along the shores of Doheny Beach. Because the tsunami wave begins shoaling in much deeper water than typical storm-induced waves, it causes seabed scour and erosion to occur out to very deep water depths. Therefore all run-up and total water level solutions are based eroded beach profile conditions. Tsunami TWL inundation calculations are summarized Tables ES-2 and indicate that all of the shore facilities of the Doheny Desalination Project are above tsunami inundation levels at present sea level; as well as safe for the low range projections for sea level rise out to year 2070 or 2100. However, well heads #1, #2, #3 and well pump electrical north would suffer some degree of tsunami overtopping if concurrent with the high-range predictions of sea level at year 2070 or 2100. Only well pump electrical south is safe from flooding and overtopping by tsunami for all sea level rise scenarios anticipated out to year 2100. These findings are consistent with the FEMA tsunami flood map which show that all of the Doheny Beach/San Juan Creek corridor extending several miles inland will be inundate by a shoaling tsunami solitary wave. Again, all of the shore facilities in Figure ES-1 are in close proximity to one another with nothing to support water surface gradients between each installation. Therefore, no significant differences occur in tsunami total water levels at each facility for any given sea level.

Finally, we provide guidance for how to apply the extreme ocean water level data developed in this study to the compound frequency calculation of the 1% (100-year) event used in the design calculations of back-water effects for project facilities to be located along the lower reaches of the San Juan Creek. We develop *One-Percent Compound Frequency Curves* for present and future sea levels using NOAA ocean water level measurements superimposed on sea level rise predictions for 2070 and 2100. The *One-Percent Compound Frequency Curves* allow the project engineers to determine the appropriate combinations of flood hydrographs and ocean water levels at the mouth of San Juan Creek, such that the modeled backwater effects will represent a 100-year event. This result in turn will provide design guidance for the required elevations for project facilities along the lower San Juan Creek in order to avoid flooding and inundation of those facilities during river floods that happen concurrently with extreme ocean water levels.

Table ES-2: Tsunami Total Water Level (*TWL*) and Overtopping Rates (Q') Analysis

	Well Head #1 Elevation = 17 ft. NAVD	Well Head #2 Elevation = 17 ft. NAVD	Well Head #3 Elevation = 18 ft. NAVD	Well Pump Electrical South Elevation = 21 ft. NAVD	Well pump Electrical North Elevation = 18 ft. NAVD
TWL Present Sea Level	15.22 ft. NAVD status = dry	15.22 ft. NAVD status = dry	15.22 ft. NAVD status = dry	15.22 ft. NAVD status = dry	15.22 ft. NAVD status = dry
Q' Present Sea Level	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.
TWL 2070 Sea Level Low Range Projection	15.92 ft. NAVD status = dry	15.92 ft. NAVD status = dry	15.92 ft. NAVD status = dry	15.92 ft. NAVD status = dry	15.92 ft. NAVD status = dry
Q' 2070 Sea Level Low Range Projection	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.
TWL 2070 Sea Level High Range Projection	18.43 ft. NAVD status = flooded	18.43 ft. NAVD status = flooded	18.43 ft. NAVD status = flooded	18.43 ft. NAVD status = dry	18.43 ft. NAVD status = flooded
Q' 2070 Sea Level High Range Projection	0.07 cfs/ft.	0.07 cfs/ft.	0.01 cfs/ft.	0.0/0.0 cfs/ft.	0.01 cfs/ft.
TWL 2100 Sea Level Low Range Projection	16.59 ft. NAVD status = dry	16.59 ft. NAVD status = dry	16.59 ft. NAVD status = dry	16.59 ft. NAVD status = dry	16.59 ft. NAVD status = dry
Q' 2100 Sea Level Low Range Projection	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.
TWL @ 2100 Sea Level High Range Projection	20.71 ft. NAVD status = flooded	20.71 ft. NAVD status = flooded	20.71 ft. NAVD status = flooded	20.71 ft. NAVD status = dry	20.71 ft. NAVD status = flooded
Q' 2100 Sea Level High Range Projection	0.31 cfs/ft.	0.31 cfs/ft.	0.19 cfs/ft.	0.0/0.0 cfs/ft.	0.19 cfs/ft.

*Evaluated for 2m high tsunami deep water wave height approaching Doheny State Beach from 165 degrees true

Coastal Hazards Analysis for the Doheny Desalination Project

by Scott A. Jenkins, Ph.D.

1) Introduction: The source water for the Doheny Desalination Project will be drawn from an array of slant wells that extract pore water from marine sediments that were deposited in a paleo-valley cut by the San Juan Creek across the continental shelf during the previous low-stand of sea level (ca. 18,000 yr B.P.; Inman et al, 2003). With the subsequent rise in sea level during the Flandrian Transgression, the paleo valley in-filled with fluvial sediments from the San Juan Creek and littoral sediments from the adjacent nearshore, (Jenkins and Wasyl, 2005), leaving only the expression of a modern sand delta at the mouth of the San Juan Creek (denoted by light brown contours in Figure 1). Thus a large formation of marine valley-fill sediments is available seaward of the mouth of the San Juan Creek to provide sub-bottom filtration of ocean source water harvested by slant wells. Desalination of this source water by reverse osmosis (RO) is expected to present several possible discharge scenarios for disposal of the concentrated seawater by-product (brine), depending upon the production rate and recovery ratio. The Doheny Desalination Project will blend brine with treated wastewater and will discharge the combined effluent through the San Juan Creek Ocean Outfall (SJCOO). The SJCOO extends seaward 10,334 ft. from the mouth of the San Juan Creek, (Figure 1), in a 1,488 ft. total length L-shaped linear diffuser with a 216 ft long shoreline-normal section and a right angle dog-leg with a 1,272 ft diffuser section employing 125 discharge ports. The diffuser discharges at local depths of 95 ft MSL (29 m MSL), at a distance of roughly 4,415 ft (1,346 m) from the edge of the continental shelf.

The coastal hazards analysis evaluates potential impacts of combinations of extreme waves and ocean water levels on these structures at both present and future sealevels; and conversely, potential impacts of these structures on nearshore erosion, sediment transport and shoreline stability. The study includes assimilation of long-term wave climate data bases to evaluate inundation by extreme wave and tsunami run-up that may affect stability and operations of subsurface desalination plant intake structures, (slant wells), as well as supporting shore facilities. The essential requirements for this study, as stated in the California Coastal Commission guidance document for Coastal Development Permits Applications are: 1) quantify the magnitude and extent to which the subsurface intake and associated shore zone structures could be subject to sea level rise, erosion, wave attack or wave run-up due to wave refraction/diffraction over local nearshore and shelf bathymetry over a projected lifespan; 2) quantify the of the frequency of such events; and 3) evaluate the consequences of such events should they be determined significant, and pose remedial options for avoiding such consequences. In evaluating these potential hazards for this study, the study will also: 4) evaluate potential impacts to the adjacent shoreline due to sea level rise, erosion and wave diffraction and reflection from the subsurface intake structures. The latter requirement entails a sediment budget and transport analysis of both the near- and far-field of the study area.

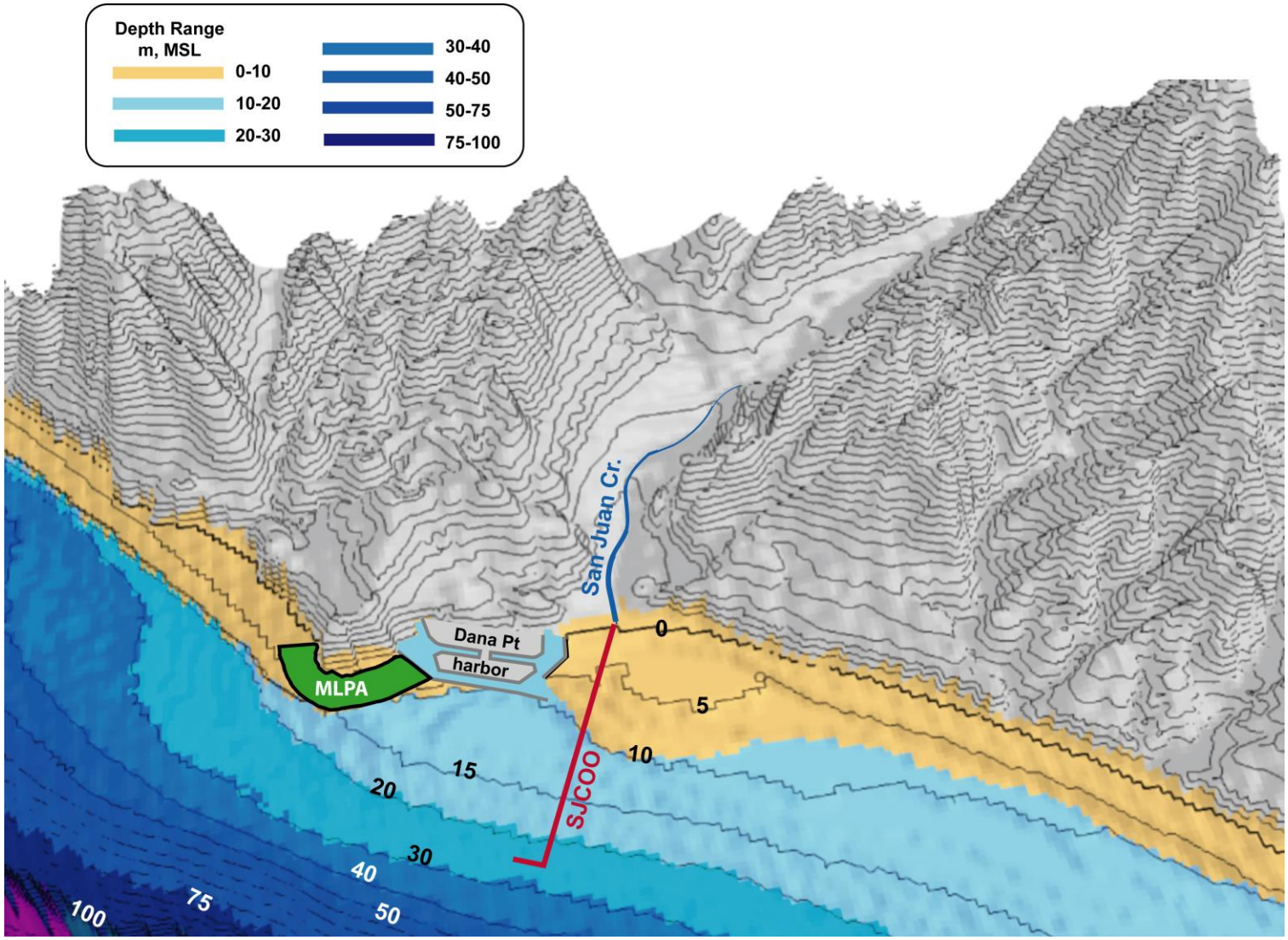


Figure 1.1: Project site map in GIS. Bathymetry contours in meters MSL. Data from GEODAS 3 arc-second database

2) Regulatory Requirements:

Appendix-B of the *California Coastal Commision Sea Level Rise Guidance Policy Guidance* document (CCC, 2015) provides specific guidance on the analysis protocals of a sea level rise/coastal hazards analysis. These are

Step 1 – Develop temporally- and spatially-appropriate sea level rise projections

Two methods are recommended for establishing a projection value for a specific year: 1) conduct a linear interpolation100, or 2) use the "best fit" equations that are provided below. At this time, both are acceptable for Coastal Commission purposes

Step 2 – Determine tidal range and future inundation

This step requires the determination the future intersections of mean sea level or other tidal datums with the shoreline. Erosion must be accounted for in these determinations.

Step 3 – Determine still water changes from surge, El Niño events, and PDOs

Estimates of surge, El Niño, and PDO water elevation changes are to be developed primarily from historical records. There are no state-wide resources for this information,

Step 4 – Estimate beach, bluff, and dune change from erosion

There is no single specific accepted method for predicting future beach erosion. At a minimum, projects should assume that there will be inundation of dry beach and that the beach will continue to experience seasonal and inter-annual changes comparable to historical amounts. When there is a range of erosion rates from historical trends, the high rate should be used to project future erosion with rising sea level conditions (unless future erosion will encounter more resistant materials, in which case lower erosion rates may be used). For beaches that have had a relatively stable long-term width, it would be prudent to also consider the potential for greater variability or even erosion as a future condition.

Step 5 – Determine wave, storm wave, wave runup, and flooding conditions

Wave impacts to the coast, to coastal bluff erosion and inland development, should be analyzed under the conditions most likely to cause harm. Those conditions normally occur in winter when most of the sand has moved offshore leaving only a reduced dry sand beach to dissipate wave energy (this seasonal change in beach width is often referred to as short-term or seasonal erosion). On beaches that will experience long-term erosion, trends expected to occur over the entire expected life of the development should also be considered. Since water levels will increase over the life of the development due to rising sea level, the development should be examined for the amount of sea level rise (or a scenario of sea level rise conditions) that is likely to occur throughout the expected life of the development. Then, the wave impact analysis should examine the consequences of a 100-year design storm event using the combined water levels that are likely to occur with high water conditions and sea level rise, as well as a long-term and seasonally eroded beach.

Step 6 – Examine potential flooding from extreme events

Extreme events, by their very nature, are those beyond the normal events that are considered in most shoreline studies. Tsunami should be among the extreme events evaluated. Planning and project analysis need to consider and anticipate the consequences of these outlier events. Projections of potential flooding from extreme events are the principle outcome of Step-6.

3) Temporally- and Spatially-Appropriate Sea Level Rise Projections

This section adresses Step-1 of a sea level rise/coastal hazards analysis as outlined in Section 2. Appendix-B of the *California Coastal Commision Sea Level Rise Guidance Policy Guidance* document (CCC, 2015) requires that coastal hazards analyses consider sea level rise impacts over the project lifetime. Precedence from antecedant desalination projects have typically used project lifspans of 50 years (SEIR, 2010). With a potential start date of 2020, a fifty year project life for the Doheney Desalination Project (DDP) would extend the sea level rise analysis out to 2070. However, the Chapte 6 guidance of (CCC, 2015) recommends 100 year planning horizons for critical infrastructure projects. The difficulty with implementing this recommendation follows from the fact that the NRC sea level projections (NRC, 2012) on which the the Appendix-B guidance is based, do not extend beyond year 2100. This makes an analysis of sea level impacts for a 100 year project life not possible with the Doheny Desalination Project. Furthermore, the Appendix-B guidance admits, "The uncertainty associated with any projections for sea level grows significantly as the time period increases and there are large uncertainties in projections for sea level rise in the year 2100 and beyond." But, Appendix-B guidance also states, "Since there has been little, if any, measureable rise in sea level since 2000 for most locations in California (Bromirski *et al.* 2011; NOAA 2013), there is little reason or justification for adjusting sea level rise projections from the year 2000 baseline to a more current date when analyzing projects with start dates prior to about Year 2015 or 2020". Therefore, the present analysis will use 2100 as the ultimate planning horizon for a critical infrastructure project.

Appendix-B of CCC, (2015) permits either of two methods derived from the NRC report (NRC, 2012) for making sea level projections, 1) the *linear interpolation method*, and 2) the *best fit equation.* Sea level projection estimates using the "best-fit" equation are slightly less than estimations based on linear interpolation because the NRC's sea level curves are concave upward (sea level rise is expected to accelerate over the 21st Century). Therefore, we select the best-fit equation method for the sea level rise projections used in this study. Since the Doheny Desalination Project is located well south of Cape Mendocino, the appropriate best fit equation for use in the DDP coastal hazards analysis is:

$$SLR = 0.0093t^2 + 0.7457t \quad \text{(upper-range projection)} \qquad (1)$$

$$SLR = 0.0038t^2 + 0.039t \quad \text{(lower-range projection)} \qquad (2)$$

Here, *SLR* is the sea level rise in centimeters (cm) and *t* is the time in years after the year 2000 baseline. Figure 3.1 plots the sea level rise projections from equations (1) & (2), with the 2070 and 2100 planning horizons indicated by various symbols on the upper and lower range curves. For the 2070 planning horizon (50 year project life) sea level rise projections range from 0.70 ft to 3.21 ft.; while for the critical infrastructure planning horizon at 2100, sea level rise is projected range from 1.37 ft to 5.50 ft. These values will be used in the calculations of extreme total water levels (TWL's) in the following sections.

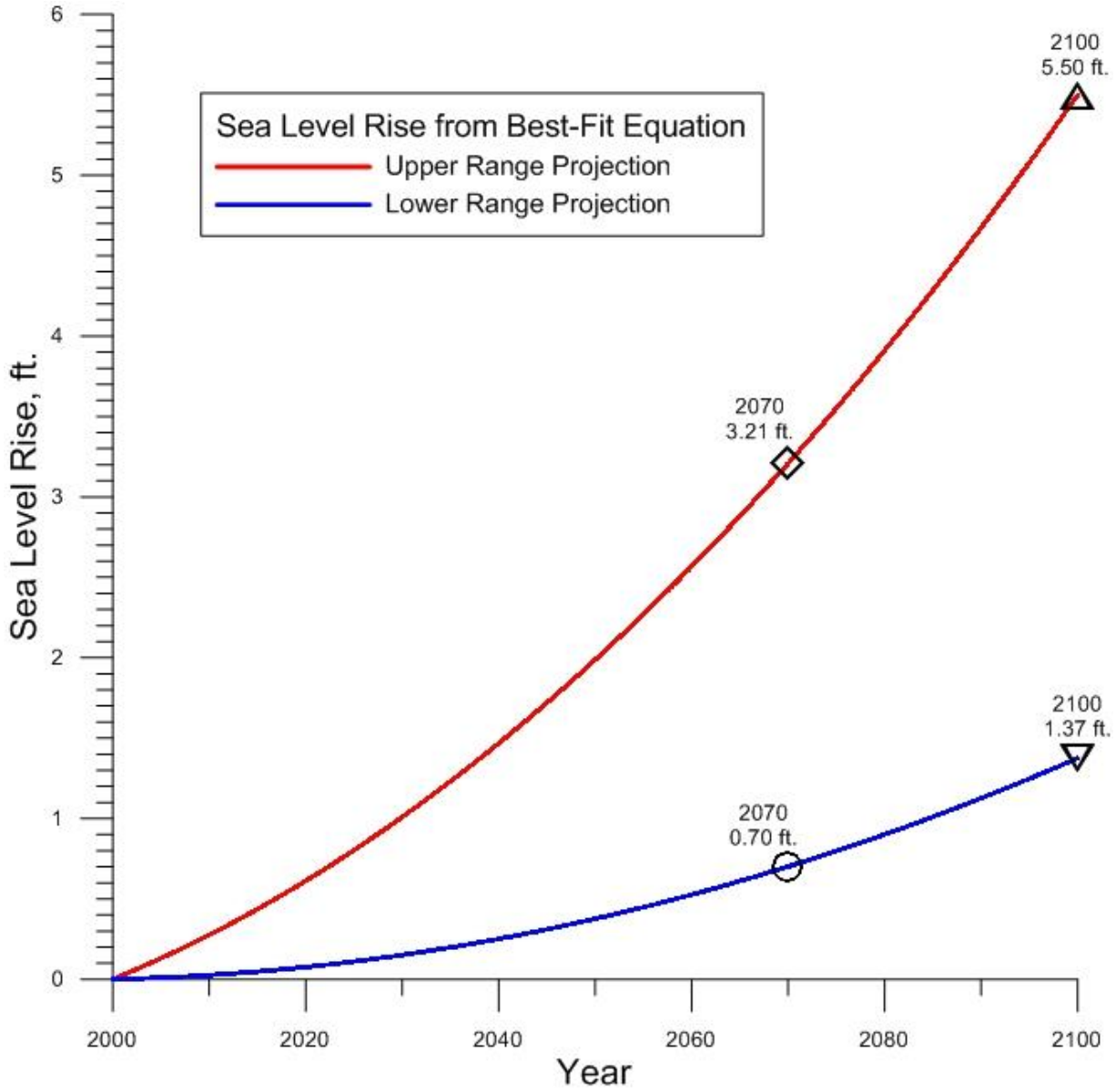


Figure 3.1: Range of sea level rise projections from the best-fit equation, (CCC, 2015, Appendix-B). 2070 and 2100 planning horizons indicated by symbols on the upper and lower range curves.

4) Tidal range and Still Water Levels

This section adresses Steps-2 & 3 of a sea level rise/coastal hazards analysis as outlined in Section 2. This is accomplished by leveraging a long standing effort of NOAA who has deployed tide gages up and down the California coast (NOAA, 2016) to continuously monitor ocean and bay water levels, and who has periodically verified those water levels for multi-decadal periods referred to as "*tidal epochs"*. NOAA has deployed continuously active tide gages along the California coast, which typically record water levels every 6 minutes, and those measurements account for all the combined astronomical, meteorological and climatic effects that have effected water levels in the coastal regions of California since the tide gages were installed. These effects include climate cycles such as El Niño /Southern Oscillation (ENSO) and the longer period Pacific Decadal Oscillation (PDO), as specifically cited for consideration in a coastal hazards analysis in Chapter 6 and Appendix-B of CCC, (2015). The two closest NOAA tide gage stations to the Doheny Desalination Project site are at Newport (NOAA #9410580) and Scripps Pier in La Jolla (NOAA#9410230). The period of record for the Newport tide gage ends in 1994, and fails to capture El Niño effects thereafter, including the El Niño s in 1995 and 1997-98. Therefore we base our tidal range and static water level analysis on the Scripps Pier tide gage, whose period of record extends from 1924 until present, and its vertical datum elevations have also been verified by NOAA for the most recent tidal epoch 1983-2001. Those vertical datum elevations are listed in Table-1.

Water level recurrence statistics are derived from the record of ocean water levels at the NOAA Scripps Pier tide gage based on calculating a stage frequency curve called a "*hydroperiod function"*. The hydroperiod function provides a continuous relationship between ocean water levels measured at 6 minute time intervals and the recurrence probability for each observed water level increment. The computations involves N_0 time steps in the NOAA water level files. Each time sep is at 6 minute intervals, over the period of record (1924-2016). Conditional if statements embedded in counting loops of the *hydro-pr_caltrans* software (developed for Caltrans coastal culvert design, cf. Jenkins and Taylor, 2016) calculate the number time steps, $N(\eta \le Z_i)$, for which the ocean water level, η, was at least as high as a potential still-water elevation Z_i at or above mean sea level. The percent time that elevation Z_i is wet due to ocean inundation is calculated as:

$$\hat{E}_i = \frac{100\%}{\hat{N}_o} \sum N(\eta \ge Z_i) \qquad (3)$$

where : $\hat{N}_0 = \sum_i N_i(\eta \ge MSL)$

Time averaging Equation (3) over yearly increments and then ensemble averaging the yearly averages gives an *annualized hydroperiod function* $H_{i,j}$ that represents the annualized probability of ocean water levels reaching a still-water elevation Z_i

Table 4.1: Tidal Datums at Scripps Pier NOAA Tide Gage Station:
Elevations on Station Datum
Station: 9410230, La Jolla, CA
Status: Accepted (Oct 6 2011)
Units: Feet
T.M.: 120
Epoch: 1983-2001
Datum: STND

Datum	Value	Description
MHHW	9.69	Mean Higher-High Water
MHW	8.97	Mean High Water
MTL	7.12	Mean Tide Level
MSL	7.10	Mean Sea Level
DTL	7.03	Mean Diurnal Tide Level
MLW	5.27	Mean Low Water
MLLW	4.37	Mean Lower-Low Water
NAVD88	4.56	North American Vertical Datum of 1988
STND	0.00	Station Datum
GT	5.33	Great Diurnal Range
MN	3.69	Mean Range of Tide
DHQ	0.73	Mean Diurnal High Water Inequality
DLQ	0.91	Mean Diurnal Low Water Inequality
HWI	5.01	Greenwich High Water Interval (in hours)
LWI	11.07	Greenwich Low Water Interval (in hours)
Maximum	12.03	Highest Observed Water Level
Max Date & Time	01/11/2005 17:00	Highest Observed Water Level Date and Time
Minimum	1.50	Lowest Observed Water Level
Min Date & Time	12/17/1933 23:36	Lowest Observed Water Level Date and Time
HAT	11.51	Highest Astronomical Tide
HAT Date & Time	08/09/1987 03:54	HAT Date and Time
LAT	2.49	Lowest Astronomical Tide
LAT Date & Time	01/28/1987 22:48	LAT Date and Time

Tidal Datum Analysis Periods : 01/01/1983 - 12/31/2001

Tidal Datums:
EHW = 7.47 ft NAVD
HAT = 6.95 ft. NAVD
MHHW = 5.13 ft NAVD
MHW = 4.41 ft NAVD
MSL = 2.54 ft NAVD
MTL = 2.56 ft NAVD
MLLW = 0.00 ft. NAVD
ELW = -3.06 ft NAVD
NGVD 1929 = 2.35 ft. NAVD

$$P_{i,j} = \frac{1}{k} \sum_{j=1}^{j=k} \left[\frac{1}{\tau_j} \int_0^{\tau_j} \hat{E}_i \, dt \right] \qquad (4)$$

Here τ_j is the length of tidal record in *year-j* and k is the number of years in the period of record of the tide gage. The annualized hydroperiod function of still-water level elevations at present sea level is plotted in Figure 4.1, based on the NOAA Scripps Pier ocean water level data (surrogate for the Doheny Desalination Project site). Inspection of Figure 4.1 indicates that recurrence probability for mean higher high water levels are *P*(MHHW) = 13% and *P*(MHW) = 28% for mean high water levels; while intuitively the recurrence probability for mean sea level is *P*(MSL) = 100%. The extreme high water level event is a less than 1% event at *P*(EHW) = 0.06%.

Table 1 reveals that the extreme high water level, (EHW = 7.47 ft. NAVD, occurring 1 November 2005) exceeds the highest astronomical tide, (HAT = 6.95 ft NAVD, occurring 9 August 1987). The largest exceedance of daily high water levels above the astronomic tides in the period of record of the NOAA #9410230 occurred during the 1997-98 El Niño on 13 November 1997, when the daily high water level was 1.47 ft above the astronomic tides (Figure 4.2). This discrepancy occurs as a result of climate cycle effects that warm the coastal ocean creating an increase in *steric* sea level due to thermal expansion of the water mass, which can persist for as long as 8-10 months. Climate cycles involve intense global modifications that are signaled by anomalies in the pressure fields between the tropical eastern Pacific Ocean and Australia/Malaysia known as the ***Southern Oscillation***. The intensity of the oscillation is often measured in terms of the ***Southern Oscillation Index (SOI)***, defined as the monthly mean sea level pressure anomaly in mb normalized by the standard deviation of the monthly means for the period 1951-1980 at Tahiti minus that at Darwin, Australia. The Southern Oscillation is in turn, modulated over multi-decadal periods by the ***Pacific Decadal Oscillation***, which results in alternating decades of strong and weak El Niño.

The long-term variability of the Pacific Decadal Oscillation (PDO) is shown in Figure 4.3 and the cumulative residual of the Southern Oscillation Index, between 1882 and 1996, is plotted in Figure 4.4, where cumulative residuals SOI_n are taken as the continued cumulative sum of departures of annual values of a time series SOI_j from their long-term mean values $\overline{SOI}$, such that :

$$SOI_n = \sum_{o}^{n} (SOI_i - \overline{SOI}) \qquad (5)$$

Here n is the sequential value of a time series of n years. Southern Oscillation effects give rise to enhancements and protractions of the inter-annual seasonal cycles, and their two extremes are referred to as El Niño (SOI negative) and La Niña (SOI positive). Inspection of Figure 5a reveals a number of large positive oscillations in the SOI between 1944 and 1978 corresponding to La Niña dominated climate; and a series of very large negative oscillations occurring between 1978 and 1995 which correspond with El Niño dominated climate.

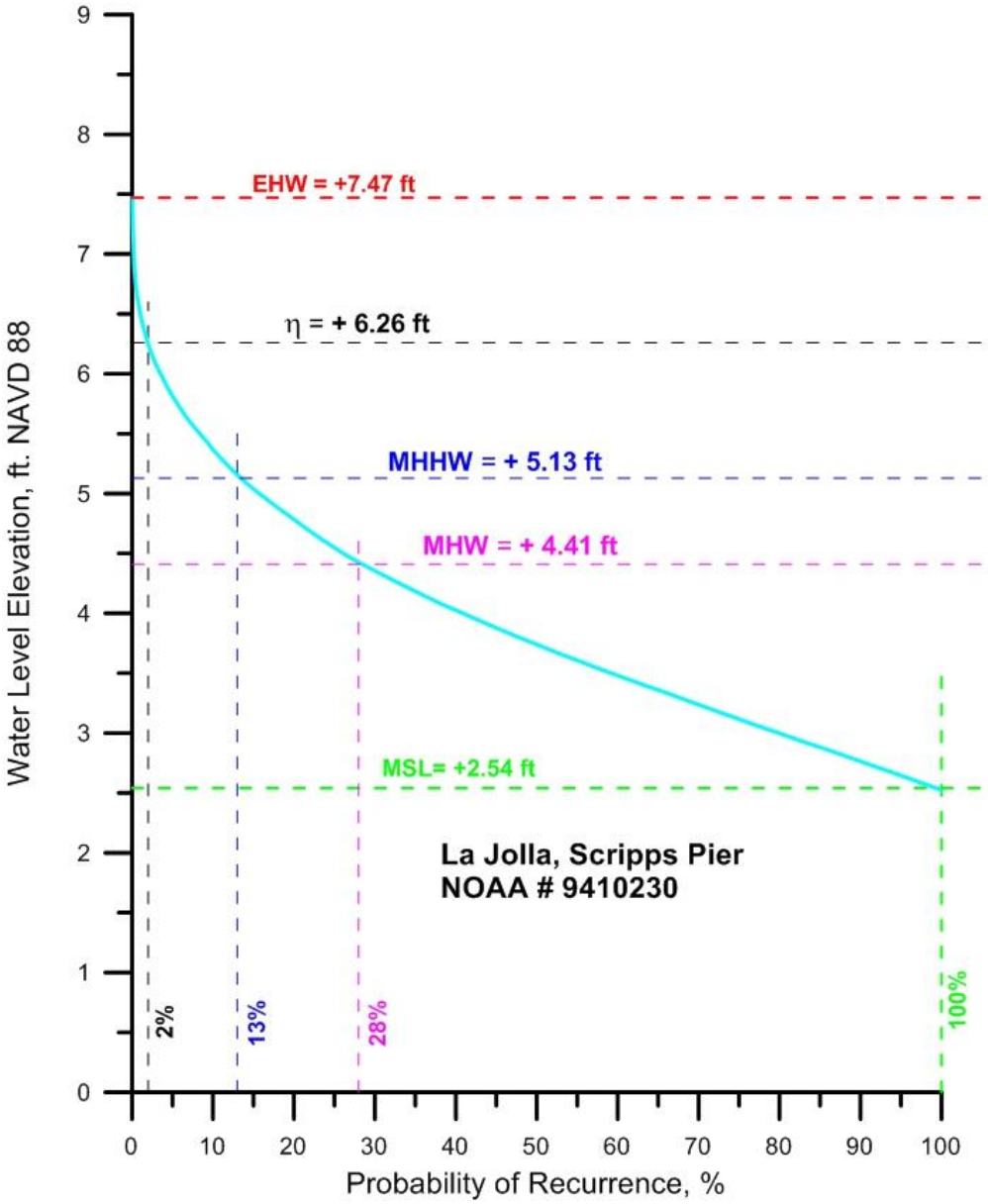


Figure 4.1: Hydroperiod function of still-water level elevations at present sea level, based on ocean water level measurements at the Scripps Pier tide gage station, NOAA #9410230, for the period of record 1924-2016. Tidal datums based on the 1983-2001 tidal epoch (latest datum analysis period).

Along the southern California coast, a period of mild-stable La Niña dominated pressure systems prevailed between 1944 and 1978. The average SOI for this period was +0.1,with strong La Niña events in 1950, (SOI = +1.4); 1955/56, (+1.2); 1970/71, (+1.0); 1973/74, (+1.0); and 1975/76 (+1.4). Winters were moderate with low rainfall, and winds were predominantly from the west-northwest. The principal wave energy was from Aleutian lows having storm tracks which usually did reach southern California. Summers were mild and dry with sea surface temperatures seldom exceeding 20^0C. The North Pacific High dominated the coastal transport by strengthening the California Current and promoting coastal upwelling of cold bottom water. The effect of these cool dry La Niña dominated climate periods was to promote negative anomalies in the steric sea level, augmented by depression of sea level by the inverse barometer effects of a strong North Pacific High.

The climate in southern California changed, beginning with the El Niño years of 1978/79 and extending at least until 1999. The average SOI for this period was -0.5, with the 1978/79 El Niño averaging -1.2, the 1982/83 El Niño averaging a record -1.7 and the 1993/94 El Niño recording a mean of -1.0. During these periods, the North Pacific High was weakened and transport of warm equatorial water masses into the Southern California Bight were promoted by topographically trapped Kelvin waves. The North Pacific High was weak and the prevailing north-westerly winter waves were replaced by high energy waves approaching from the west or southwest, while the previous southern hemisphere swell waves of summer were replaced by shorter period tropical storm waves during late summer months from the more immediate waters off Central America. These dynamics promoted positive sea level anomalies in steric sea level as a consequence of thermal expansion of the warm coastal ocean water mass, augmented by inverse barometer effects under strong frontal cyclones during winter.

These climate effects on the hydroperiod function are proportioned schematically in Figure 4.5. Basically, ocean water levels result from the astronomic tides oscillating around the steric sealevel, which itself varies slowly in response to seasonal warming and cooling of the coastal water mass, and longer term warming and cooling from ENSO and PDO. While the highest astronomic tides have reached HAT = +6.95 ft NAVD, astronomic tides typically do not exceed η = +6.0 ft NAVD during a typical spring-neap cycle. Seasonal warming of the coastal ocean can cause an increase in steric sea levels by as much as $\Delta\eta$ = +0.5 ft. As Figure 4.2 reveals, a strong El Niño event can create as much as $\Delta\eta$ =+1.47 ft. increase in steric sea level, but more typically El Niño events cause positive sea level anomalies on the order of $\Delta\eta$ =+1.47 ft. Because PDO re-enforces El Niño events during a multi-decadal warm wet climate period as occurred during the 1978-1998 epoch, just how much of these anomalies is due to PDO is uncertain, but generally it is believed that about 10% to 15% of an El Niño sea level anomaly is due to a positive PDO cycle. On the other hand, La Niña events depress steric sea levels and typically produce negative sea level anomalies on the order of $\Delta\eta$ = -0.6 ft.

Because the hydroperiod function in Figure 4.1 is based on multi decadal ocean water level measurements (1924-2016), it captures the combined effects of PDO, ENSO, and astronomic tides. It also captures the transient storm surge events. Storm surge is a wind-set-up phenomena, but because California is a collision coastline with a very narrow continental shelf, it does not develop the large storm surges of tens of feet that occur on the broad shelf environments of the Gulf and Atlantic coastlines during hurricanes. Storm surge on the California coast is primarily due to the inverse barometer effect, which causes the sea surface to bulge upwards under low pressure weather systems approaching the coastline, and typically lasts a few days during the passage of

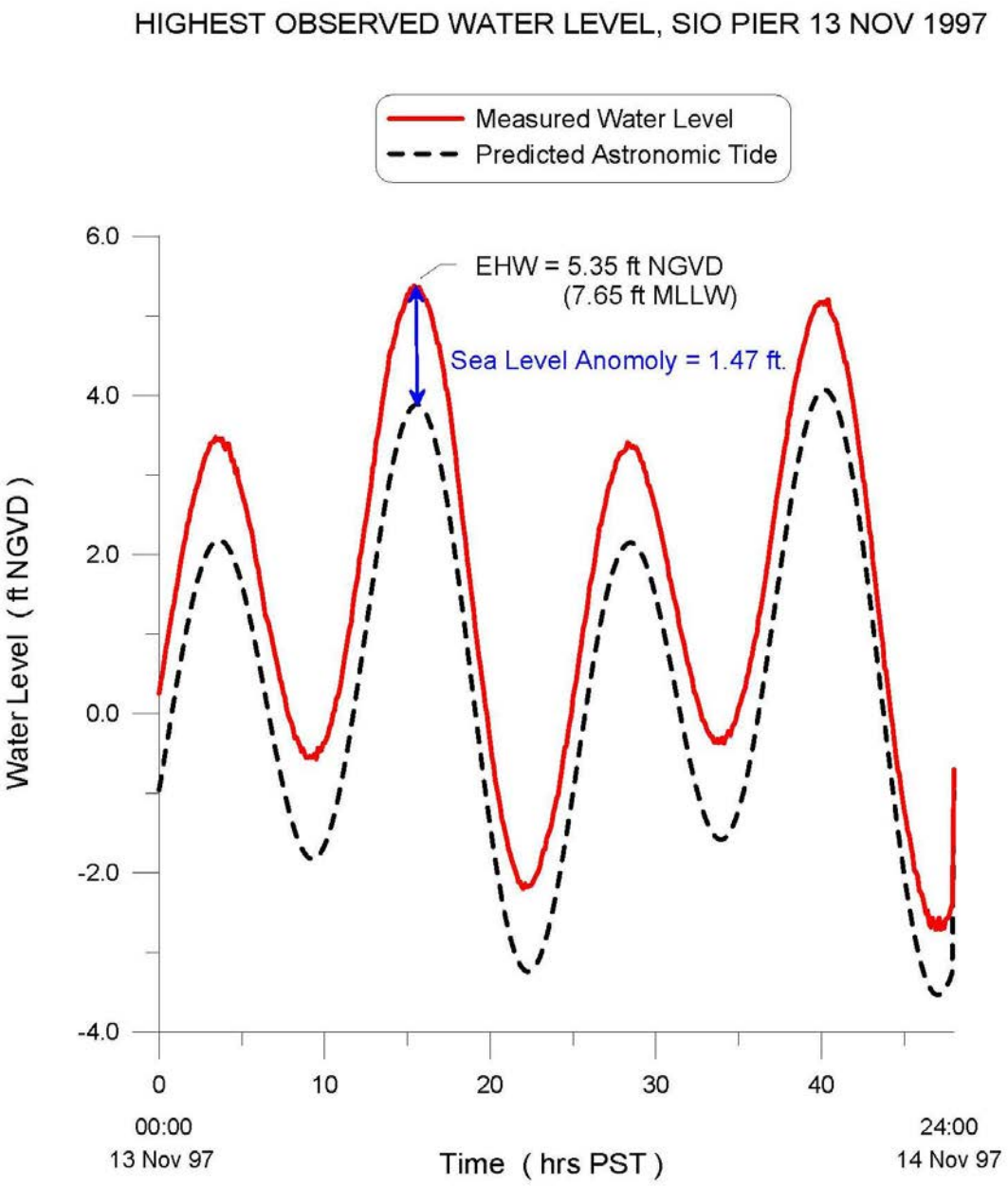


Figure 4.2: Comparison of measured ocean water level (red) at the Scripps Pier tide gage vs. predicted water level based on tidal constituents (black dashed) for the extreme high water event of 13 November 1997 (from Jenkins and Wasyl, 2005).

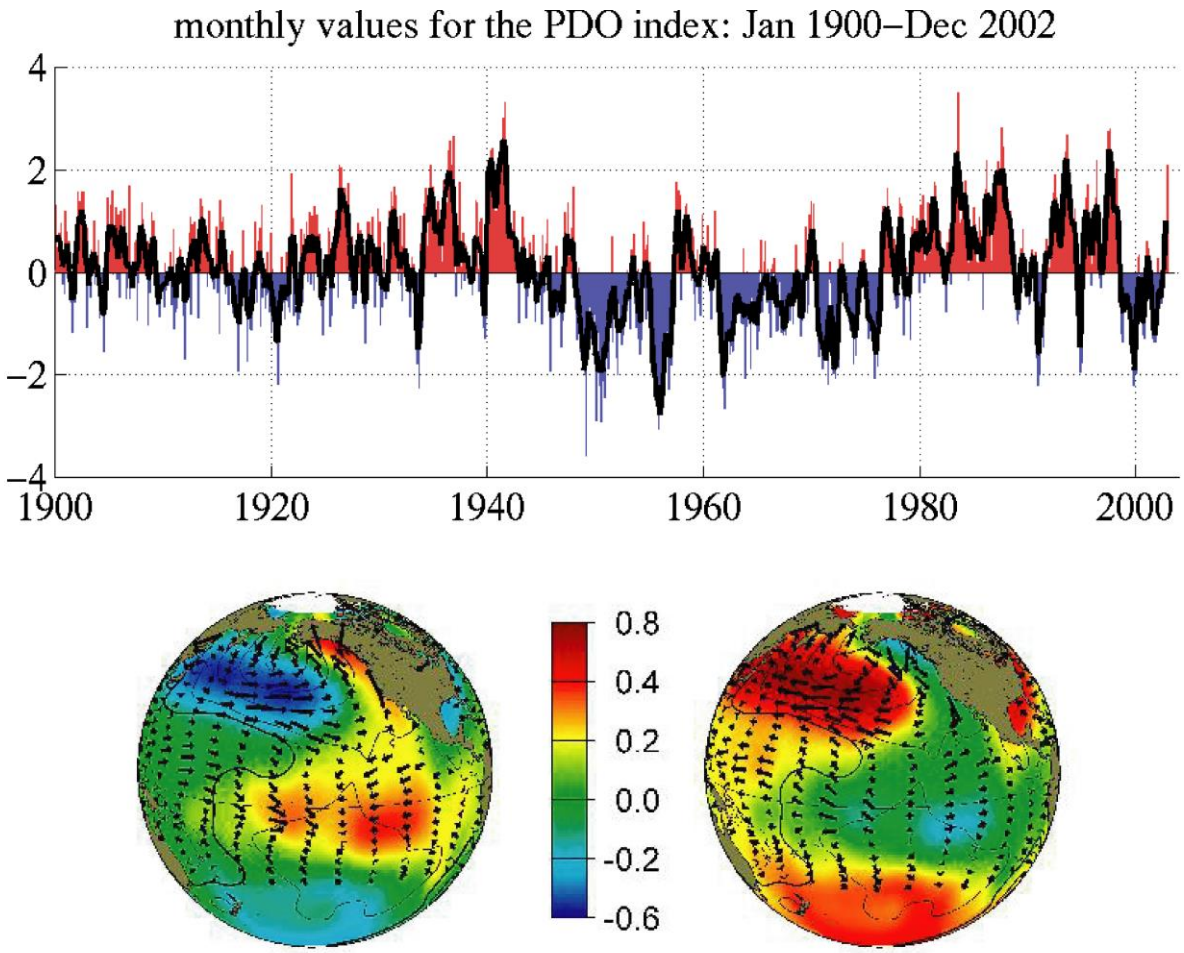


Figure 4.3. Typical wintertime Sea Surface Temperature (colors), Sea Level Pressure (contours) and surface wind stress (arrows) anomaly patterns during warm and cool phases of PDO.

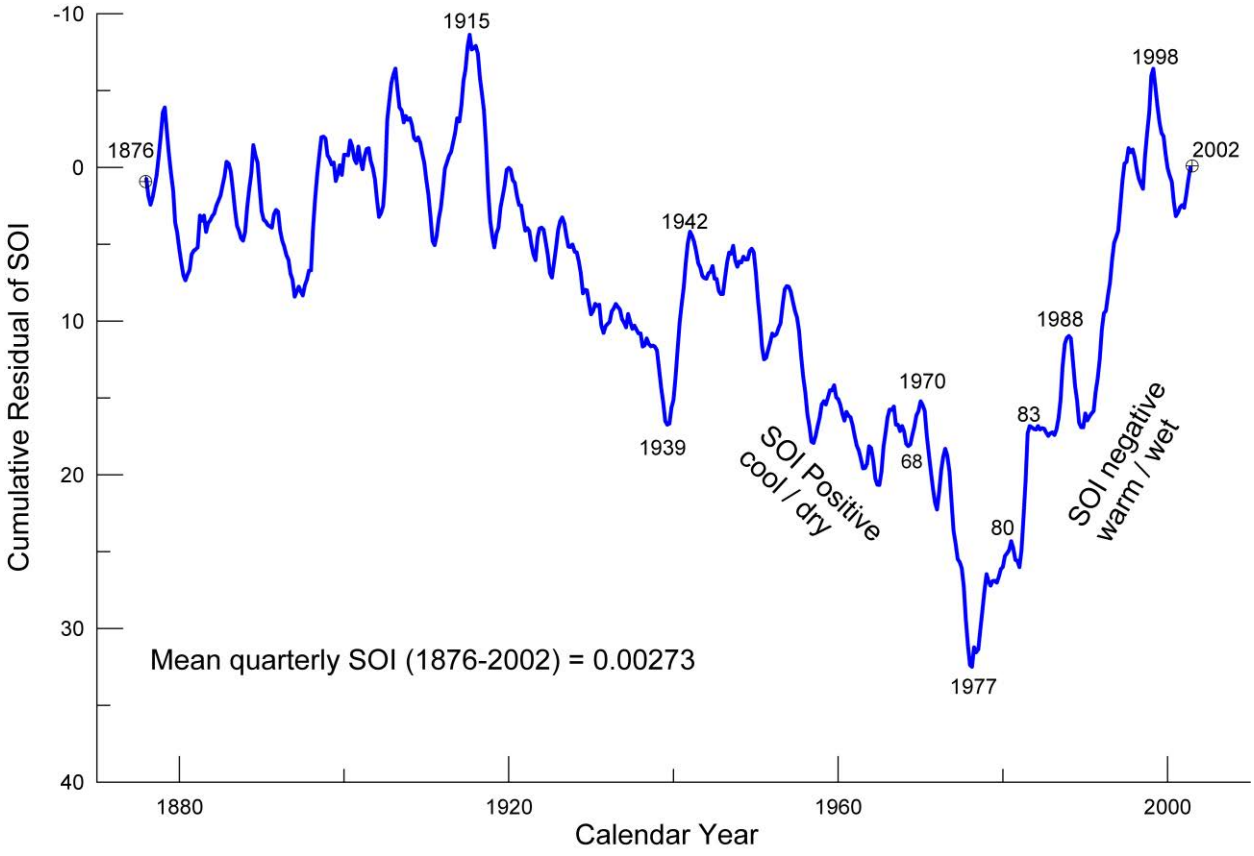


Figure 4.4. Cumulative residual of quarterly values of Southern Oscillation Index (SOI) [data from Australian Commonwealth Bureau of Meteorology].

