4.0 Final EIR Appendices

4.1 Draft EIR Distribution

4.2 Final EIR Technical Analyses

4.2.1 Coastal Hazards Analysis for Final EIR

4.2.2 Brine Discharge Analysis for Final EIR

4.2.3 Hydrogeologic Analyses

4.2.3.1 Groundwater Modeling for Final EIR

4.2.3.2 San Juan Creek Lagoon Technical Memo

4.2.4 Local Hazard and Drainage Calculations for Final EIR

4.2.5 Marine Biology Technical Memos

4.2.5.1 Diffuser Entrainment Memo for Final EIR

4.2.5.2 Brine Discharge Memo for Final EIR

Attachment A: Comment Letter O1 Exhibits

- *Exhibit D – Water Well Standards*
- *Exhibit E – Water Well Standards*
- *Exhibit F – IDA Technical Paper (Dennis Williams, 2015)*
- *Exhibit G – Extended Pumping and Pilot Test (MWDOC, 2014)*
- *Exhibit H – CalEEMod User Manual (2017)*

Section 4.1

Draft EIR Distribution

DOHENY OCEAN DESALINATION PROJECT

Notice of Availability and Public Meeting Notice

Affidavit of Distribution

Kimley-Horn and Associates, Inc.
3880 Lemon Street, Suite 420
Riverside, CA 92501

Table of Contents

Affidavit of Distribution

Kimley»Horn

AFFIDAVIT OF MAILING

Date: May 23, 2018

Subject: Doheny Ocean Desalination Project NOA and Public Meeting Notice

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AFFIDAVIT OF POSTING

I, Amanda McCallum, do hereby certify that a copy of the attached Notice of Availability and Public Meeting Notice was posted at the City of Dana Point Public Library, 33841 Niguel Road, Dana Point, CA 92629, the South Coast Water District, 31592 West Street, Laguna Beach, CA 92651, and with the County Clerks for Orange, Los Angeles, San Diego, San Bernardino, and Riverside Counties, May 23, 2018. I declare under penalty of perjury that the foregoing is true and correct.

Amanda McCallum
Kimley-Horn and Associates

AFFIDAVIT OF MAILING

I, Amanda McCallum, do hereby certify that a copy of the attached Notice of Availability and Public Meeting Notice was mailed via USPS or sent via FedEx to each and every person on the attached distribution lists on May 23, 2018. Copies of the NOA distribution lists are attached. FedEx delivery receipts are on file. I declare under penalty of perjury that the foregoing is true and correct.

Amanda McCallum
Kimley-Horn and Associates

AFFIDAVIT OF NEWSPAPER PUBLICATION

I, Amanda McCallum, do hereby certify that a copy of the attached Notice of Availability and Public Meeting Notice was published in the OC Register on May 24, 2018 and the Dana Point Times on May 25, 2018. I declare under penalty of perjury that the foregoing is true and correct.

Amanda McCallum
Kimley-Horn and Associates

NOA and Public Meeting Notice Posting

- Orange County Clerk Filing Copy

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NOTICE OF AVAILABILITY OF A DRAFT EIR & PUBLIC MEETING NOTICE

SCH# 2016031038

POSTED
MAY 22 2018
ORANGE COUNTY CLERK-RECORDER DEPARTMENT
BY: HB DEPUTY

Date: May 23, 2018

To: Reviewing Agencies, Organizations, and Interested Parties

Lead Agency: South Coast Water District

Subject: ***Notice of Availability & Public Meeting Notice***

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State Clearinghouse No. 2016031038

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POSTED
MAY 22 2018
ORANGE COUNTY CLERK-RECORDER DEPARTMENT
BY: HB DEPUTY

NOTICE OF AVAILABILITY OF A DRAFT EIR & PUBLIC MEETING NOTICE

SCH# 2016031038

Date: May 23, 2018

To: Reviewing Agencies, Organizations, and Interested Parties

Lead Agency: South Coast Water District

Subject: ***Notice of Availability & Public Meeting Notice***

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FILED
Ernest J Dronenburg, Jr. Recorder County Clerk
MAY 22 2018
BY Carmelo Mendoza
DEPUTY