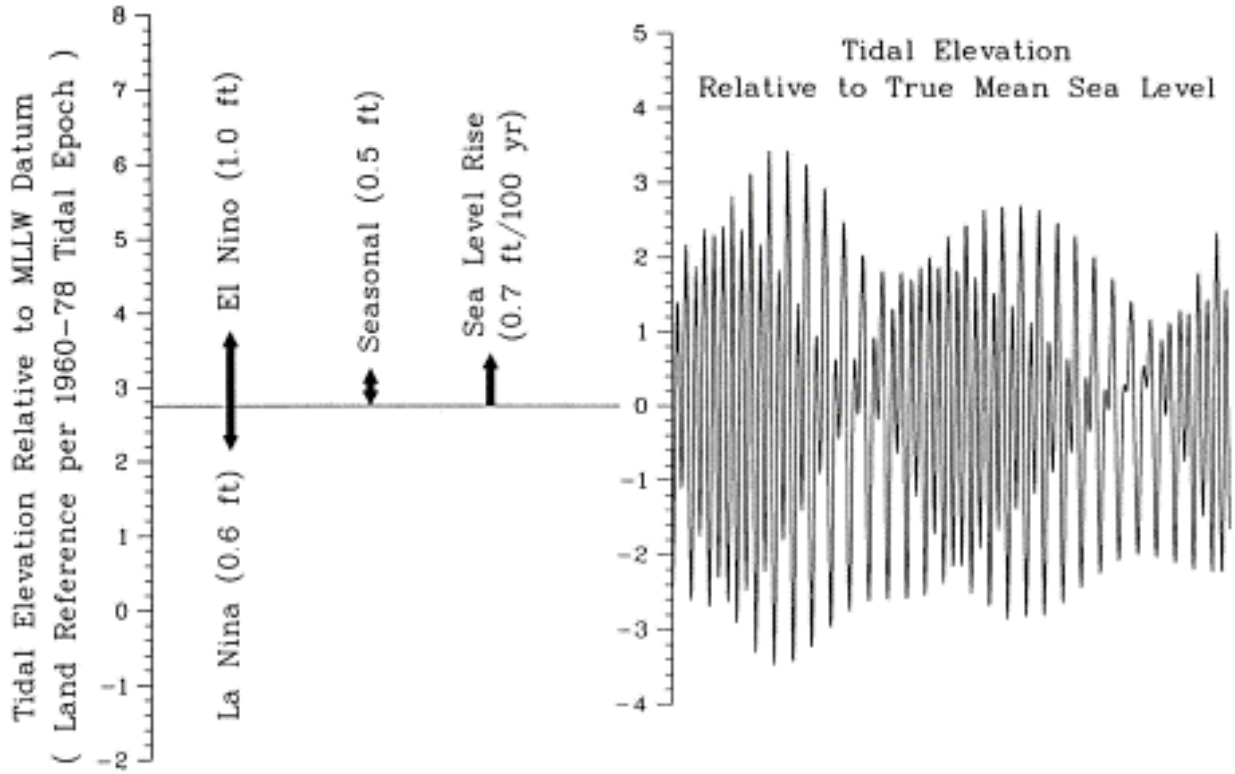


Figure 4.5 Schematic decomposition ocean water levels according to astronomic tides, and seasonal and ENSO/PDO cycle effects on steric sea levels.

winter cold fronts. The sea surface rises 1 cm for every millibar drop in atmospheric pressure. The atmospheric pressure during strong El Niño storms may drop to as low as 993 millibars, (as compared to 1,013 millibars standard atmospheric pressure); which equates to a 20 cm rise in ocean water level during the passage of the storm due to the inverse barometer effect. That short term rise is captured by the NOAA tide gage at the end of Scripps Pier, and is built into the hydroperiod function in Figure 4.1. But, because the Scripps Pier tide gages is located in a stilling well considerably seaward of the surf zone, the hydroperiod function derived from its water level measurements do not include dynamic effects from wave set-up or runup. Consequently the hydroperiod function maps the probabilities of still water levels at or above mean sea level.

Because both the Scripps Pier NOAA tide gage and the Doheny Desalination Project are sited in locations with narrow continental shelfs of only about 4.5 km in width, it is reasonable to assume that the local tidal dynamics will not be altered by higher future sea levels (ie, sea level rise will not cause any new resonance or damping effects of the astronomic tides across the continental shelf). It is not known how ENSO or PDO climate cycles might be altered by global warming and higher sealevels, but for now it is resonable to assume that the hydroperiod function of still water elevations at future sealevels can be obtained by linear superposition of the present hydroperiod function in Figure 4.1 and the sea level rise projections in Figure 3.1. By that approach, the hydroperiod function of still-water level elevations was obtained at 2070 sea level in Figure 4.6 and at 2100 sea level in Figure 4.7. For the 2070 low range projections in Figure 4.6, mean sea level

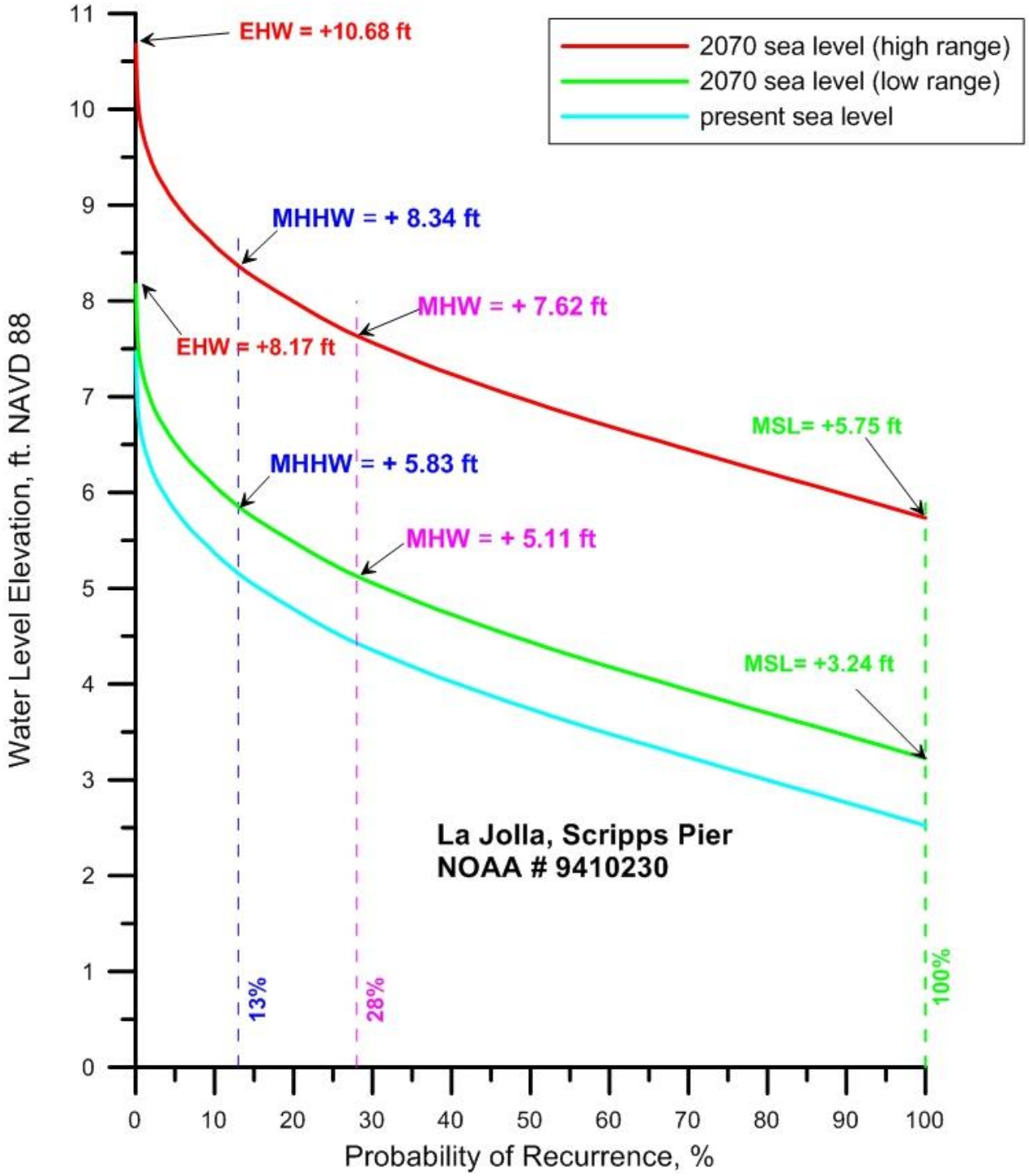


Figure 4.6: Hydroperiod function of still-water level elevations at 2070 sea level, based on ocean water level measurements at the Scripps Pier tide gage station, NOAA #9410230, for the period of record 1924-2016. Tidal datums based on the 1983-2001 tidal epoch (latest datum analysis period).

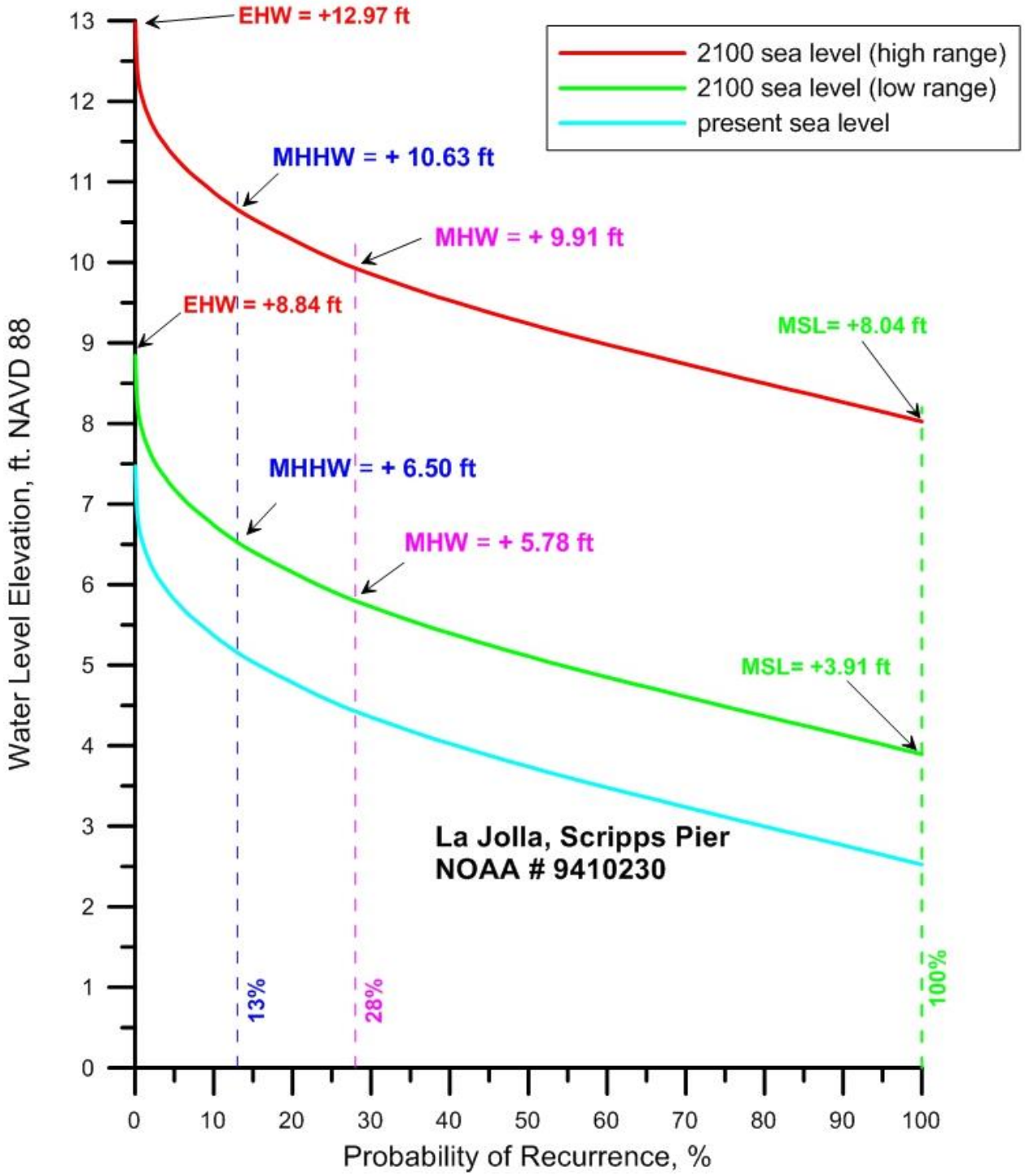


Figure 4.7: Hydroperiod function of still-water level elevations at 2100 sea level, based on ocean water level measurements at the Scripps Pier tide gage station, NOAA #9410230, for the period of record 1924-2016. Tidal datums based on the 1983-2001 tidal epoch (latest datum analysis period).

increases to MSL = +3.24 ft NAVD while extreme high water increases to EHW = +8.17 ft. NAVD. Based on recurrence probabilities, mean higher high water increases to MHHW = + 5.83 ft. NAVD for the 2070 low range projections. At the high range 2070 projections, mean sea level increases to MSL = +5.75 ft. NAVD; extreme high water increases to EHW = +10.68 ft. NAVD, and mean higher high water increases to MHHW = + 8.34 ft. NAVD. At the year 2100 planning horizon for critical infrastructure, low range projections in Figure 4.7 indicate that mean sea level increases to MSL = +3.91 ft NAVD while extreme high water increases to EHW = +8.84 ft. NAVD, while mean higher high water increases to MHHW = + 6503 ft. NAVD. At the high range 2100 projections, mean sea level increases to MSL = +8.04 ft. NAVD; extreme high water increases to an astonishing EHW = +12.97 ft. NAVD, and mean higher high water increases to MHHW = + 10.63 ft. NAVD. The still water elevations inferred at future sea levels from the linear superposition assumption are summarized in Table 4.2 below.

Table 4.2: Still Water elevations at present and future sea levels from the linear superposition assumption.

Tidal Datums	Present Sea Level (ft. NAVD 88)	2070 Sea Level Low Range Projection (ft. NAVD 88)	2070 Sea Level High Range Projection (ft. NAVD 88)	2100 Sea Level High Range Projection (ft. NAVD 88)	2100 Sea Level High Range Projection (ft. NAVD 88)
Mean Sea Level (MSL)	2.54	3.24	5.75	3.91	8.04
Mean High Water (MHW)	4.41	5.11	7.62	5.78	9.91
Mean Higher-HighWater (MHHW)	4.13	5.83	8.34	6.50	10.63
Extreme High Water (EHW)	7.47	8.17	10.68	8.84	12.97

5) Technical Approach for Erosion and Dynamic Water Level Analysis:

This section establishes the technical approach for evaluating Steps-4 & 5 of a sea level rise/coastal hazards analysis as outlined in Section 2. The total run-up, R, is composed of three main components: Static wave setup, $\overline{\eta}$, Dynamic wave setup, η_{rms}; Incident wave run-up, R_{inc}. The total water level (TWL) is defined as the sum of the total run-up and the SWL, referenced to an established vertical datum.

5.1 Models: To quantitatively evaluate the problems of implementing subsurface intake technology at SJCOO, we invoke a numerical seabed stability analysis utilizing the *Coastal Evolution Model* (Figure 5.1) applied to the Oceanside Littoral Cell (Figure 5.2). The Coastal Evolution Model was commissioned by the Kavli Foundation to make forecast predictions of the effects of sea level rise on the coastline of California (Jenkins and Wasyl, 2005).

The Coastal Evolution Model (CEM) is a process-based numerical model. It consists of a Littoral Cell Model (LCM) and a Bedrock Cutting Model (BCM), both coupled and operating in varying time and space domains (Figure 5.1.) determined by sea level and the coastal boundaries of the littoral cell at that particular sea level and time. At any given sea level and time, the LCM accounts for erosion of uplands by rainfall and the transport of mobile sediment along the coast by waves and currents, while the BCM accounts for the cutting of bedrock by wave action in the absence of a sedimentary cover.

In both the LCM and BCM, the coastline of the Oceanside Littoral Cell (the region of coastline between Dana Point and Point La Jolla, Figure 5.2) is divided into a series of coupled control cells. Each control cell is a small coastal unit of uniform geometry where a balance is obtained between shoreline change and the inputs and outputs of mass and momentum. The model sequentially integrates over the control cells in a down-drift direction so that the shoreline response of each cell is dependent on the exchanges of mass and momentum between cells, giving continuity of coastal form in the down-drift direction. Although the overall computational domain of the littoral cell remains constant throughout time, there is a different coastline position at each time step in sea level. For each coastline position there exists a similar set of coupled control cells that respond to forcing by waves and current. Time and space scales used for wave forcing and shoreline response (applied at 6 hour intervals) and sea level change (applied annually) are very different. To accommodate these different scales, the model uses multiple nesting in space and time, providing small length scales inside large, and short time scales repeated inside of long time scales. The LCM (Figure 5.1, upper) has been used to predict the change in shoreline width and beach profile resulting from extreme wave run-up, sea level rise, erosion, accretion and longshore transport of sand by wave action, where sand source is from river runoff or from tidal exchange at lagoon and bay inlets (e.g., Jenkins and Inman, 1999). More recently it has been used to compute the sand level change (Farfield Effect) in the prediction of mine burial (Jenkins and Inman, 2002; Inman and Jenkins, 2002). Time-splitting logic and feedback loops for climate cycles and sea level change were added to the LCM together with long run time capability to give numerically stable long term predictions.

5.2: Computational Approach: The presently adopted procedure for wave run-up analysis for the design of coastal structures, (as set forth in the *U.S. Army Corps of Engineers Coastal Engineering Manual* (USACE, 2006), and its software counterpart, the *Automated Coastal Engineering System*, known as *ACES*), is based on the assumption of rigid boundaries. The Coastal Evolution Model described in Section 3.1 is utilized for this analysis and employs algorithms consistent with the U.S. Army Corps of Engineers Coastal Engineering Manual, but employs the

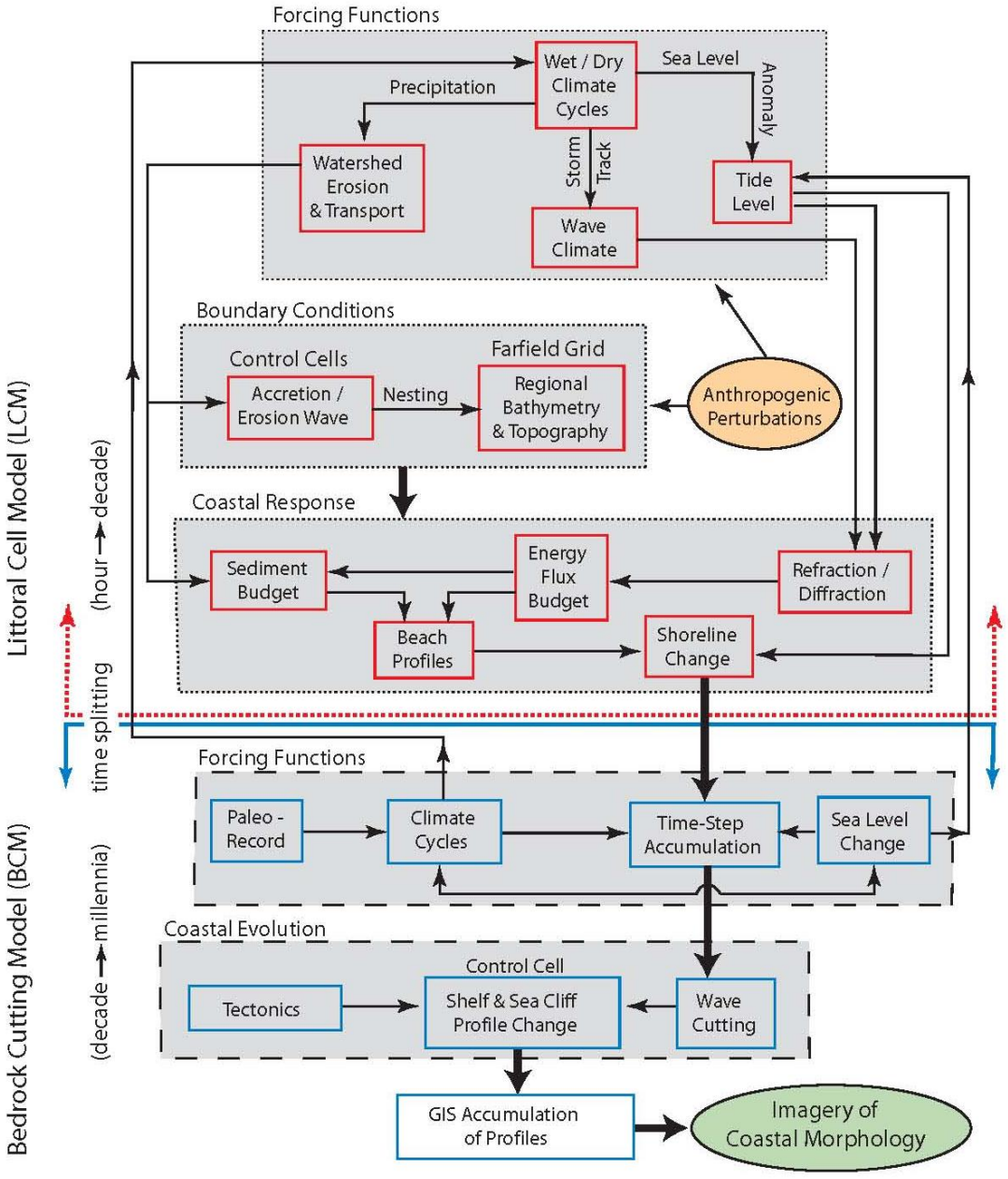


Figure 5.1: Architecture of the Coastal Evolution Model consisting of the Littoral Cell Model (above) and the Bedrock Cutting Model (below). Modules (shaded) are formed of coupled primitive process models. (Jenkins and Wasyl, 2005).

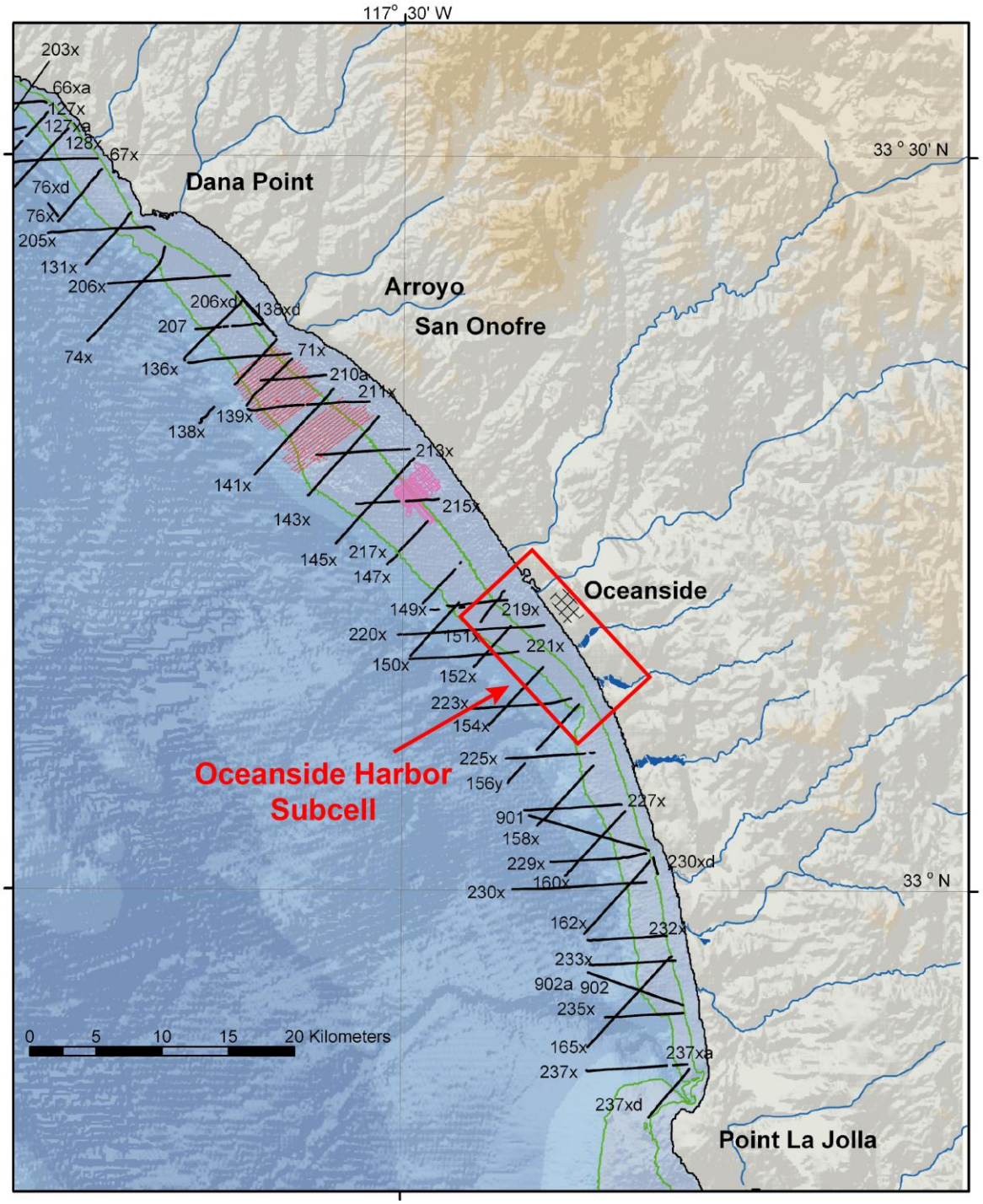


Figure 5.2: Oceanside Littoral Cell and Oceanside Harbor Sub-Cell. Composite bathymetry from NOS data base and equilibrium profiles after Jenkins and Inman (2006) for wave conditions of wet weather scenario. Depth contours shown in meters mean sea level. USGS cross-shelf survey tracks shown as numbered black line segments.

add-on features of latest generation equilibrium beach profile algorithms from Jenkins and Inman (2006) and supporting bathymetric data bases for the entire shore and continental shelf of California.

5.3) Wave Setup and Run-up: Wave setup is an increased elevation of the water level due to the effects of wave momentum being transferred to the surf zone. In wave systems composed of more than one wave component, as occurs in the Pacific Ocean, the setup oscillates and comprises a static and a dynamic component. Wave runup is the culmination of the wave breaking process, whereby the wave surges up the beach, bluff, or structure face along the shoreline. Overtopping occurs when the wave runup exceeds the profile crest elevation, which can result in flooding landward of the crest. Runup is a function of several key parameters. These include the wave height, H the wave period, T , the wave length, L , the profile slope, m , and the surf similarity parameter (Iribarren number), ξ defined as: $\xi = m/\sqrt{H/L}$. The total water level (TWL) is defined as the sum of the total runup and the SWL, referenced to an established vertical datum. The results for this study are referenced to the North American Vertical Datum of 1988 (NAVD88) vertical datum. The total runup, R , is composed of three main components: Static wave setup, $<\eta>$, Dynamic wave setup, η_{rms} ; Incident wave runup, R_{inc}.

Wave setup and runup are typically computed at hourly time steps from an historic record of wave monitoring, (see Section 6.0). Wave setup and runup are combined with still water level values (from hydroperiod functions, see Jenkins, 2015) to develop the TWL values. It should be noted that the increase in sea level for future scenarios should be added to each hourly SWL over the 32-year wave record (see Section 4.2) for the analysis of TWLs, with the 1-percent-annual-chance results derived statistically from the resultant 32 annual maxima as explained in Section 2.6.

Annual maxima TWLs are computed for each sea level rise (SLR) scenario, and a statistical Generalized Extreme Value (GEV) analysis is performed on these values to determine the 1-percent-annual-chance TWL for two example problems. The overtopping rate is calculated for instances where the TWL exceeded the engineered barrier crest and overtopping occurred. Each step used to evaluate hazards is described in detail in the following subsections.

Both static and dynamic components of wave setup were calculated using the Direct Integration Method (DIM) which uses a parameterized set of equations that consider wave and bathymetric characteristics, specifically the shape of the wave energy spectrum and the nearshore shorerise and bar-berm beach slope (m_{DIM}). The wave setup equations include factors for wave height (F_H and G_H), wave period (F_T and G_T), JONSWAP spectral narrowness factor (F_{Gamma} and G_{Gamma}), and nearshore slope (F_{Slope} and G_{Slope}).

Static wave setup is calculated as:

$$<\eta> = 4.0 F_H F_T F_{Gamma} F_{Slope} = 4.0 \left(\frac{H_0'}{26.2} \right)^{0.8} \left(\frac{T_P}{20.0} \right)^{0.4} \left(\frac{m_{DIM}}{0.01} \right)^{0.2} \quad (6)$$

Dynamic wave setup is calculated as:

$$\eta_{rms} = 4.0 G_H G_T G_{Gamma} G_{Slope} = 4.0 \left(\frac{H'_0}{26.2} \right)^{0.8} \left(\frac{T_P}{20.0} \right)^{0.4} (Gamma)^{0.16} \left(\frac{m_{DIM}}{0.01} \right)^{0.2} \qquad (7)$$

The wave parameters required as input for DIM are the deepwater equivalent significant wave height, in feet, (H'_0) and the spectral peak wave period (T_P), as well as a measure of the spectral shape (*Gamma*). The spectral peak parameter, *Gamma*, was computed via a polynomial fit between the spectral width parameter ν and *Gamma,* according to:

$$Gamma = 2047\nu^4 - 3083\nu^3 + 1782\nu^2 - 4769.9\nu + 507.1 \qquad (8)$$

Values of are computed directly from the spectral moments ($\beta_0, \beta_1, \beta_2$) based on the Longuet-Higgins (1973) definition of the spectral narrowness:

$$\nu = \left[\frac{\beta_0 \beta_2}{\beta_1} - 1 \right]^{1/2} \qquad (9)$$

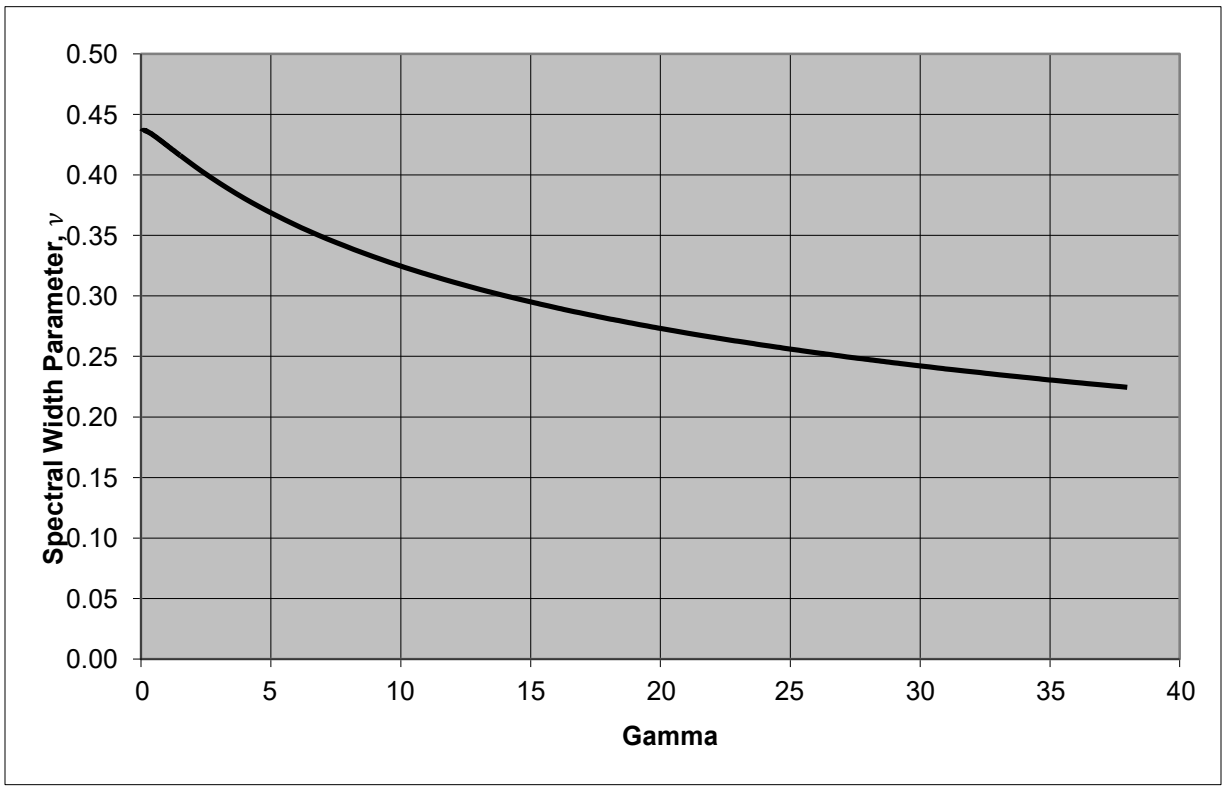


Figure 5.3 *Gamma* values are limited from 1 to 38, based on the range of wave data used (Section 4.4) to relate the spectral narrowness, ν, to the peak parameter, *Gamma*, as shown in Figure 3.3.

The deepwater equivalent significant wave height, H'_0, and the peak wave period, T_P, are provided as output from the CDIP wave monitoring data (CDIP, 2015) and are input directly into Equations 8 and 9. The nearshore slope, m_{DIM}, is taken from nearshore and beach surveys by Coastal Environments, et al., (2014) that were used to calibrate extreme event computations of profile slope using the elliptic cycloid algorithms of Jenkins and Inman (2006). The slope term, m_{DIM}. Used in the TWL computations is calculated from the average slope between the landward limit of wave runup and the location offshore where the water depth is two times the depth at which the deepwater significant wave height would be subject to depth-limited breaking (van der Meer, 2002). The landward limit of wave runup is calculated iteratively, with the initial approximation being the SWL.

5.4 Wave Runup: Wave runup was calculated using either the DIM or the Technical Advisory Working Group (TAW) method (van der Meer, 2002), depending upon the dynamic water level relative to the toe of the coastal structure and the shoreline (bar-berm) slope, m_{TAW}, calculated iteratively across the surf zone. The DIM is used to calculate runup for transects with natural, gently sloping ($m_{DIM} < 0.125$) profiles. For shorelines with shore protection structures and steeply sloping ($m_{TAW} \geq 0.125$) natural shorelines where the dynamic water level exceeds the toe of the structure, the TAW method was used to calculate runup. If the dynamic water level does not reach the toe of the structure or bluff face, the DIM is used. The total runup, including wave setup and incident wave runup, is added to the *still water level* (SWL) to determine the *total water level*, (TWL), see Figure 5.4). Each of these methods is described in detail in the following subsections.

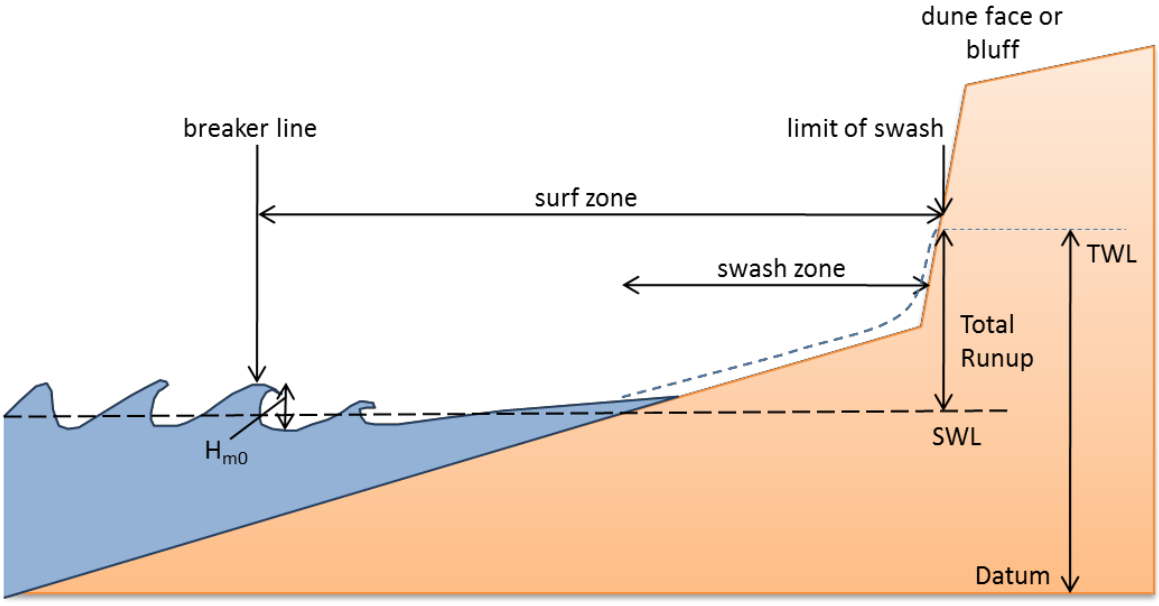


Figure 5.4: Conceptual Model Showing the Components of Wave Runup Associated with Incident Waves.

5.5 DIM Runup Calculations: Runup on gently sloping, natural shorelines, and beaches seaward of a structure or bluff toe, is calculated using the *direct integration method* (DIM). The runup calculation is based on the standard deviations of the oscillating wave setup and the incident wave runup components, and is a continuation of the DIM approach for wave setup. The dynamic setup η_{rms} is defined as the standard deviation of setup fluctuations, calculated from Equation 2. The standard deviation of the incident wave oscillations (wave runup), σ_2 on natural beaches is:

$$\sigma_2 = 0.3\xi_0 H_0' \tag{10}$$

Where, H'_0 is the deep water significant wave height, m_{DIM} is the nearshore (shorerise) bottom slope, $L_0 = gT_P^2 / 2\pi$ is the deep water wave length, and ξ_0 is the deep water Iribarren number:

$$\xi_0 = \frac{m_{DIM}}{\sqrt{H_0' / L_0}}$$

The oscillating component of the total wave runup or *swash*, $\hat{\eta}_T$, is determined from the combination of the two standard deviations of the fluctuating components:

$$\hat{\eta}_T = 2.0\sqrt{\eta_{rms}^2 + \sigma^2} \tag{11}$$

Combining the results from Equations 6 & 11 yields the total wave runup, which when superimposed with the SWL yields the total water level, TWL:

$$TWL = <\eta> + \hat{\eta}_T + SWL \tag{12}$$

Where SWL is the still water level derived from the hydroperiod function given by Jenkins, (2015).

5.6 TAW Runup Calculations: Runup on barriers, including steep ($m_{TAW} >$ 0.125) dune features, bluffs, and coastal armoring structures such as revetments, are calculated using the TAW method (van der Meer, 2002). Wave runup on barriers is a function of the geometry and roughness of the structure, as well as the height and steepness of the incident wave. The TAW method provides a mechanism for calculating wave runup with adjustments made through reduction factors to account for surface roughness and the effects associated with the angle of wave approach.

With the TAW methodology the wave setup component of the TWL is calculated at the toe of the structure, and wave setup landward of the toe of the structure is not included. Wave setup seaward of the toe of the structure is computed with the DIM, using the nearshore slope, m_{DIM} . Wave setup is not included for cases where waves would not have broken prior to reaching the toe of the structure.

The reference water level at the toe of the structure for runup calculations using the TAW method is defined as the 2-percent Dynamic Water Level (DWL2%). The dynamic water level is the sum of the measured SWL, the static wave setup, $\overline{\eta}$, and the dynamic wave setup, η_{rms}. Because DIM provides the static setup at the shoreline and not the barrier toe, and the magnitude of static wave setup varies significantly with depth across the surf zone, from a maximum at the shoreline to approximately zero seaward of the breaking point, a reduction to the static setup component is applied for cases where the barrier toe elevation is inundated by the SWL and the TAW method is used for computing wave runup. The dynamic setup, however, varies insignificantly across the surf zone and requires no adjustment.

This procedure involves computing the static wave setup at the shoreline and at the toe location to determine a static setup reduction factor to be applied to the static wave setup calculated using DIM. The wave setup at the shoreline and toe location and subsequent reduction factor are based on the root mean square of the breaking significant wave height $(H_b)_{rms}$, and the depth at the toe of the barrier relative to SWL, h. The $(H_b)_{rms}$ is determined using the deepwater equivalent significant wave height, H'_0, and the peak wave period, T_P , according to:

$$(H_b)_{rms} = 0.714 \left(\frac{\kappa}{g}\right)^{1/5} \left(\frac{H_0'^2 C_0}{2}\right)^{2/5} \tag{13}$$

Where κ is the breaker criterion equal to 0.78 and C_0 is the deepwater wave celerity, $C_0 = L_0 / T_P$. The static wave setup at the SWL shoreline is:

$$\overline{\eta}_0 = 0.189 (H_b)_{rms} \tag{14}$$

And the static wave setup at the toe of the engineered barrier is:

$$\overline{\eta}(h) = 0.189 (H_b)_{rms} - 0.186h \tag{15}$$

The static wave setup reduction factor, γ_η is then a ratio of the static wave setup at the toe to the static wave setup at the SWL shoreline, or:

$$\gamma_\eta = \frac{\overline{\eta}(h)}{\overline{\eta}_0} \tag{16}$$

This reduction factor is then applied to the DIM static wave setup to compute a depth-adjusted static wave setup at the toe of the engineered barrier,

$$\bar{\eta}' = \gamma_\eta \bar{\eta} \tag{17}$$

The 2-percent Dynamic Water Level ($DWL_{2\%}$) is thus calculated as:

$$DWL_{2\%} = \bar{\eta}' + 2\eta_{rms} + SWL \tag{18}$$

The next step is to compute the wave height at the toe of the barrier and the resultant wave runup on the barrier. Let H_{m0} represent the spectral significant wave height at the toe of the structure. If the $DWL_{2\%}$ depth at the structure toe is found to be too shallow to support the calculated wave height, the wave was assumed to be depth-limited and the incident wave height was calculated using a breaker index of 0.78, whence $H_{m0} = 0.78\ h_{toe}$. The average slope for use in the TAW methodology, m_{TAW}, is calculated iteratively across the surf zone between the still water line minus $1.5H_{m0}$ and the runup limit. The lower slope point must never be below the toe, however, even if SWL - $1.5H_{m0}$ falls below the toe (van der Meer, 2014). In these cases, the lower slope point is set at the toe. Since the runup limit is initially unknown, the still water level plus $1.5H_{m0}$ is chosen as a first estimate (Figure 5.5). If the runup limit exceeded the selected crest, the runup limit was set at the crest. The general formula of TAW for calculating the 2-percent wave runup on barriers is

$$R_{2\%} = 1.77 H_{m0} \gamma_r \gamma_b \gamma_\beta \xi_{0m} \qquad \text{if: } 0.5 \le \gamma_\beta \xi_{0m} < 1.8$$

or:

$$R_{2\%} = H_{m0} \gamma_r \gamma_b \gamma_\beta \left(4.3 - \frac{1.6}{\sqrt{\xi_{0m}}} \right) \qquad \text{if: } 1.8 \le \gamma_\beta \xi_{0m} \tag{14}$$

Where, $R_{2\%} = 2\sigma_2$ is the wave runup height exceeded by 2 percent of the incoming waves; H_{m0} is the spectral significant wave height at the structure toe; γ_r is the influence coefficient for roughness element of slope; γ_b is the influence coefficient for a berm; γ_β is the influence coefficient for oblique wave attack; $\xi_{0m} = m_{TAW} / (H_{m0} / L_m)^{0.5}$ is the Iribarren number based on wave parameters at the toe of the structure. Influence factors for roughness, the presence of a berm, and oblique wave attack are selected according to Table D.4.5-3 in the Final Draft *Guidelines for Coastal Flood Hazard Analysis and Mapping for the Pacific Coast of the United States* (FEMA, 2005), hereafter referred to as the Pacific Guidelines. The roughness reduction factor is set to 1.0 for a smooth concrete seawall or sheet pile barrier.

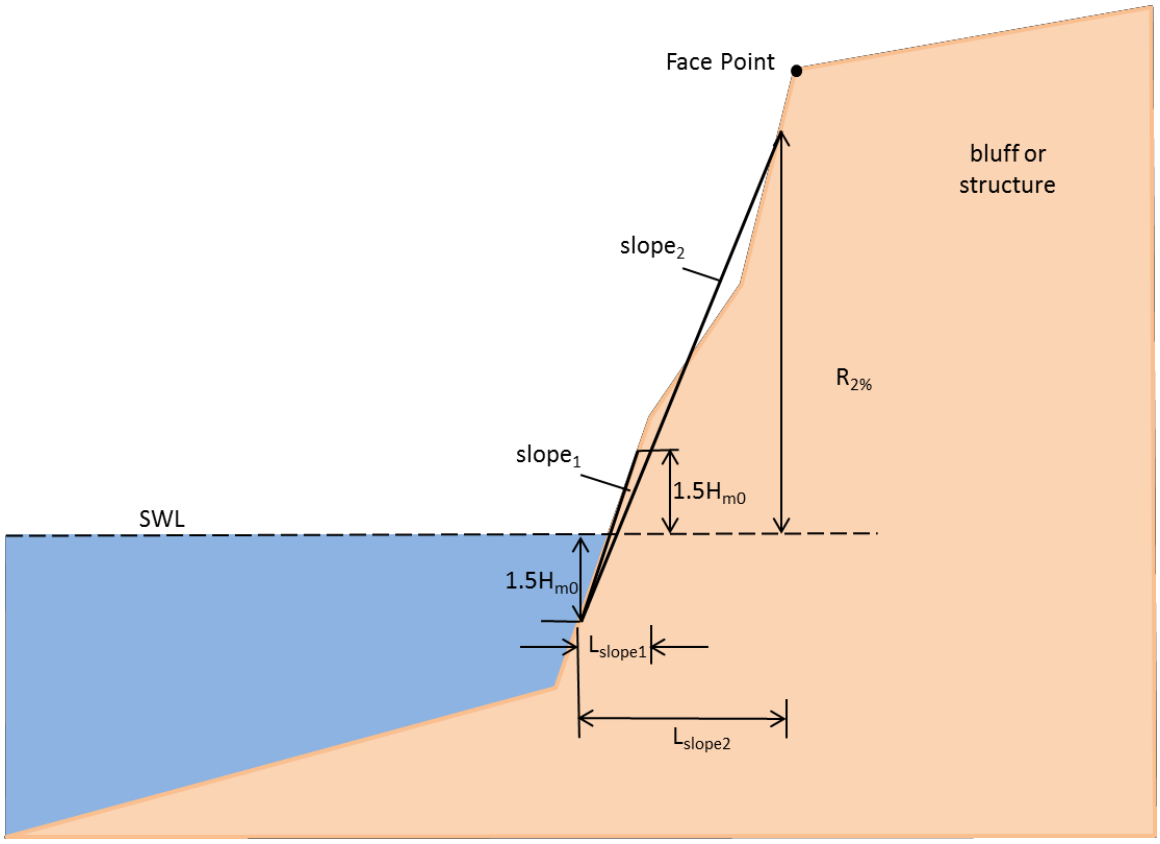


Figure 5.5: Determination of an Average Slope of Hard Back-Shore Formations (Bluff or Barriers) Based on an Iterative Approach, (Corrected from van der Meer, 2002)

The influence factor for oblique wave attack is calculated at each time step in the CDIP wave record (see Section 6). The spectral significant wave height H_{m0} is shoaled and refracted from a deep water point to the structure toe. The wave direction at the toe is compared to the transect orientation, perpendicular to the shoreline, to determine the angle of wave attack. For cases in which waves break seaward of the structure toe, the wave direction is taken from the point of breaking; i.e., where the incident wave height at the toe is depth-limited and calculated using a breaker index of 0.78, whence: $H_{m0} = 0.78\, h_{toe}$.

Incident wave runup, $R_{2\%} = 2\sigma_2$ is then statistically combined with the reduced dynamic wave setup as with the application of DIM, and added to SWL and static wave setup to yield the total water level, TWL, or:

$$TWL = SWL + \bar{\eta}' + 2.0\sqrt{\eta_{rms}^2 + \left(\frac{R_{2\%}}{2}\right)^2} \qquad (15)$$

For non-vertical structures with slopes greater than 1:1, the TAW manual after van der Meer (2002) suggests using the TAW method with an additional vertical wall reduction factor, γ_v, to account for runup on very steep (but not vertical) slopes. With steep slopes, the Iribarren number $\xi_{0m} = m_{TAW} / \left(H_{m0} / L_m\right)^{0.5}$ becomes large which means that the waves will not break. To keep the relationship between the type of breaking and the Iribarren number, the vertical wall must be schematized as a 1:1 slope. Therefore, the barrier slope was set to 1:1 for the Iribarren number calculation, and a vertical wall reduction factor for steep slopes was applied:

$$\gamma_v = 1.35 - 0.0078 \tan^{-1} m_{face} \tag{16}$$

where the face slope, m_{face} measured between the selected toe and face locations, is the angle of the actual slope in degrees (van der Meer, 2002). While this approach is based on work done for vertical walls atop dikes, sensitivity testing showed that it compared well with the TAW method and the Shore Protection Method (SPM) (USACE, 1984) for vertical walls as an intermediate approach to calculating runup on steep slopes. The use of this vertical wall reduction factor accounts for wave reflection expected on slopes greater than 45 degrees, and this approach generates results that fall between those for a 45- degree slope and those for a vertical wall.

Wave overtopping occurs when a potential runup elevation exceeds a structure's profile crest elevation. When wave runup is shown to exceed the barrier crest, the severity of wave overtopping is evaluated based on the mean overtopping rate, *q*. The required input parameters for computing the mean overtopping discharge are the wave height and freeboard, defined as the difference between the DWL2% and the structure crest. The 1-percent-annual- chance TWL available from the wave runup and extreme value analyses is a statistical value and is not associated with either a specific wave height or DWL2%. Therefore, the maximum wave height at the structure toe and the maximum and average DWL2% associated with the 32 annual maximum TWLs were chosen for use with the 1-percent TWL to estimate the 1-percent overtopping hazard.

Mean overtopping rates, *q*, were computed following Table VI-5-13 in the Coastal Engineering Manual (USACE, 2006) which presents an overtopping formula for impermeable and permeable barriers and structures according to:

$$Q' = 0.82\sqrt{gH_s^3} \exp\left(-\frac{3.0R_c}{H_s \gamma_\beta \gamma_s}\right) \tag{17}$$

Where H_s, is the significant wave height at the structure, R_c is the freeboard, γ_β is the influence factor for oblique wave attack, and γ_s is the influence factor for porosity. To conservatively maximize the overtopping potential, H_s and R_c are selected as the maximum wave height at the structure and the minimum freeboard between the highest DWL2% and the barrier crest elevation.

5.7) Beach Profile Calculations: A critical set of inputs to the wave setup, total runup and total water level (TWL) computations are the profile slope terms, m_{DIM}, m_{TAW}, and m_{face}. These are calculated from the beach and shore rise profiles during extremal wave events. Since there are only a limited set of beach profile measurements at Doheny State Beach, (and virtually none of these measurements have been performed during extremal wave events), the beach profile and its slope must be represented by model calculations that have been calibrated using the available set of beach profile measurements. Beach profile measurements at Doheny State Beach have been conducted by the US Army Corps of Engineers, USACOE (1991), Coastal Environments, (2014), and Coastal Frontiers (2014).

It is well known that beach and nearshore bottom profiles change seasonally in response to seasonal wave climate variations as shown in Figure 5.6, (cf: Inman et al, 1993; Jenkins and Inman 2006); and that seasonal transitions between summer and winter equilibrium states cause seasonal changes in the mean shoreline (Equation 7).

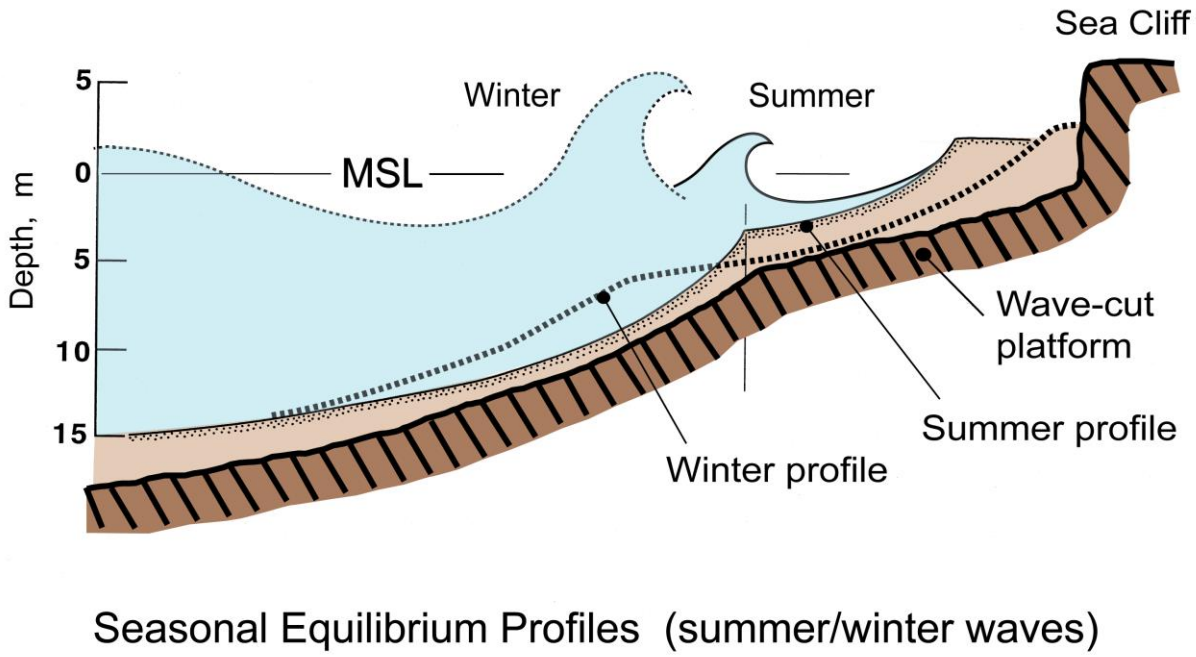


Figure 5.6: Schematic of summer and winter equilibrium beach profiles, from Inman, et al (1993).

Short period waves during summer (from the spin up of winds from the local North Pacific High) cause the inner bar-berm section of the beach profile to build up and steepen; while long period storm swells during winter from the Aleutian low cause the

bar-berm profile to flatten, and transfer beach sand to the outer shore-rise profile. These changes between summer and winter equilibrium states are predicted from the long-term wave record (Section 6) applied to the well-tested elliptic cycloid solutions after Jenkins and Inman (2006). The elliptic cycloid represents the equilibrium beach profile with a curve that is traced out by following a point on the circumference of a rolling ellipse (Figure 5.7)

The elliptic cycloid solutions were developed for beach profiles by Jenkins and Inman, (2006) using equilibrium principles of thermodynamics applied to very simply representations of the nearshore fluid dynamics. Equilibrium beaches are posed as isothermal shorezone systems of constant volume that dissipate external work by incident waves into heat given up to the surroundings. By the maximum entropy production formulation of the second law of thermodynamics (the law of entropy increase), the shorezone system achieves equilibrium with profile shapes that maximize the rate of dissipative work performed by wave-induced shear stresses. Dissipative work is assigned to two different shear stress mechanisms prevailing in separate regions of the shorezone system, an outer solution referred to as the *shorerise* and a *bar-berm* inner solution (Figure 5.7a). The equilibrium shorerise solution extends from closure depth (zero profile change) to the breakpoint, and maximizes dissipation due to the rate of working by bottom friction. In contrast, the equilibrium bar-berm solution between the breakpoint and the berm crest maximizes dissipation due to work by internal stresses of a turbulent surf zone. Both shorerise and bar-berm equilibria were found to have an exact general solution belonging to the class of elliptic cycloids.

The elliptic cycloid solution is a curve allows all the significant features of the equilibrium profile to be characterized by the eccentricity and the size of one of the two ellipse axes. These two basic ellipse parameters are related herein to both process-based algorithms and to empirically based parameters for which an extensive literature already exists. The elliptic cycloid solutions reproduce realistic and validated wave height, period and grain size dependence and demonstrated generally good predictive skill in point-by-point comparisons with measured profiles (Jenkins and Inman, 2006 display).

To understand the formulation of the elliptic cycloid representation of the nearshore bottom profile and sensitivity to ocean conditions, we first review the nomenclature of the shorezone as shown schematically in Figure 5.7a. The seaward boundary of the shorezone is a vertical plane at the critical closure depth $\hat{h}_c$ (Figure 8a) corresponding to the maximum incident wave [e.g., *Kraus and Harikai*, 1983]. The landward boundary is a vertical plane at the berm crest (cross), a distance $\hat{X}_1$ from a bench mark. The cross-shore length of the system from the berm crest to closure depth is $\hat{X}_c$. The distance from the point of wave breaking to closure depth is $\hat{X}_{c2}$ such that $\hat{X}_c = \hat{X}_{c2} + \hat{X}_2$, where $\hat{X}_2$ is the distance from the berm crest to the origin of the shorerise profile near the wave breakpoint.

We consider equilibrium over time scales that are long compared with a tidal cycle and profiles that remain in the wave dominated regime where the relative tidal range (tidal range/H) < 3 [*Short*, 1999]. Under these conditions, the curvilinear solution to the bottom profile which satisfies the maximum entropy production formulation of the *Second Law of Thermodynamics* can be expressed in polar coordinates (r, θ) as:

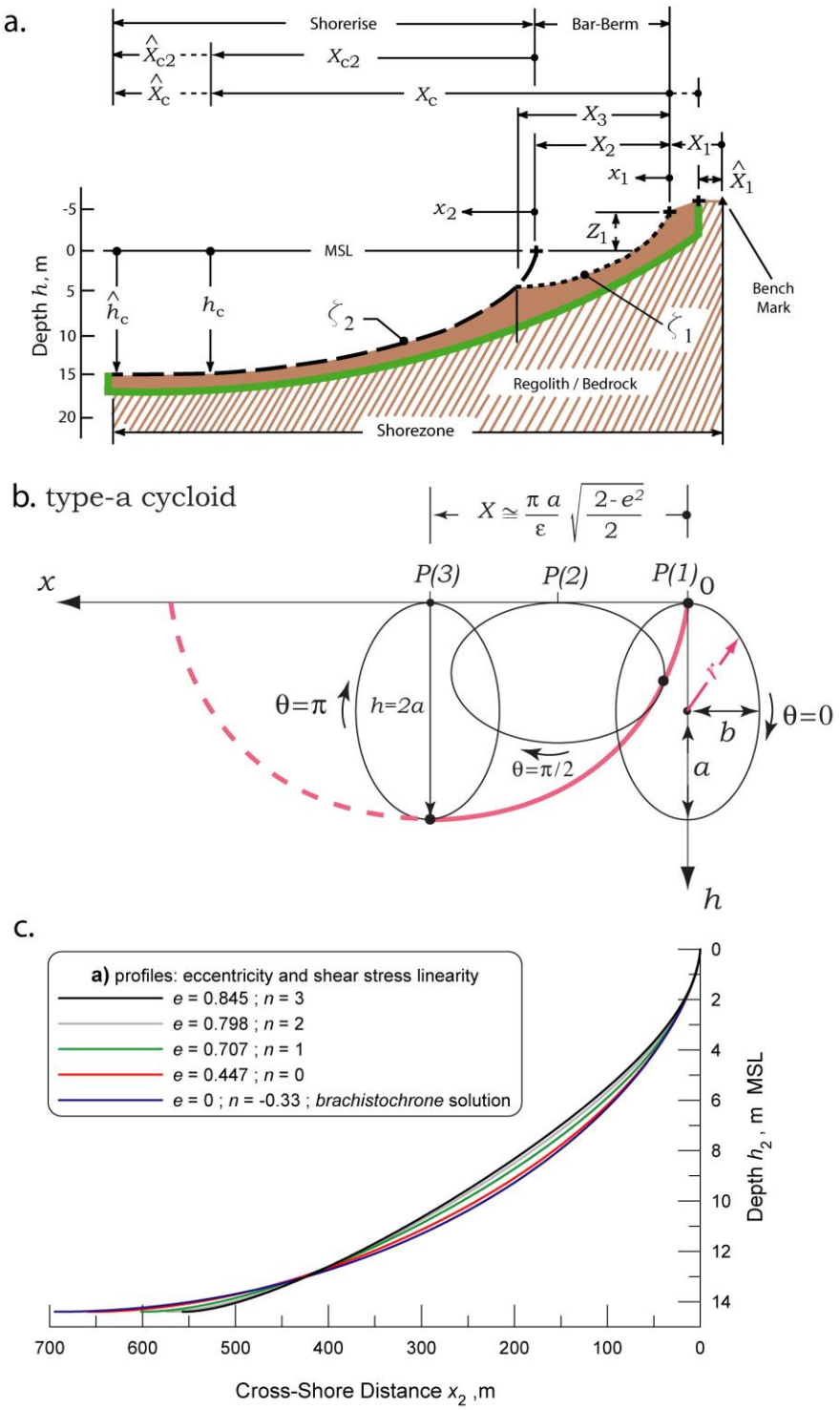


Figure 5.7. Equilibrium beach profile a) nomenclature, b) elliptic cycloid, c) Type-a cycloid solution.

$$x = x_2 = \frac{2r\, I_{\mathrm{e}}^{(k_{1,2})}}{\pi\, \varepsilon}\,(\theta - \sin\theta) \tag{18}$$

where r is the radius vector measured from the center of an ellipse whose semi-major and semi-minor axes are a, b and $I_{\mathrm{e}}^{(k)}$ is the elliptic integral of the first or second kind. This curve is what a point on the circumference of an ellipse would trace by rolling through some angle θ , (Figure 3.8b); hence the name elliptic cycloid. The polar equivalent of the type-a cycloid shown in Figure 3.8b has a radius vector whose magnitude is:

$$r = r_{\mathrm{a}} = \left[\frac{a^2\, b^2}{a^2 \sin^2\theta + b^2 \cos^2\theta}\right]^{1/2} = \frac{a\sqrt{1-e^2}}{\sqrt{\sin^2\theta + (1-e^2)\cos^2\theta}} \tag{19}$$

where e is the eccentricity of the ellipse given by $e = \sqrt{1-(b^2/a^2)}$. The polar form of the type-a cycloid in Figure 5.7b is based on the elliptic integral of the second kind that has an analytic approximation, $I_{\mathrm{e}}^{(2)} = (\pi/2)\sqrt{(2-e^2)/2}$, see *Hodgman* [1947]. The inverse of (18) for the type-a elliptic cycloid gives the companion solution in terms of local water depth, h, as:

$$h = h_2 = \frac{\pi\varepsilon\; x_2}{2I_e^{(k_{1,2})}}\left(\frac{1-\cos\theta}{\theta-\sin\theta}\right) = r\left(1-\cos\theta\;\right) \tag{20}$$

The depth of water at the seaward end of the profile ($\theta = \pi$) is $h = 2a$ in the case of the type-a cycloid. The length of the profile X is equal to the semi-circumference of the ellipse,

$$X = \frac{2aI_{\mathrm{e}}^{(2)}}{\varepsilon} \cong \frac{\pi\; a}{\varepsilon}\sqrt{\frac{2-e^2}{2}} \qquad \text{at} \quad \theta = \pi \quad \text{(type-a cycloid)} \tag{21}$$

With (21) the bottom slope can be solved as:

$$m = \frac{\sin\theta_b + e^2(\cos\theta_b - 1)\sin\theta_b\cos\theta_b}{1-\cos\theta_b + e^2(\sin\theta_b - \theta_b)\sin\theta_b\cos\theta_b} \tag{22}$$

$$\text{Where: } \theta_b = \arccos\left[1 - 2\left(\frac{H_0'}{\Lambda\gamma\, h_c}\right)^{\alpha}\right] \tag{23}$$

The shoaling factor assumed for these bar-berm solutions ($\Lambda = 0.81$) was based on uniform shoaling of the incident wave conditions, while a mean value was chosen for gamma ($\gamma = 0.8$) from the data reported by *Raubenheimer et al.* [1996]. In equation (23) the term h_{c} is the *closure depth*, which represents the closest point to the shoreline where a stable seabed can be found, because it is the point beyond which all changes in the

beach profiles cease. It is calculated from Jenkins and Inman (2006) by the following parametric relation:

$$h_c = \frac{K_e H_\infty}{\sinh k h_c}\left(\frac{D_o}{D_2}\right)^{\psi} \tag{24}$$

where K_e and ψ are non-dimensional empirical parameters, D_2 is the shorerise median grain size; and D_o is a reference grain size. With $K_e \sim 2.0, \psi \sim 0.33$ and $D_o \sim 100\mu m$, the empirical closure depths reported in *Inman et al.* [1993] are reproduced by Figure 5.8. From Figure 5.8 we find closure depth increases with increasing wave height and decreasing grain size, as shown in Figure 3.7. Because of the wave number dependence of (8), closure depth also increases with increasing wave period.

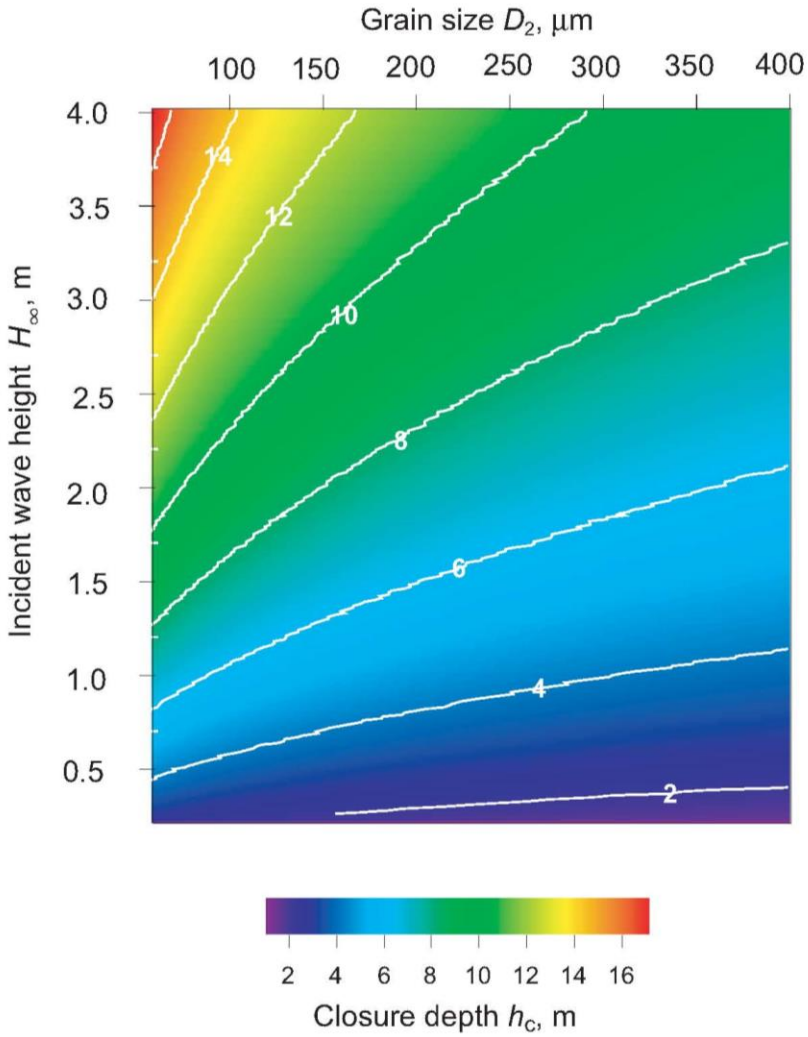


Figure 5.8: Closure depth contoured versus incident wave height and sediment grain size for waves of 15 second period, with $K_e \sim 2.0, \psi \sim 0.33$ and $D_o \sim 100\mu m$. D_2 is the shorerise median grain size; and D_o is a reference grain size.

6) Model Initialization:

This section develops the data bases necessary to evaluate Steps-4 & 5 of a sea level rise/coastal hazards analysis as outlined in Section 2.

6.1) Bathymetry: Bathymetry provides a controlling influence on all of the coastal processes that affect dispersion and dilution. The bathymetry consists of two parts: 1) a stationary component in the offshore where depths are roughly invariant over time; and 2) a non-stationary component in the nearshore where depth variations do occur over time. The stationary bathymetry generally prevails at depths that exceed closure depth which is the depth at which net on/offshore transport vanishes. Closure depth is typically -12 m to -15 m MSL in the Oceanside Littoral Cell, (Inman et al. 1993). The stationary bathymetry was derived from the National Ocean Survey (NOS) digital database. Gridding is by latitude and longitude with a 1 x 1 arc second grid cell resolution yielding a computational domain of 30.9 km x 18.5 km. Grid cell dimensions along the x-axis (longitude) are 25.7 meters and 30.9 meters along the y-axis (latitude).

For the non-stationary bathymetry data inshore of closure depth (less than -15 m MSL) nearshore and beach surveys were conducted by the US Army Corps of Engineers in 1985, 1990, 1996, 2001 and have been compiled in USACE (2001). These nearshore and beach survey data were used to update the NOS database for contemporary nearshore and shoreline changes that have occurred following the most recent NOS surveys.

To perform both the required wave shoaling and transport computations in the farfield of the SJCOO outfall diffusers, a large-domain grid is required to compute the effects of island sheltering and regional scale refraction and circulation due to the shallow banks of the continental margin (Figure 6.1). A nearfield grid (Figure 6.2) in the immediate neighborhood of the diffuser is nested inside the farfield grid and is used to calculate the brine discharge dilution and dispersion.

6.2 Shore-side Structures: Wave runup, and overtopping were analyzed at the shore-side facilities associated with the Doheny Desalination Project assuming present conditions and two future scenarios including sea level rise. These facilities included: Wellhead 1, elevation 17 ft. MLLW, at 33 27 43.99834 N, 117 41 13.25899 W; Wellhead 2, elevation 17 ft. MLLW, at 33 27 44.16980 N, 117 41 06.01901 W; Wellhead 3, elevation 18 ft. MLLW at 33 27 44.07064 N, 117 40 53.96544 W; Well Pump Electrical (North), at elevation 18 ft. MLLW at 33 27 48.54582 N, 117 41 02.09570 W; and Well Pump Electrical (South), at elevation 21 ft. MLLW at 33 27 47.44890 N, 117 40 54.92019 W; (see Figure 6.3).

6.3) Wave Forcing: Waves in deep water generally do not cause significant mixing. But shoaling waves produces bottom currents (referred to as *bottom wind*), cause scrubbing action against intake and discharge structures that result in vertical mixing of the nearfield water mass, and cause longshore and rip current circulation as a result of along shore variation in shoaling wave heights due to refraction over shelf bathymetry.

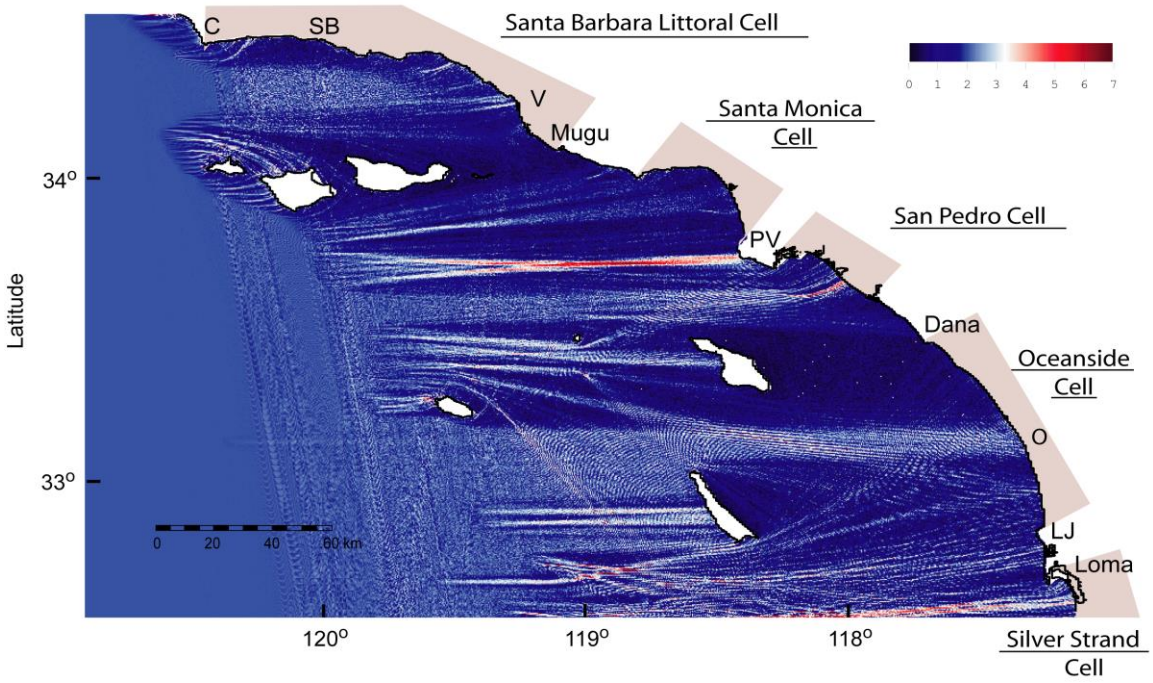


Figure 6.1: Far-field refraction/diffraction grid to simulate shoaling waves entering the Southern California Bight and Oceanside Littoral Cell. Results based on the 5 largest storms of the 1998 El Nino winter (from Jenkins and Wasyl, 2008b).

Figure 6.2: Near-field refraction/diffraction grid to simulate shoaling waves in the immediate neighborhood of Dana Point, SJCOO and Doheny Beach.

Figure 6.3: Shore-side infrastructure of the Doheny Desalination Project Evaluated for wave and tsunami inundation potential

Wave forcing to the SEDXPORT and VORTEX Lattice Models were derived from archival measurements of waves for the period 1980-2010, supplemented by wave burst measurements from the Acoustic Doppler Current Profiler (ADCP) measurements taken under the MBC *Applied Environmental Sciences* (MBC) marine environment studies. The archival wave records were obtained from the Oceanside, Dana Point, San Clemente, and Huntington Beach monitoring stations maintained by the Coastal Data Information Program, [CDIP, 2012, http://cdip.ucsd.edu]. To correct the archival data from widely spaced offshore monitoring sites to the nearshore of the SJCOO, raw data were entered into a refraction/diffraction numerical code, back-refracted out into deep water to remove local refraction and island sheltering effects, and subsequently forward refracted into the immediate neighborhood of the proposed Project. The backward and forward refractions of CDIP data were done using a numerical refraction-diffraction computer code called OCEANRDS. The primitive equations for this code are lengthy, but a listing of the codes for OCEANRDS are in Jenkins and Wasyl (2005).

An example of a reconstruction of the wave field throughout the Bight from the CDIP Oceanside buoy data is shown in Figures 6.1 for the 5 largest storms of the 1998 El Nino winter. Wave heights are contoured in meters according to the color bar scale and represent 6 hour averages, not an instantaneous snapshot of the sea surface elevation. Note how the sheltering effects of Catalina and San Clemente Islands have induced considerable variations in the neighborhood of the SJCOO and Dana Point Harbor. The wave height and direction parameters inside the Channel Islands are the values used as the deep water boundary conditions along the seaward face of the nearfield grid for the SJCOO Dana Point shoaling analysis.

Figure 6.4 gives the local forward refraction calculation into the nearfield domain of the SJCOO and the Doheny Desalination Project site (green box), due to the 100-year storm-wave event of 17-18 January 1998 after passing through the gaps in the continental margin and Channel Islands, (island sheltering effects, cf.Figure 6.1). Figure 6.4 gives extremal wave height variations along an 18.5 km section of coastline in the in the Dana Point region, including wave shoaling and reflection effects induced by the Dana Pt Harbor breakwater. Replication of the backward/forward refraction analysis on each of the 3 hour increments of the CDIP monitoring data produced continuous, unbroken records of the wave height, period and direction in the nearfield of the Doheny Desalination Project throughout the 1980-2010 period of record, as shown in Figure 6.5. The data in Figure 6.5 were supplemented by wave burst measurements from the Acoustic Doppler Current Profiler (ADCP) measurements taken at the SJCOO monitoring stations (MBC, 1998). Figure 6.6 gives the wave refraction/diffraction field in the SJCOO/Dana Pt. Harbor Littoral Sub-cell derived from these ADCP wave burst measurements. We find in Figures 6.4 & 6.6 that the refraction effects over local bathymetry create areas (indicated by red) where wave heights increase locally to 4 -5 m. In these areas, the shelf bathymetry has focused the incident wave energy and these regions of intensified wave energy are referred to as “bright spots”. The increased wave heights in these bright spots increases the wave run-up and induces local wave erosion. Conversely, the dark areas in Figures 6.4 & 6.6 (indicated by dark blue) where wave heights have been diminished are termed “shadows,” and represent areas of reduced run up and potential beach accretion For the January 1998 storm in Figure 6.6, the area around the SJCOO discharge site is indeed a bright spot in the local refraction pattern while the slant well sites for the Doheny Desalination Project are located in a shadow zone. Another wave shoaling phenomena at the slant well site is divergence of drift. Wave-driven longshore currents flow away from areas of high waves (away from bright spots) and converge on shadow regions. This convergence of the longshore current leads induces seaward flowing rip currents. Rip currents are advantages to shallow nearshore intake sites

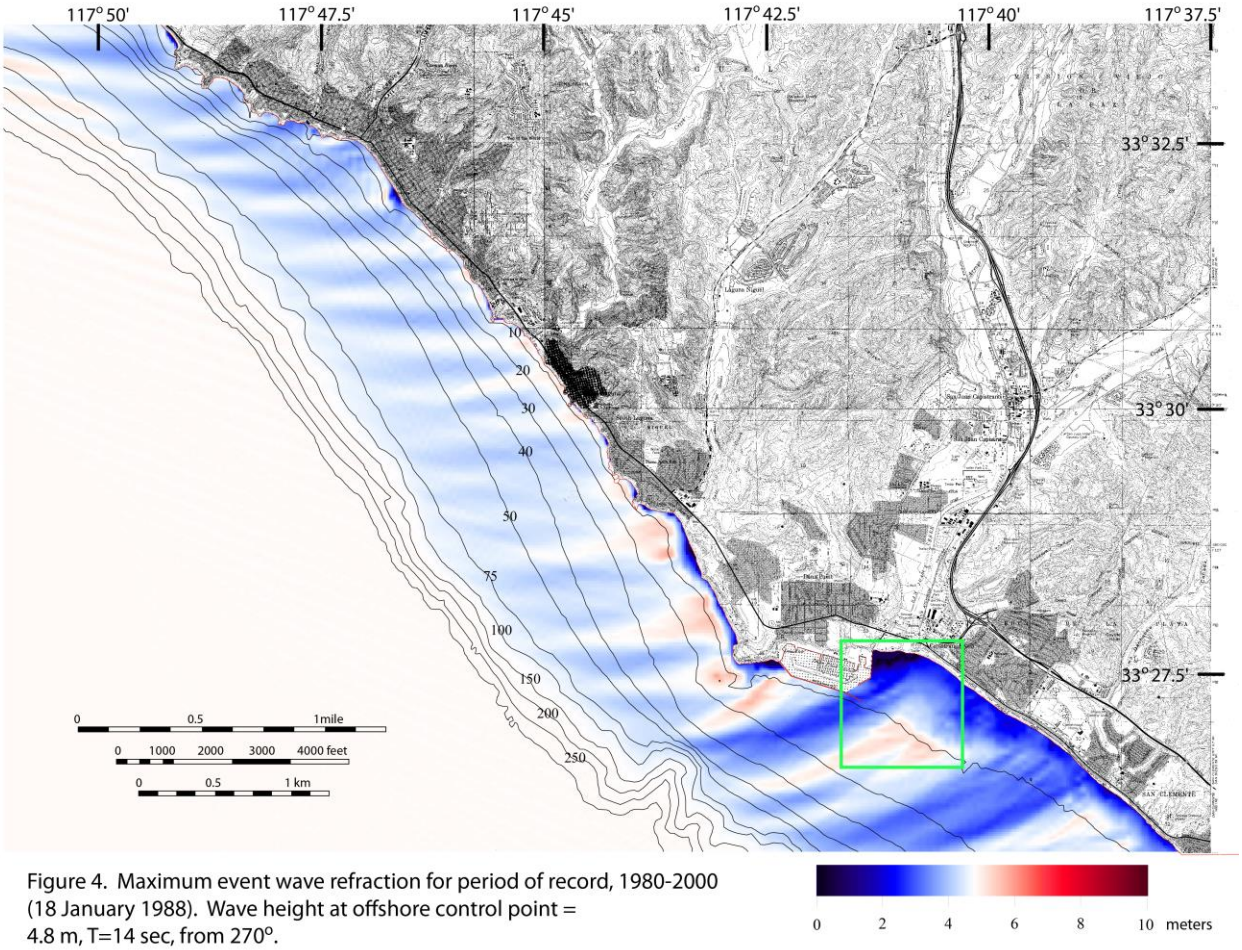


Figure 4. Maximum event wave refraction for period of record, 1980-2000 (18 January 1988). Wave height at offshore control point = 4.8 m, T=14 sec, from 270°.

Figure 6.4: Forward wave refraction/diffraction for the 100-year storm-wave event of 17-18 January 1998. These local refraction results are used to provide the point-to-point initializations for the wave setup and runup inputs to the total water level problem. The nearfield domain of the SJCOO and the Doheny Desalination Project is designated by the green box.

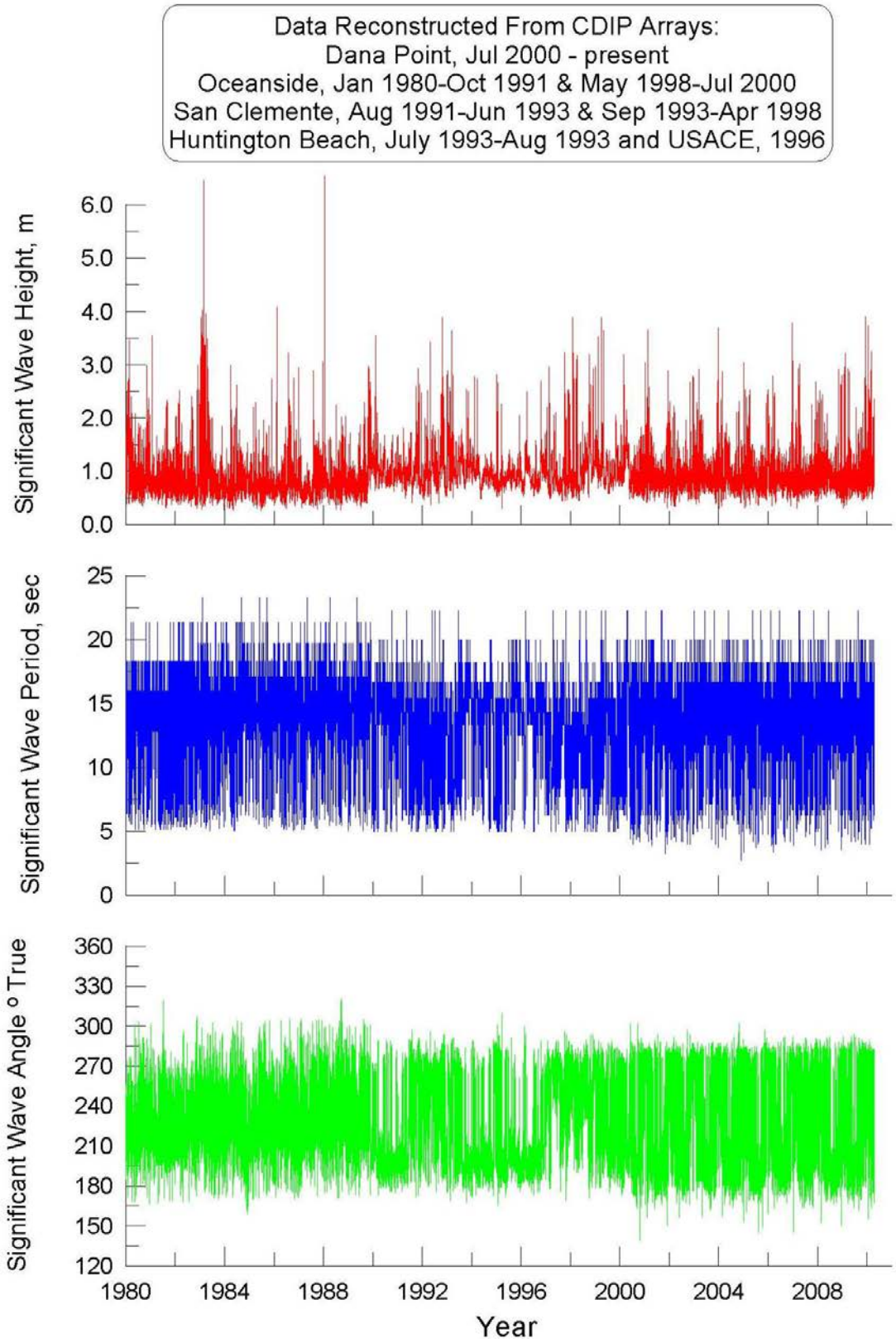


Figure 6.5: Archival wave forcing data 1980-2010 reconstructed for the SJCOO and Doheny Desalination Project modeling, from backward/forward refraction of regional CDIP wave monitoring data.

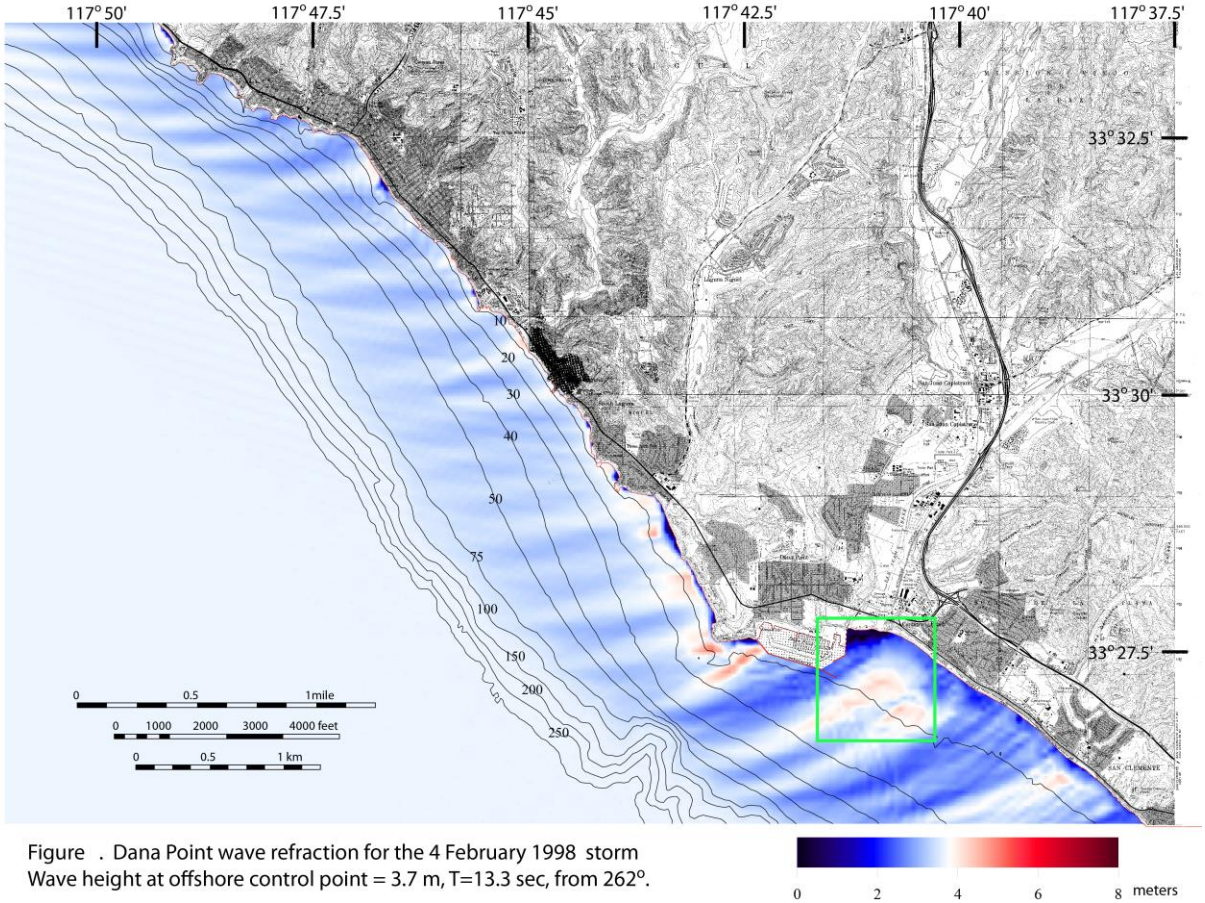


Figure 6.6: Wave refraction/diffraction field around the SJCOO site and the Doheny Desalination Project site derived from wave burst measurements from the Acoustic Doppler Current Profiler (ADCP) records taken under the MBC Applied Environmental Sciences (MBC) NPDES monitoring studies. Nearfield domain of the SJCOO and the Doheny Desalination Project site designated by green box.

because rip currents would advect storm water and urban run-off away from the shoreline and disperse it offshore in deeper water, thereby reducing potential for marine life impacts to nearshore and beach ecology. On the other hand, these same seaward flowing rip currents can also carry beach sand offshore, resulting in local beach erosion. Wave refraction/diffraction analyses of the 15 largest storm events in the 1980-2010 period of record are presented in Appendix-A. The 100 year event (1% event) was the two day storm of 17-18 January, 1988, and refraction/diffraction patterns for both days are also included in Appendix-A.