FILED IN THE OFFICE OF THE COUNTY CLERK
San Diego County on MAY 22 2018
Posted MAY 22 2018 Removed
Returned to agency on
Deputy Carmelo Mendoza

NOTICE OF AVAILABILITY OF A DRAFT EIR & PUBLIC MEETING NOTICE

SCH# 2016031038

ORIGINAL FILED
MAY 22 2018
LOS ANGELES, COUNTY CLERK

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- **A concentrate (brine) disposal system** that would utilize the existing San Juan Creek Ocean Outfall (SJCOO), to return brine and treated process waste streams to the ocean with negligible impact on coastal and marine water quality. This would be achieved in part through blending in the outfall pipe with the existing wastewater stream from the J.B. Latham Wastewater Treatment Plant, and other regional treatment plants. Mixing desalination brine with existing wastewater treatment plant flow (a "comingled discharge") is the preferred method by state and federal regulators and is consistent with the State Board's Ocean Plan Amendment.
- **A product water storage tank and distribution system** that would feed into the District's local distribution system and, depending on plant capacity and District demands, other adjacent local and regional transmission pipelines that are located adjacent to the site. Desalinated product water from the Phase I Project could be conveyed entirely using existing District and local infrastructure with no off-site improvements other than a short connection to the District's existing local transmission lines.
- **All appurtenant facilities** (e.g. pump stations, valves and metering) as well as all construction, operation and maintenance activities associated with all Project facilities.
- **Offsite Electrical Transmission Facilities** provided by San Diego Gas & Electric Company (SDG&E). At this time, SDG&E has indicated that electrical service can be provided to the Phase I Project using existing facilities, with a short connection from the desalination site to underground electrical lines in Stonehill Drive.

SIGNIFICANT ENVIRONMENTAL IMPACTS:

The Draft EIR addresses Aesthetics, Air Quality, Biological Resources, Cultural Resources, Geology and Soils, Greenhouse Gas Emissions, Hazards and Hazardous Materials, Hydrology and Water Quality, Land Use and Planning, Noise, Public Services, Recreation, Transportation and Traffic, Tribal Cultural Resources and Utilities and Service Systems, as well as energy conservation, alternatives, potential growth-inducing impacts, and cumulative impacts. With implementation of various Project Design Features and EIR mitigation measures, the EIR has concluded that the Phase I Project (up to 5 MGD) would not have any "unavoidable significant impacts." The Regional Project, if pursued at a later date, could result in unavoidable significant impacts, although this is speculative at this time due to lack of Regional Project details.

CEQA also requires this NOA to specify if the Project site contains any listed toxic sites. The Project site does not contain sites identified as meeting the "Cortese List" requirement (Government Code Section 65962.5).

AGENCIES:

The District requests each Responsible and Trustee agency review the Draft EIR relevant to the agency's statutory responsibilities in connection with the proposed Project, in a manner consistent with California Code of Regulations, Title 14, Section 15087. Each agency may use the EIR prepared by the District when considering any permits that the agency must issue, or other approvals for the Project.

PUBLIC REVIEW PERIOD:

The Draft EIR is available for public review for a period of 60 days. In accordance with CEQA, should you have any comments, please provide written comments on the Draft EIR within the 60-day period between **May 23, 2018 to July 23, 2018.**

LOCATIONS WHERE DRAFT EIR IS AVAILABLE FOR PUBLIC REVIEW

An electronic PDF of the Draft EIR is available for download on the District's Project website at www.scwd.org/desal. In addition, during the 60-day public review period, hard copies of the Draft EIR and the documents referenced in the EIR will be available at the following locations:

- South Coast Water District Offices, address noted below
- Orange County Public Library, Dana Point Branch, 33841 Niguel Rd. Dana Point, CA 92629

PUBLIC COMMENTS:

The District requests your careful review and consideration of the Draft EIR, and invites ***written comments*** from interested agencies, persons, and organizations regarding environmental issues identified in the Draft EIR. Please indicate a contact person for your agency or organization. You may also provide oral or written comments in person at the ***Public Meeting*** noted below. Comments in response to this notice must be submitted to the District through close of business (5:00 PM) on **July 23, 2018**.

LEAD AGENCY CONTACT:

All comments should be submitted in writing to:

South Coast Water District
Attn: Mr. Rick Shintaku, PE – Acting General Manager, Chief Engineer
31592 West Street, Laguna Beach, CA 92651
(949) 499-4555

PUBLIC MEETING:

The District will conduct a public meeting in order to receive public comments on the Draft EIR. The meeting will be held at the following location, date and time:

Tuesday, June 26, 2018
6:00 p.m.
(ending no later than 8:00 p.m. or when discussion concludes)
Capistrano Unified School District
CUSD Education Center
33122 Valle Road, San Juan Capistrano, CA 92675
Phone: (949) 234-9200

Special Accommodations. Should you require special accommodations at the public meeting, such as for the hearing impaired or an English translator, please contact South Coast Water District no later than **June 19, 2018** (see contact information above).

SOUTH COAST WATER DISTRICT
Partnering With The Community

Notic

NOTICE OF AVAILABILITY OF A DRAFT EIR & PUBLIC MEETING NOTICE

SCH# 2016031038

Date: May 23, 2018

To: Reviewing Agencies, Organizations, and Interested Parties

Lead Agency: South Coast Water District

Subject: ***Notice of Availability & Public Meeting Notice***

DOHENY OCEAN DESALINATION PROJECT DRAFT ENVIRONMENTAL IMPACT REPORT
State Clearinghouse No. 2016031038

The South Coast Water District (District) has prepared an Environmental Impact Report (EIR) pursuant to the California Public Resources Code and the California Environmental Quality Act (CEQA) to evaluate the environmental effects associated with the proposed Doheny Ocean Desalination Project (Project). This Notice of Availability (NOA) has been issued to notify interested parties that a Draft EIR is publicly available for review and comment. The District is requesting comments on the Draft EIR from Responsible and Trustee agencies, interested public agencies, organizations, and the general public (pursuant to CEQA Guidelines §15087).

PROJECT SUMMARY:

The Draft EIR assesses the potential environmental effects of implementing a proposed ocean water desalination facility located in Dana Point, including subsurface intake wells proposed at Doheny State Beach and Capistrano Beach, and various conveyance lines connecting the intake and discharge facilities to existing District property located approximately ½ mile inland, adjacent to San Juan Creek. The District is only proposing to pursue permits and approvals for the initial Phase I Project, which would provide up to 5 million-gallons-per-day (MGD) of potable water, and is therefore addressed at a "project level" of CEQA review. The EIR also evaluates a potential future Regional Project of up to 15 MGD, at a programmatic level, since specific Regional Project partners, financing, and facilities have yet to be defined. Specific Project component descriptions are provided below and on the District's website at: **www.scwd.org/desal.**

- **A subsurface water intake system** consisting of subsurface slant wells that draw ocean water from offshore subsurface alluvial material (located below the ocean floor), while providing natural sand bed filtration and eliminating the entrainment and impingement of marine biota. This subsurface intake system is the recommended approach by state and federal regulators, and is consistent with the State Water Resource Control Board's (State Board or SWRCB) recently adopted Ocean Plan Amendment. The slant wells would be located and fully buried near the beach, in a study area encompassing Doheny State Beach and Capistrano Beach Park.