The composite 30-year wave record obtained from the CDIP archival data for 1980-2010 (Figure 6.5) was iteratively fit to Weibull (Type III) distributions with a range of *K-values* to find the best overall fit (highest correlation coefficient). A *K-value* of K = 1 was found to give an R-squared = 0.98, resulting in the extremal analysis curve shown in Figure 6.7. The red-line in Figure 6.5 is the Weibull Type III best fit and the crosses are the data points at the control point in 12 m water depth from Appendix-A refraction/diffraction analyses used to produce the best fit distribution. The Weibull Type III best fit projects a maximum significant wave height of $H_0' =$ 19.9 ft. with a probability of recurrence of 0.04% (return period = 2,500 yr); but such a wave has never been measured. The highest wave that was recovered from the refraction analysis in 12 m

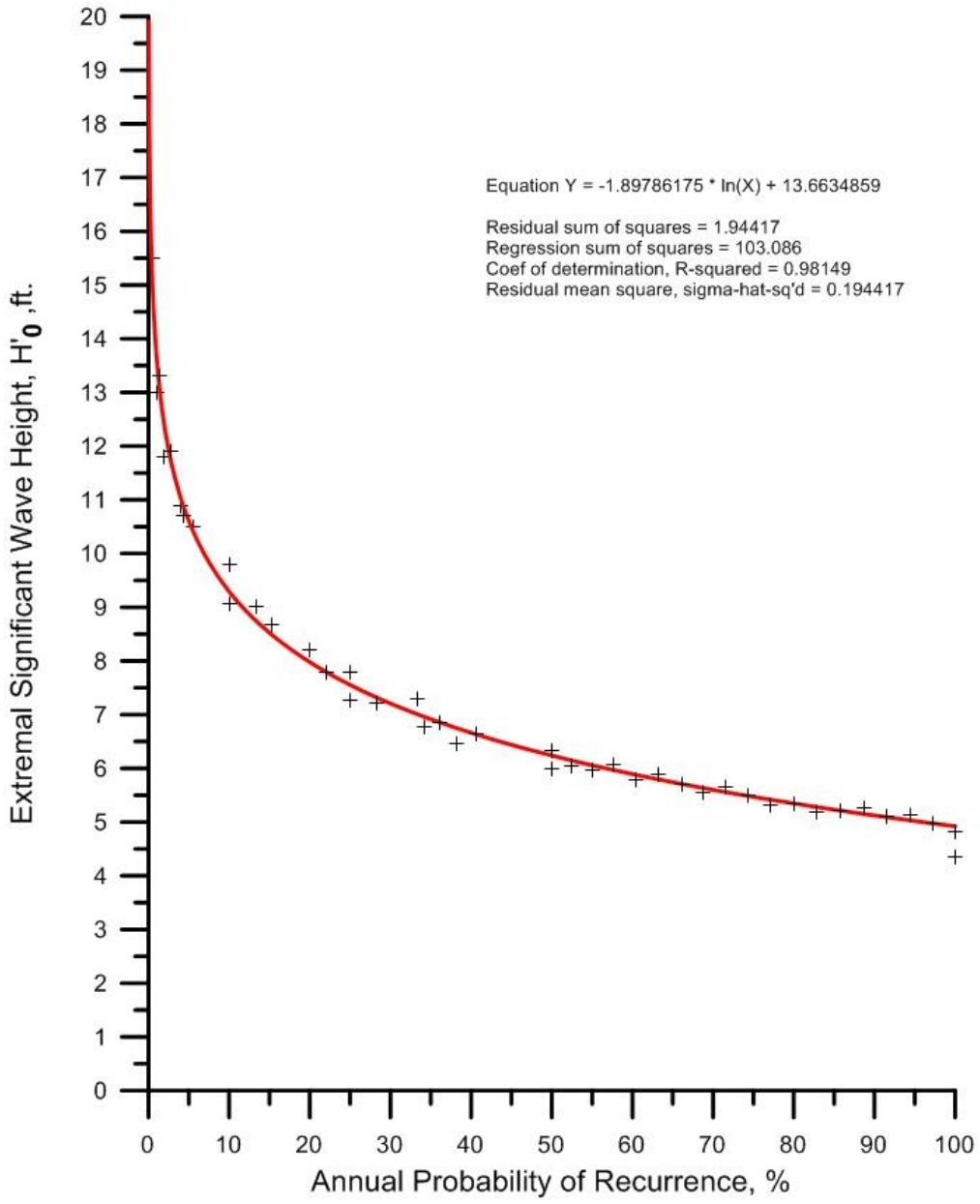


Figure 6.7 : Probability of recurrence of design wave heights based on Weibull extremal analysis of significant wave heights at Doheny State Beach. Analysis based on Weibull Type III distribution applied to 12 m local water depth with K = 1.0. Recurrence Probability $P(H)$ = 100%/T, where T = return period

of water depth was due to the 18, January, 1988 storm (Figure 6.4) with a significant wave height H_0' = 15.5 ft. and a probability of recurrence of 1.0% (return period = 100 yr). The extremal analysis curve in Figure 6.7 will be the computational basis of the extreme value analysis of wave setup, total runup and total water level (TWL) in Section 7.

6.4) Beach Erosion: Another critical set of inputs to the wave setup, total runup and total water level (TWL) computations are the profile slope terms, m_{DIM}, m_{TAW}, and m_{face}. These are calculated from equations (22) – (24) using measured beach profiles to calibrate the empirical factors in these equations, which include the shoaling factor, Λ, and the non-dimensional empirical parameters: K_e and ψ. Beach profile measurements at Doheny State Beach have been conducted by the US Army Corps of Engineers, USACOE (1991), Coastal Environments, (2014), and Coastal Frontiers (2014). Plots of the beach profiles measured by the US Army Corps of Engineers, USACOE (1991) and Coastal Environments, (2014) are shown in Figures 6.8 & 6.9. Figure 6.8 shows the shore rise and bar berm sections of the beach profiles immediately west of Well Heads # 2 and #1; where profile ranges R4 & R5 bracket beach slope conditions in front of Well Head # 2, and range DB 1890 measured by the US Army Corps of Engineers give slope conditions in front of Well Head # 1 (cf. Figure 6.3). Figure 6.9 shows the shore rise and bar berm sections of the beach profiles immediately west of Well Heads # 2 and #3; where profile range R5 provides beach slope conditions in front of Well Head # 2, and range R7 gives slope conditions in front of Well Head # 3 (cf. Figure 6.3).

Figures 6.10 and 6.11 give seasonal profile changes at Doheny State Beach immediately west of Well Head #3 over a number of years between 2001 and 2007. Figure 6.10 provides a generalization of the winter profiles, indicating an average nearshore slope, $m_{DIM} = 0.066$. Figure 6.11 indicates that the average nearshore slope in summer steepens to $m_{DIM} = 0.10$. Using these values to calibrate the elliptic cycloidal slope algorithms in equations (22)-(24), the variation of beach slope with on/offshore position in response to the potential range of extremal wave height was calculated according to Figure 6.12. Generally, across the inner portion of the beach profile closest to the DDP well heads the beach slopes become flatter in winter and steeper in summer, while both types of seasonal profiles develop offshore bars offshore during higher extremal wave conditions. This response is consistent with the well-known response of sandy beaches to increasing levels of incident wave energy; whereby the exposed inner section of the beach profile (the bar-berm profile) erodes and flattens in slope during winter or periods of high waves, while outer submerged portion of the profile (the shore-rise profile) develops offshore sand bar formations. Review of the composite surveys in Figures 6.8-6.11 reveals that variations in the beach widths around the well heads between summer and winter profiles are on the order of 50 ft. to150 ft. These relatively small range of seasonal variation in beach width indicates that Doheny State Beach is stable, as a consequence of being located at a sediment source, i.e. the San Juan Creek. The San Juan Creek is the second largest source of sediment for the Oceanside Littoral Cell and provides an average of 51,000 metric tons of beach grade sand to Doheny State Beach annually (Figure 6.13). This supply of new sediment provides adequate sediment cover for the beach to establish and maintain equilibrium profile adjustments throughout the most high energy El Nino winter/summer seasonal cycles.

Variations in the beach widths and sediment cover with time are modeled in the LCM module of the Coastal Evolution Model (Figure 5.1) using time-stepped solutions to the sediment continuity equation (otherwise known as the *sediment budget*) applied to the boundary conditions of the coupled control cell mesh diagramed schematically in Figure 6.14. The sediment continuity equation is written (Jenkins, et al, 2007):

$$\frac{\partial q}{\partial t} = \frac{\partial}{\partial y}\left(\varepsilon \frac{\partial q}{\partial y}\right) - V_l \frac{\partial q}{\partial y} + J(t) - R(t) \qquad (25)$$

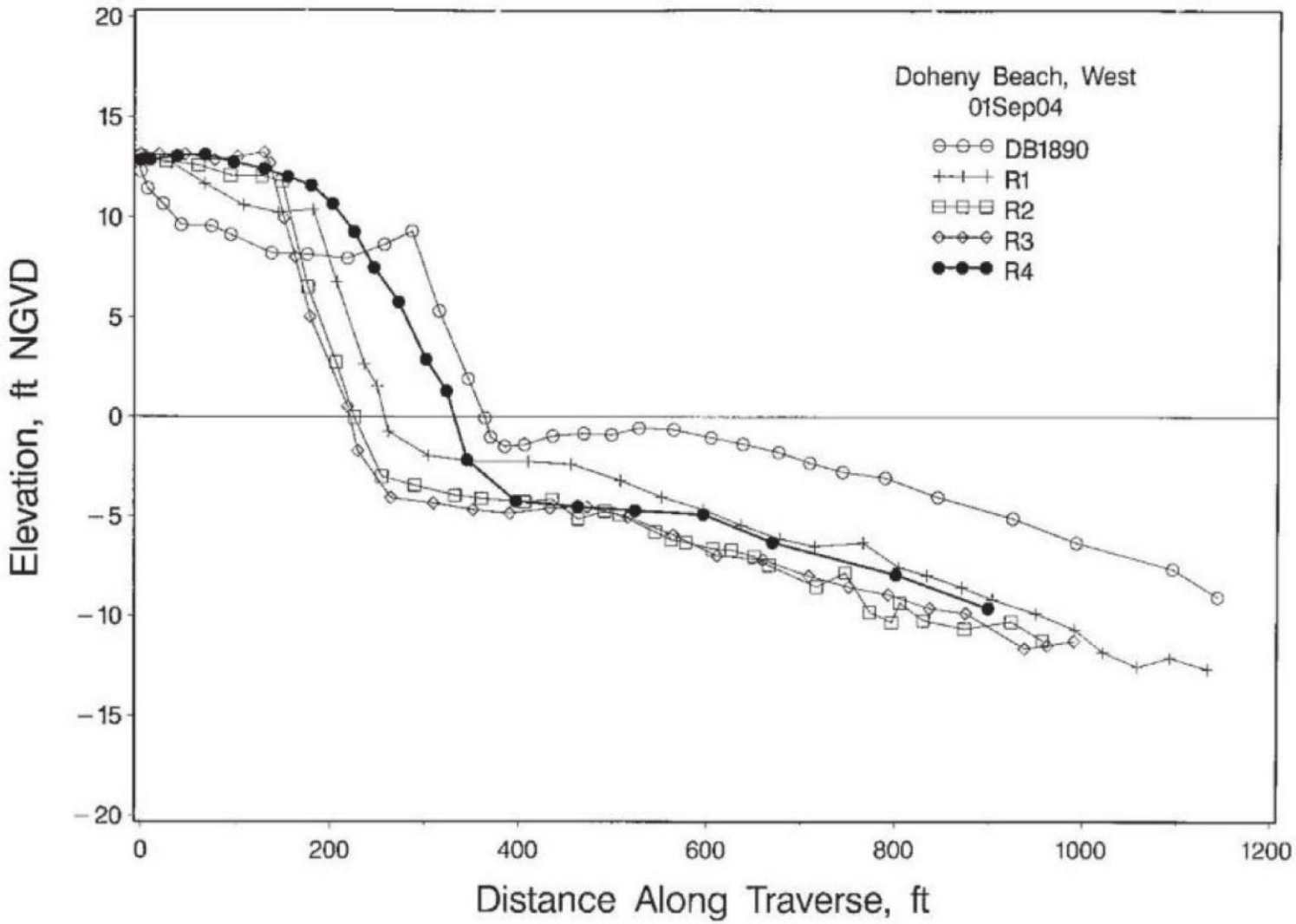


Figure 6.8: Beach profile surveys of Doheny Beach range lines adjacent to Well heads # 2, and #1. Data from Coastal Environments, (2014).

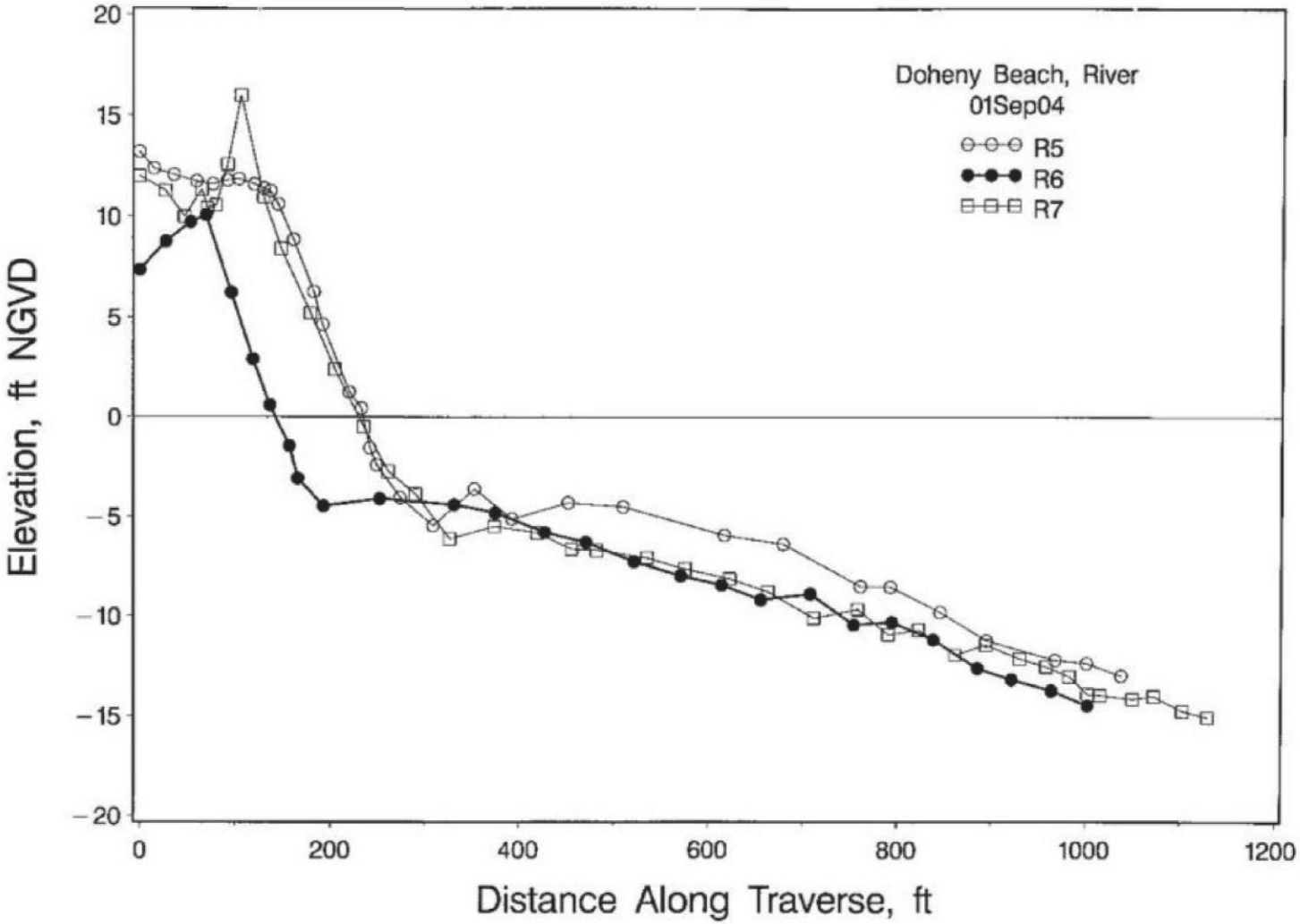


Figure 6.9: Beach profile surveys of Doheny Beach range lines adjacent to Well heads # 2, and #3. Data from Coastal Environments, (2014).

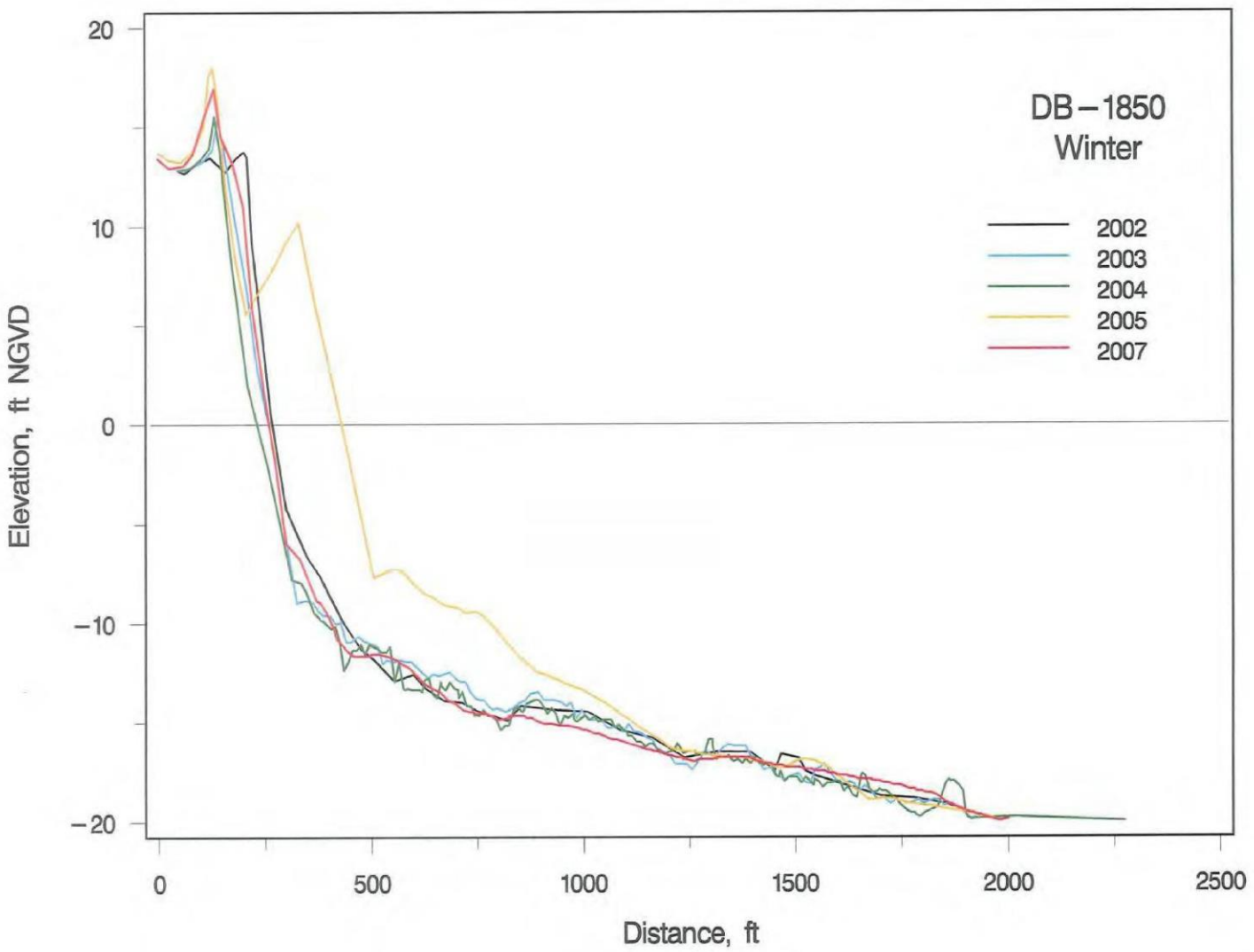


Figure 6.10: Winter beach profile surveys of Doheny Beach range lines adjacent to Well head # 3. Surveys due to the US Army Corps of Engineers. Data provided by Coastal Frontiers, (2014).

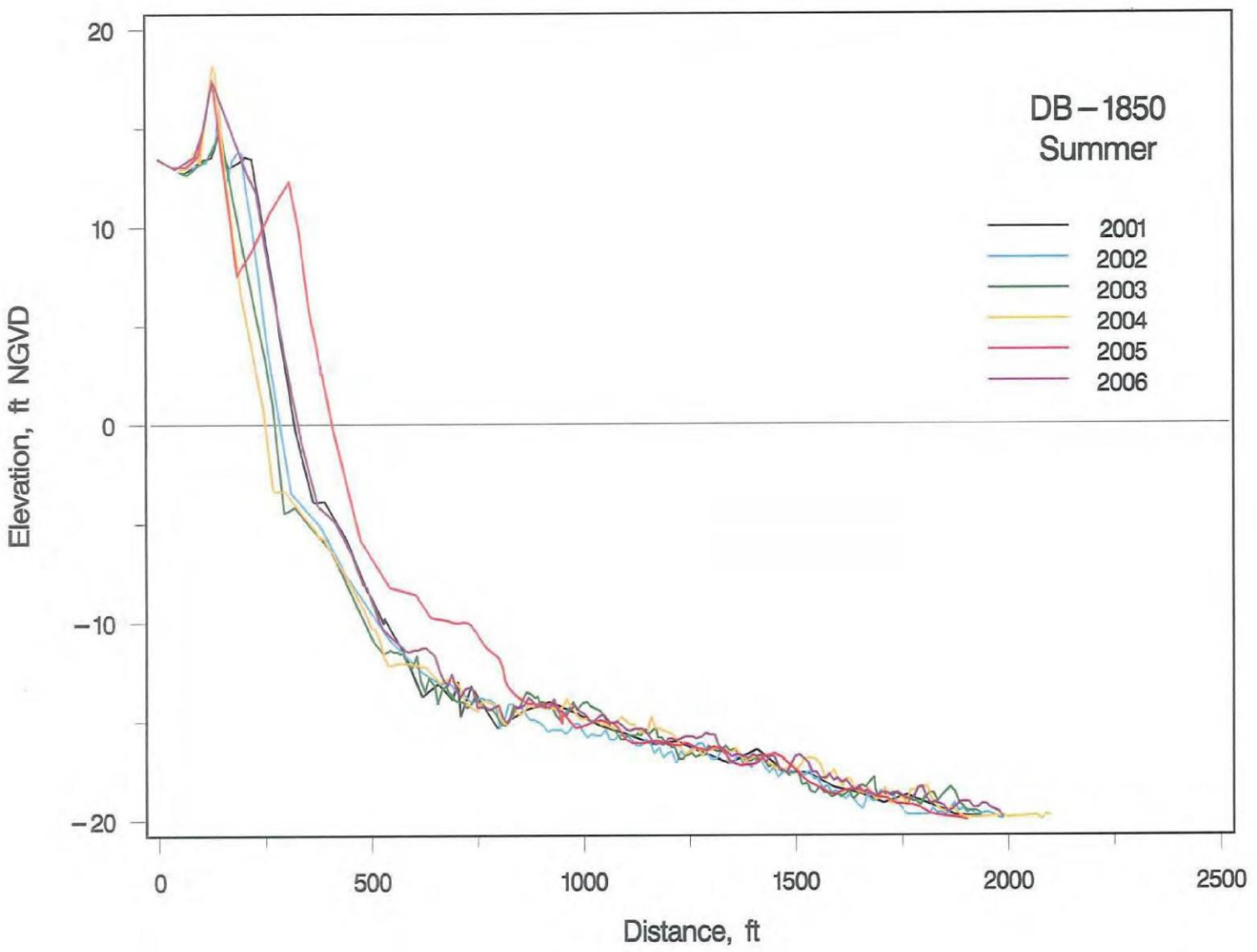
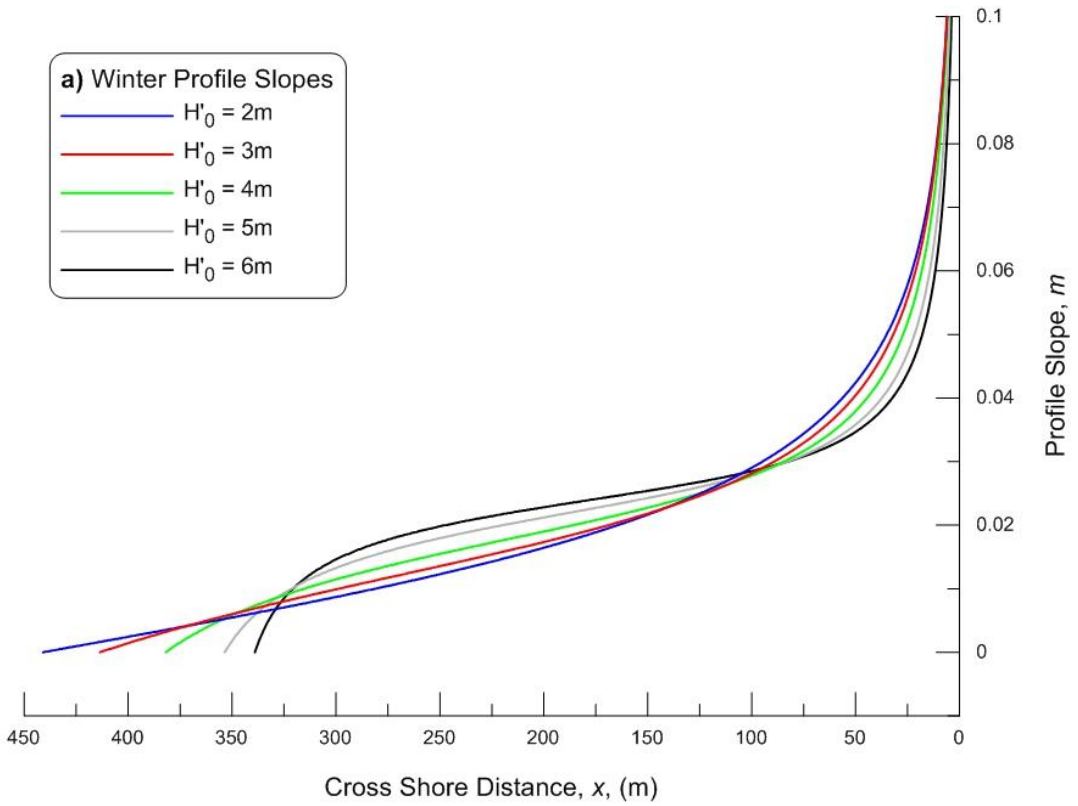


Figure 6.11: Summer beach profile surveys of Doheny Beach range lines adjacent to Well heads # 3. Surveys due to the US Army Corps of Engineers. Data provided by Coastal Frontiers, (2014).



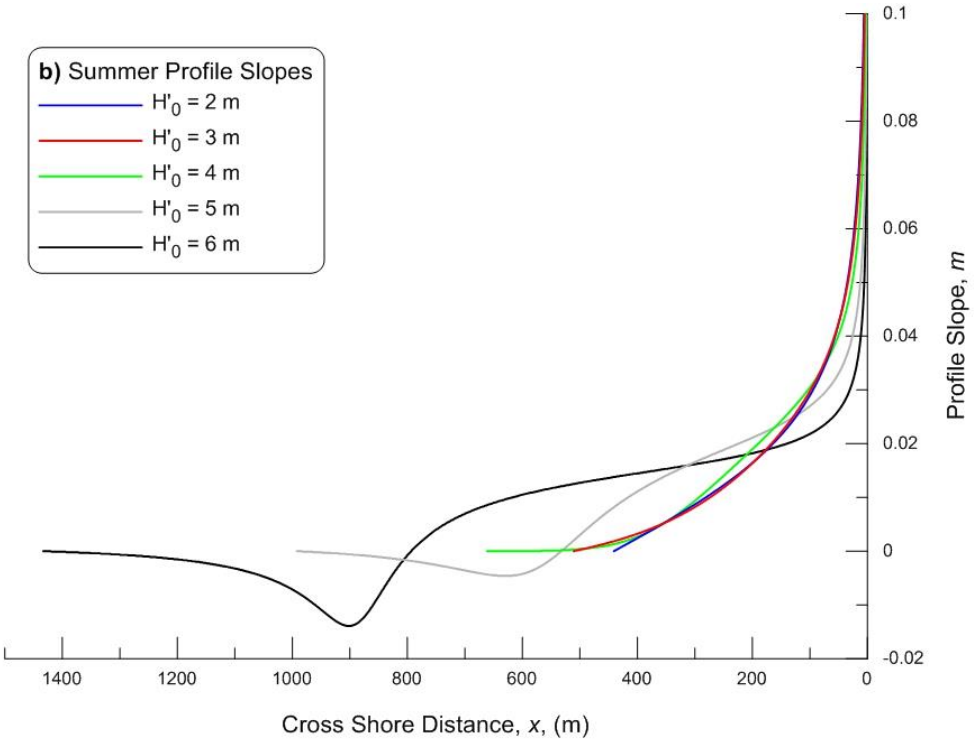


Figure 6.12. Family of elliptic cycloid slope solutions in the bar berm: a) type-a cycloids; b) type-b cycloids. Cycloids scaled for : H'_0 = 2 - 6 m; T = 15 sec; m_{DIM} = 0.06 (winter); m_{DIM} = 0.1 (summer); γ = 0.8; Γ = 0.76; Λ = 0.81

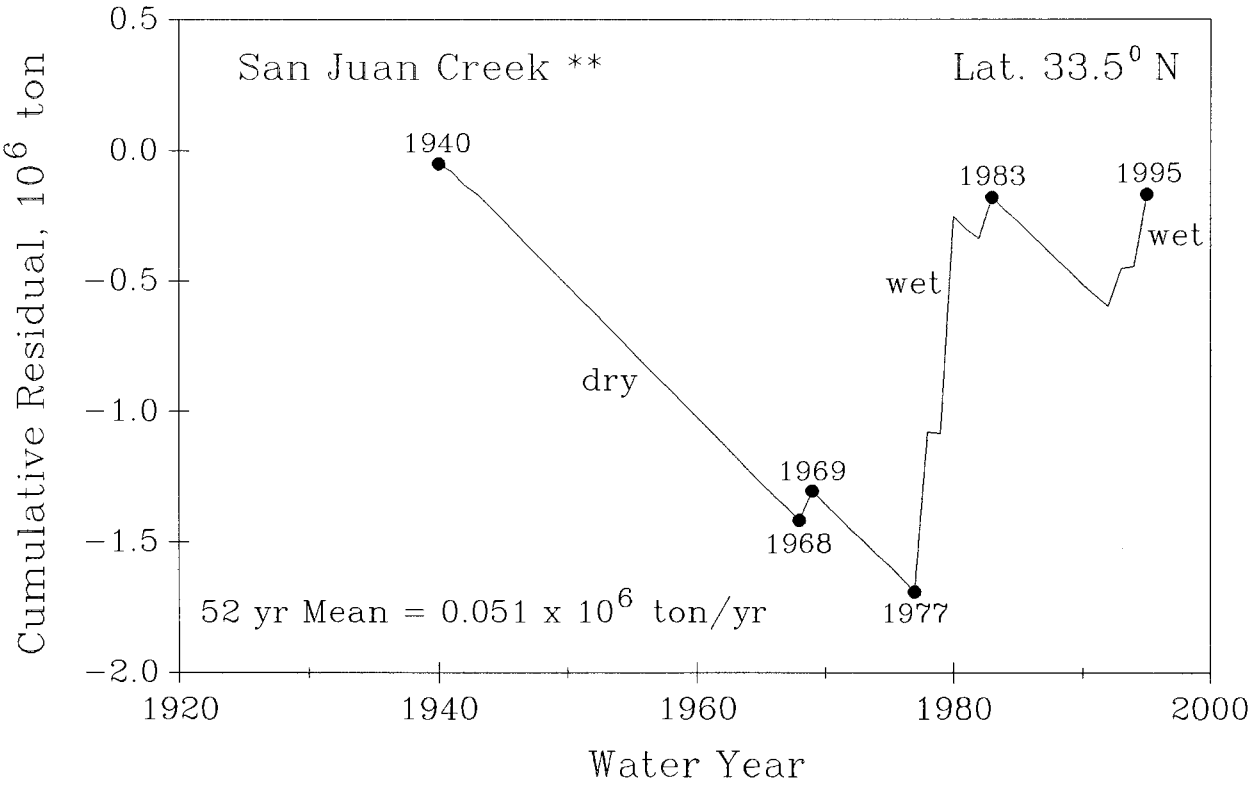


Figure 6.13. Cumulative residual time series of sediment flux for the San Juan Creek calculated using a 56-year mean (1940-1995), from Inman and Jenkins (1999).

In equation (25) q is the sediment volume per unit length of shoreline (m^3/m) and *dq/dt* is the sediment volume flux ($m^3/m/day$), ε is the mass diffusivity, V_l is the longshore current, *J(t)* is the flux of new sediment from the San Juan Creek, and *R(t)* is the flux of sediment lost to sinks , in this case, the scour holes near the mouth of the San Juan Creek following river floods. The first term in (1) is the surf diffusion term while the second is the advective term due to the longshore current. For any given control cell along Doheny State Beach, equation (25) may be discretized in terms of the rate of change of "beach volume", Λ, in time increment Δt , given by:

$$\frac{d\Lambda}{dt} = J(t) + \frac{q_{in} + q_{out}}{\Delta t} \qquad (26)$$

Sediment is supplied to the control cells in Figure 6.14 by the sediment yield from the rivers and beach nourishment, $J(t)$ by the influx of sediment volume due to littoral drift from up-coast sources, q_{in} (beach-fill). Sediment is lost from the control cell due to the action of wave erosion

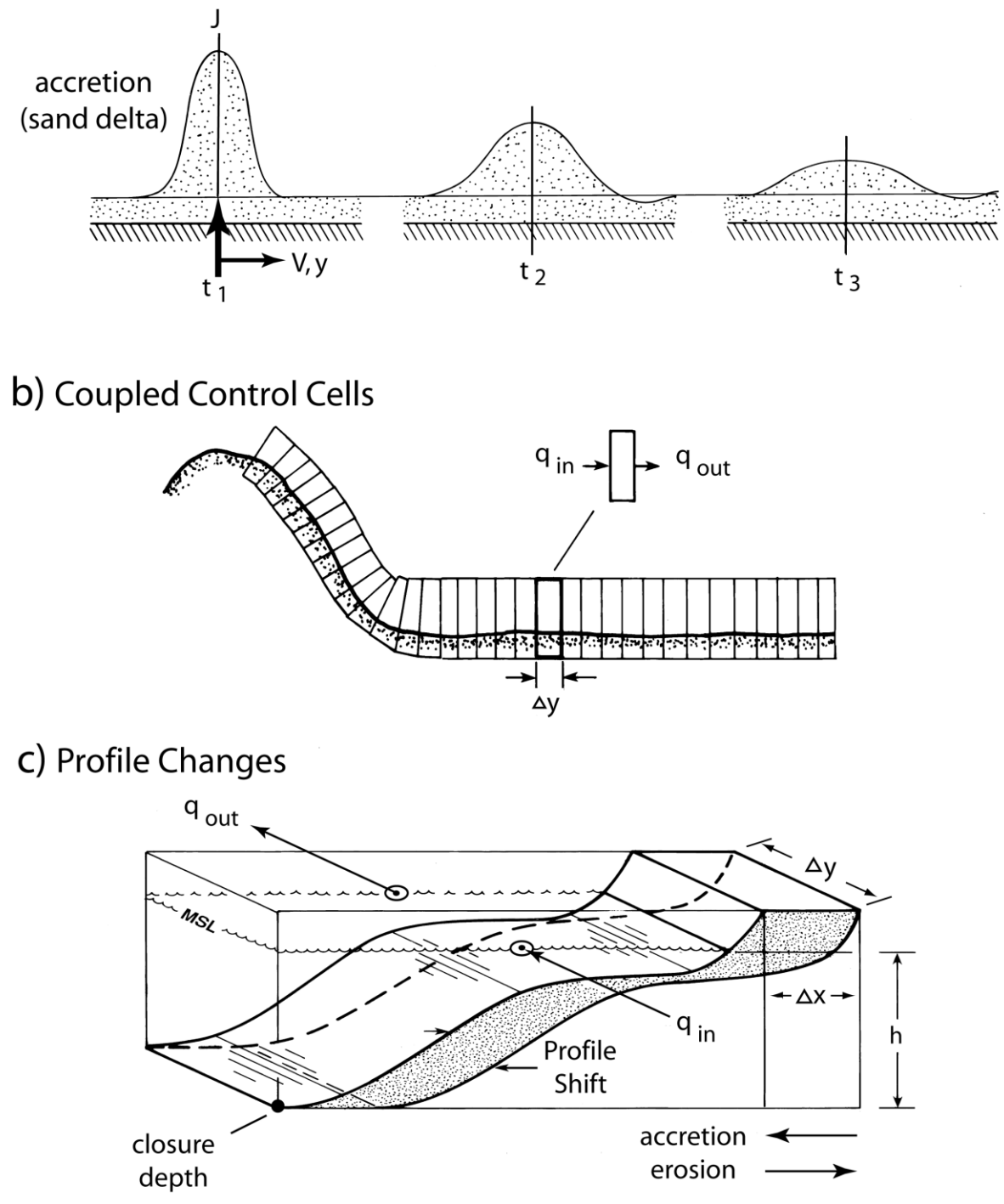


Figure 6.14: Computational approach for modeling changes in beach width and shoreline positon after Jenkins, et. al., (2007).

and expelled from the control cell by exiting littoral drift, q_{out}. Here fluxes into the control cell ($J(t)$ and $q_{in}/\Delta t$) are positive and fluxes out of the control cell, ($q_{out}/\Delta t$), are negative.

The beach and nearshore sand volume change, *dq/dt*, is related to the change in shoreline position, *dX/dt*, according to:

$$\frac{dV}{dt} \cong \frac{d\Lambda}{dt} = \frac{dX}{dt} \cdot Z \cdot l \tag{27}$$

where $$Z = Z_1 + h_c \tag{28}$$

Here, Z is the height of the shoreline flux surface equal to the sum of the closure depth below mean sea level, h_c, (equation 24), and the height of the berm crest, Z_1, above mean sea level; and l is the length of the shoreline flux surface. Hence, beaches and the offshore bottom profile out to closure depth remain stable if a mass balance is maintained such that the flux terms on the right-hand side of equation (2) sum to zero; otherwise the shoreline will move during any time step increment as:

$$\Delta x(t) = \frac{1}{\Delta y(Z_1 + h_c)} \int \left(\frac{\partial}{\partial y}\left(\varepsilon \frac{\partial q}{\partial y} \right) - V \frac{\partial q}{\partial y} + J(t) \right) dt \tag{29}$$

where ε is the mass diffusivity, V is the longshore drift , J is the flux of sediment from river sources, Δy is the alongshore length of the control cell, and Z_1 is the maximum run-up elevation from Hunt's Formula. River sediment yield, *J*, from is calculated from streamflow, *Q*, based on the power law formulation of that river's sediment rating curve after Inman and Jenkins, (1999), or

$$J = \xi Q^{\omega} \tag{30}$$

where ξ, ω are empirically derived power law coefficients of the sediment rating curve from best fit (regression) analysis (Inman and Jenkins,1999). When San Juan Creek floods produce large episodic increases in *J*, a river delta is initially formed. Over time the delta will widen and reduce in amplitude under the influence of surf diffusion and advect (move) down-coast with the longshore drift, forming an accretion erosion wave (Figure 6.14a). The local sediment volume varies in response to the net change of the volume fluxes, between any given control cell and its neighbors, referred to as divergence of drift = q_{in} - q_{out} , see Figure 6.14b and 6.14c. The mass balance of the control cell responds to a non-zero divergence of drift with a compensating shift, Δx , in the position of the equilibrium profile (Jenkins and Inman, 2006). This is equivalent to a net change in the beach entropy of the equilibrium state. The divergence of drift is given by

the continuity equation of volume flux, requiring that *dq/dt* is the net of advective and diffusive fluxes of sediment plus the influx of new sediment, *J*. The rate of change of volume flux through the control cell causes the equilibrium profile to shift in time according to (29), producing the net change in beach widths shown by the surveys in Figures 6.8 – 6.11. Changes in sea level also cause the shoreline to move (retreat) which are calculated in the LCM module of the Coastal Evolution Model using *Bruun's Rule*, (Bruun, 1962, 1983):

$$\Delta x = X_c \left(\frac{SLR}{h_c + Z_1} \right) \tag{31}$$

Where SLR is the increment of sea level rise, and X_c is the distance offshore to closure depth given by the elliptic cycloid formulation to the equilibrium profile (Jenkins and Inman, 2006) according to:

$$X_{c2} = \frac{h_c I_e^{(2)}}{\varepsilon} \cong \frac{\pi\, h_c}{2\varepsilon} \sqrt{\frac{2-e^2}{2}} \tag{32}$$

$$\text{With: } \varepsilon = \frac{\sigma}{N} \left(\frac{H_b}{\gamma\, g} \right)^{1/2} \cong \frac{\sigma^{4/5}}{2^{1/5} N} \left(\frac{H_0'}{g\gamma} \right)^{2/5}$$

Because Bruuns Rule merely produces a self-similar landward shift to profile in response to sea level rise (with no change to the shape of the profile or to the elliptic cycloid parameters); sea level rise does not effect the intrinsic slope parameters of the profile on which the total run-up elevation depends. This response is based on an assumption that the beach has adequate sand volume and sediment cover to execute the profile shift required under Bruun's Rule. This assumption appears to be well founded at Doheny State Beach due to the fact that it is continually re-nourished by the flux of new sediment from San Juan Creek, (*J* = 51,000 ton/yr)**.**

7.0 Wave Run-up and Overtopping Statistical Analysis:

This section uses the data bases described in Section 6 to evaluate Steps-4 & 5 of a sea level rise/coastal hazards analysis as outlined in Section 2. We seek to quantify the probability of occurrences of *extremal total water levels* where the total water level (TWL) is the sum of the total run-up and the still water level (SWL). The total run-up, *R*, is a dynamic water level variation caused by wave shoaling and breaking, and is composed of three components: wave setup, $<\eta>$, dynamic wave setup, η_{rms} ; and incident wave run-up, R_{inc}. We will begin in Section 7.1 by setting the still water level equal to present or future mean sea level, which will allow us to isolate the total runup as an independent dynamic process whose probability is uniquely determined by the extremal wave height curve in Figure 6.7. We will then solve for *extremal total water levels* (TWL_{max}) by admitting to probability of occurrences of still water levels higher than mean sea level; which results in a joint probability analysis of occurrence of extremal wave heights concurrent with extreme ocean water levels.

7.1) Total Water Level Analysis for Constant Still Water Levels: Total water level is a multi-variant function determined by the combined effects of stationary processes (processes vary slowly in time) and dynamic processes (processes that vary rapidly with time). The still

water level component of the total water level is a relatively stationary process when compared to the total run-up component, where the latter varies rapidly in time at the frequency of surface gravity waves. At lowest order approximation, we can solve for the probability of recurrence of potential total water levels by assuming the stationary processes are fixed in time. By that approach, we adopt a common practice in coastal engineering by setting the still water level at mean sea level and then solve for the potential total water levels as a conditional probability using Bayes' theorem:

$$P(TWL_{\max}) = P[R, Z_i] = P[R(H'_T] \bullet P_{i,j}(Z_i = MSL) \qquad (33)$$

Here, $P_{i,j,}(Z_i)$ is the annualized probability of ocean water levels reaching an elevation of Z_i feet NAVD 88 from equations (3) and (4), where $P_{i,j,}(Z_i = MSL)$ = 1, (cf. Figures 4.1, 4.6 & 4.7); $P[R(H'_T)]$ is the annualized probability of total run-up from the sum of equations (6) and (11) based on the probability of extremal wave heights with return frequency of once every T years, $P(H'_T) = 1/T$, (cf. Figure 6.7). The total run-up calculations using extremal wave heights are based on the direct integration method (DIM) from Section 5.5 because the beach slopes at Doheny State Beach for both eroded (winter) and summer (accreted) conditions are always than 12.5%. (Here beach slope, m_{DIM} , is taken as the average slope between the landward limit of wave run-up and the location offshore where the water depth is two times the depth at which the deep water significant wave height would be subject to depth-limited breaking, cf. Van der Meer, 2002). Figures 6.8 – 6.12 show generally that average nearshore beach slopes at Doheney State Beach range from $\overline{m}_{DIM}$ = 0.006 for eroded beach profiles, and steepen to $\overline{m}_{DIM}$ = 0.10 for accreted beach profiles. One advantage of the approach taken by equation (25) is that it allows us to separate the individual dynamic components to the total water level solutions.