- **A raw (ocean) water conveyance pipeline** that would deliver the subsurface intake system's ocean water to the desalination facility site.
- **A desalination facility** that would receive ocean feedwater at approximately 10 to 30 MGD, with a recovery rate of ~50% resulting in up to 5 to 15 MGD of potable drinking water (for the Phase I and Regional Project, respectively). The proposed desalination facility is located on the District's existing San Juan Creek Property site, on an industrial site located away from the beach but in close proximity to the subsurface intake wells. This facility siting is also consistent with state and federal regulator preference to minimize desalination facilities on the coast while being close enough to avoid lengthy raw water and brine conveyance pipelines. The desalination facility includes a variety of typical desalination process equipment and appurtenant facilities, such as pretreatment, seawater reverse osmosis (SWRO) membranes, an energy recovery system, post-treatment conditioning, solids handling and disposal, product water storage, electrical equipment, staff facilities, and connections to off-site brine disposal, sanitary sewer, and product water conveyance facilities. It is assumed there will be a utility power connection required; however, the District is also evaluating the feasibility of supplementing or replacing that supply with an alternative energy source. The desalination facility will include solar photovoltaic panels on flat rooftops where feasible. Other alternative energy sources being evaluated include natural-gas turbines and fuel cells to maximize efficiency and minimize energy costs.
- **A concentrate (brine) disposal system** that would utilize the existing San Juan Creek Ocean Outfall (SJCOO), to return brine and treated process waste streams to the ocean with negligible impact on coastal and marine water quality. This would be achieved in part through blending in the outfall pipe with the existing wastewater stream from the J.B. Latham Wastewater Treatment Plant, and other regional treatment plants. Mixing desalination brine with existing wastewater treatment plant flow (a "comingled discharge") is the preferred method by state and federal regulators and is consistent with the State Board's Ocean Plan Amendment.
- **A product water storage tank and distribution system** that would feed into the District's local distribution system and, depending on plant capacity and District demands, other adjacent local and regional transmission pipelines that are located adjacent to the site. Desalinated product water from the Phase I Project could be conveyed entirely using existing District and local infrastructure with no off-site improvements other than a short connection to the District's existing local transmission lines.
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- **Offsite Electrical Transmission Facilities** provided by San Diego Gas & Electric Company (SDG&E). At this time, SDG&E has indicated that electrical service can be provided to the Phase I Project using existing facilities, with a short connection from the desalination site to underground electrical lines in Stonehill Drive.

SIGNIFICANT ENVIRONMENTAL IMPACTS:

The Draft EIR addresses Aesthetics, Air Quality, Biological Resources, Cultural Resources, Geology and Soils, Greenhouse Gas Emissions, Hazards and Hazardous Materials, Hydrology and Water Quality, Land Use and Planning, Noise, Public Services, Recreation, Transportation and Traffic, Tribal Cultural Resources and Utilities and Service Systems, as well as energy conservation, alternatives, potential growth-inducing impacts, and cumulative impacts. With implementation of various Project Design Features and EIR mitigation measures, the EIR has concluded that the Phase I Project (up to 5 MGD) would not have any "unavoidable significant impacts." The Regional Project, if pursued at a later date, could result in unavoidable significant impacts, although this is speculative at this time due to lack of Regional Project details.

CEQA also requires this NOA to specify if the Project site contains any listed toxic sites. The Project site does not contain sites identified as meeting the "Cortese List" requirement (Government Code Section 65962.5).

AGENCIES:

The District requests each Responsible and Trustee agency review the Draft EIR relevant to the agency's statutory responsibilities in connection with the proposed Project, in a manner consistent with California Code of Regulations, Title 14, Section 15087. Each agency may use the EIR prepared by the District when considering any permits that the agency must issue, or other approvals for the Project.

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- Orange County Public Library, Dana Point Branch, 33841 Niguel Rd. Dana Point, CA 92629

PUBLIC COMMENTS:

The District requests your careful review and consideration of the Draft EIR, and invites ***written comments*** from interested agencies, persons, and organizations regarding environmental issues identified in the Draft EIR. Please indicate a contact person for your agency or organization. You may also provide oral or written comments in person at the ***Public Meeting*** noted below. Comments in response to this notice must be submitted to the District through close of business (5:00 PM) on **July 23, 2018.**

LEAD AGENCY CONTACT:

All comments should be submitted in writing to:

South Coast Water District
Attn: Mr. Rick Shintaku, PE – Acting General Manager, Chief Engineer
31592 West Street, Laguna Beach, CA 92651
(949) 499-4555

PUBLIC MEETING:

The District will conduct a public meeting in order to receive public comments on the Draft EIR. The meeting will be held at the following location, date and time:

Tuesday, June 26, 2018
6:00 p.m.
(ending no later than 8:00 p.m. or when discussion concludes)
Capistrano Unified School District
CUSD Education Center
33122 Valle Road, San Juan Capistrano, CA 92675
Phone: (949) 234-9200

Special Accommodations. Should you require special accommodations at the public meeting, such as for the hearing impaired or an English translator, please contact South Coast Water District no later than **June 19, 2018** (see contact information above).