Figures 7.1-7.3 give the annualized probability of recurrence of total run-up and its components of static wave setup, dynamic wave setup, and the total oscillatory swash component based on the extremal wave analysis curve in Figure 6.7 as applied to equations (6)- (12). For each component of total wave runup, there are two sets of curves, representing eroded and accreted conditions at Dohenty State Beach survey ranges #R@, #R4, #R5, #R7 and # DB1850. Ina all cases, the maximum water elevations are greater for the accreted beach conditions than for the eroded beach conditions. This is due to the fact that eroded beaches have flatter slopes in the bar-berm section of the profile where waves are breaking and producing run-up. Flatter beach slopes are intrinsically more dissipative, resulting in less residual energy after breaking to produce runup. Inspection of Figure 7.3 indicates that maximum run-up is 15.4 ft. for the accreted beach conditions and 13.1 ft. for the eroded beach conditions, with a probability of recurrence of 0.04% (return period = 2,500 yr). But the maximum wave run-up is based on a statistical projection from the Weibull Type III best fit to the extremal wave results from refraction/diffraction analysis in Figure 6.7. The highest wave that was recovered from the refraction analysis in 12 m of water depth was due to the100-year storm of 18, January, 1988 (Figure 6.4) with a significant wave height H'_0 = 15.5 ft. and a probability of recurrence of 1.0%. The 1% runup up event in Figure 7.3 actually gives maximum total wave run-up of 11.9 ft. for the accreted beach conditions and 9.98 ft. for the eroded beach conditions.

The annualized probability of recurrence of total water level is plotted in Figure 7.4 under the stationary hypothesis for still water level according to equation (33). Under this assumption (where still water level is fixed at present mean sea level), the maximum total water level is *TWL* = 17.98 ft. NAVD for the accreted beach conditions and *TWL* = 15.69 ft. NAVD for the eroded

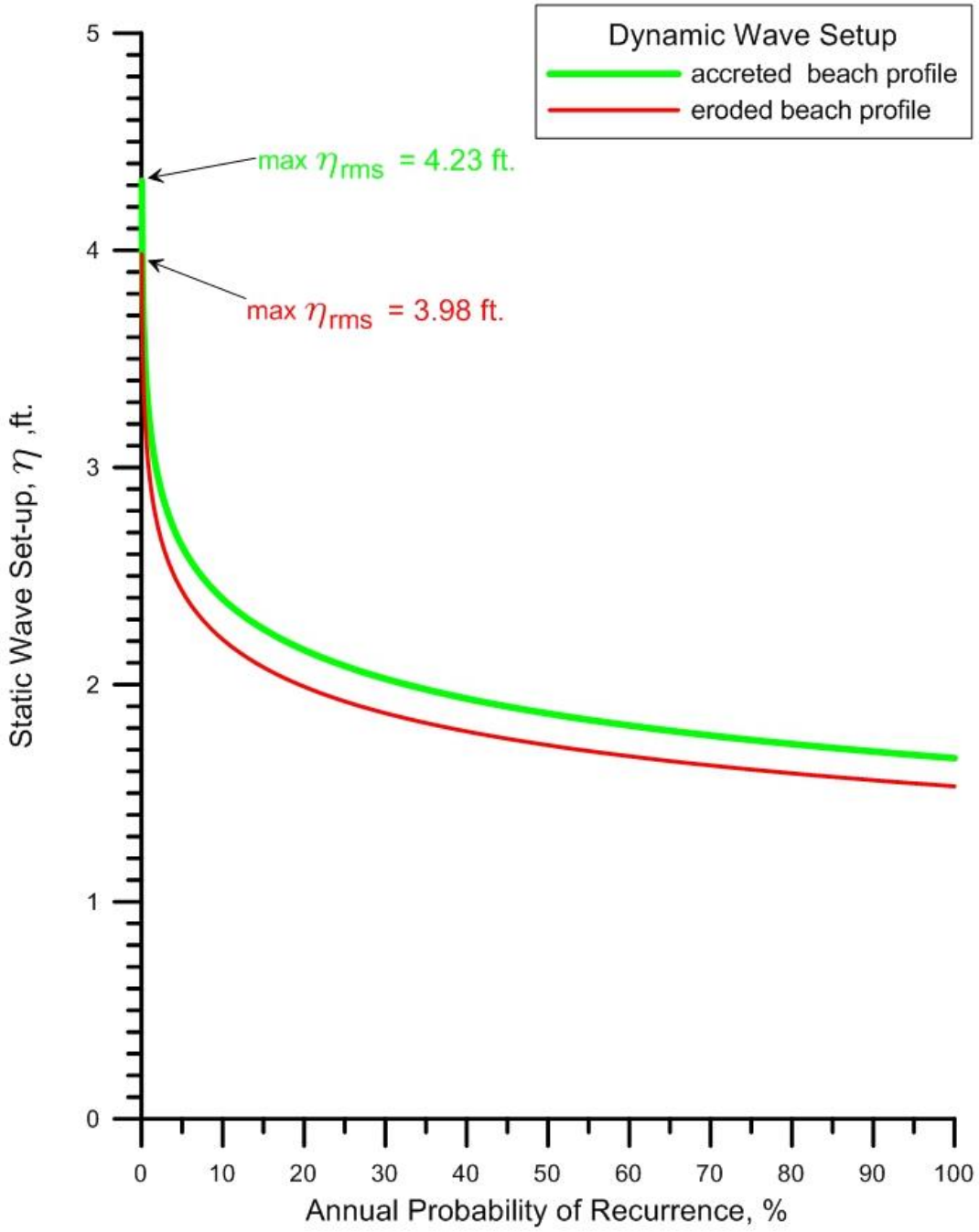


Figure 7.2: Annualized probability of recurrence of dynamic wave setup at Doheny State Beach based on extremal design wave heights from Weibull Type III distribution (cf. Figure 6.7)

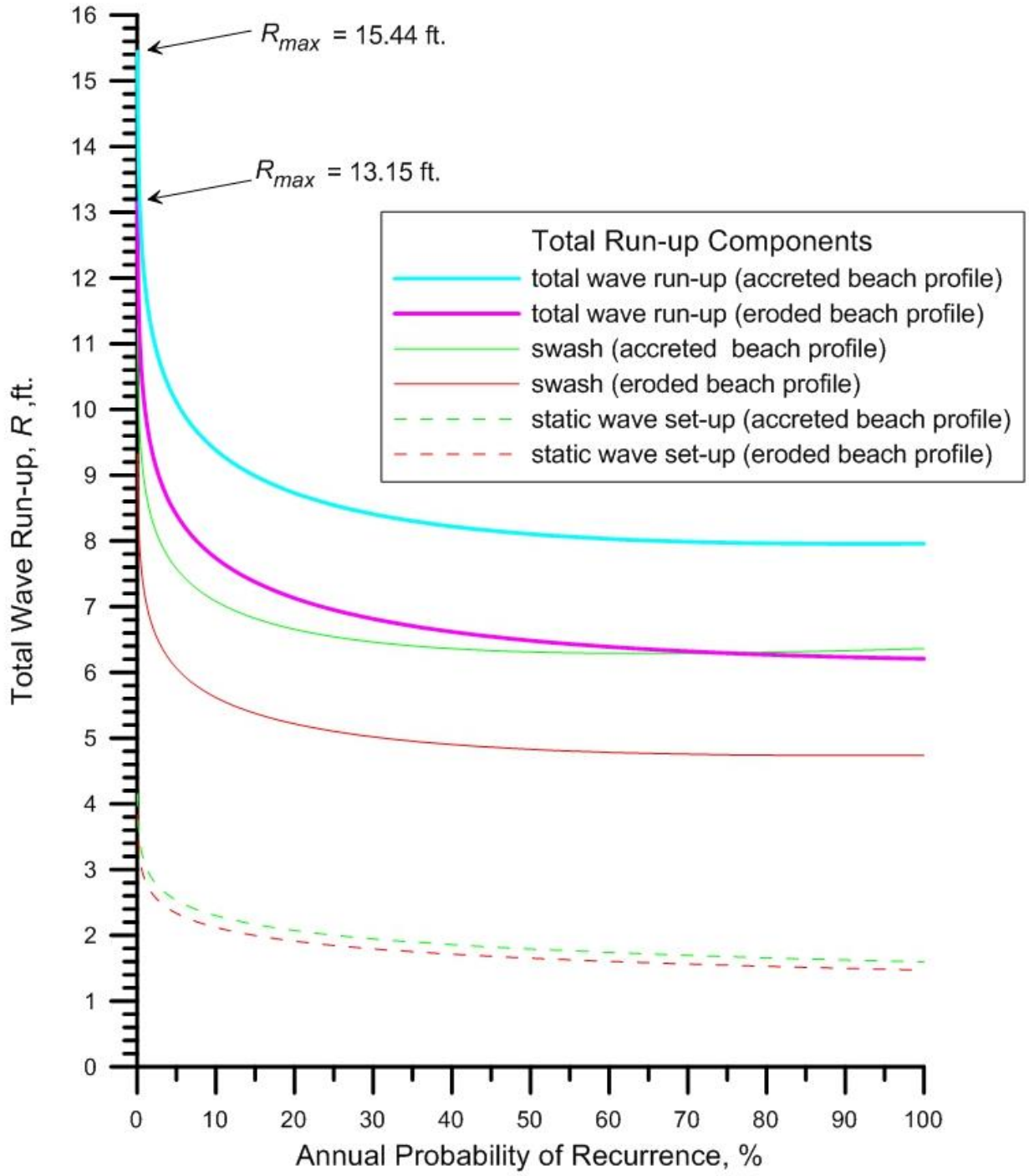


Figure 7.3: Annualized probability of recurrence of total wave run-up (and its components) at Doheny State Beach based on extremal design wave heights from Weibull Type III distribution, (cf. Figure 6.7)

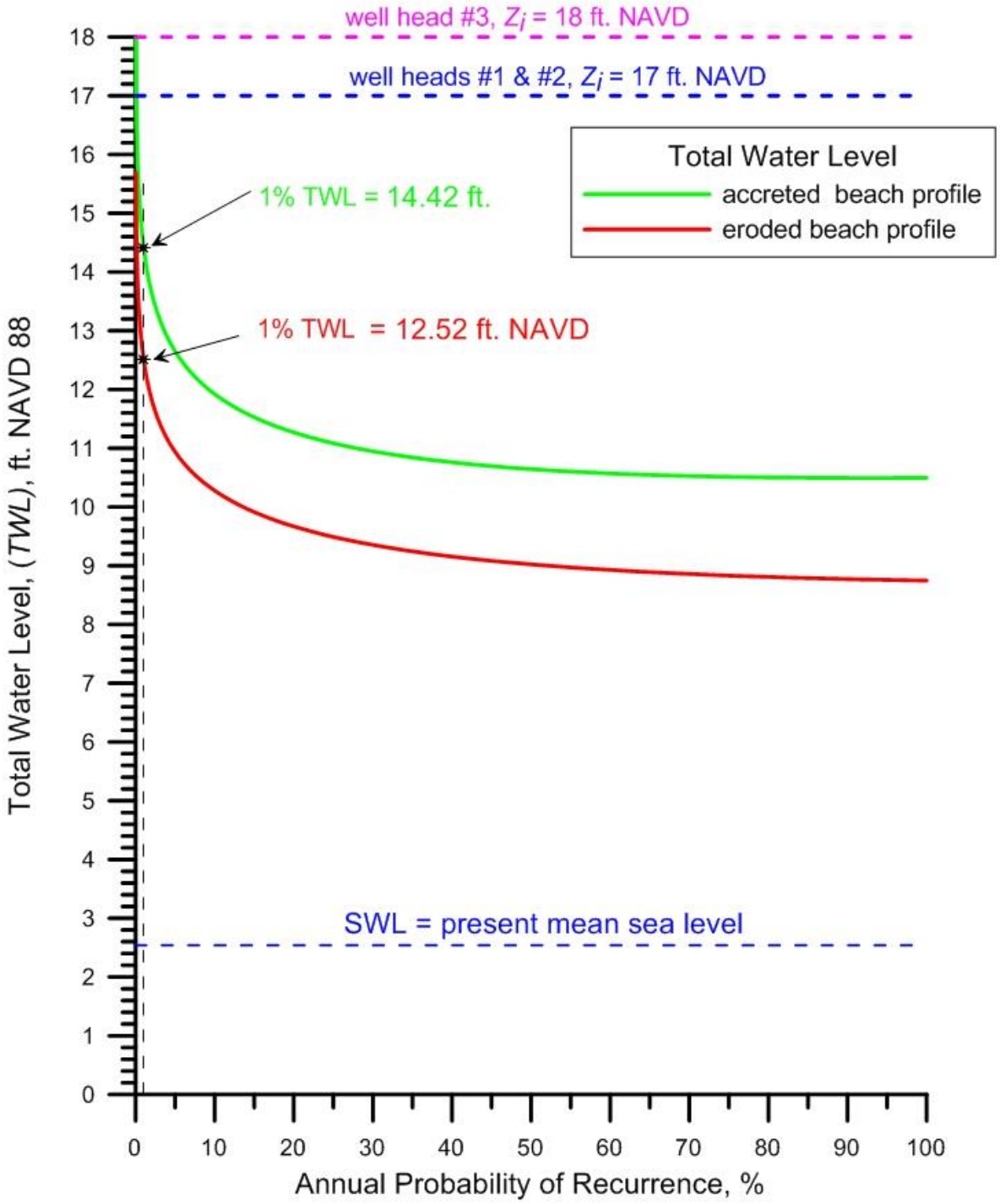


Figure 7.4: Annualized probability of recurrence of total water level at Doheny State Beach based on present sea level and extremal design wave heights from Weibull Type III distribution. SWL = MSL

beach conditions. The total water level achieved under accreted beach conditions at present sea level exceeds the elevations of well heads #1 and #2, which are located at Z_i == 17 ft. NAVD; but the probability of this occurring is only 0.04% (return period = 2,500 yr). Appendix-B of the *California Coastal Commision Sea Level Rise Guidance Policy Guidance* document (CCC, 2015) provides no specific guidance on the redline frequency for flooding or inundation. In the absence of such guidance we will adopt Federal Emergency Management Agency (FEMA) standards for flooding frequency and set redline planning frequency at the 100 year event (1% probability of recurrence). Accordingly, Figure 7.4 has been annotated to highlight the 1% total water level events which indicate is *TWL*(1%) = 14.42 ft. NAVD for the accreted beach conditions and *TWL*(1%) = 12.52 ft. NAVD for the eroded beach conditions. Consequently we conclude that all the beach front facilities for the Doheny Desalination Project (Figure 6.3) are safe from flooding or inundation by extreme event waves under the stationary hypothesis for still water level at present mean sea level.

We repeat the total water level analysis in Figures 7.5 and 7.6 for 2070 sea levels under the stationary hypothesis for still water level (where still water level is fixed at 2070 mean sea level for the low and high range projections). For the low-range 2070 sea level projections, (Figure 7.5) the 1% total water level events reach *TWL*(1%) = 15.12 ft. NAVD for the accreted beach conditions and *TWL*(1%) = 13.22 ft. NAVD for the eroded beach conditions; indicating that all the beach front facilities for the Doheny Desalination Project (Figure 6.3) are safe from flooding or inundation by extreme event waves. But, for the high-range 2070 sea level projections, (Figure 7.6) the 1% total water level events reach *TWL*(1%) = 17.63 ft. NAVD for the accreted beach conditions and *TWL*(1%) = 15.73 ft. NAVD for the eroded beach conditions. This indicates that well heads #1 and #2 are vulnerable to flooding by 0.63 ft of overtopping during accreted beach conditions, which according to equation (17) results in an overtopping rate of Q' = 0.02 cfs/ft under the hypothesis that still water level is fixed at the high range of 2070 mean sea level.

Figures 7.7 and 7.8 present the total water level analysis for 2100 sea levels under the stationary hypothesis for still water level (where still water level is fixed at 2100 mean sea level for the low and high range projections). For the low-range 2100 sea level projections, (Figure 7.7) the 1% total water level events reach *TWL*(1%) = 15.79 ft. NAVD for the accreted beach conditions and *TWL*(1%) = 13.89 ft. NAVD for the eroded beach conditions; again indicating that all the beach front facilities for the Doheny Desalination Project (Figure 6.3) are safe from flooding or inundation by extreme event waves. But, for the high-range 2100 sea level projections, (Figure 7.8) the 1% total water level events reach *TWL*(1%) = 19.91 ft. NAVD for the accreted beach conditions and *TWL*(1%) = 18.01 ft. NAVD for the eroded beach conditions. Consequently all beach front facilities for the Doheny Desalination Project except well pump electrical south are vulnerable to flooding. Well heads #1 and #2 are vulnerable to flooding by as much as 2.91 ft of overtopping, which according to equation (17) results in an overtopping rate of Q' = 0.21 cfs/ft under the hypothesis that still water level is fixed at the high range of 2100 mean sea level. Similarly well head #3 and well pump electrical south could be flooded by 1.91 ft of overtopping, resulting in an overtopping rate of Q' = 0.11 cfs/ft under the hypothesis that still water level is fixed at the high range of 2100 mean sea level.

7.2) Total Water Level Analysis for Extremal Still Water Levels: In this section we relax the stationary hypothesis for still water level and allow it to vary according to the hydroperiod functions for present and future sea levels in Figures 4.1, 4.6, and 4.7. This will provide an analysis of total water levels due to extreme waves concurrent with extreme ocean water levels (extremal TWL's). The recurrence frequency (or return period) for these extremal

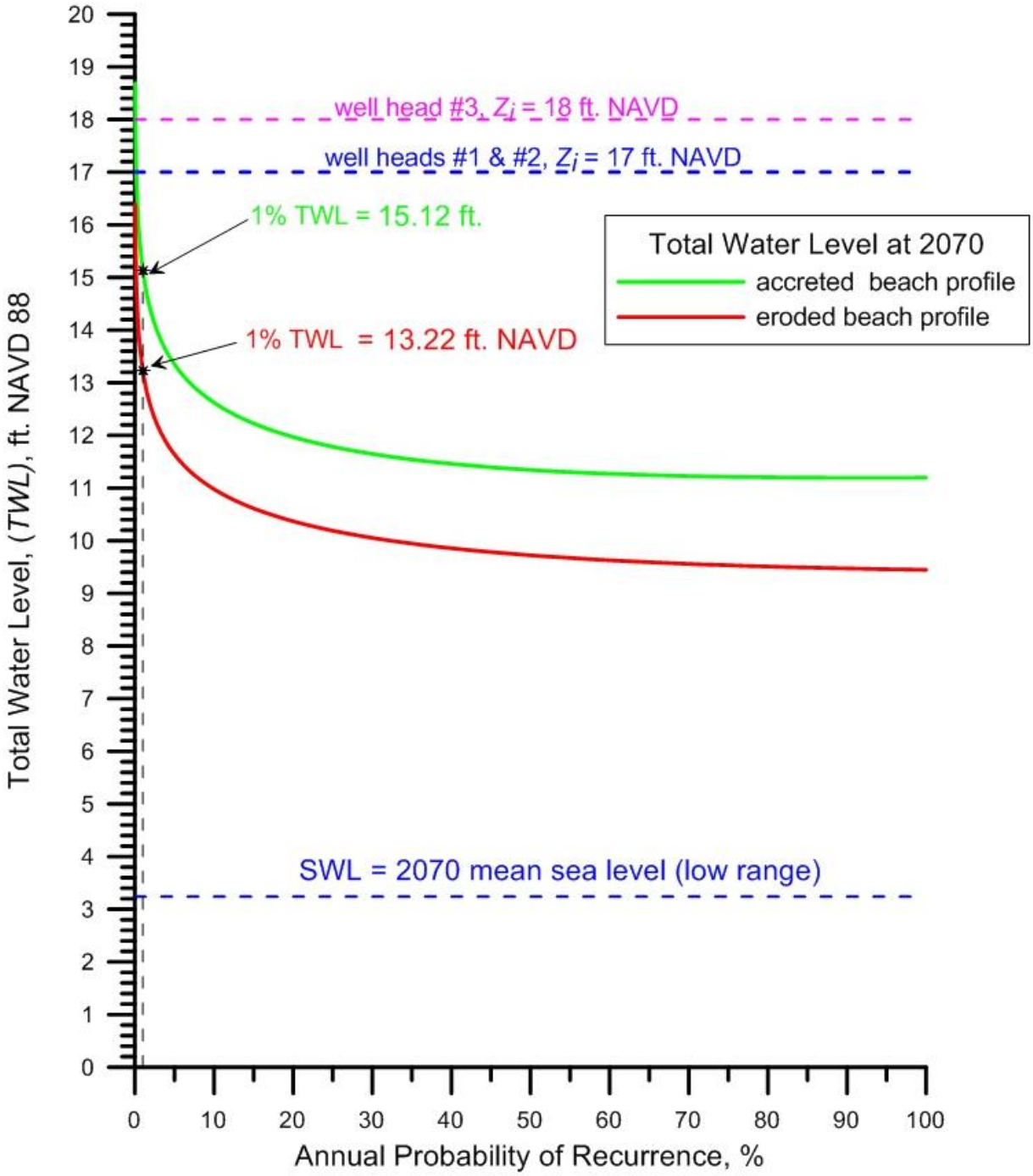


Figure 7.5: Annualized probability of recurrence of total water level at Doheny State Beach for still water level at 2070 (low range) mean sea level, based on extremal design wave heights from Weibull Type III distribution

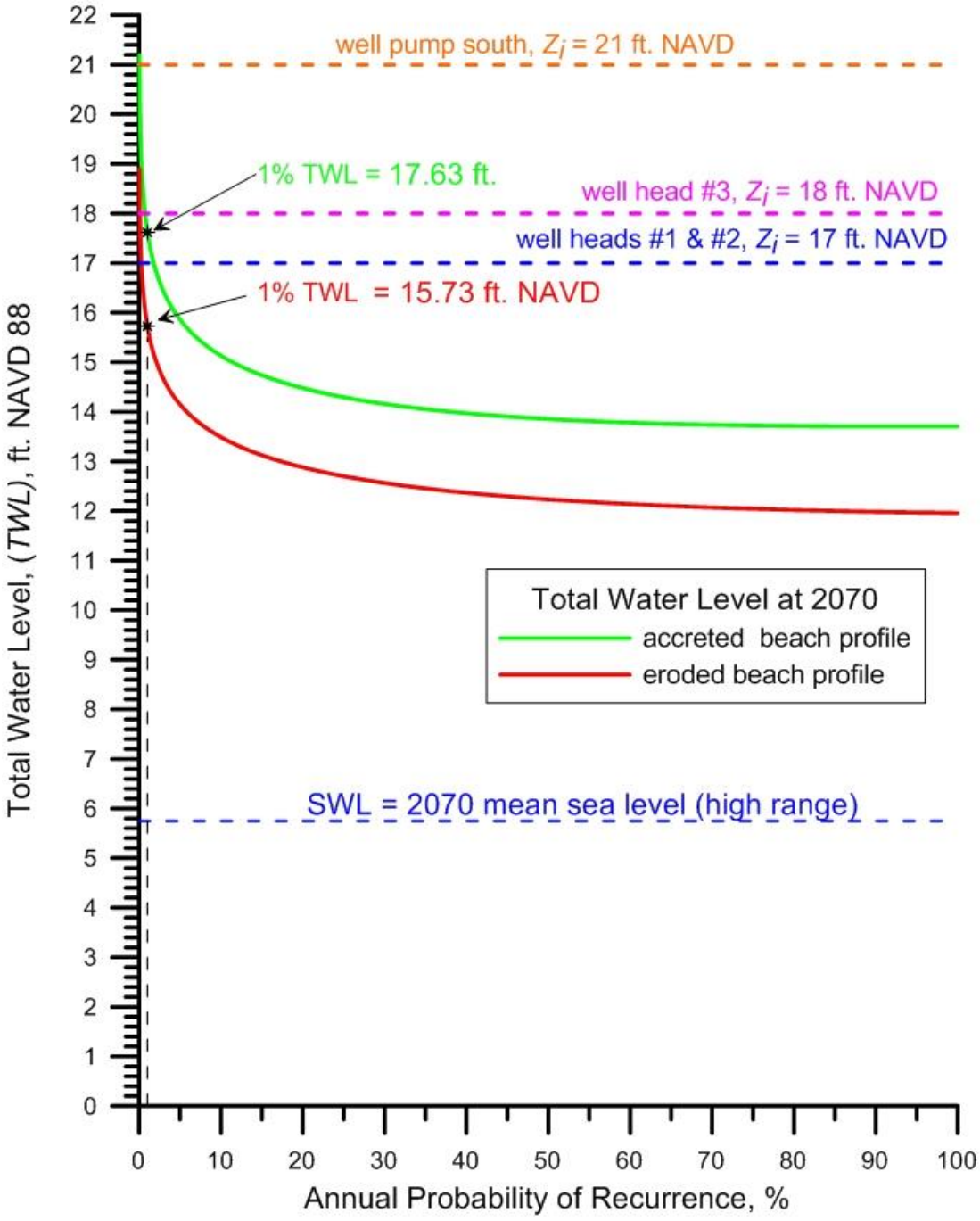


Figure 7.6: Annualized probability of recurrence of total water level at Doheny State Beach for still water level at 2070 (high range) mean sea level, based on extremal design wave heights from Weibull Type III distribution

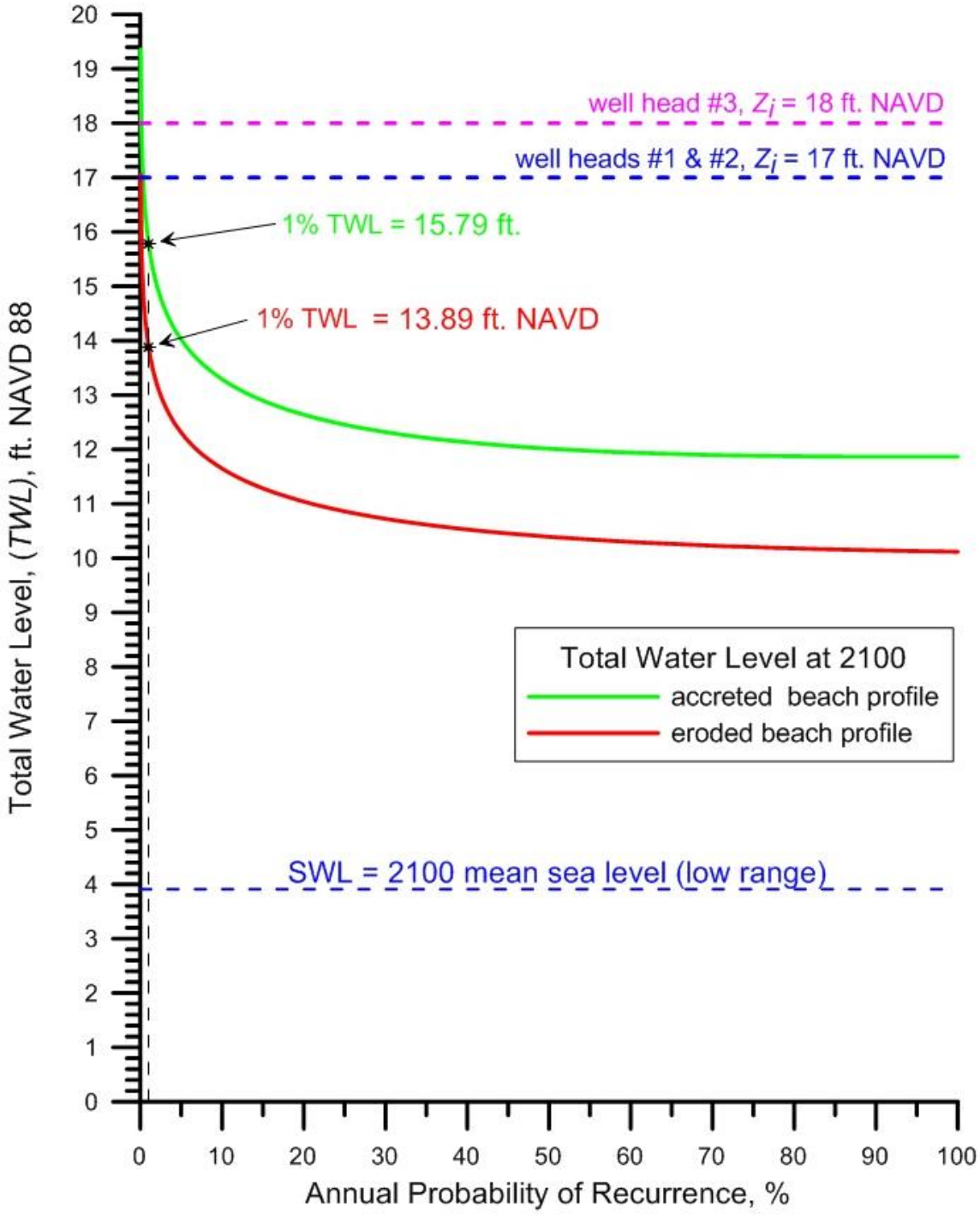


Figure 7.7: Annualized probability of recurrence of total water level at Doheny State Beach for still water level at 2100 (low range) mean sea level, based on extremal design wave heights from Weibull Type III distribution

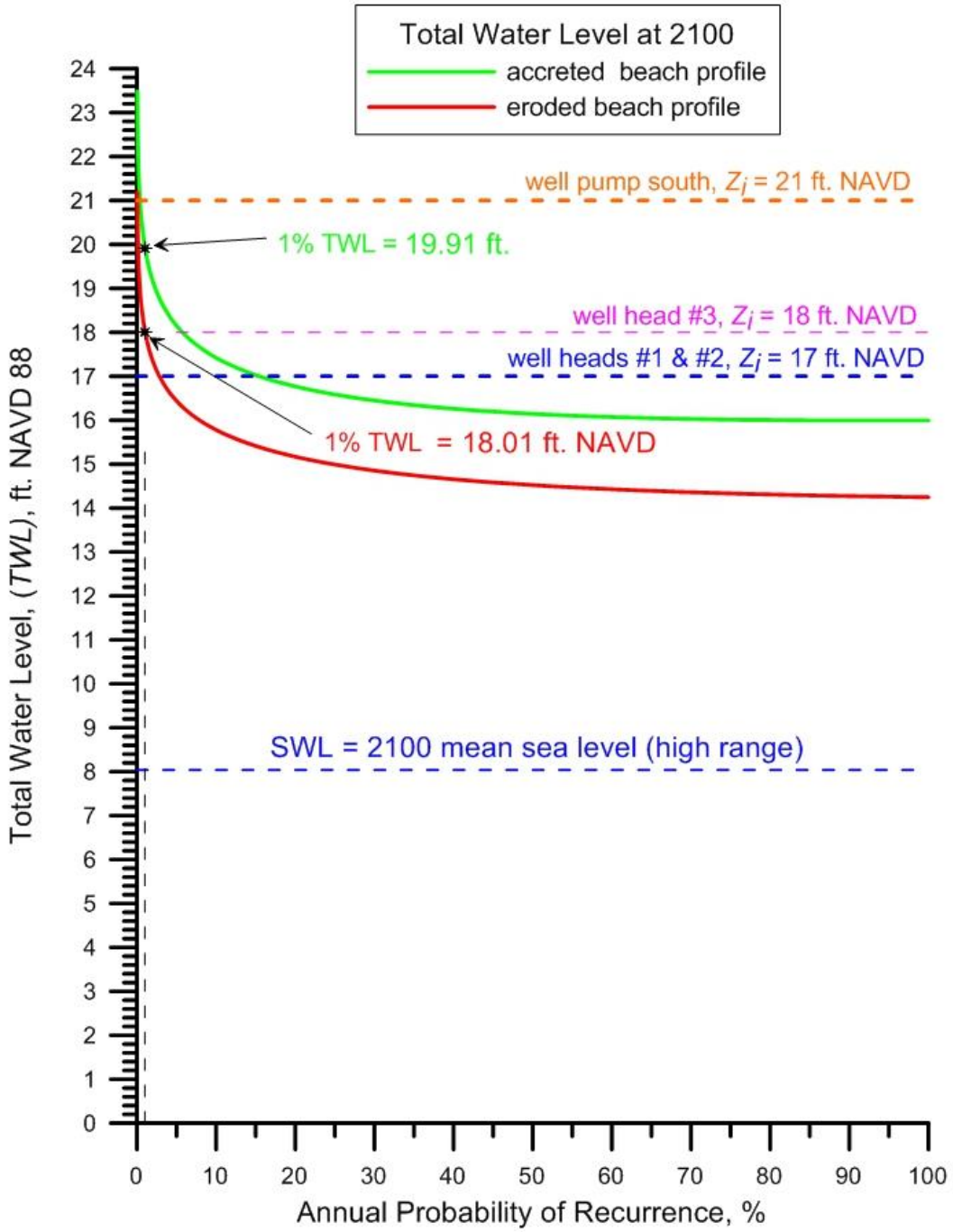


Figure 7.8: Annualized probability of recurrence of total water level at Doheny State Beach for still water level at 2100 (high range) mean sea level, based on extremal design wave heights from Weibull Type III distribution

TWL's is given by the joint probability of occurrence of extremal wave heights concurrent with extreme ocean water levels, or:

$$P(TWL_{\max}) = P[R, Z_i] = P[R(H'_T] \bullet P_{i,j}(Z_i) \qquad (34)$$

where H'_T is the extremal significant wave height with return period of T years, and $P_{i,j}(Z_i)$ is the annualized probability of ocean water levels η reaching an elevation of Z_i feet NAVD 88 at or above mean sea level, as derived from the annualized hydroperiod function, equations (3) and (4). The results for return periods $T_r = 1/P[R, Z_i]$ of extremal total water levels at present sea level are plotted in Figure 7.9, while those for 2070 sea levels are found in Figures 7.10 & 7.11; and results for 2100 sea levels are found in Figures 7.12 & 13. Comparing these results with the total water level results in Figures 7.4-7.8 (that were based on the stationary hypothesis for still water level) indicates that the joint probability analysis for extreme waves concurrent with extreme ocean water levels gives TWL's that are about 0.5 ft. higher for the 1% recurrence event (100 year return period). For example the extremal TWL's in Figure 7.9 give the TWL(100) = 13.1 ft for eroded conditions and TWL(100) = 14.8 ft. for accreted conditions at present sea levels. On the other hand, when SWL is set at mean sea level per Section 7.1, as shown in Figure 7.4, the 1% TWL = 12.5 ft for eroded conditions and 1% TWL = 14.4 ft. for accreted conditions at present sea levels. Therefore, we adopt the extremal still water formulation per equation (34) as the redline analysis method for assessing Steps-4 & 5 of a sea level rise/coastal hazards analysis as outlined in Section 2.

Inspection of Figure 7.9 indicates that all the beach front facilities for the Doheny Desalination Project (Figure 6.3) are safe from flooding or inundation by extreme event waves, even for event return periods as long as 500 yr, when extreme waves happen concurrently with extreme ocean water levels in an environment of present sea levels. In fact, according to Figures 7.10 & 7.12, all these facilities are safe from flooding by the 100-year event in environments of the low-range predictions for 2070 and 2100 sea levels. However, once we admit to the high-range sea level rise projections, a number of the beach front facilities for the Doheny Desalination Project will suffer some flooding and overtopping to varying degrees. For the high-range 2070 sea level projections, (Figure 7.11) the 100-yr total water level events reach $TWL(100)$ = 18.0 ft. NAVD for the accreted beach conditions and $TWL(100)$ = 16.3 ft. NAVD for the eroded beach conditions. This indicates that well heads #1 and #2 are vulnerable to flooding by any event that exceeds a return period of 36 years; and could be flooded by as much as 1.0 ft of overtopping during accreted beach conditions, with overtopping rates of as much as Q' = 0.04 cfs/ft. Flooding and overtopping of well heads #1 and #2 under eroded beach conditions would not happen until events exceeding 225 year return periods when extreme waves happen concurrently with extreme ocean water levels under high-range 2070 sea level projections.

For the high-range 2100 sea level projections, (Figure 7.13) the 100 year total water level events reach $TWL(100)$ = 20.3 ft. NAVD for the accreted beach conditions and $TWL(100)$ = 18.5 ft. NAVD for the eroded beach conditions. Consequently all beach front facilities for the Doheny Desalination Project except well pump electrical south are vulnerable to flooding by the 100-year event. Well heads #1 and #2 would be flooded by as little as a 3-year event under accreted beach conditions and by a 16-year event under eroded beach conditions. . Well heads #1 and #2 are vulnerable to flooding by as much as 3.3 ft of overtopping during a 100 year event, which according to equation (17) results in an overtopping rate of Q' = 0.26 cfs/ft under conditions when extreme waves happen concurrently with extreme ocean water levels that are superimposed

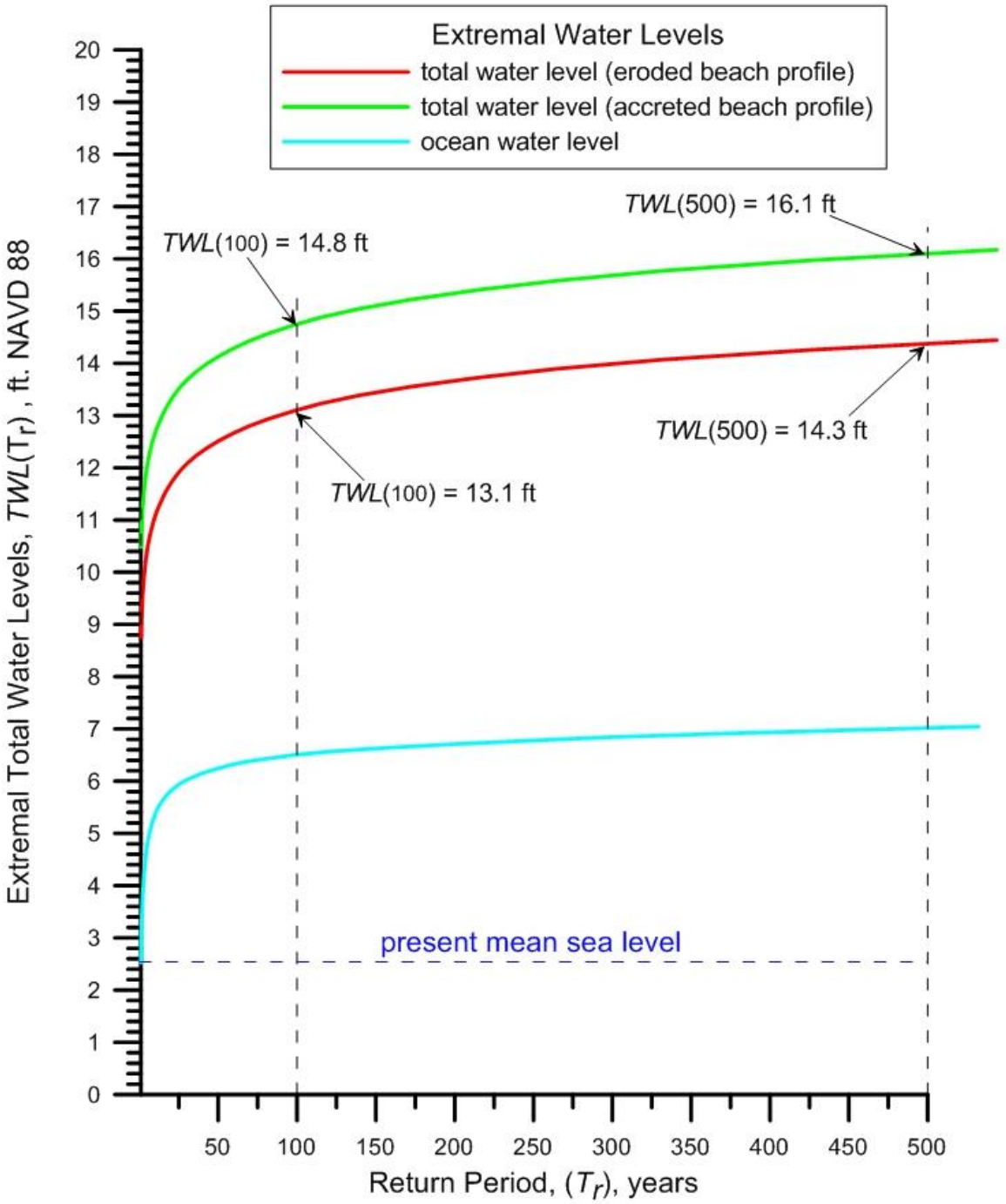


Figure 7.9: Return period of extremal total water level (TWL) at Doheny State Beach based on extremal design wave heights from Weibull Type III distribution and still water levels set at extremal ocean water levels for present sea level.

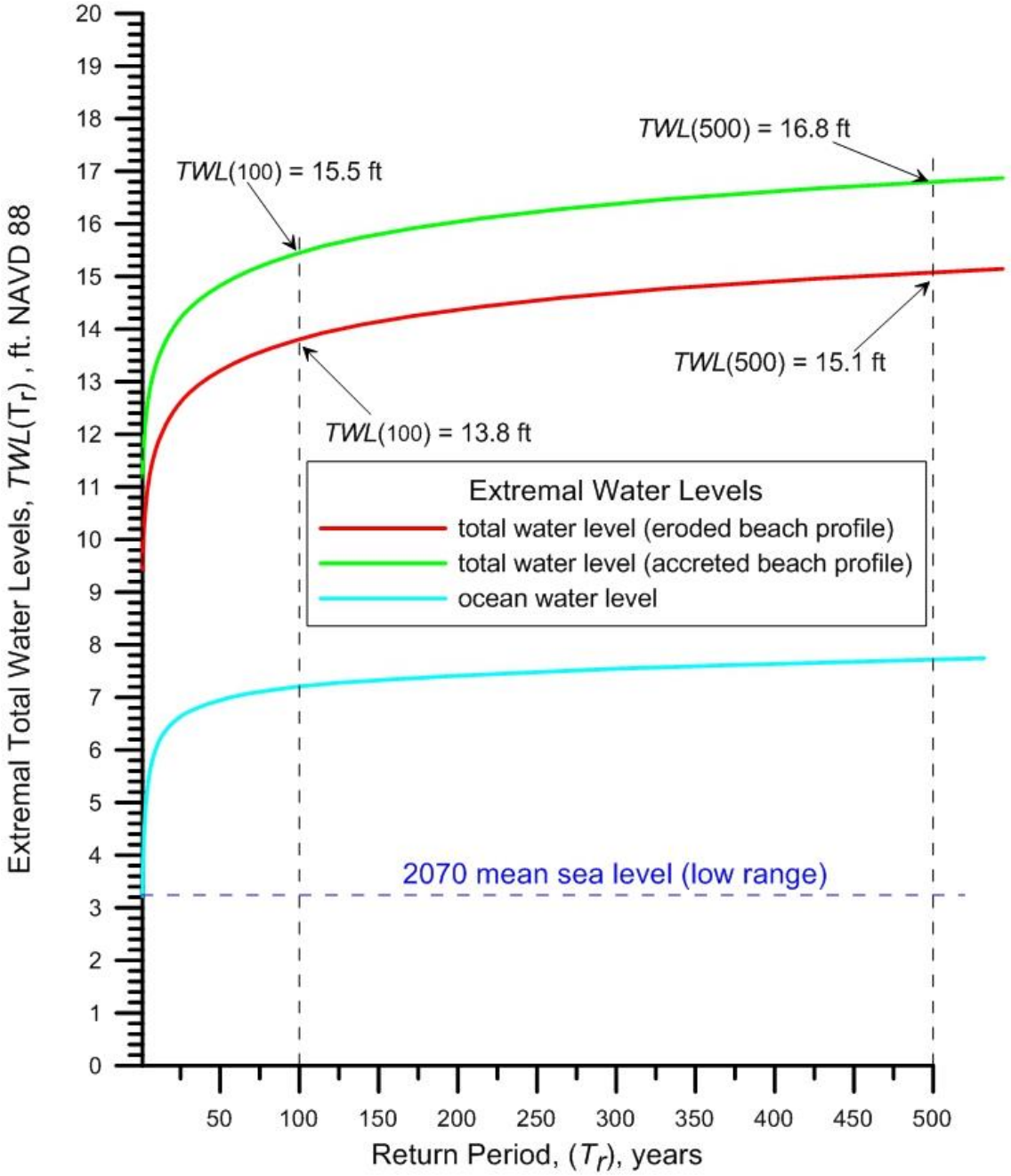


Figure 7.10: Return period of extremal total water level (TWL) at Doheny State Beach based on extremal design wave heights from Weibull Type III distribution and still water levels set at extremal ocean water levels for 2070 sea level (low range prediction).

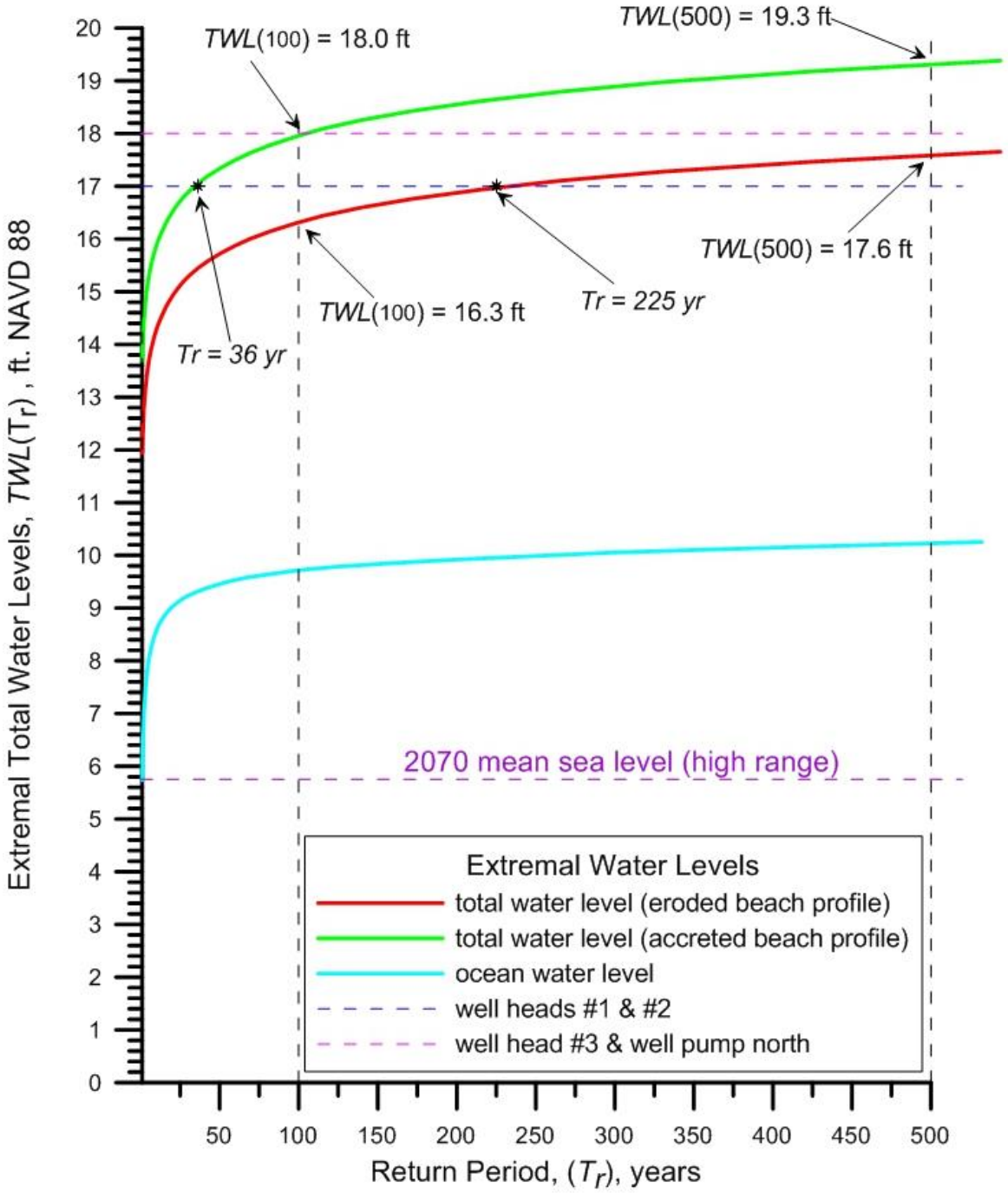


Figure 7.11: Return period of extremal total water level (TWL) at Doheny State Beach based on extremal design wave heights from Weibull Type III distribution and still water levels set at extremal ocean water levels for 2070 sea level (high range prediction).

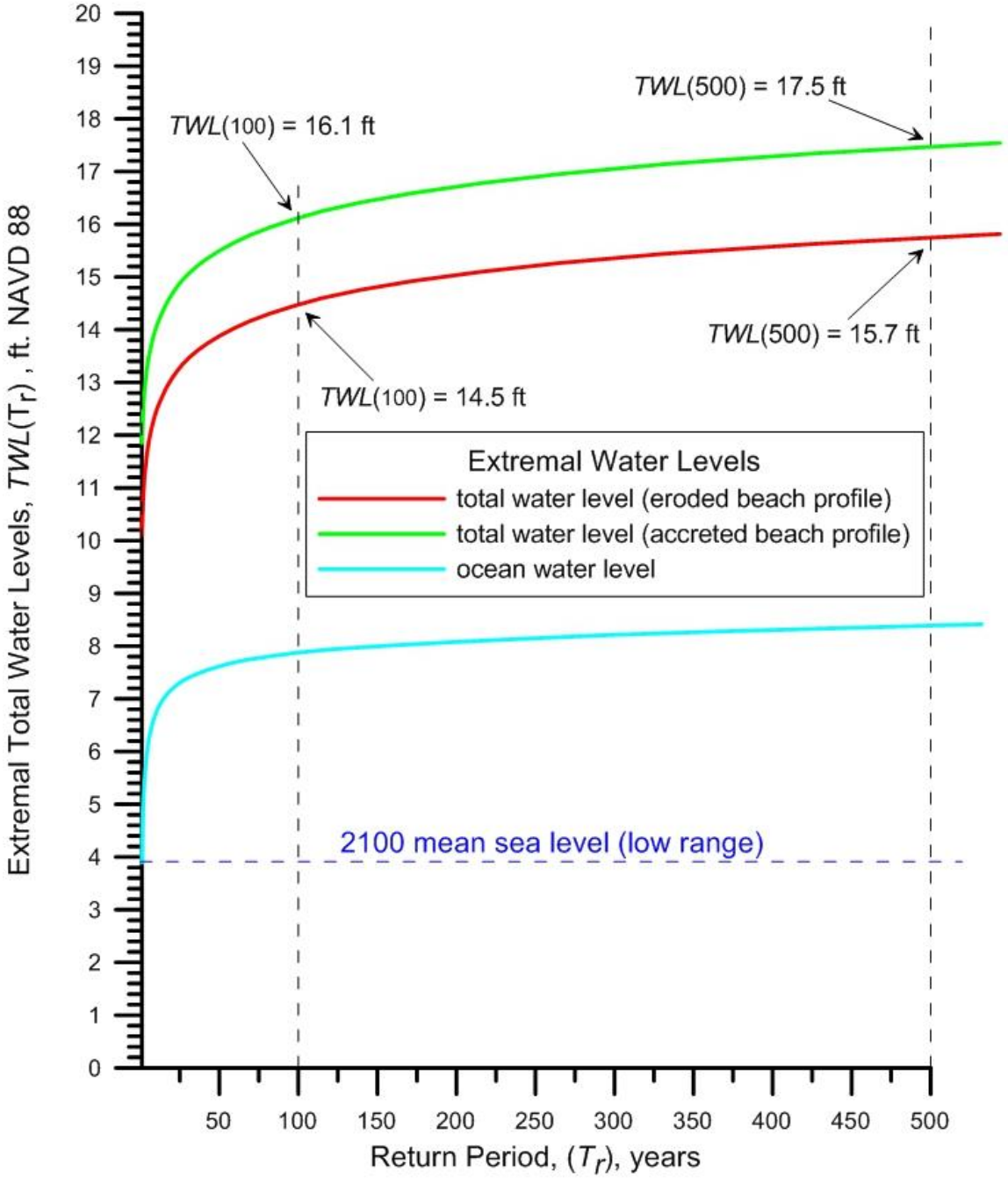


Figure 7.12: Return period of extremal total water level (TWL) at Doheny State Beach based on extremal design wave heights from Weibull Type III distribution and still water levels set at extremal ocean water levels for 2100 sea level (low range prediction).

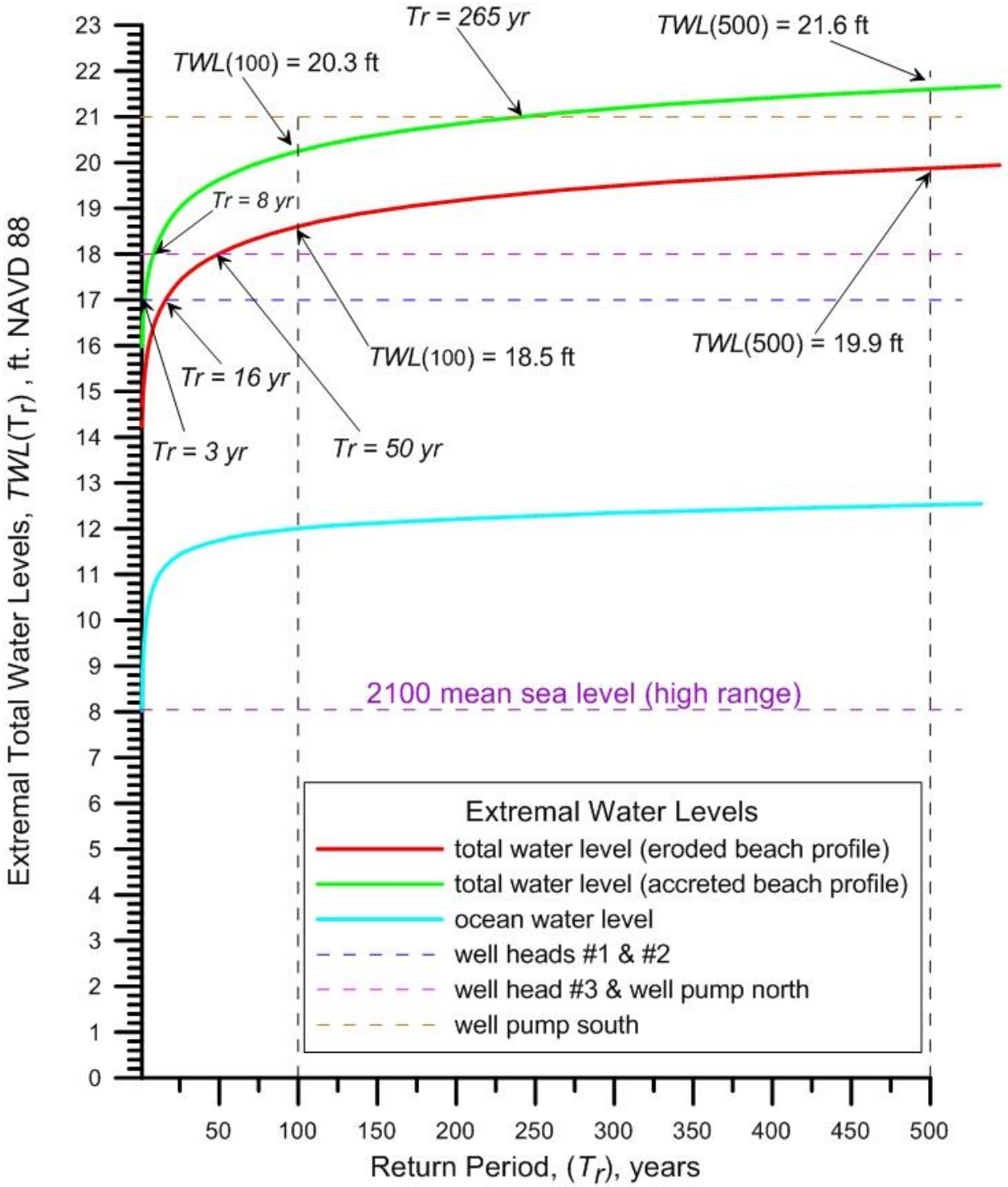


Figure 7.13: Return period of extremal total water level (TWL) at Doheny State Beach based on extremal design wave heights from Weibull Type III distribution and still water levels set at extremal ocean water levels for 2100 sea level (high range prediction).

on the high range of 2100 sea levels. Similarly well head #3 and well pump electrical north could be flooded by 2.3 ft of overtopping, resulting in an overtopping rate of $Q' = 0.15$ cfs/ft when extreme waves happen concurrently with extreme ocean water levels that are superimposed on the high range of 2100 sea levels. Well pump electrical south is safe from flooding by any event less than 265-year event in environments of the high-range predictions for 2100 sea levels. The results of the Extremal Total Water Level Analysis are summarized in Table 7.1 below. Because all of the shore facilities in Table 7.1 are in close proximity to one another, (clustered around the mouth of the San Juan Creek with nothing to support water surface gradients between each facility), there are insignificant differences in extremal total water levels occurring at each facility for any given event.

Table 7.1: Extremal Total Water Level (TWL) and Overtopping Rates (Q') Analysis

	Well Head #1 Elevation = 17 ft. NAVD	Well Head #2 Elevation = 17 ft. NAVD	Well Head #3 Elevation = 18 ft. NAVD	Well Pump Electrical South Elevation = 21 ft. NAVD	Well pump Electrical North Elevation = 18 ft. NAVD
$*TWL(100)$ Present Sea Level (eroded/accreted)	13.1/14.8 ft. NAVD status = dry	13.1/14.8 ft. NAVD status = dry	13.1/14.8 ft. NAVD status = dry	13.1/14.8 ft. NAVD status = dry	13.1/14.8 ft. NAVD status = dry
$*Q'(100)$ Present Sea Level (eroded/accreted)	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.
$*TWL(100)$ 2070 Sea Level Low Range Projection (eroded/accreted)	13.8/15.5 ft. NAVD status = dry	13.8/15.5 ft. NAVD status = dry	13.8/15.5 ft. NAVD status = dry	13.8/15.5 ft. NAVD status = dry	13.8/15.5 ft. NAVD status = dry
$*Q'(100)$ 2070 Sea Level Low Range Projection (eroded/accreted)	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.
$*TWL(100)$ 2070 Sea Level High Range Projection (eroded/accreted)	16.3/18.0 ft. NAVD status = flooded	16.3/18.0 ft. NAVD status = flooded	16.3/18.0 ft. NAVD status = dry	16.3/18.0 ft. NAVD status = dry	16.3/18.0 ft. NAVD status = dry
$*Q'(100)$ 2070 Sea Level High Range Projection (eroded/accreted)	0.0/0.04 cfs/ft.	0.0/0.04 cfs/ft.	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.
$*TWL(100)$ 2100 Sea Level Low Range Projection (eroded/accreted)	14.5/16.1 ft. NAVD status = dry	14.5/16.1 ft. NAVD status = dry	14.5/16.1 ft. NAVD status = dry	14.5/16.1 ft. NAVD status = dry	14.5/16.1 ft. NAVD status = dry
$*Q'(100)$ 2100 Sea Level Low Range Projection (eroded/accreted)	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.
$*TWL(100)$ @ 2100 Sea Level High Range Projection (eroded/accreted)	18.5/20.3 ft. NAVD status = flooded	18.5/20.3 ft. NAVD status = flooded	18.5/20.3 ft. NAVD status = flooded	18.5/20.3 ft. NAVD status = dry	18.5/20.3 ft. NAVD status = flooded
$*Q'(100)$ 2100 Sea Level High Range Projection (eroded/accreted)	0.08/0.26 cfs/ft.	0.08/0.26 cfs/ft.	0.01/0.15 cfs/ft.	0.0/0.0 cfs/ft.	0.01/0.15 cfs/ft.

8.0 Tsunami Run-up and Overtopping Analysis:

Tsunami induced erosion, runup, and inundation were analyzed for the Doheny State Beach bottom profiles (Figures 6.8- 6.12) and shore-side facilities associated with the Doheny Desalination Project for present and future sea levels according to low and high range sea level rise predictions as shown in Figure 3.1. Because of the uncertainty of the probability of occurrence of such a tsunami event, and the absense of specific guidance on the redline frequency for flooding considerations in Appendix-B of the *California Coastal Commision Sea Level Rise Guidance Policy Guidance* document (CCC, 2015), we will carry forward the total water level analysis based on the stationary still water level hypothesis; whereby the still water level in the shoaling and runup equations is fixed at whatever mean sea level is for each sea level rise scenario.

The tsunami event scenario is based on a 2m high solitary wave approaching Doheny Beach from 165 degrees true, as could be anticipated for a catastrophic tsunami event arising from a major landside on the east side of San Clemente Island. The local refraction/diffraction pattern from the solitary wave is calculated in Figure 8.1 for present mean sea level. Inspection of Figure 8.1 reveals the tsunami wave height begins to increase at 50 m of water depth due to shoaling and reaches 6m of height before breaking along the shores of Doheny Beach. Because the tsunami wave begins shoaling in much deeper water than typical storm-induced waves, it causes seabed scour and erosion to occur out to very deep water depths. Therefore all run-up and total water level solutions are based eroded beach profile conditions. The critical mass thickness computed by the CEM in Figure 8.2 for this tsunami shoaling scenario reveals that seabed erosion occurs offshore to depths of -124 to -137 ft. MSL; and the volume of eroded sediment can be as high as 1,827 m^3 per meter of shoreline. Figure 8.2 also shows that a tsunami of this magnitude is capable of eroding as much as 4 ft to 6 ft of seabed offshore, to depths of -120 to -130 ft. MSL, and could erode as much as 12 ft . of beach sediment cover in a single tsunami wave breaking event.

Tsunami runup and TWL inundation calculations in Tables 8.1 also indicate that all of the shore facilities of the Doheny Desalination Project are above tsunami inundation levels at present sea level, as well as safe for the low range projections for sea level rise out to year 2070 or 2100. However, well heads #1, #2, #3 and well pump electrical north would suffer some degree of tsunami overtopping if concurrent with the high-range predictions of sea level at year 2070 or 2100. Only well pump electrical south is safe from flooding and overtopping by tsunami for all sea level rise scenarios anticipated out to year 2100. These findings are consistent with the FEMA tsunami flood map which show that all of the Doheny Beach/San Juan Creek corridor extending several miles inland will be inundate by a shoaling tsunami solitary wave. Again, all of the shore facilities in Figure 6.3 are in close proximity to one another with nothing to support water surface gradients between each installation. Therefore, no significant differences occur in tsunami total water levels at each facility for any given sea level.

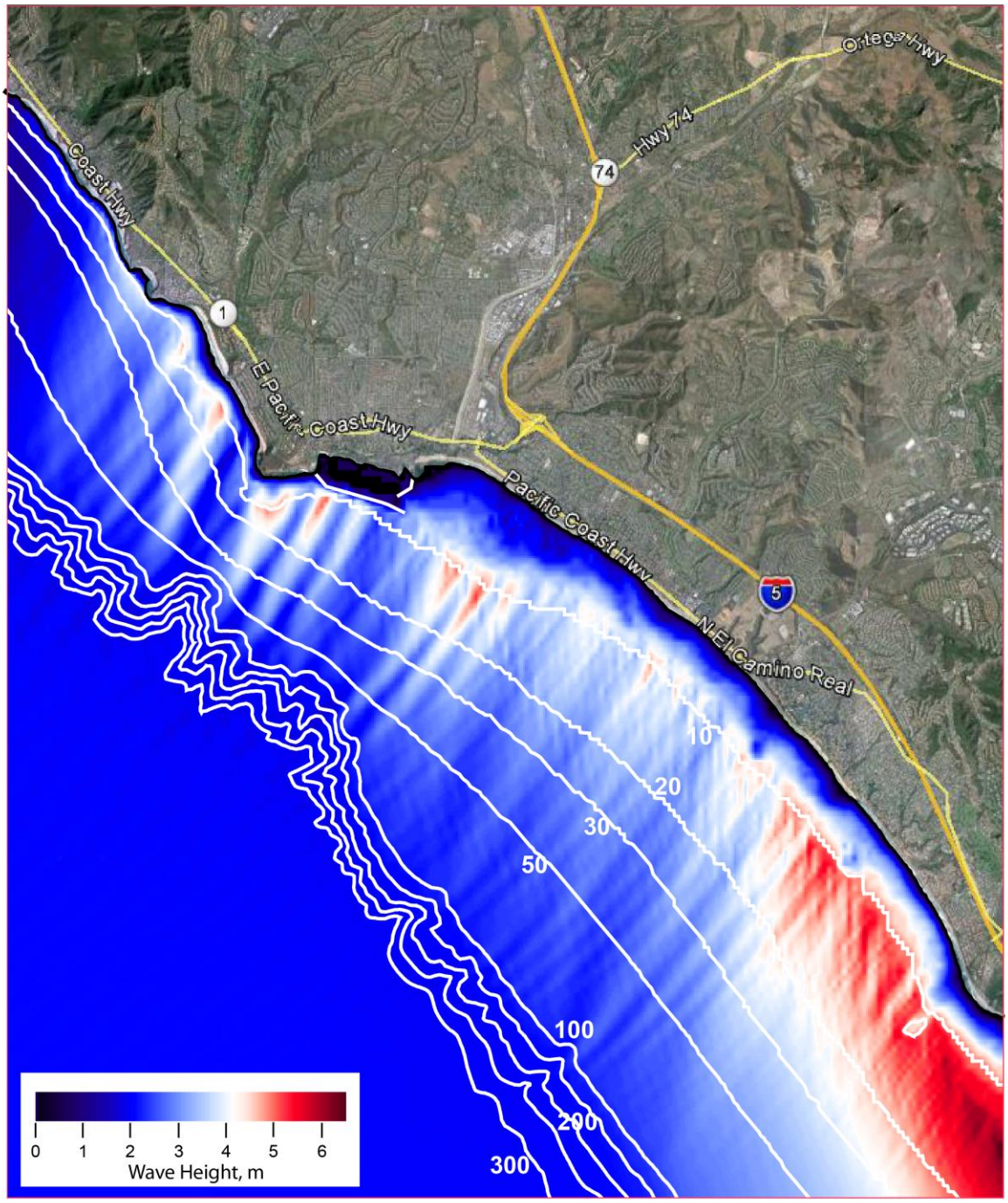


Figure 8.1: High resolution refraction/diffraction computation for a 2m high solitary tsunami wave approaching Doheny Beach from 165 degrees true.

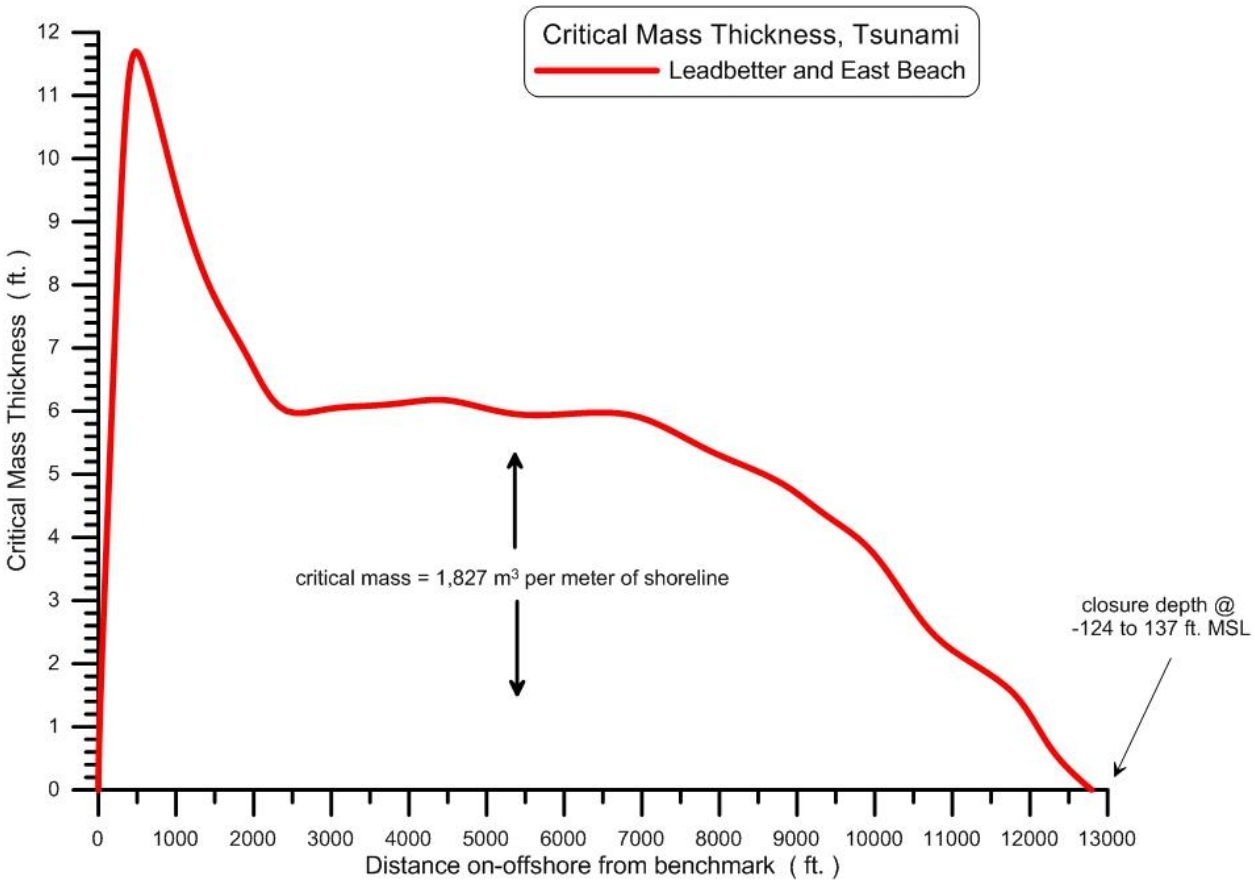


Figure 8.2: Thickness of critical mass envelope at historic survey ranges Doheny Beach, calculated by the calibrated CEM sediment budget based a 2m high solitary tsunami wave approaching Doheny Beach from 165 degrees true. Closure depth = -124 to -137 ft. MSL; critical mass volume = 1,827 m^3 per meter of shoreline.

Table 8.1: Tsunami Total Water Level (*TWL*) and Overtopping Rates (Q') Analysis

	Well Head #1 Elevation = 17 ft. NAVD	Well Head #2 Elevation = 17 ft. NAVD	Well Head #3 Elevation = 18 ft. NAVD	Well Pump Electrical South Elevation = 21 ft. NAVD	Well pump Electrical North Elevation = 18 ft. NAVD
TWL Present Sea Level	15.22 ft. NAVD status = dry	15.22 ft. NAVD status = dry	15.22 ft. NAVD status = dry	15.22 ft. NAVD status = dry	15.22 ft. NAVD status = dry
Q' Present Sea Level	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.
TWL 2070 Sea Level Low Range Projection	15.92 ft. NAVD status = dry	15.92 ft. NAVD status = dry	15.92 ft. NAVD status = dry	15.92 ft. NAVD status = dry	15.92 ft. NAVD status = dry
Q' 2070 Sea Level Low Range Projection	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.
TWL 2070 Sea Level High Range Projection	18.43 ft. NAVD status = flooded	18.43 ft. NAVD status = flooded	18.43 ft. NAVD status = flooded	18.43 ft. NAVD status = dry	18.43 ft. NAVD status = flooded
Q' 2070 Sea Level High Range Projection	0.07 cfs/ft.	0.07 cfs/ft.	0.01 cfs/ft.	0.0/0.0 cfs/ft.	0.01 cfs/ft.
TWL 2100 Sea Level Low Range Projection	16.59 ft. NAVD status = dry	16.59 ft. NAVD status = dry	16.59 ft. NAVD status = dry	16.59 ft. NAVD status = dry	16.59 ft. NAVD status = dry
Q' 2100 Sea Level Low Range Projection	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.
TWL @ 2100 Sea Level High Range Projection	20.71 ft. NAVD status = flooded	20.71 ft. NAVD status = flooded	20.71 ft. NAVD status = flooded	20.71 ft. NAVD status = dry	20.71 ft. NAVD status = flooded
Q' 2100 Sea Level High Range Projection	0.31 cfs/ft.	0.31 cfs/ft.	0.19 cfs/ft.	0.0/0.0 cfs/ft.	0.19 cfs/ft.

*Evaluated for 2m high tsunami deep water wave height approaching Doheny State Beach from 165 degrees true

9) Calculus for Backwater Effects with Extremal Still Water Levels:

A review of the Caltrans District 4 protocols indicates that the "Standard Method" used in the design calculations of back-water effects in open channel coastal drainage systems requires that the design frequency (one percent recurrence interval) must equal the compound frequency of the discharge rate and the ocean water level or:

$$P[Q_T, \eta] = P(Q_T) \bullet P(\eta) = \frac{1}{100} \tag{35}$$

Where Q_T is the flood discharge rate with return period of T years, η_i is the ocean water level equivalent to District 4 usage of "tide stage", $P(Q_T)=1/T$ is the probability of flood discharge rates with return frequency of once every T years; and $P(\eta)$ is the annualized probability of ocean water levels reaching an elevation of η feet NAVD 88. The Standard Method typically poses a 1% compound frequency analysis by combining the discharge rate of the 100-year flood event (Q_{100}) with the mean sea level (MSL), or by combining the extreme high water level (EHW) event with the discharge of the annual flood event (Q_1):

$$P(Q_{100}) \bullet P[\eta = MSL] = \frac{1}{100}$$
$$P(Q_1) \bullet P[\eta = EHW] = \frac{1}{100} \tag{36}$$

Here, *MSL* is *mean sea level* tidal datum, and *EHW* is *extreme high water* tidal datum. But a 1% compound frequency analysis can also be based on other combinations of tidal datums and flood events. Some of these intermediate combinations may be more appropriate design criteria because the duration of these compounded intermediate events may be longer. Intermediate cases under the Standard Method based could also include:

$$P(Q_{28}) \bullet P[\eta = MHW] \cong \frac{1}{100}$$

(3) $P(Q_{13}) \bullet P[\eta = MHHW] \cong \frac{1}{100}$

(37)

$$P(Q_2) \bullet P[\eta = 6.26 ft] \cong \frac{1}{100}$$

Here, *MHW* is *mean high water* tidal datum, and *MHHW* is *mean higher high water* tidal datum.

9.1) Implementation of the Annualized Hydroperiod Approach: The one-percent compound frequency backwater problem is based on two annualized independent probability functions, $P(Q_T)$, $P(\eta)$ and has units of reciprocal years (years $^{-1}$) based on the flood recurrence period, T . Although dimensionless, the annualized probability of ocean water levels, $P(\eta)$, is numerically equivalent to flood recurrence interval when expressed as an annualized hydroperiod function, using the calculus of equations (3) and

(4). This is apparent by the Standard Method exercise in equations (36) & (37) whereby it was found that the annualized probability of wetting for a particular ocean water elevation, e.g. $P(MHW) = 25\% \Leftrightarrow 25/100$, combines exactly with a flood recurrence interval of the same numeric value $P(Q_{25}) = 1/25$, to give a one-percent compound frequency from equation (35). This fact (unique to the one-percent frequency calculus) allows the annualized hydroperiod in Figure 4.1 to be transposed into a *One-Percent Compound Frequency Curve* in Figure 9.1, which establishes a direct relationship between the flood recurrence period and the elevation of a project facility such that there is only a 1% chance a backwater effect will occur as a result of high ocean water levels with a flood that occurs once every *T* years. Figure 9.1 gives a *One-Percent Compound Frequency Curve* at present sea level that was derived from equations (3) and (4) and transposed from Figure 4.1. The corresponding *One-Percent Compound Frequency Curves* for 2070 and 2100 sea levels are shown in Figures 9.2 & 9.3.

9.2 The Direct Design Problem: Suppose the project engineer specifies a project facility at elevation of + 4.0 ft NAVD in the lower reach of San Juan Creek and wants to know how severe a flood that facility can incur without a greater than 1% chance of flooding from a back water effect as a consequence high ocean water levels. Figure 9.4 provides a graphic example of how to solve this problem using a *One-Percent Compound Frequency Curve* from either Figure 9.1 at present sea level or from Figures 9.2 or 9.3 for future sea levels. Step-1: make a horizontal projection from the y-axis at + 4.0 ft NAVD to an intersection point on the *One-Percent Compound Frequency Curve.* Step-2: make a vertical projection from this intersection on the compound frequency curve to an intersection with the x-axis. Read the flood recurrence period of 70 years. Hence the engineer needs to design the facility with sufficient safeguards for a Q_{70} discharge rate to avoid damage from back-water problems.

9.3 The Inverse Design Problem: : Suppose the project engineer is designing a project facility in the lower reach of San Juan Creek and wants to know what the minimum elevation of that facility could be without incurring a greater than 1% chance of a back water effect from high ocean water levels during San Juan Creek floods. Figure 9.5 provides a graphic example of how to solve this problem using a *One-Percent Compound Frequency Curve*. Step-1: make a vertical projection from the x-axis at a flood return period equal to project life (say 50 years) and continue that projection to an intersection point on the *One-Percent Compound Frequency Curve.* Step-2: make a horizontal projection from this intersection on the compound frequency curve to an intersection with the y-axis. Read project facility elevation of + 4.5 ft NAVD. Hence the engineer needs to design the facility elevation of + 4.5 ft NAVD to avoid back-water problems with a Q_{50} discharge rate from the San Juan Creek. Other combinations of flood hydrograph and facility elevations at future sea levels are indicated on the *One-Percent Compound Frequency Curves* in Figures 9.2 & 9.3.

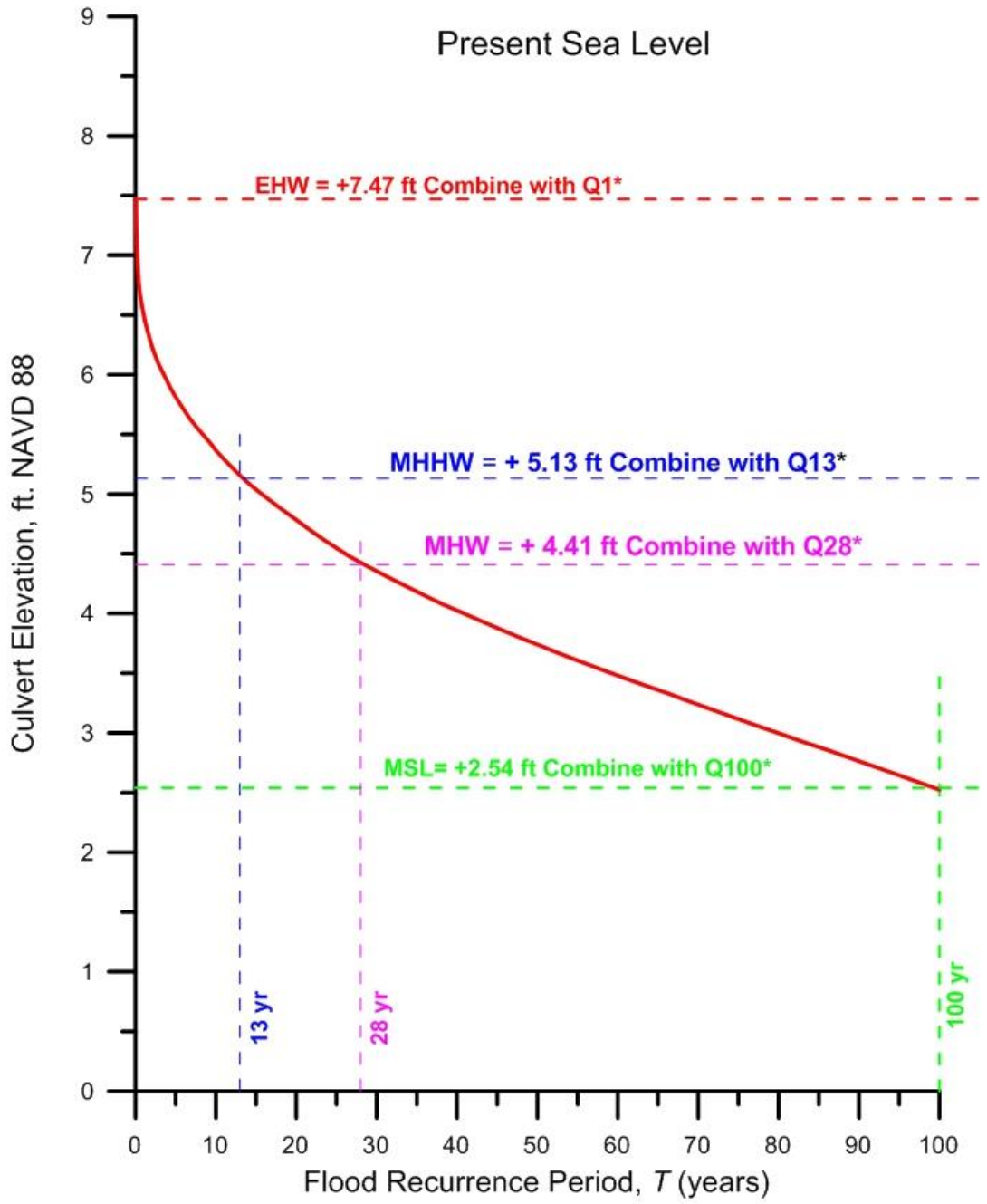


Figure 9.1: One-Percent Compound Frequency Curve at present sea level for determining the appropriate combinations of San Juan Creek flood events and ocean water levels in a 1% compound frequency analysis of potential project related back-water effects at the Doheny Desalination Project site, (based on NOAA # 9410230, La Jolla, Scripps Pier)

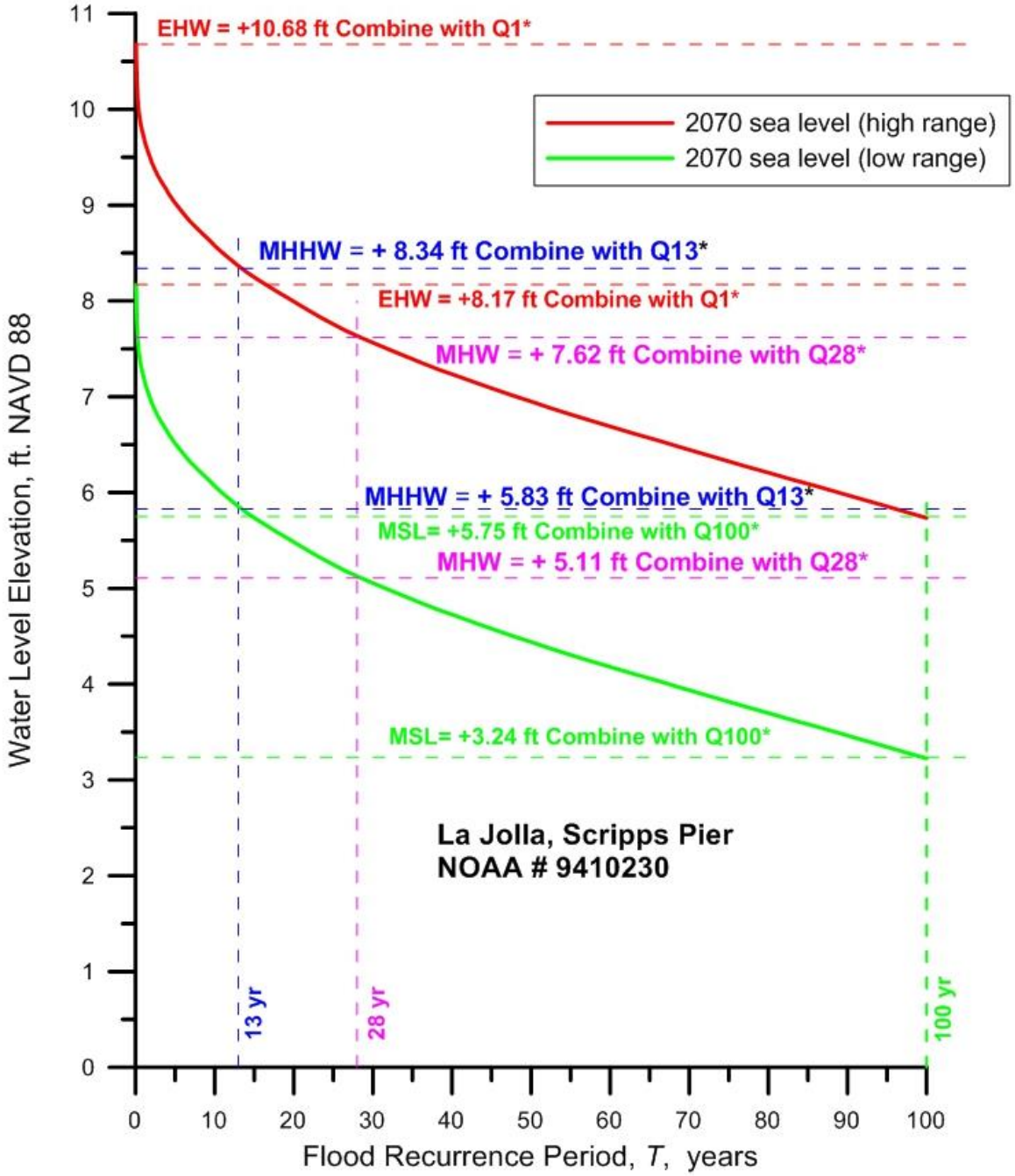


Figure 9.2: One-Percent Compound Frequency Curve at 2070 sea level for determining the appropriate combinations of San Juan Creek flood events and ocean water levels in a 1% compound frequency analysis of potential project related back-water effects at the Doheny Desalination Project site, (based on NOAA # 9410230, La Jolla, Scripps Pier)

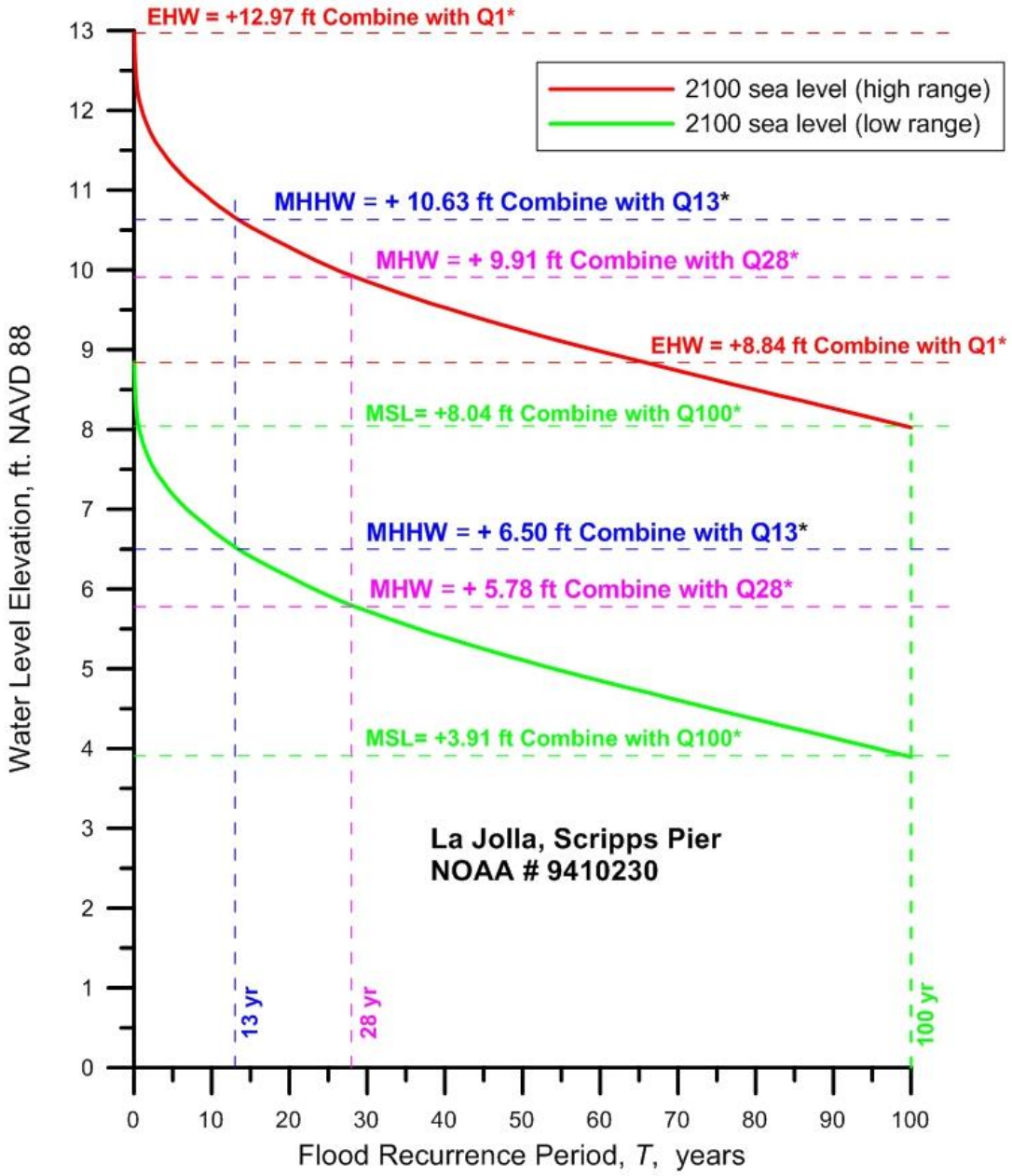


Figure 9.3: One-Percent Compound Frequency Curve at 2100 sea level for determining the appropriate combinations of San Juan Creek flood events and ocean water levels in a 1% compound frequency analysis of potential project related back-water effects at the Doheny Desalination Project site, (based on NOAA # 9410230, La Jolla, Scripps Pier)

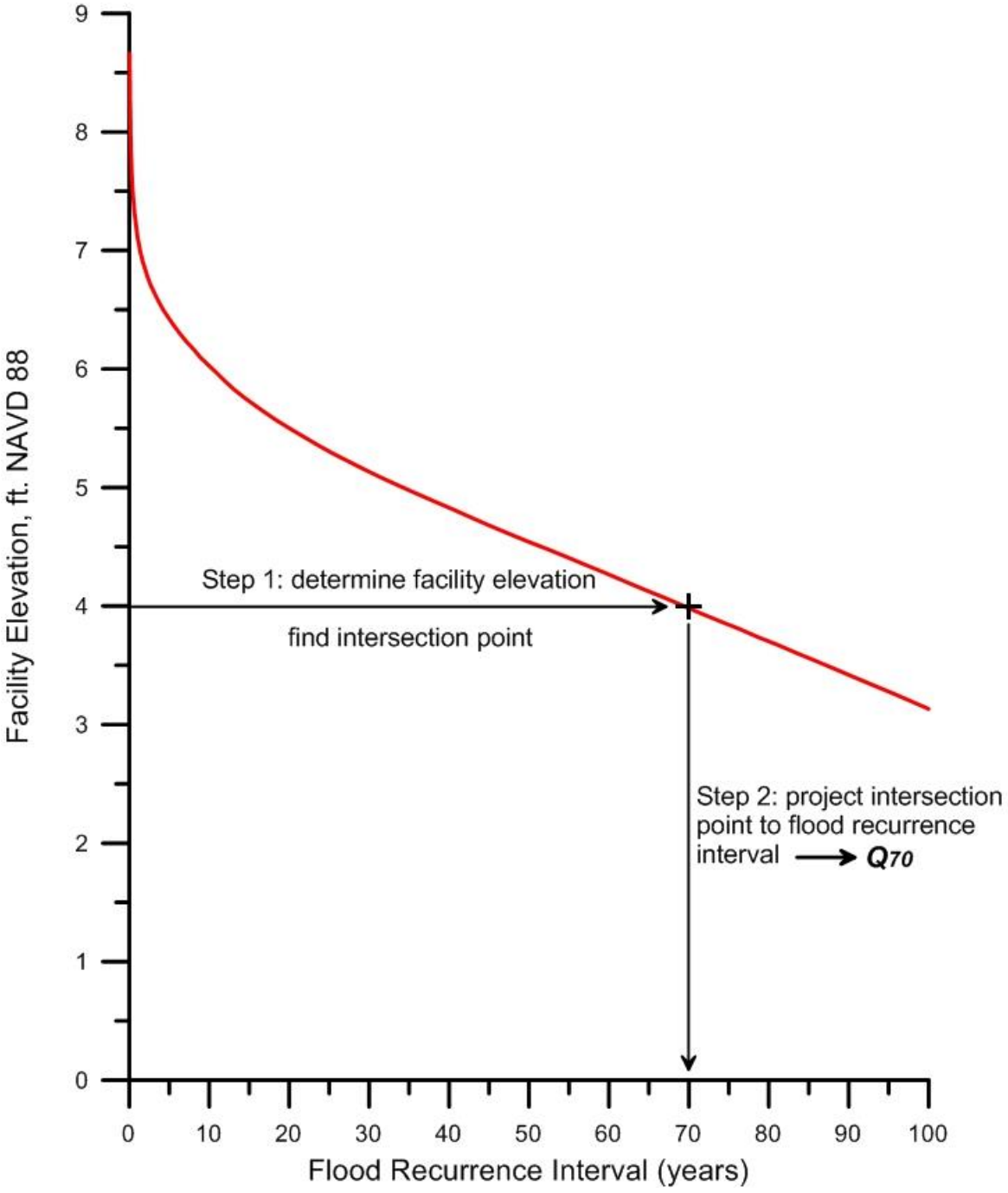


Figure 9.4: Graphic example of how to solve the *Direct Design Problem* using a *One-Percent Compound Frequency Curve* derived from the annualized hydroperiod function for the Doheny Desalination Project.