Distribution Lists

- Government Agencies

- Non-Government Organizations

- Interested Parties

Government Agencies

Organization	Attention	Address	Phone Number	Email
FEDERAL				
NOAA National Marine Fisheries Services	Bryant Chesney	501 West Ocean Blvd. Suite 4200 Long Beach, CA 90802	562-980-4000 562-980-4197	bryant.chesney@noaa.gov Eric.Chavez@noaa.gov
NOAA National Marine Fisheries Services	Anthony P. Spina	501 West Ocean Blvd. Suite 4200 Long Beach, CA 90802		
U.S. Army Corps of Engineers	Regulatory Permitting-Orange County	U.S. Army Corps of Engineers 915 Wilshire Boulevard Suite 930 LOS ANGELES, CA 90017	213-452-3417	
U.S. Army Corps of Engineers	Therese O'Rourke-Bradford ACOE, Carlsbad Field Office	5900 La Place Court, Carlsbad, California, 92008		therese.o.bradford@usace.army.mil
U.S. Army Corps of Engineers	Cori Farrar ACOE Carlsbad Field Office	5900 La Place Court, Carlsbad, California, 92008		Corice.J.Farrar@usace.army.mil
U.S. Fish & Wildlife Service, Region 8	Environmental Services, April Evenas	Federal Building 2800 Cottage Way, Room W-2606 Sacramento, CA 95825	916-414-6464	april_evans@usace.army.mil stephen.M.Estes@usace.army.mil
Federal Emergency Mangement Agency	CEQA Review	1111 Broadway, Suite 1200, Oakland, CA 94607-4052		
STATE				
California Coastal Commission	Tom Luster	45 Fremont St #1900, San Francisco, CA 94105	(415) 904-5248	tluster@coastal.ca.gov
California Coastal Commission, South Coast District Office	Karl Schwing, South Coast District Manager	South Coast Area Office 200 Oceangate, Suite 1000 Long Beach, California 90802-4302	562-590-5071	Karl.Schwing@coastal.ca.gov
California Coastal Commission, South Coast District Office	Deborah Lee	South Coast Area Office 200 Oceangate, Suite 1000 Long Beach, California 90802-4302		DLee@coastal.ca.gov
California Department of Fish & Wildlife	Bill Paznokas	4949 Viewridge Avenue San Diego, CA 92123	(858) 467-4218	William.Paznokas@wildlife.ca.gov WPaznokas@dfg.ca.gov
California Department of Fish & Wildlife	Jennifer Edwards	3883 Ruffin Rd. San Diego, CA 92123	858-467-2717	jennifer.edwards@wildlife.ca.gov
California Department of Fish & Wildlife	Jennifer Turner	3883 Ruffin Rd. San Diego, CA 92123		Jennifer.Turner@wildlife.ca.gov
California Department of Fish & Wildlife	Loni Adams, Marine Environmental Scientist	3883 Ruffin Rd. San Diego, CA 92123	858-627-3985	loni.adams@wildlife.ca.gov
California Department of Fish & Wildlife, South Coast Region	Ed Pert, Regional Manager	3883 Ruffin Rd. San Diego, CA 92123	858-467-4201	epert@dfg.ca.gov
California Department of Parks and Recreation, Orange Coast District	Steve Scott	3030 Avenida del Presidente San Clemente, CA 92672	949-492-0802	sscott@parks.ca.gov
California Department of Transportation (Caltrans), Division 12	Maureen El Harake, Branch Chief	3347 Michelson Dr. Suite 100 Irvine, CA 92612	949-724-2000	Maureen.el.harake@dot.ca.gov
	Yatman Kwan			
California Department of Water Resources	Richard Mills, Section Chief, Water Recycling and Desalination	901 P Street, Room 313A , Third Floor. Sacramento, CA 94236-0001	(916) 651-0715	richard.mills@water.ca.gov
California Natural Resources Agency	Amy Vierra, Deputy Director, Ocean Protection Council	1416 Ninth Street, Suite 1311 Sacramento, CA 95814		Amy.Vierra@resources.ca.gov