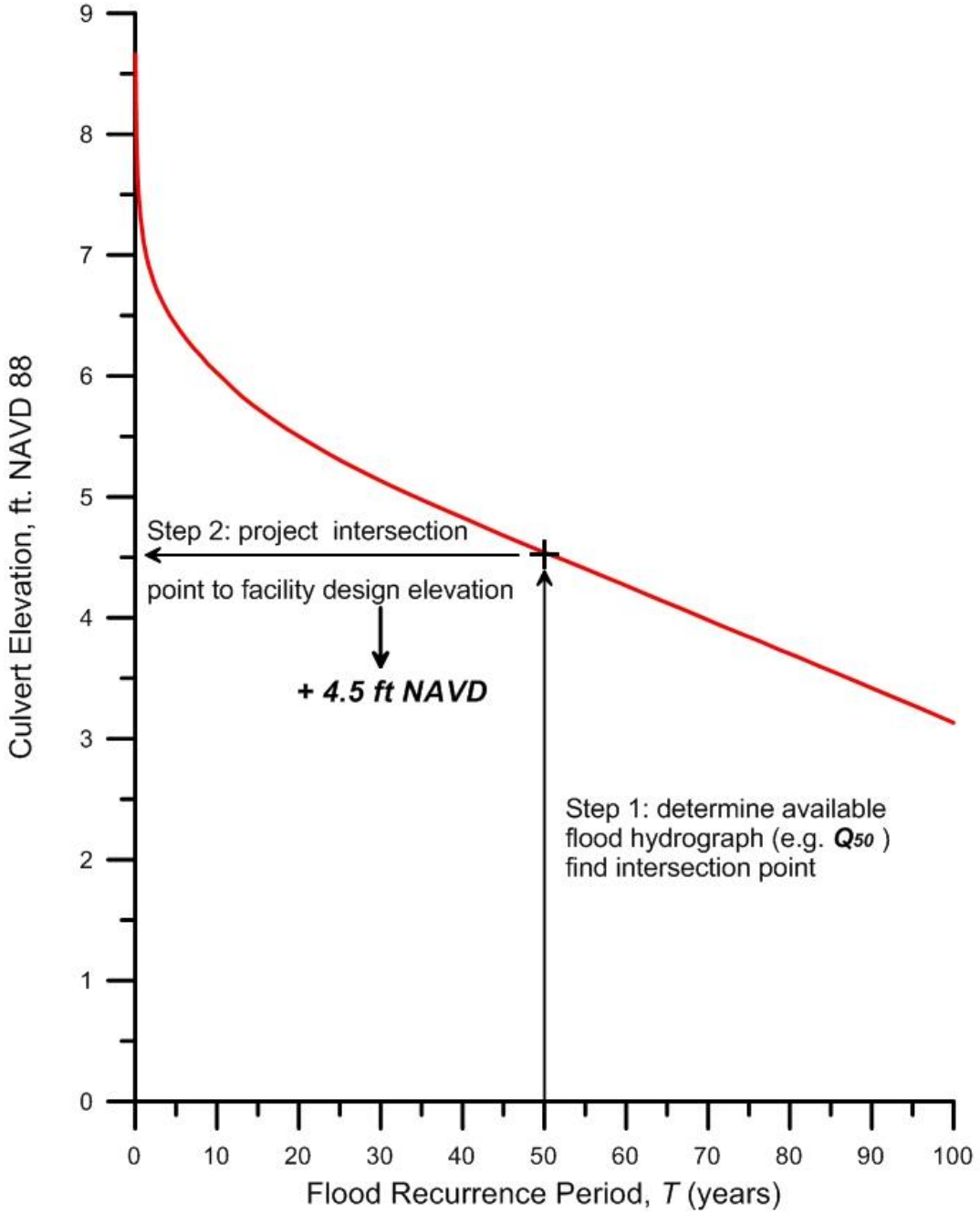


Figure 9.5: Graphic example of how to solve the *Inverse Design Problem* using a *One-Percent Compound Frequency Curve* derived from the annualized hydroperiod function for the Doheny Desalination Project.

10) Summary and Conclusions:

The *Coastal Evolution Model* (CEM) developed at the Scripps Institution of Oceanogrphy was used to evaluate Appendix-B requirements of the *California Coastal Commision Sea Level Rise Policy Guidance* document (CCC, 2015) for a sea level rise/coastal hazards analysis of the Doheny Desalination Project (DDP). The Coastal Evolution Model is public domain and available from the University of California Digital Library at: http://repositories.cdlib.org/sio/techreport/58/. The Coastal Evolution Model employs algorithms consistent with the U.S. Army Corps of Engineers Coastal Engineering Manual, (USACE, 2006), but employs the latest generation equilibrium beach profile algorithms from Jenkins and Inman (2006) that provide 3-dimensional predictive and mapping capability of the wave run-up field, beach erosion and shoreline recession under the effects of wave climate variability, climate cycles and sea level rise. The CEM input files were populated with National Ocean Survey (NOS) digital bathymetry in the offshore domain; beach profiles sediment gran size measurements by the U.S. Army Corps of Engineers, Coastal Environments and Coastal Frontiers; long-term wave data from the Coastal Data Information Program; long-term ocean water level measurements by the National Atmospheric and Oceanic Administration; and stream flow and sediment flux for the San Juan Creek from the United States Geological Survey and the Federal Emergency Management Agency. Sea level rise projections used in this study were based on the *best fit equation* from Appendix-B of the *California Coastal Commision Sea Level Rise Policy Guidance* document for a 50 year project planning horizon (year 2070) and for a *critical infrastructure* planning horizon (year 2100). Critical project infrastructure subject to potential flooding by extreme event waves concurrent with extreme ocean water levels and sea level rise include: ***Wellhead 1***, elevation 17 ft.NAVD, at 33 27 43.99834 N, 117 41 13.25899 W; ***Wellhead 2***, elevation 17 ft. NAVD, at 33 27 44.16980 N, 117 41 06.01901 W; ***Wellhead 3***, elevation 18 ft. NAVD at 33 27 44.07064 N, 117 40 53.96544 W; ***Well Pump Electrical (North)***, at elevation 18 ft. NAVD at 33 27 48.54582 N, 117 41 02.09570 W; and **Well Pump Electrical (South)**, at elevation 21 ft. NAVD at 33 27 47.44890 N, 117 40 54.92019 W; (see Figure 6.3).

Appendix-B of the *California Coastal Commision Sea Level Rise Policy Guidance* document (CCC, 2015) provides no specific guidance on the redline frequency for flooding or inundation. In the absence of such guidance we have adopted Federal Emergency Management Agency (FEMA) standards for flooding frequency and set redline planning frequency at the 100 year event (1% probability of recurrence). The 100 year wave event was the two day storm of 17-18 January, 1988, which produced deep water significant wave heights off Doheny State Beach reaching 15.5 ft., approaching the beach from 270^0 with 15 second significant wave periods. An analysis of extremal total water levels, (TWL's), based on the occurrence of extreme waves concurrent with extreme ocean water levels, is summarized in Table 7.1. Inspection of Table 7.1 reveals that all the beach front facilities for the Doheny Desalination Project (Figure 6.3) are safe from flooding or inundation at present sea levels by extreme event waves concurrent with extreme ocean water levels, even for event return periods as long as 500 yr. In fact, all these facilities are safe from flooding by the 100-year event in environments of the low-range predictions for 2070 and 2100 sea levels. However, once we admit to the high-

range sea level rise projections, a number of the beach front facilities for the Doheny Desalination Project will suffer some flooding and overtopping to varying degrees.

For the high-range 2070 sea level projections, the 100-yr total water level events reach $TWL(100)$ = 18.0 ft. NAVD for the accreted beach conditions and $TWL(100)$ = 16.3 ft. NAVD for the eroded beach conditions. This indicates that well heads #1 and #2 are vulnerable to flooding by any event that exceeds a return period of 36 years; and could be flooded by as much as 1.0 ft of overtopping during accreted beach conditions, with overtopping rates of as much as Q' = 0.04 cfs/ft. Flooding and overtopping of well heads #1 and #2 under eroded beach conditions would not happen until events exceeding 225 year return periods when extreme waves happen concurrently with extreme ocean water levels under high-range 2070 sea level projections. Total water levels for eroded beach conditions are always less, because these beaches have flatter slopes and are more dissipative of wave set-up and run-up than the steeper accreted beaches.

For the high-range 2100 sea level projections, Table 7.1 indicates the 100 year total water level events reach $TWL(100)$ = 20.3 ft. NAVD for the accreted beach conditions and $TWL(100)$ = 18.5 ft. NAVD for the eroded beach conditions. Consequently all beach front facilities for the Doheny Desalination Project except well pump electrical south are vulnerable to flooding by the 100-year event. Well heads #1 and #2 would be flooded by even a 3-year event under accreted beach conditions and by a 16-year event under eroded beach conditions. Well heads #1 and #2 are vulnerable to flooding by as much as 3.3 ft of overtopping during a 100 year event, which results in an overtopping rate of Q' = 0.26 cfs/ft under conditions when extreme waves happen concurrently with extreme ocean water levels that are superimposed on the high range of 2100 sea levels. Similarly well head #3 and well pump electrical north could be flooded by 2.3 ft of overtopping, resulting in an overtopping rate of Q' = 0.15 cfs/ft when extreme waves happen concurrently with extreme ocean water levels that are superimposed on the high range of 2100 sea levels. Well pump electrical south is safe from flooding by any event less than 265-year event in environments of the high-range predictions for 2100 sea levels. Because all of the shore facilities in Table 7.1 and Figure 6.3 are in close proximity to one another, (clustered around the mouth of the San Juan Creek with nothing to support water surface gradients between each installation), there are insignificant differences in extremal total water levels occurring at each facility for any given event.

Tsunami induced erosion, runup, and inundation were analyzed for the Doheny State Beach bottom profiles and shore-side facilities associated with the Doheny Desalination Project for present and future sea levels according to low and high range sea level rise predictions. The tsunami event scenario is based on a 2m high solitary wave approaching Doheny Beach from 165 degrees true, as could be anticipated for a catastrophic tsunami event arising from a major landside on the east side of San Clemente Island. The local refraction/diffraction pattern from the solitary wave reveals the tsunami wave height begins to increase at 50 m of water depth due to shoaling, and reaches 6m of height before breaking along the shores of Doheny Beach. Because the tsunami wave begins shoaling in much deeper water than typical storm-induced waves, it causes seabed scour and erosion to occur out to very deep water depths. Therefore all run-up and total water level solutions are based eroded beach profile conditions. Tsunami TWL inundation calculations are summarized Tables 8.1 and indicate that all of the shore

facilities of the Doheny Desalination Project are above tsunami inundation levels at present sea level; as well as safe for the low range projections for sea level rise out to year 2070 or 2100. However, well heads #1, #2, #3 and well pump electrical north would suffer some degree of tsunami overtopping if concurrent with the high-range predictions of sea level at year 2070 or 2100. Only well pump electrical south is safe from flooding and overtopping by tsunami for all sea level rise scenarios anticipated out to year 2100. These findings are consistent with the FEMA tsunami flood map which show that all of the Doheny Beach/San Juan Creek corridor extending several miles inland will be inundate by a shoaling tsunami solitary wave. Again, all of the shore facilities in Figure 6.3 are in close proximity to one another with nothing to support water surface gradients between each installation. Therefore, no significant differences occur in tsunami total water levels at each facility for any given sea level.

Finally, we provide guidance for how to apply the extreme ocean water level data developed in this study to the compound frequency calculation of the 1% (100-year) event used in the design calculations of back-water effects for project facilities to be located along the lower reaches of the San Juan Creek. We develop *One-Percent Compound Frequency Curves* for present and future sea levels using NOAA ocean water level measurements superimposed on sea level rise predictions for 2070 and 2100. The *One-Percent Compound Frequency Curves* allow the project engineers to determine the appropriate combinations of flood hydrographs and ocean water levels at the mouth of San Juan Creek, such that the modeled backwater effects will represent a 100-year event. This result in turn will provide design guidance for the required elevations for project facilities along the lower San Juan Creek in order to avoid flooding and inundation of those facilities during river floods that happen concurrently with extreme ocean water levels.

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Appendix-A: Refraction/Diffraction Plots for Extreme Event Waves:

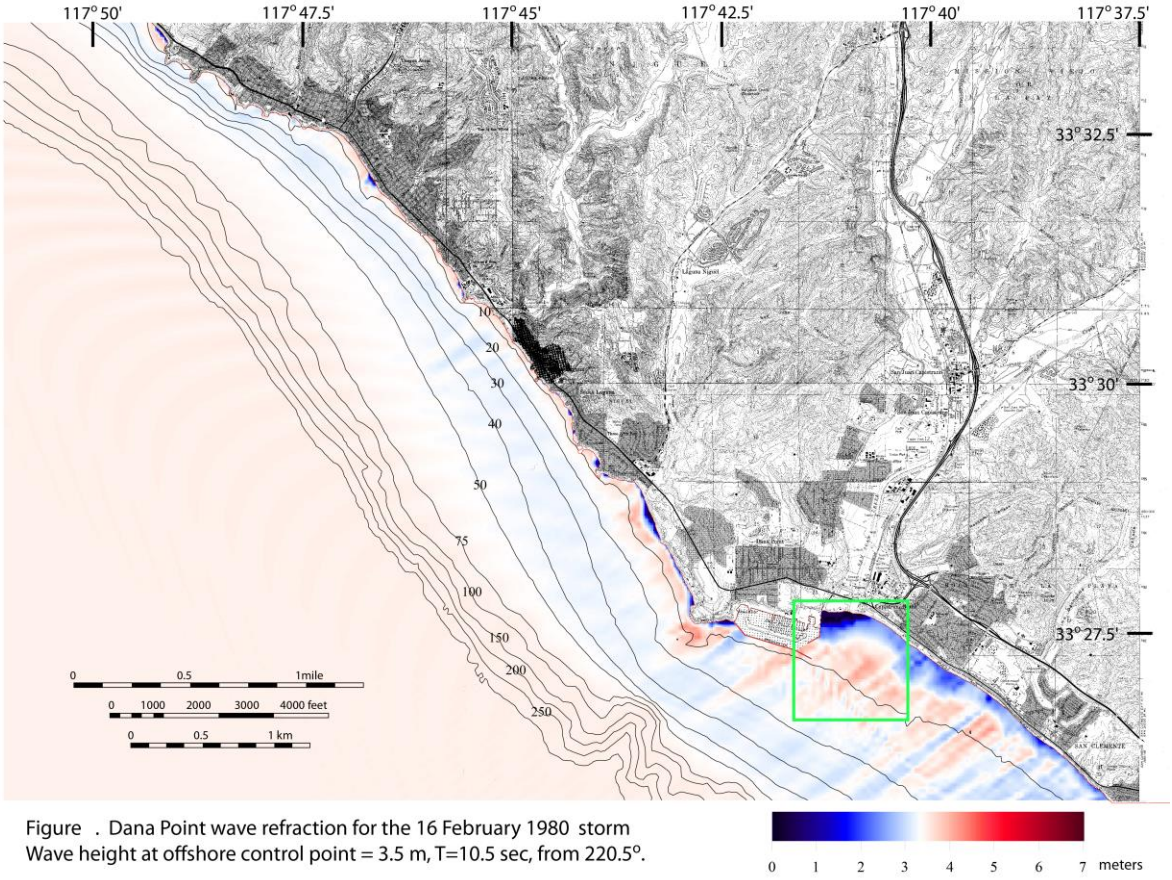

Figure . Dana Point wave refraction for the 16 February 1980 storm
Wave height at offshore control point = 3.5 m, T=10.5 sec, from 220.5°.

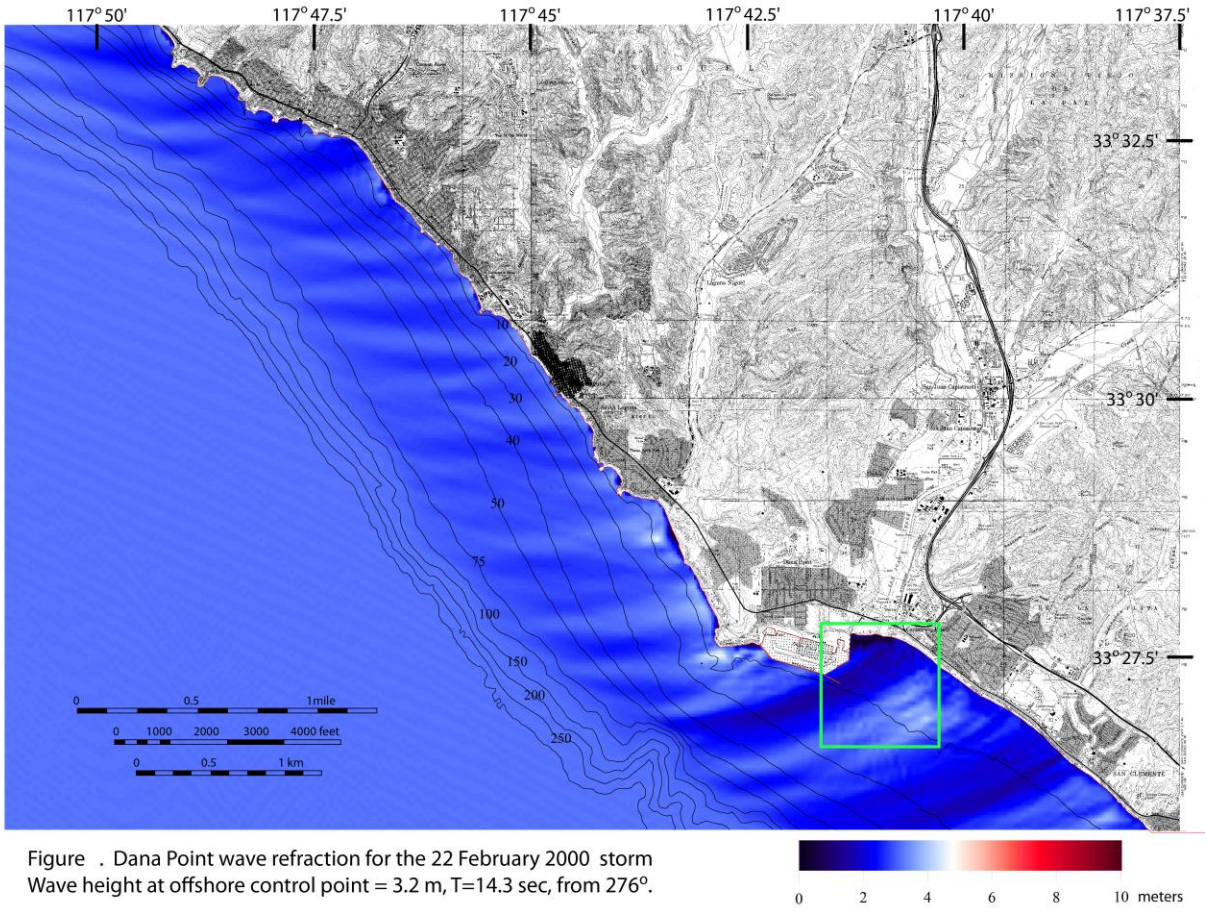

Figure . Dana Point wave refraction for the 22 February 2000 storm
Wave height at offshore control point = 3.2 m, T=14.3 sec, from 276°.

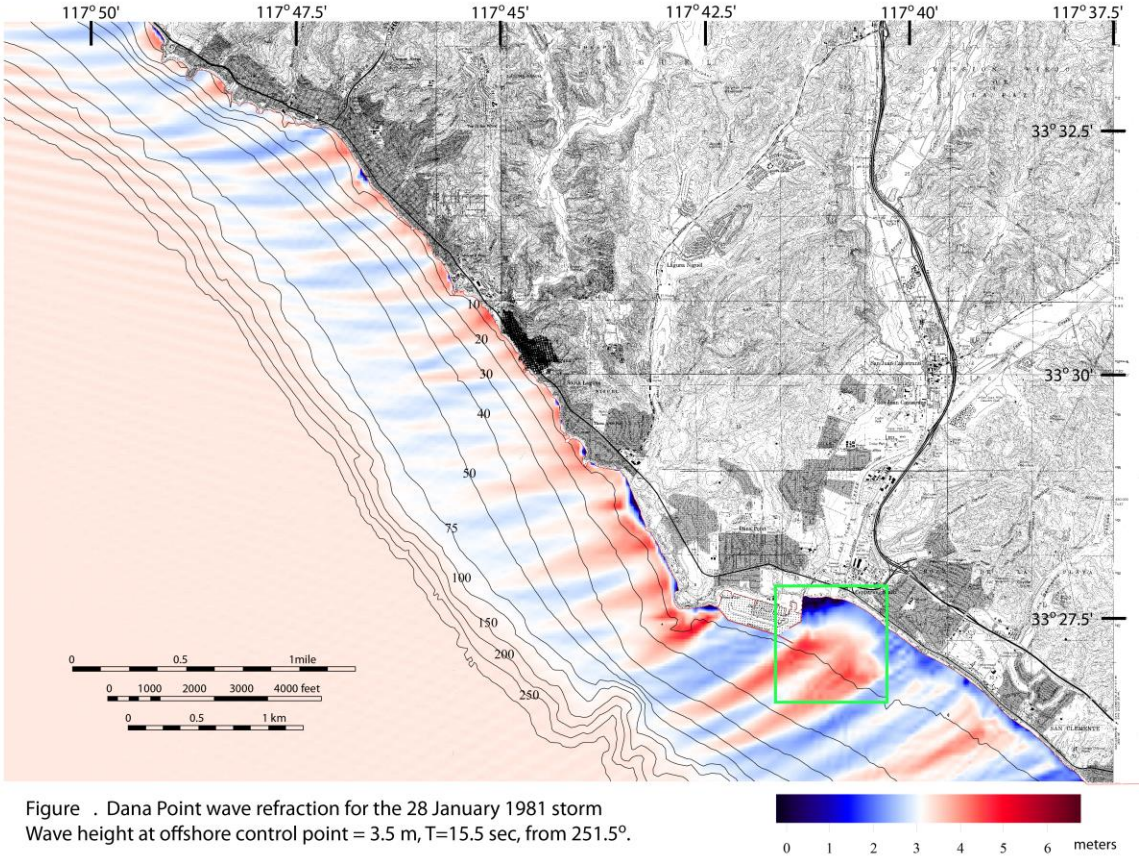

Figure . Dana Point wave refraction for the 28 January 1981 storm
Wave height at offshore control point = 3.5 m, T=15.5 sec, from 251.5°.

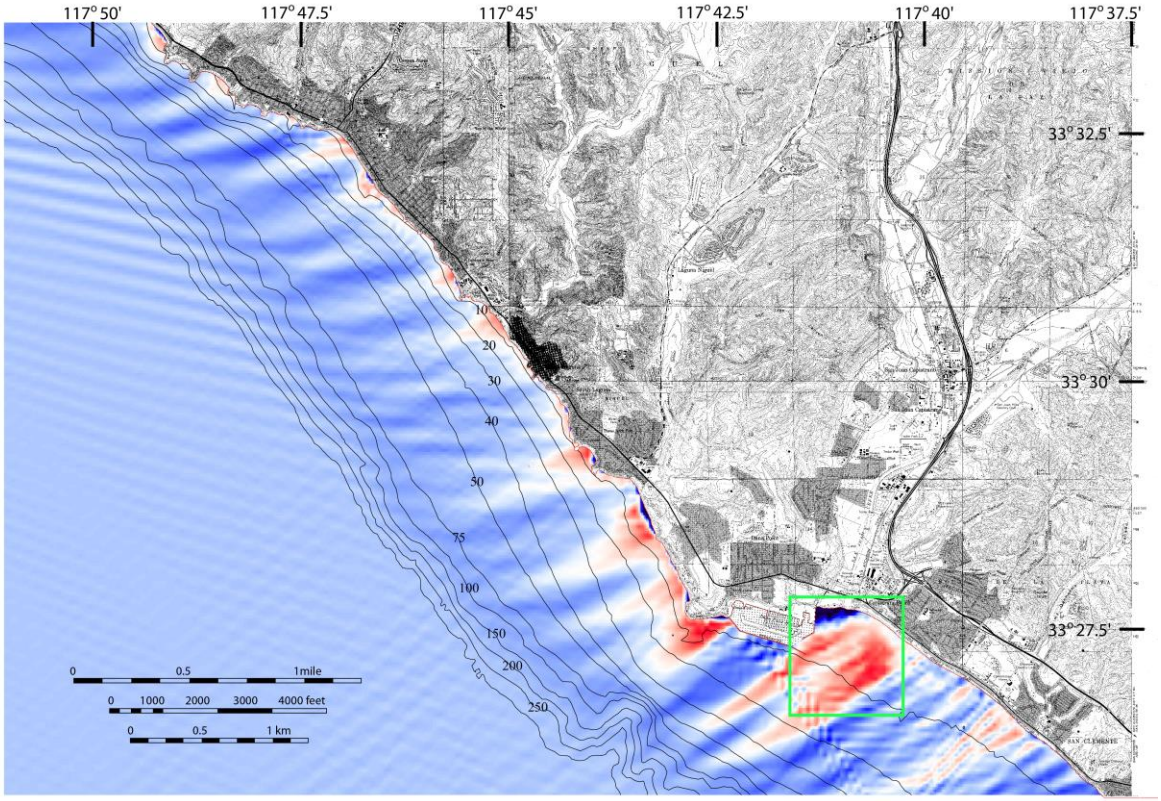

Figure . Dana Point wave refraction for the 25 January 1983 storm
Wave height at offshore control point = 2.1 m, T=19.5 sec, from 246°.

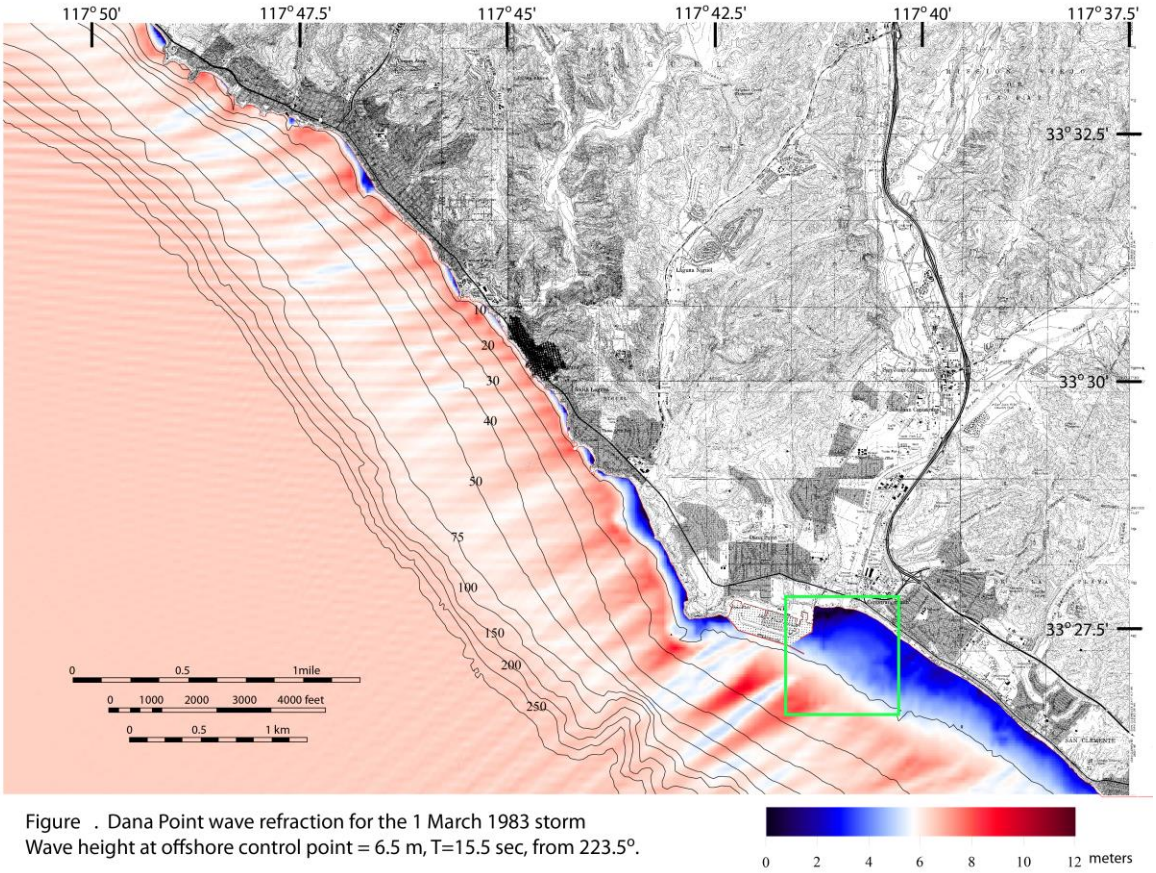

Figure . Dana Point wave refraction for the 1 March 1983 storm
Wave height at offshore control point = 6.5 m, T=15.5 sec, from 223.5°.

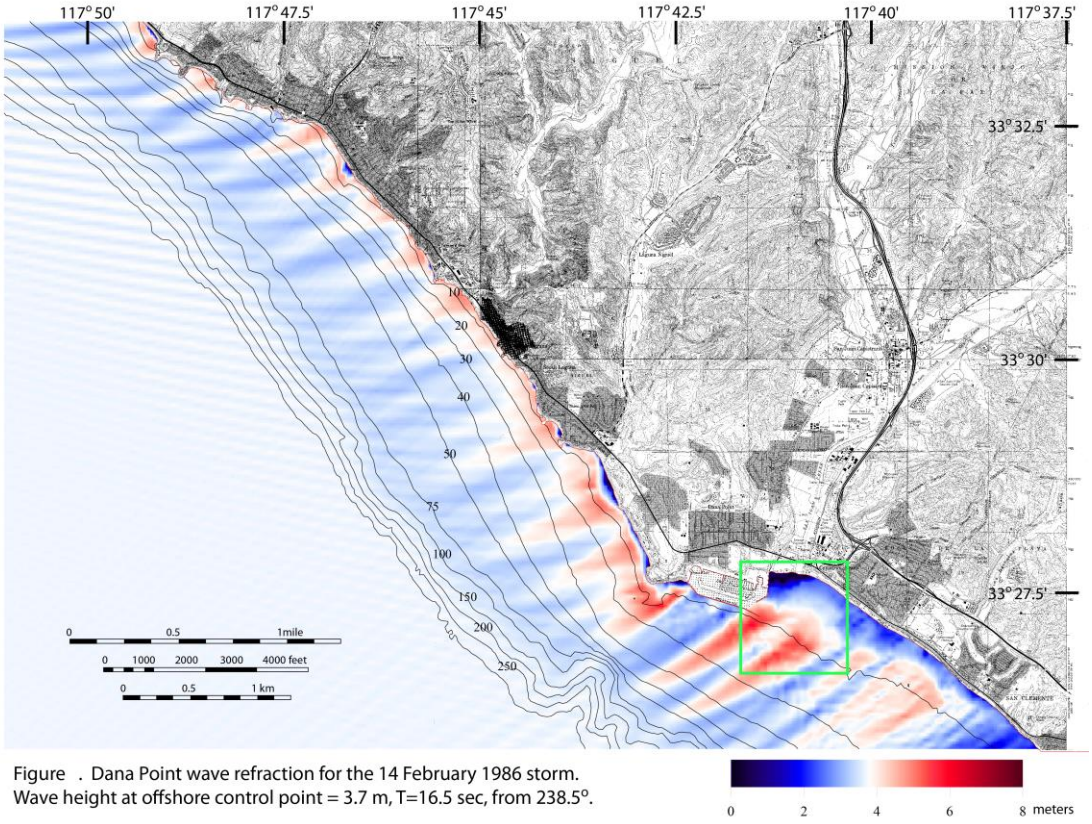

Figure . Dana Point wave refraction for the 14 February 1986 storm.
Wave height at offshore control point = 3.7 m, T=16.5 sec, from 238.5°.

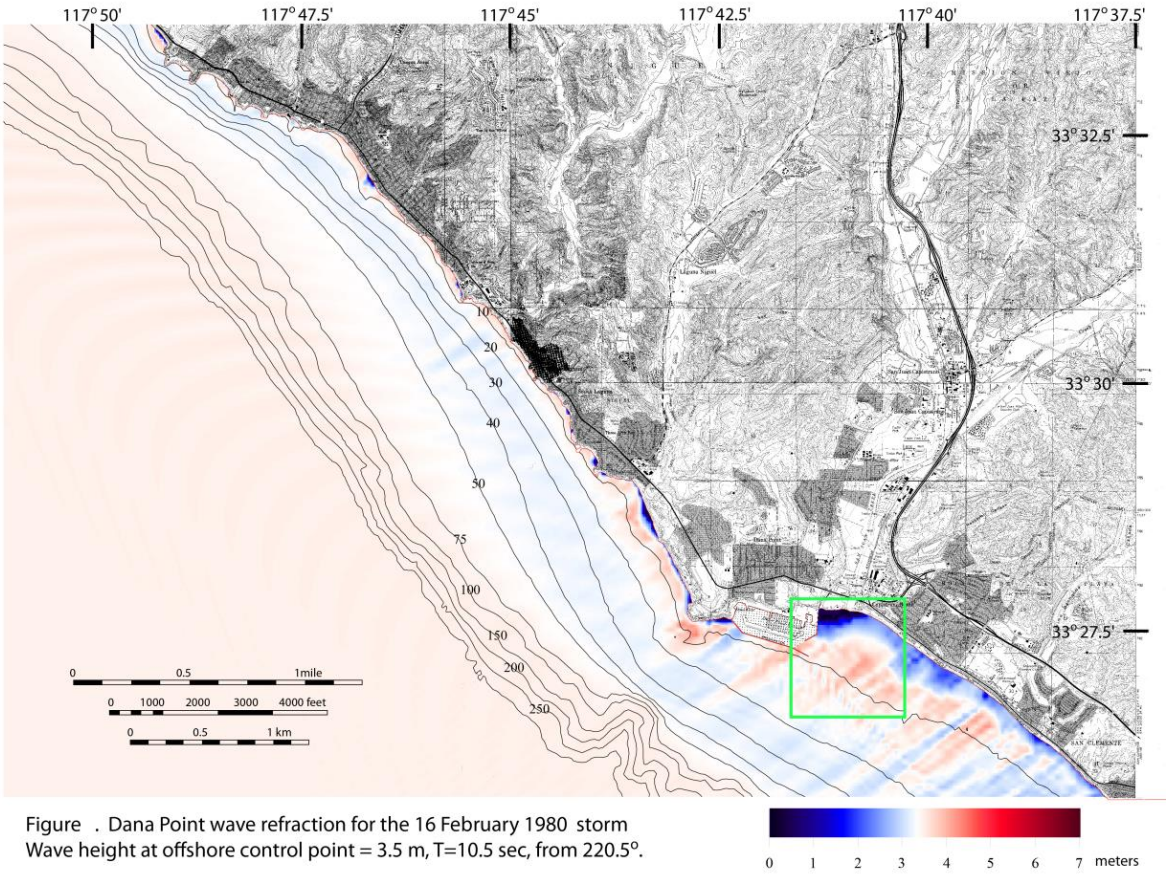

Figure . Dana Point wave refraction for the 16 February 1980 storm
Wave height at offshore control point = 3.5 m, T=10.5 sec, from 220.5°.

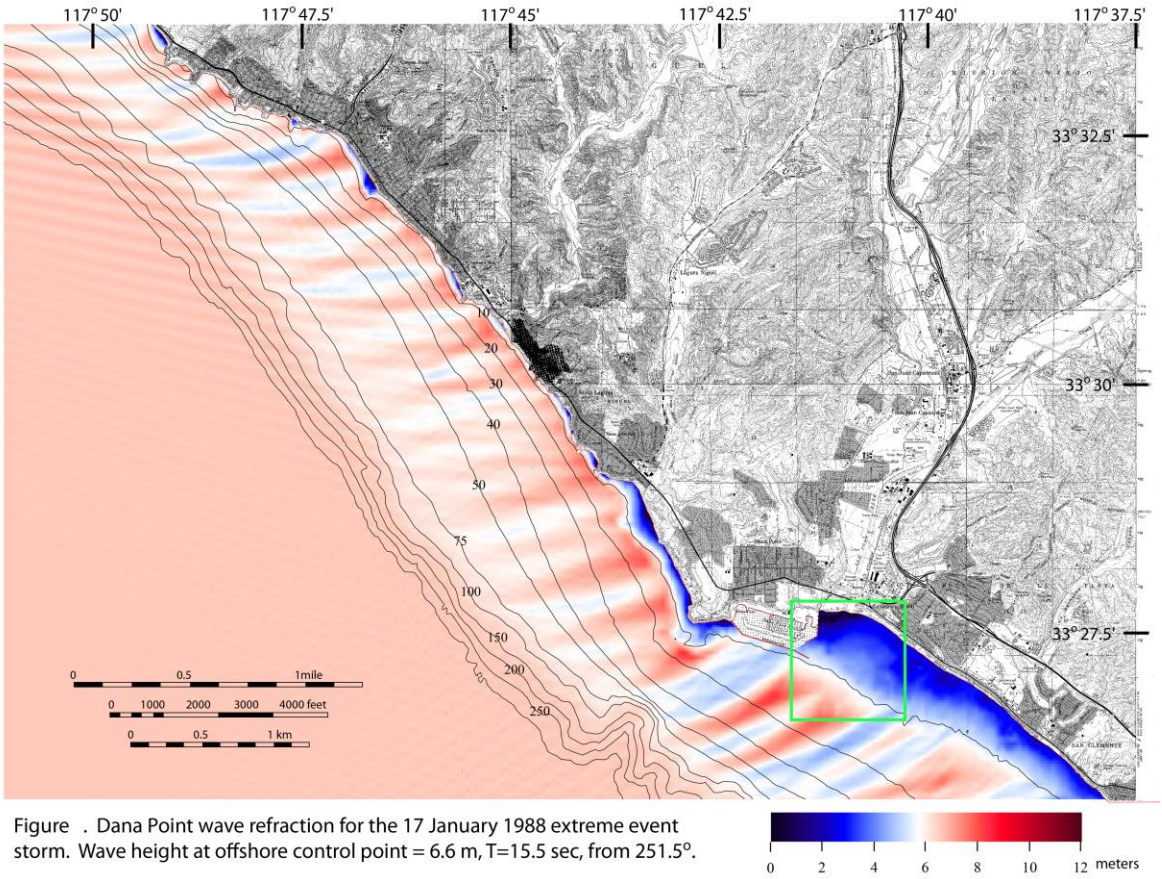

Figure . Dana Point wave refraction for the 17 January 1988 extreme event storm. Wave height at offshore control point = 6.6 m, T=15.5 sec, from 251.5°.

The 100 year (1%) Storm, Day-1

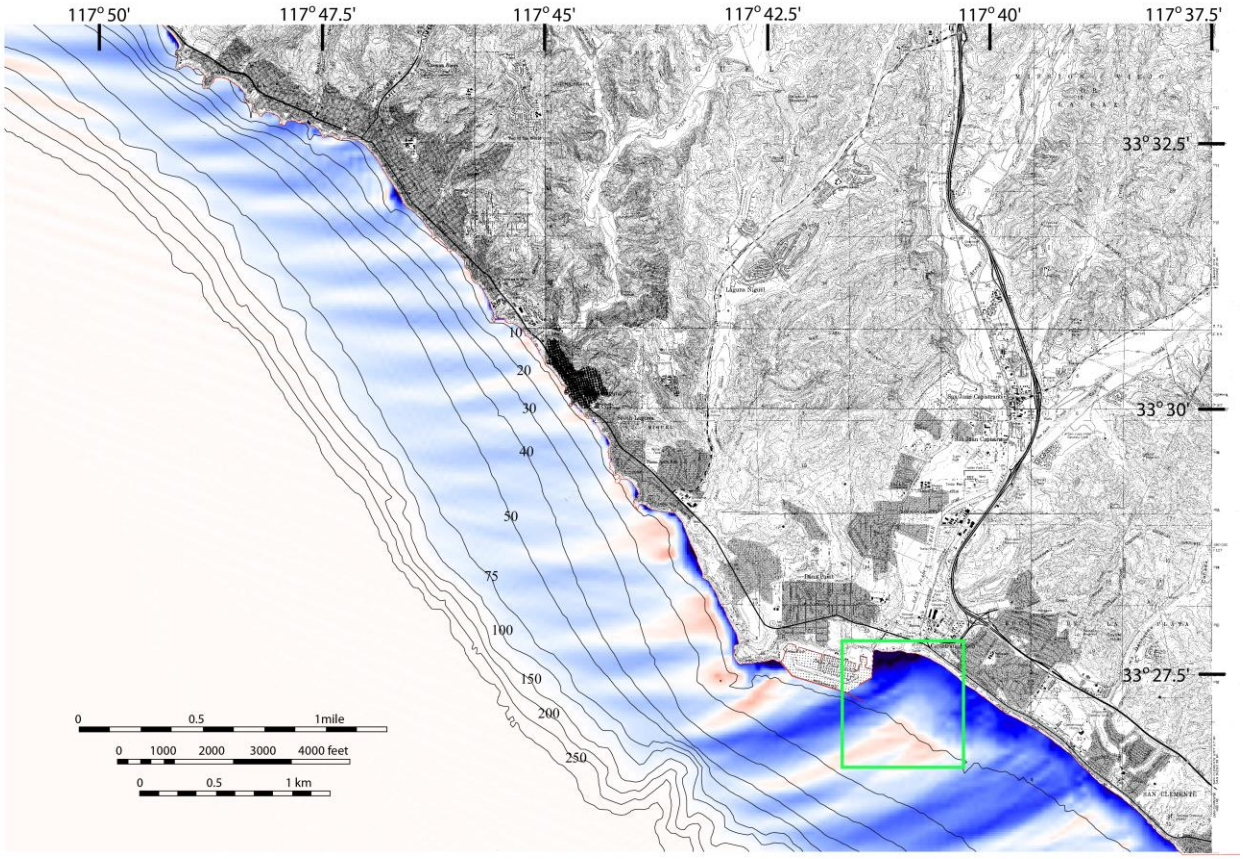

Figure 4. Maximum event wave refraction for period of record, 1980-2000 (18 January 1988). Wave height at offshore control point = 4.8 m, T=14 sec, from 270°.