California Public Utilities Commission	Chi Cheung To, P.E. Utilities Engineer	505 Van Ness Ave San Francisco, CA 94102	213.576.5766	cct@cpuc.ca.gov
California State Parks	Rich Haydon, State Park Superintendent	3030 Avenida del Presidente San Clemente, CA 92672	949-366-4895	rhaydon@parks.ca.gov; rich.haydon@parks.ca.gov
California State Parks, Orange County District	James Newland, Park and Recreation Specialist	3030 Avenida del Presidente San Clemente, CA 92672	949-607-9510	james.newlandd@parks.ca.gov
California Toxic Substances Control Department		5796 Corporate Ave, Cypress, CA 90630		
Native American Heritage Commission	Gayle Totton, Associate Analyst	1550 Harbor Blvd, Suite 100 West Sacramento, CA 95691	916-373-3710	nahc@nahc.ca.gov
Regional Water Quality Control Board, San Diego	David Gibson, Executive Officer	2375 Northside Drive, Suite 100 San Diego, CA 92108-2700	619-516-1990	David.Gibson@waterboards.ca.gov
Regional Water Quality Control Board, San Diego	Ben Neill, Brandi Outwin-Beals	2375 Northside Drive, Suite 100 San Diego, CA 92108-2700	(619) 521-3376	Ben.Neill@waterboards.ca.gov; Brandi.Outwin-Beals@waterboards.ca.gov
Southern California Regional Rail Authority		2703 Melbourne Ave, Pomona, CA 91767		
State Clearinghouse	Scott Morgan, Director	1400 Tenth Street, PO Box 3044 Sacramento, CA 95812-3044	916-445-0613	Scott.Morgan@opr.ca.gov
State Lands Commission	Cy Oggins, Chief, Environmental Planning and Management	100 Howe Avenue, Suite 100 South Sacramento, CA 95825	916-574-1900	cy.oggins@slc.ca.gov
State Lands Commission	Eric Gillies	100 Howe Avenue, Suite 100 South Sacramento, CA 95825		eric.gillies@slc.ca.gov
State Lands Commission	Alexandra Borack	100 Howe Avenue, Suite 100 South Sacramento, CA 95825		Alexandra.Borack@slc.ca.gov
State Office of Historic Preservation	CEQA Notice	1725 23rd Street, Suite 100 Sacramento, CA 95816	916-445-7000	
State Water Resources Control Board	Daniel Ellis	1001 I Street - P.O. Box 2815 Sacramento, CA 95812-2815		Daniel.Ellis@waterboards.ca.gov
State Water Resources Control Board	Scott Seyfried	1001 I Street - P.O. Box 2815 Sacramento, CA 95812-2815		Scott.Seyfried@waterboards.ca.gov
State Water Resources Control Board	Claire Waggoner and Kimberly Tenggardjaja	1001 I Street - P.O. Box 2815 Sacramento, CA 95812-2815	(916) 341-5858	Claire.Waggoner@waterboards.ca.gov Kimberly.Tenggardjaja@waterboards.ca.gov
State Water Resources Control Board	Mariela Carpio Obeso, Chief, Ocean Standards Unit	1001 I Street - P.O. Box 2815 Sacramento, CA 95812-2815		MarielaPaz.Carpio-Obeso@waterboards.ca.gov
State Water Resources Control Board - Division of Drinking Water	Oliver Pacifico	605 West Santa Ana Blvd, Building #28, Room 325 Santa Ana CA 92701	(714) 558-4410	oliver.pacifico@waterboards.ca.gov
State Water Resources Control Board - Drinking Water Revolving Fund	James Garrett	1001 I Street - P.O. Box 2815 Sacramento, CA 95812-2815		James.Garrett@waterboards.ca.gov
State Water Resources Control Board	Carol Atkins	1001 I Street - P.O. Box 2815 Sacramento, CA 95812-2815		Carol.Atkins@Waterboards.ca.gov
LOCAL				
California Air Resources Board	CEQA Notice	P.O. Box 2815 Sacramento, CA 95812		
Capistrano Bay District		35000 Beach Road, Capistrano Beach, CA 92624	9149-496-6576	drussell@capobay.org