The 100 year (1%) Storm, Day-2

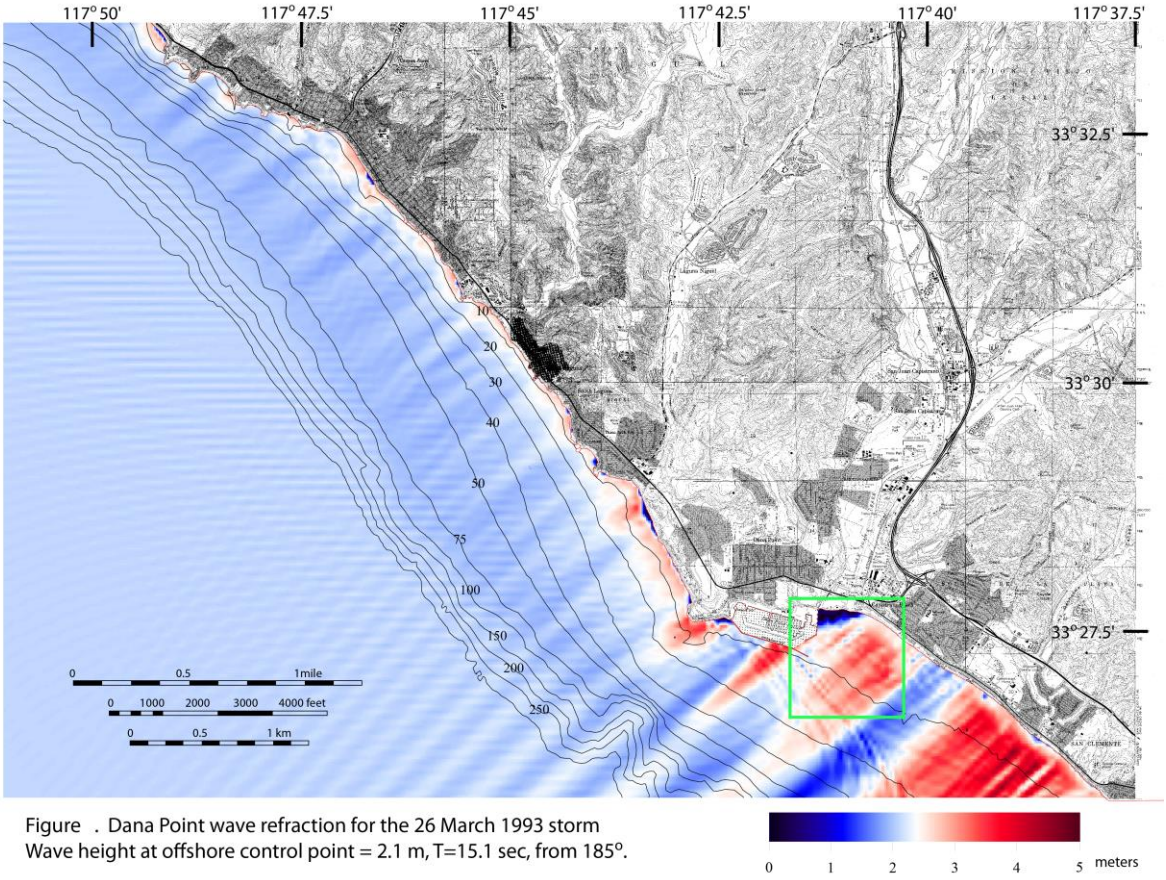

Figure . Dana Point wave refraction for the 26 March 1993 storm
Wave height at offshore control point = 2.1 m, T=15.1 sec, from 185°.

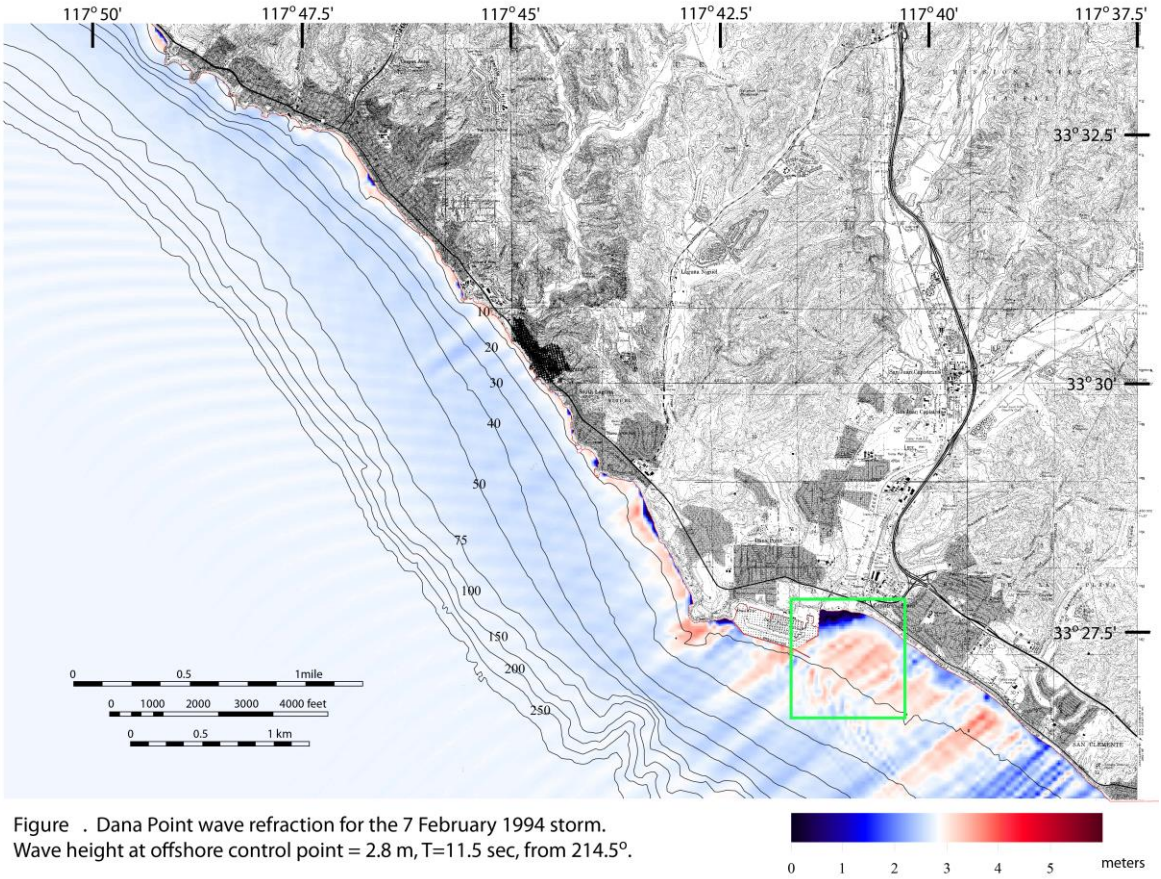

Figure . Dana Point wave refraction for the 7 February 1994 storm.
Wave height at offshore control point = 2.8 m, T=11.5 sec, from 214.5°.

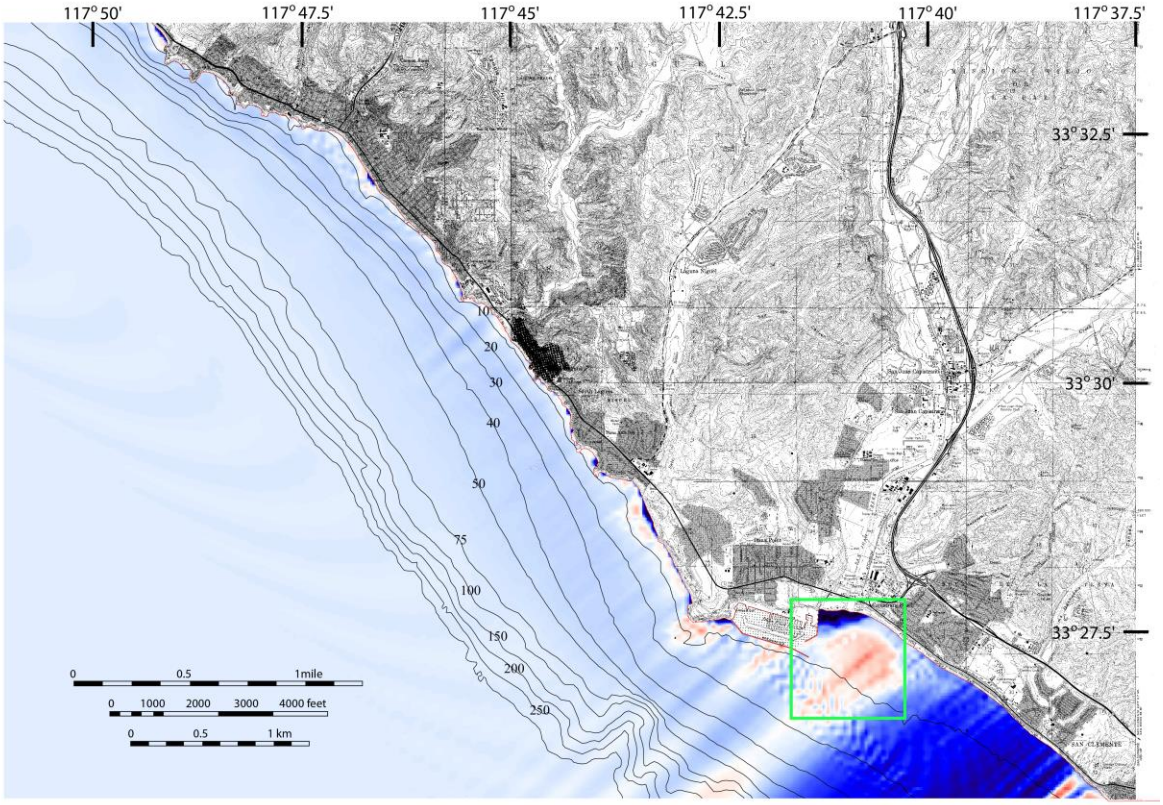

Figure . Dana Point wave refraction for the 5 January 1995 storm
Wave height at offshore control point = 2.7 m, T=10.2 sec, from 199°.

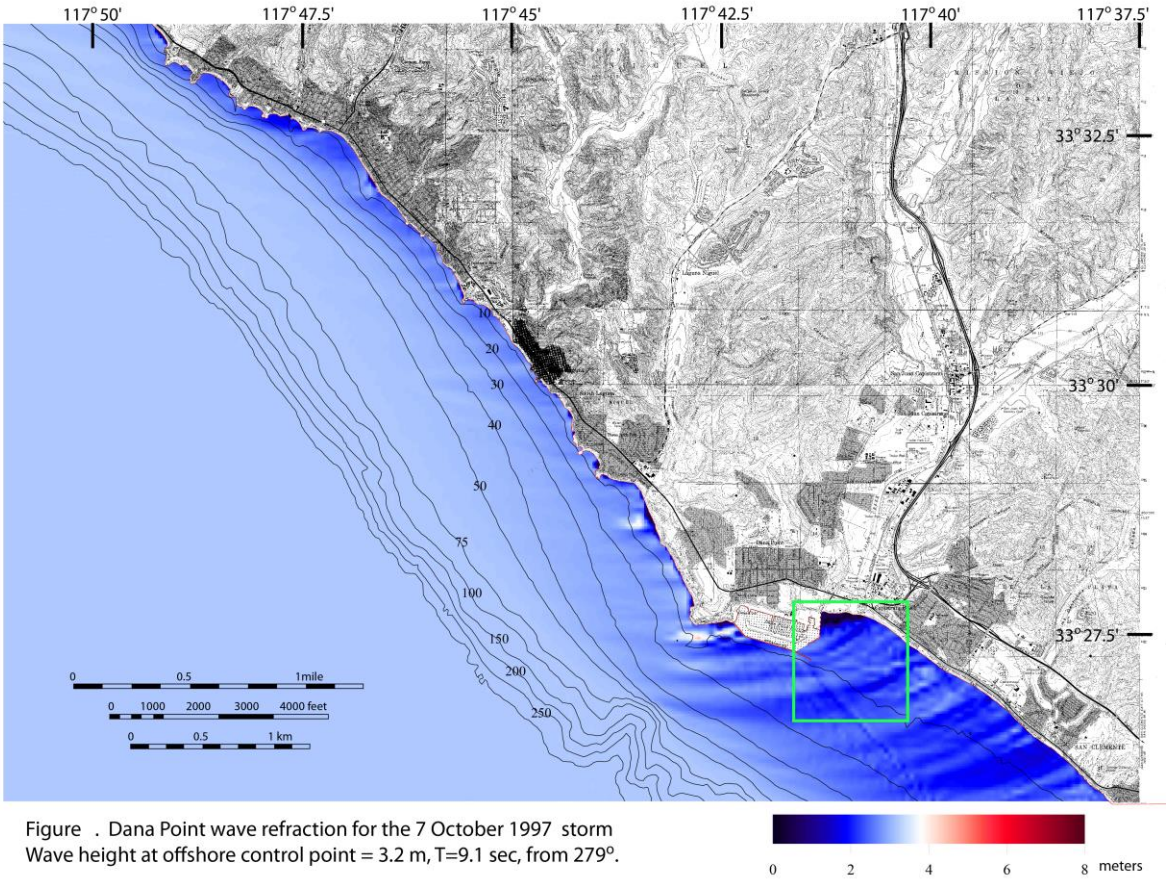

Figure . Dana Point wave refraction for the 7 October 1997 storm
Wave height at offshore control point = 3.2 m, T=9.1 sec, from 279°.

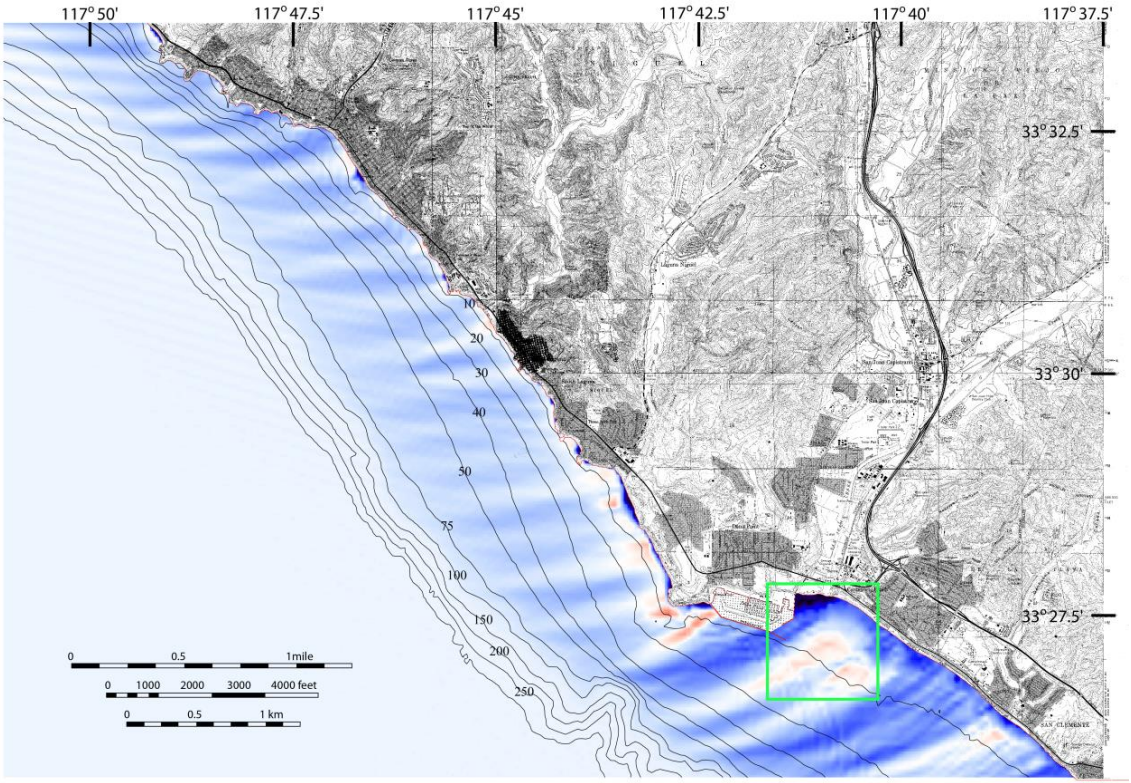

Figure . Dana Point wave refraction for the 4 February 1998 storm
Wave height at offshore control point = 3.7 m, T=13.3 sec, from 262°.

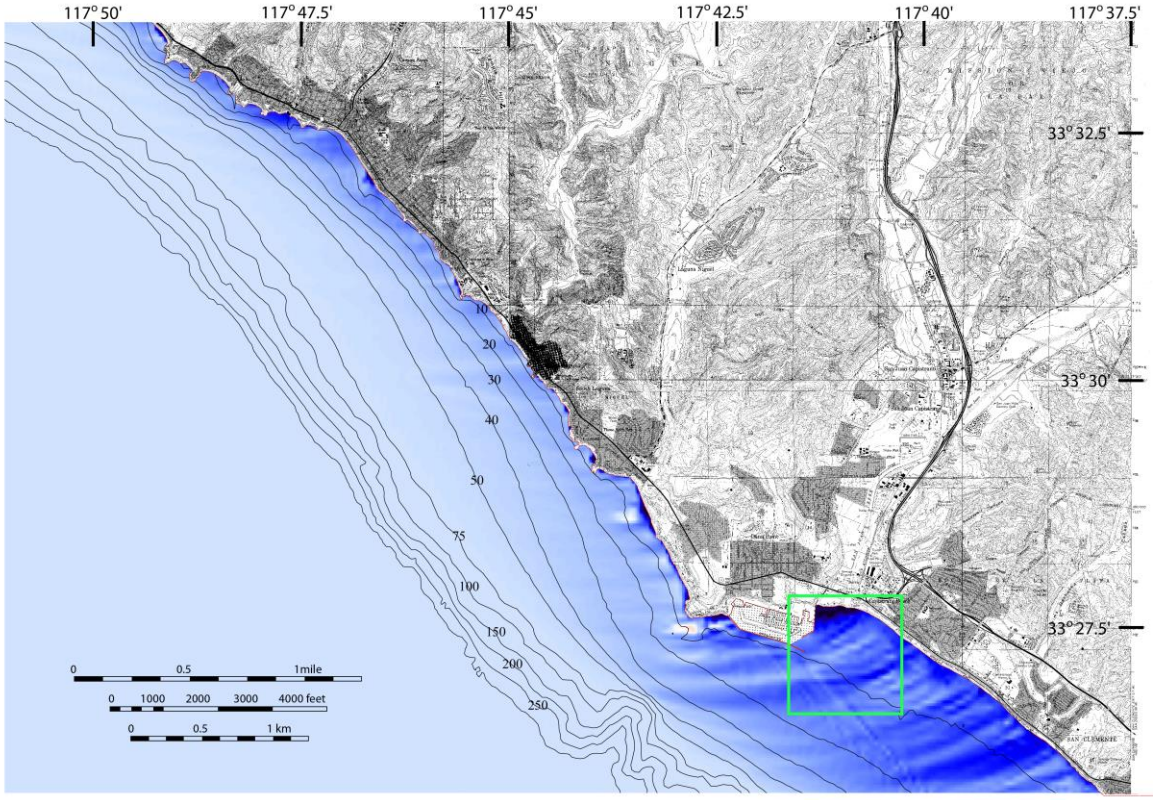

Figure . Dana Point wave refraction for the 4 April 1999 storm
Wave height at offshore control point = 3.9 m, T=9.1 sec, from 276°.

Appendix **C**:

Project Cost Estimates

Doheny Desalination Project Cost Estimates

Alternative 1

Item		Description	Unit	Quantity (GHD)	Unit Price	Item Total
A	**Mobilization & Demobilization**		LS	1	$161,900	$161,900
					Subtotal Construction Cost A	$161,900
B	**Site Improvements**					
	1	Clearing & Grubbing	LS	1	$20,000	$20,000
	2	Grading and Excavation	LS	1	$60,000	$60,000
	3	Earthwork (imported)	CY	63100	$25	$1,577,500
	4	Remove Existing Storm Drain Inlet	EA	1	$1,400	$1,400
	5	Cap Existing Manhole	EA	1	$1,200	$1,200
	6	Install Junction Structure (W6' x L6' x H20')	LS	1	$30,000	$30,000
	7	Shoring Equipment (6 weeks)	LS	1	$1,800	$1,800
	8	Miscellaneous Site Work	LS	1	$50,000	$50,000
					Subtotal Construction Cost B	$1,741,900
C	**Detention Basin**					
	1	Construct Stormwater Detention Basin (0.5 acres X 2' deep)	LS	1	$87,000	$87,000
	2	Miscellaneous Basin Work	LS	1	$20,000	$20,000
					Subtotal Construction Cost C	$107,000
					Subtotal Construction Cost (A+B+C)	$2,010,800
					15% Contingency	$301,620
					Total Construction Cost (A+B+C)	$2,312,420
		Grand Total				**$2,312,420**

Note:

1. Mobilization and demobilization is estimated to be 7% of construction cost.

Doheny Desalination Project Cost Estimates

Alternative 2

Item		Description	Unit	Quantity (GHD)	Unit Price	Item Total
A	**Mobilization & Demobilization**		LS	1	$264,500	$264,500
					Subtotal Construction Cost A	$264,500
B	**Site Improvements**					
	1	Clearing & Grubbing	LS	1	$20,000	$20,000
	2	Grading and Excavation	LS	1	$120,000	$120,000
	3	Earthwork (imported)	CY	27100	$25	$677,500
	4	Demolish Existing Channel Wall with Footing	LF	255	$600	$153,000
	6	Miscellaneous Site Work	LS	1	$50,000	$50,000
					Subtotal Construction Cost B	$1,020,500
C	**Sheet Pile Wall**					
	1	Furnish and Install Sheet Piles	LF	500	$4,000	$2,000,000
					Subtotal Construction Cost C	$2,000,000
					Subtotal Construction Cost (A+B+C)	$3,285,000
					15% Contingency	$492,750
					Total Construction Cost (A+B+C)	$3,777,750
		Grand Total				**$3,777,750**

Note:

1. Mobilization and demobilization is estimated to be 7% of construction cost.

Doheny Desalination Project Cost Estimates

Alternative 3

Item		Description	Unit	Quantity (GHD)	Unit Price	Item Total
A	**Mobilization & Demobilization**		LS	1	$277,500	$277,500
					Subtotal Construction Cost A	$277,500
B	**Site Improvements**					
	1	Clearing & Grubbing	LS	1	$20,000	$20,000
	2	Grading and Excavation	LS	1	$120,000	$120,000
	3	Earthwork (imported)	CY	27100	$25	$677,500
	4	Remove Existing Storm Drain Inlet	EA	1	$1,400	$1,400
	5	Cap Existing Manhole	EA	1	$1,200	$1,200
	6	Install Junction Structure (W6' x L6' x H20')	LS	1	$30,000	$30,000
	7	Shoring Equipment(6 weeks)	LS	1	$1,800	$1,800
	8	Demolish Existing Channel Wall with Footing	LF	255	$600	$153,000
	9	Gravel Bag Berm	LF	1050	$5	$5,250
	11	Miscellaneous Site Work	LS	1	$50,000	$50,000
					Subtotal Construction Cost B	$1,060,150
C	**Detention Basin**					
	1	Construct Stormwater Detention Basin (0.5 acres X 2' deep)	LS	1	$87,000	$87,000
	2	Miscellaneous Basin Work	LS	1	$20,000	$20,000
					Subtotal Construction Cost C	$107,000
D	**Sheet Pile Wall**					
	1	Furnish and Install Sheet Piles	LF	500	$4,000	$2,000,000
					Subtotal Construction Cost C	$2,000,000
					Subtotal Construction Cost (A+B+C+D)	$3,444,650
					15% Contingency	$516,698
					Total Construction Cost (A+B+C+D)	$3,961,348
		Grand Total				**$3,961,348**

Note:

1. Mobilization and demobilization is estimated to be 7% of construction cost.

Introduction

This Hydrology and Water Quality Technical Study is a technical engineering study/evaluation and program-level document supporting the Doheny Desalination Plant Specific Plan Environmental Impact Report (EIR). It presents issues related to drainage, surface hydrology and water quality. The level of analysis prepared is compatible with the level of planning information available.

All assessments and technical analysis in this report comply with the local drainage policies and requirements of the Orange County Hydrology Manual.

Project Description

The project site is located in Dana Point, San Clemente, California approximately 1,200 linear feet from the mouth of San Juan Creek and the Pacific Ocean, along the southeast bank of the creek. The overall site is approximately 10 acres and is separated from San Juan Creek by an earthen berm levee. See Figure 1 - Site Vicinity Map.

Definition of Level of Significance

The purpose of this technical evaluation is to determine the impact of the proposed project on surface water drainage and storm water quality within the City of San Clemente. If the analysis indicates that it will result in significant impacts, then appropriate mitigation measures will be identified to minimize the potential project's impacts to a less significant level.

Federal, State, and local drainage laws and regulations govern the evaluation of impacts to surface water drainage. For this evaluation, the impacts to surface water drainage would be considered if any of the following were to occur:

- Violation of any water quality standards or waste discharge requirements.
- Interference with groundwater recharge such that there would be a net deficit in aquifer volume or lowering of the local groundwater table level.
- Alteration of the drainage patterns of the site which would result in substantial erosion, siltation, or runoff that would result in increased flooding on-, or off-site.
- Creation of runoff that would exceed the capacity of existing or planned storm water drainage systems or provide substantial additional sources of polluted runoff.
- Substantial degradation of water quality.
- The placing of housing within a 100-year flood hazard area structures which would impede or redirect flood flows.

Existing Condition

The purpose of the existing condition evaluation is to establish a baseline to compare the potential post-project impacts to the project site. The existing project site is generally made up of sub-divided parcels of land that is currently being used for storage by separate tenants. The majority of the current land use is decomposed granite, with very little impervious area. See Figure 2 – Project Area. The site generally drains from northwest to the south east and includes two drainage courses to convey on-site flows to San Juan Creek. Flows from off-site areas to the north are conveyed in a vegetated trapezoidal channel. The site drain towards an existing basin located at the south western end of the project site. Runoff is conveyed through the basin and into San Juan Creek. The surrounding are consists of commercial retail. There is a residential subdivision and public park on the western bank of the creek.

The area north of the project site in made up of an earthen berm and sub-divided parcels. This area is intended to be the construction staging area for the project. Fill from the large earthen berm will be used to balance the project site.

The baseline conditions investigation include land use, hydrology, floodplain mapping, and surface water quality. Please see the Flood Inundation Report for hydrology, and floodplain mapping. This report will address water quality requirements.

Proposed Impacts on Stormwater Quality

The Doheney Desalination Plant Plan will include approximately 4.2 acres of open space, and 5.8 acres of impervious area. The development of the proposed Doheney Desalination Project will affect project site imperviousness, resulting in an impact to stormwater quality both during and post-construction. Implementation of construction and post-construction Best Management Practices (BMPs), including the preparation of a Water Quality Management Plan (WQMP) or equivalent document, a Notice of Intent, and a Stormwater Pollution Prevention Plan (SWPPP), would aim to reduce water quality impacts to required levels.

Post-Construction Mitigation

The long-term operation and maintenance of the proposed project will be a source of pollutants, including suspended solids/sediment, nutrients, heavy metals, pathogens (bacteria/virus), pesticides, oil and grease, toxic organic compounds, trash and debris, and household hazardous wastes. Preparation of a Project WQMP, when implemented, will minimize the effects of urbanization on site hydrology, runoff flow rates or velocities and pollutant loads.

The South Orange County Model Water Quality Management Plan (Model WQMP) was developed to aid project proponents in developing a Project WQMP for Priority Development Projects in accordance with requirements set forth in the South Orange County Municipal Separate Storm Sewer System (MS4) Permit R9-2009-0002. It is used in conjunction with the Orange County Technical Guidance Document (TGD) to identify, size, and implement BMPs to address stormwater runoff quality and quantity.

Because it meets several of the criteria listed in the Model WQMP for creation of impervious area and location, the project is classified as a Priority Development Project (PDP). The specific criteria met by the project that place it in this category are 1) projects that disturb greater than 1 acre of area, 2) redevelopment projects that create or replace 5,000 sq. ft. of impervious area on an already developed site, and 3) projects directly adjacent to and directly discharging to an Environmentally Sensitive Area (ESA). The Site directly discharges stormwater runoff to San Juan Creek which is classified as an ESA due to impairments that qualify it for placement on California's Clean Water Act Section 303(d) List. The listed impairments are dichlorodiphenyldichloroethylene, indicator bacteria, phosphorous, selenium, total nitrogen, and toxicity. As a PDP, the project is required to implement stormwater BMPs including site design, source control, treatment control, and hydromodification management BMPs as applicable and to the maximum extent practicable (MEP). The different types of BMPs are defined below along with an explanation of how they will be incorporated into the project design.

Site Design BMPs

Site Design practices include a wide range of potential practices that can be implemented to reduce the volume of stormwater runoff generated on a project site as well as improve the quality of runoff that leaves the site. Site Design is predominately "preventative" in nature as it consists of practices that reduce the amount of runoff and other impacts before, or immediately after, they occur. Examples of "preventative" aspects of Site Design include reduction of impervious area, preservation of drainage courses, and restoration of impacted soils. The following site design practices will be incorporated early in the preliminary design phase of the project.

- Minimize Impervious Area
    - Coverage of paving and roofs will be minimized by designing compact, taller structures, narrower and shorter drive aisles and sidewalks, and smaller parking areas
    - Runoff will be detained and retained throughout the site by interspersing landscaped areas and integrating small scale retention practices among the buildings and paving
- Maximize Natural Infiltration Capacity

    - Buildings and other proposed impervious areas will be designed so as to avoid the permeable Hydrologic Soil Group B soils on site
    - Channelization of stormwater flows will be avoided
- Disconnect Impervious Areas
    - Runoff from impervious areas will be directed into permeable areas such as turf, medians, parking islands, planter boxes, etc.
    - Vegetated swales will be used for stormwater conveyance instead of traditional pipes or hardened channels

Source Control BMPs

Source Control BMPs reduce the potential for stormwater runoff and pollutants from coming into contact with one another. Source Control BMPs are defined as any administrative action, design of a structural facility, usage of alternative materials, and operation, maintenance, inspection, and compliance of an area that aims to eliminate or reduce stormwater pollution. The following Source Control practices will be incorporated into the design of the project.

- Provide storm drain system stenciling and signage to discourage introduction of pollutants into stormwater runoff
- Design outdoor hazardous material storage area to reduce pollutant introduction
- Design trash enclosures to reduce pollutant introduction
- Use efficient irrigation systems and landscape design
- Education for property owners, occupants, and employees
- Implementation of spill contingency plans
- Common area litter control
- Sweeping of parking lots and drive areas
- BMP maintenance

LID/Treatment Control BMPs

LID/Treatment Control BMPs are engineered facilities that are designed to retain or treat stormwater runoff on the project site. The difference between LID and Treatment Control BMPs are that LID BMPs incorporate significant biological processes for pollutant removal while Treatment Control BMPs do not. In accordance with the Model WQMP and TGD, evaluation and implementation of LID/Treatment Control BMPs for the project shall use the following hierarchy of priority:

- Infiltration BMPs
- Harvest and use BMPs
- Evapotranspiration BMPs
- Biotreatment BMPs
- Treatment Control BMPs
- Gross Solids Removal BMPs

Hydromodification Management BMPs

Hydromodification Management is the mitigation of increased post-project runoff volumes and flow rates associated with urbanization or other land use changes. Increased post-construction runoff flow rates cause alteration of stream and river channels and is one of the leading sources of impairment in streams, lakes, estuaries, and other water bodies in the United States. According to data obtained from the Orange County Georesearch Tool, San Juan Creek in the vicinity of the project site is a stabilized channel that is not susceptible to hydromodification impacts. As such, hydromodification management BMPs are not required to be implemented on the project.

Construction-Phase Mitigation

During construction, the necessary soil excavation and compaction activities will impact the beneficial uses of downstream water bodies. The proposed project has the potential to produce typical pollutants such as nutrients, heavy metals, pesticides and herbicides, toxic chemicals related to construction and

cleaning, waste materials including wash water, paints, wood, paper, concrete, food containers, and sanitary wastes, fuel, and lubricants. Preparation of a SWPPP in accordance with the Construction General Permit 2009-009-DWQ (CGP) will serve to mitigate these impacts. Prior to construction, the CGP requires the following:

- Electronic submittal of the Permit Registration Documents (PRD) to the Regional Water Quality Control Board (RWQCB) at least 30 days before the start of construction, which includes submittal of a Notice of Intent (NOI), risk assessment, site map, SWPPP, annual fee, and a signed certification statement
- Preparation and implementation of a SWPPP
- Electronic submittal of a Notice of Termination (NOT) to the RWQCB upon completion of construction and stabilization of the site

Project Location
Pacific Ocean
El Toro
Lake Forest
Mission Viejo
Rancho Santa Margarita
Coto de Caza
Laguna Woods
Laguna Hills
Aliso Viejo
Laguna Niguel
Laguna Beach
Ladera Ranch
San Juan Capistrano
Dana Point
Capistrano Beach
San Clemente
San Joaquin Hills
Crystal Cove State Park
Laguna Coast Wilderness Park
Aliso & Wood Canyons Wilderness Pk
Caspers Wilderness Park
San Joaquin Hills Transportation Corridor
Pacific Coast Hwy
N Coast Hwy
S Coast Hwy
N El Camino Real
S El Camino Real
Ortega Hwy
San Diego Fwy
Crown Valley Pkwy
Oso Pkwy
Alicia Pkwy
Antonio Pkwy
Camino Capistrano
Stonehill Dr
Selva Rd
Camino Vera Cruz
E Avenida Pico
Moulton Pkwy
Aliso Creek Rd
La Paz Rd
Felipe Rd
Olympiad Rd
Golden Lantern
Niguel Rd
Del Obispo St
Cabot Rd
Rancho Viejo Rd
Paper Size ANSI A
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Map Projection: Mercator Auxiliary Sphere
Horizontal Datum: WGS 1984
Grid: WGS 1984 Web Mercator Auxiliary Sphere
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GHD
SOUTH COAST WATER DISTRICT
DOHENY OCEAN DESALINATION PROJECT
VICINITY MAP
Project No. 11125157
Revision No. -
Date MAY 2018
FIGURE 1
G:\111\11125157- SCWD TO #8B\08-GIS\Maps\Deliverables\Stormwater\Figure 1 - Vicinity Map.mxd
Print date: 01 May 2018 - 13:30
Data source: Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), NGCC, © OpenStreetMap contributors, and the GIS User Community. Created by: pwleung

Paper Size ANSI A

0 250 500 750 1,000

Feet

Map Projection: Mercator Auxiliary Sphere
Horizontal Datum: WGS 1984
Grid: WGS 1984 Web Mercator Auxiliary Sphere

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GHD

SOUTH COAST WATER DISTRICT

DOHENY OCEAN DESALINATION PROJECT

PROJECT AREA

Project No. **11125157**
Revision No. **-**
Date **MAY 2018**

FIGURE 2

G:\111\11125157- SCWD TO #8B\08-GIS\Maps\Deliverables\Stormwater\Figure 2 - Project Area.mxd
Print date: 01 May 2018 - 13:30

Data source: Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community. Created by: pwleung

Supplemental Analysis – Alternative 1a

This document summarizes a supplemental analysis to amend the Local Hazard Conditions and Drainage Study for the Doheny Desalination Project (GHD 2017). Specifically, Alternative 1, the recommended alternative in the study, is modified to exclude the proposed fill area at the south edge of the project site, adjacent to the Highway 1 northbound bridge along the Caltrans right of way. The following outlines the modified Alternative 1, identified as Alternative 1a, with a supplemental discussion of its impacts to flood inundation at the project site and the adjacent areas.

3.5.1a Alternative 1a

Alternative 1a was formulated to be a standalone option that would mitigate flooding at the project site regardless of future flood improvements along San Juan Creek. This alternative includes re-grading the project site to 28.2 feet elevation, which is approximately 1 foot higher than the 100-year flood elevation with creek overtopping. The limits of grading for this alternative is identical to Alternative 1 as shown in Figure 3.10 in GHD 2017, except the southern grading limit is shifted north to align with the South Coast Water District property boundary as shown in Figure 3.10a. The modification of the southern grading limit is to ensure the proposed grading does not encroach into the existing Caltrans right of way.

This alternative also includes a detention basin to mitigate additional runoff generated from development of the project site. The detention basin was sized to be 0.5 acres and 3 feet deep. Lastly, this alternative includes the capping of the existing DI adjacent to the 54" RCP pipe outfall at San Juan Creek, and relocating the DI eastward in the project site. Otherwise, overtopping flow from the creek could enter the existing DI, backflow up the pipe and provide a flow path to inundate areas east of the project site. See Figure 3.11a for the flood inundation map for this alternative, and Figure 3.12a for the change in flood elevations as compared to the base scenario.

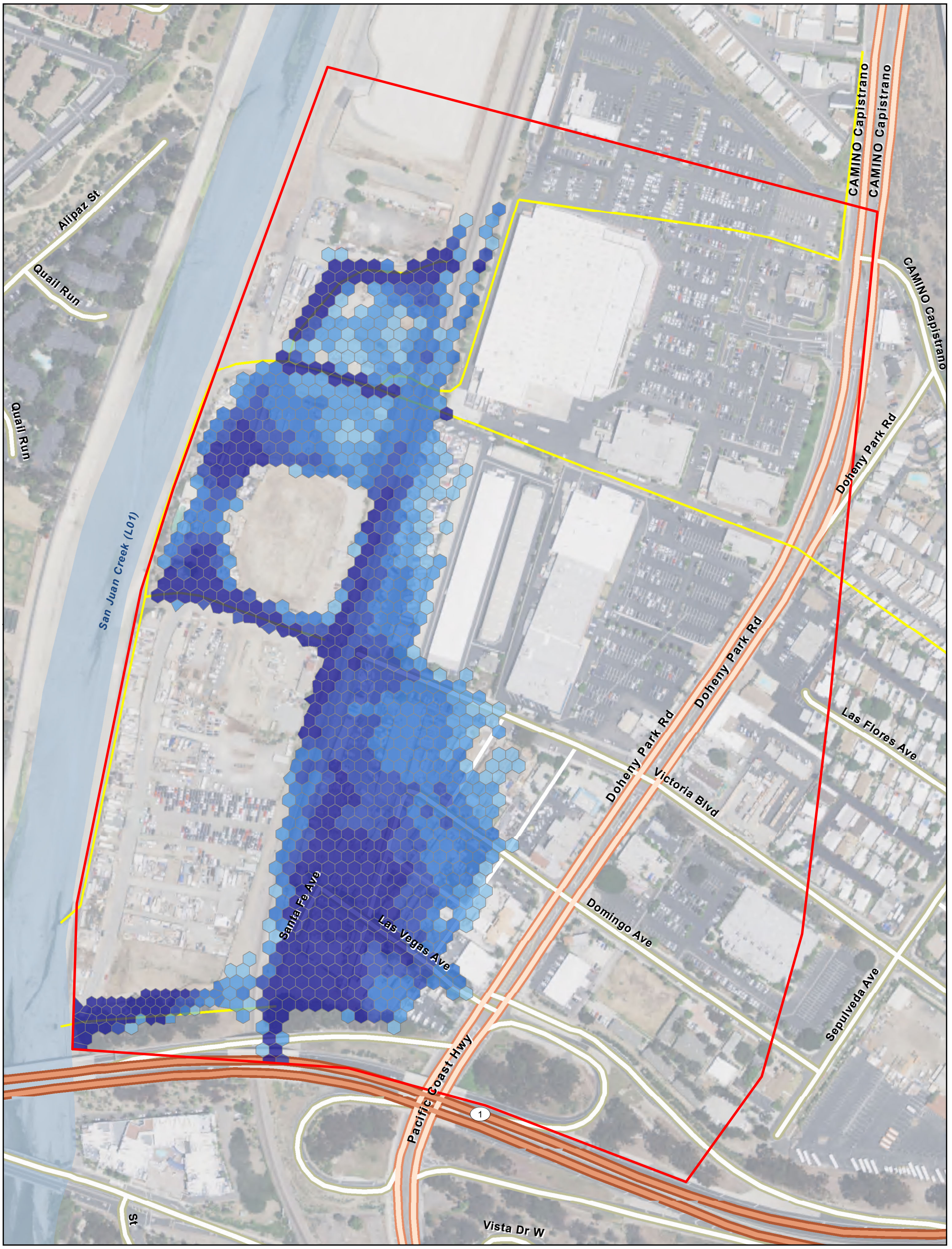
Raising the ground elevation reduces both the extent and depth of flooding. As shown in Figure 3.12a, the average flooding depth reduction in areas surrounding the project site is approximately one to three feet. Raising the ground elevation essentially acts as a flood barrier from the creek, which blocks overtopping flows from the creek to reach the project site and upstream watershed. Note that since the proposed site regrade does not include the area between the southern boundary of the project site property and the Highway 1 northbound bridge, this area provides a flood path for overtopping creek flow to inundate upstream watershed area at the east of the project site.

However, as shown in Figure 3.12a, it does not increase the level of flood inundation at the upstream watershed area at the east of the project site, as compared to the existing condition.

Table 3.10a summarizes the improvement work for Alternative 1a.

Table 3.10a Summary of Work for Alternative 1a

Alternative	Improvements
1a	• Raise project site to 28.2 feet • Demolish existing DI and cap, install new DI • Install detention basin (0.5 acres, 3 feet deep)



Paper Size ANSI B

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Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane California VI FIPS 0406 Feet

N

South Coast Water District

Doheny Desalination Project Drainage Analysis

Flood Inundation Map, 100-Year Event, Future Condition, Alternative 1a

Job Number	11125157
Revision	A
Date	12 Apr 2018

Figure 3.11a

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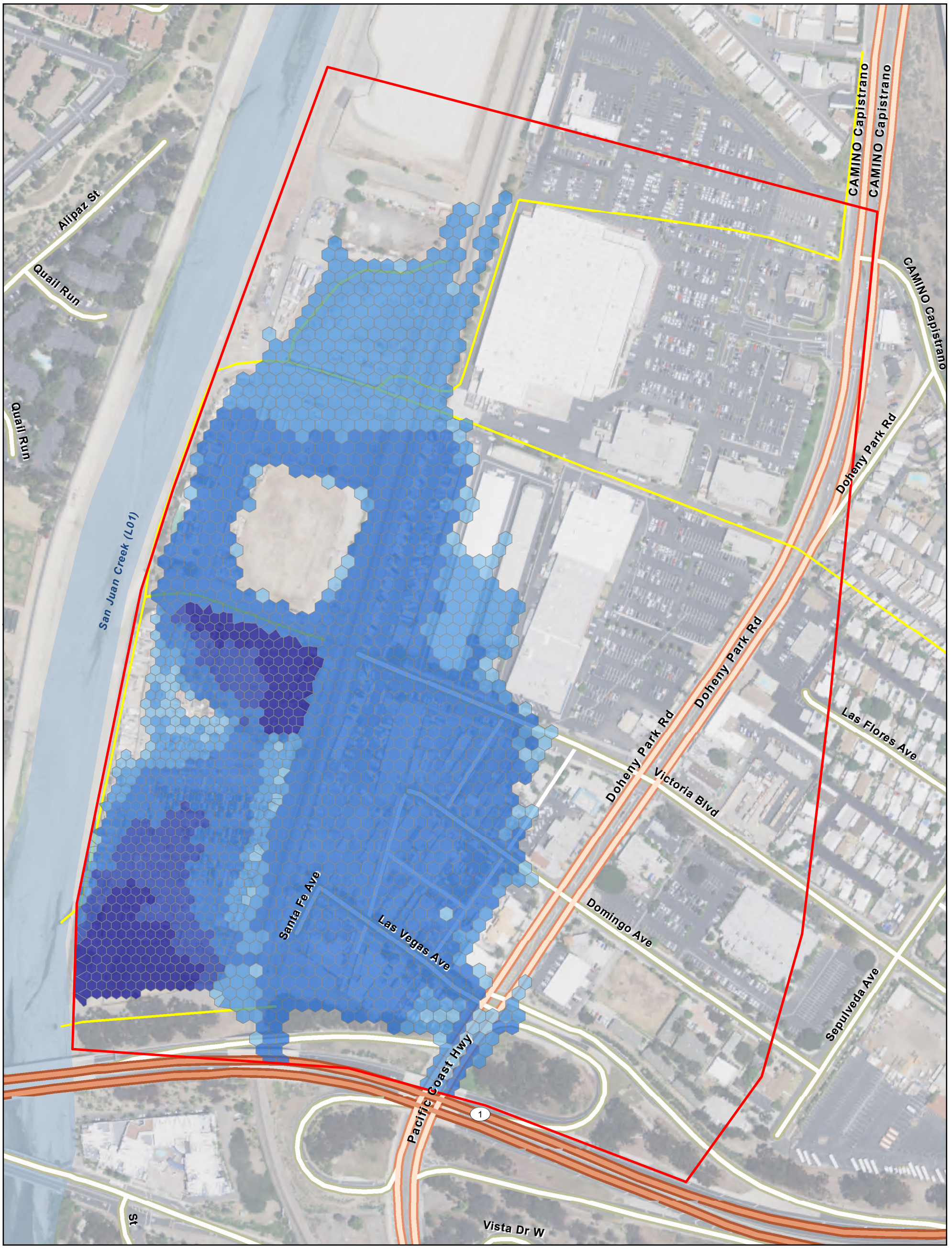
Data source: Aerial: NAIP, 2016; ESRI Street Map; GHD: elevation model. Created by:jclark2



Storm Water Conduits (in model) Two-Dimensional Model Boundary

Reduction in Maximum Flood Depth

2-6" 6-12" 1-2' 2-3' >3'

Paper Size ANSI B

0 100 200 300

Feet

Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane California VI FIPS 0406 Feet

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South Coast Water District
Doheny Desalination Project Drainage Analysis

Change in Flood Inundation, Existing Condition vs Alternative 1a

Job Number	11125157
Revision	A
Date	12 Apr 2018

Figure 3.12a

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Data source: Aerial: NAIP, 2016; ESRI Street Map; GHD: elevation model. Created by:jclark2