Government Agencies

Organization	Attention	Address	Phone Number	Email
City of Brea, Water Division	CEQA Notice	One Civic Center Circle Brea, CA 92821	714-990-7687	
City of Buena Park, Water Services	CEQA Notice	6650 Beach Blvd. Buena Park, CA 90620	714-562-3721	
City of Dana Point, Community Development	CEQA Notice c/o Ursula Luna, Community Development Director	33282 Golden Lantern Dana Point, CA 92629	949-248-3567	mschneider@danapoint.org uluna@danapoint.org msinacori@danapoint.org
City of Dana Point, City Council	Richard Viczorek, Mayor	33282 Golden Lantern, Dana Point, CA 92629	(949) 248-3500	rviczorek@danapoint.org
City of Dana Point	Matt Schneider, Planning Manager	33282 Golden Lantern Dana Point, CA 92629	949 248 3560	
City of Dana Point	Matt Sinacori, Public Works Director	33282 Golden Lantern Dana Point, CA 92629	949 248 3574	
City of Fountain Valley, Water Department	CEQA Notice	18240 Ward Street Fountain Valley, CA 92708	714-593-4420	
City of Garden Grove, Water Services	CEQA Notice	13802 Newhope Street Garden Grove, CA 92843	714-741-5395	
City of Huntington Beach, Water Division	CEQA Notice	19001 Huntington St. Huntington Beach, CA 92648	714-536-5431	
City of La Habra, Water/Sewer Division	CEQA Notice	621 W Lambert Road La Habra, CA 90633	562-383-4170	
City of La Palma, Water Division	CEQA Notice	7822 Walker Street La Palma, CA 90623	714-690-3310	
City of Laguna Beach	IGR/CEQA Review	505 Forest Avenue, Laguna Beach, CA 92651		
City of Laguna Niguel	IGR/CEQA Review	30111 Crown Valley Parkway, Laguna Niguel, CA 92677		
City of Newport Beach, Public Works	CEQA Notice	100 Civic Center Drive, Bay 2D Newport Beach, CA 92660	949-644-3330	
City of Orange, Public Works	Water Service, CEQA Notice	189 S. Water Street Orange, CA 92866	714-288-2475	
City of San Clemente	IGR/CEQA Review	910 Calle Negocio # 100, San Clemente, CA 92673		
City of San Clemente, Utilities Services	CEQA Notice	100 Avenida Presidio 3, San Clemente, CA 92672	949-361-8200	
City of San Juan Capistrano	IGR/CEQA Review	32400 Paseo Adelanto, San Juan Capistrano, CA 92675		lmaravilla@planning.lacounty.gov
City of San Juan Capistrano, Utilities Department	Steve May, Public Works and Utilities Director	32400 Paseo Adelanto, San Juan Capistrano, CA 92675	949-234-4400	
City of Seal Beach, Public Works Department	Administrative & Engineering Division, CEQA Notice	211 8th Street Seal Beach, CA 90740	562-631-2527	
City of Tustin, Water Operations	CEQA Notice	300 Centennial Way, Tustin, CA 92780	714-573-3000	
City of Westminster, Water Division	CEQA Notice	8200 Westminster Blvd. Westminster, CA 92683	714-895-2876	
County of Los Angeles	IGR/CEQA Review	320 W Temple St, Los Angeles, CA 90012		kristi lovelady
County of Orange Planning	CEQA Notice	300 N. Flower St. Santa Ana, CA 92702	(714) 667-8845	chris.uzodiribe@ocpw.ocgov.com
County of Riverside	IGR/CEQA Review, Wendel Bugtai	4080 Lemon St. Riverside, CA 92501		
County of San Bernardino	IGR/CEQA Review, Tom Hudson	385 N Arrowhead Ave, San Bernardino, CA 92415		Tom.Hudson@lus.sbcounty.gov
County of San Diego	IGR/CEQA Review, Marc Cass	5510 Overland Ave, San Diego, CA 92123		Marc.Cass@sdcounty.ca.gov
East Orange County Water District	Lisa Ohlund, General Manager	185 N. McPherson Road Orange, CA 92869 2451	714-538-5815	lohlund@eocwd.com
El Toro Water District	CEQA Notice-Dennis Cafferty	Los Alisos Blvd. Lake Forest, CA 92630	949-837-7050	dcafferty@etwd.com
Emerald Bay Service District	Michael Dunbar, General Manager	600 Emerald Bay Laguna Beach, CA 92651	949-494-8571	mdunbar@ebservicedistrict.com
Golden State Water Company, West Orange County District	Dino Orbiso	1920 West Corporate Way Anaheim, CA 92801	714-535-8010	Dino.orbiso@gswater.com
Irvine Ranch Water District	Jo Ann Corey	15600 Sand Canyon Ave. Irvine, CA 92619-7000	949-453-5300	corey@irwd.com
Laguna Beach County Water District	CEQA Notice - David Youngblood	306 Third Street Laguna Beach, CA 92651	949-494-1041	dyoungblood@lbcwd.org
Mesa Water District	CEQA Notice	1965 Placentia Ave. Costa Mesa, CA 92627	949-631-1200	
MetroLink	Christos Sourmelis	One Gateway Plaza, 12th Floor, Los Angeles, CA 90012	909.392.8463	sourmelisc@scrra.net
MetroLink	Ron Mathieu	One Gateway Plaza, 12th Floor, Los Angeles, CA 90012		
Metropolitan Water District	Dee Bradshaw	700 North Alameda Street Los Angeles, CA 90012	(213) 217-6028	VBradshaw@mwdh2o.com
Metropolitan Water District	Warren Teitz	700 North Alameda Street Los Angeles, CA 90012	(213) 217-7418	wteitz@mwdh2o.com
Moulton Niguel Water District	Matt Collings, Asst. General Manager	27500 La Paz Road Laguna Niguel, CA 92677	949-448-4032	mcollings@mnwd.com
Municipal Water District of Orange County	Karl Seckel, Assistant Manager	18700 Ward Street Fountain Valley, CA 92708	714-963-3058	kseckel@mwdoc.com
Orange County Board of Supervisors	Supervisor Lisa Bartlett	34145 Pacific Coast Highway, Suite 710 Dana Point, Ca 92629	949-232-8882	Info@lisaforsupervisor.com
Orange County Board of Supervisors		333 W. Santa Ana Blvd., Santa Ana, CA 92701		
Orange County Transportation Authority	CEQA Notice	550 S. Main Street Orange, CA 92868		dphu@octa.net
Orange County Flood Control District	Ariel Corpuz	300 N. Flower Street, Suite 716 Santa Ana, CA 92703		ariel.corpuz@ocpw.ocgov.com
Orange County Health Care Agency	Anna Peters	405 W. Fifth Street Santa Ana, CA 92701	714-834-5150	apeters@ochca.com
Orange County Public Works	Robert McLean	300 N. Flower Street, Suite 716 Santa Ana, CA 92703		Robert.McLean@ocpw.ocgov.com
Orange County Public Works	Penny Lew	300 N. Flower Street, Suite 716 Santa Ana, CA 92703	714-647-3990	Penny.Lew@ocpw.ocgov.com
Orange County Public Works	James Tyler	300 N. Flower Street, Suite 716 Santa Ana, CA 92703	714-667-3210	James.Tyler@ocpw.ocgov.com
Orange County Public Works	Jeff Dickman	300 N. Flower Street, Suite 716 Santa Ana, CA 92703		jeff.dickman@ocpw.ocgov.com
Orange County Public Works	William Fegley	300 N. Flower Street, Suite 716 Santa Ana, CA 92703	949-923-2289	william.fegley@ocparks.com
Orange County Public Works	James Volz	300 N. Flower Street, Suite 716 Santa Ana, CA 92703	714-647-3904	james.volz@ocpw.ocgov.com
Orange County Public Works	Andy Ngo	300 N. Flower Street, Suite 716 Santa Ana, CA 92703	714-726-4297	andy.ngo@ocpw.ocgov.com
Orange County Public Works	Duc Nguyen	300 N. Flower Street, Suite 716 Santa Ana, CA 92703	714-955-0676	duc.nguyen@ocpw.ocgov.com
Orange County Public Works	Richard Vuong	300 N. Flower Street, Suite 716 Santa Ana, CA 92703		
Orange County Public Works	Laree Alonso	300 N. Flower Street, Suite 716 Santa Ana, CA 92703	714-647-9649	Laree.alonso@ocpw.ocgov.com
Orange County Public Works	Nardy Khan	300 N. Flower Street, Suite 716 Santa Ana, CA 92703	714-647-3906	nardy.khan@ocpw.ocgov.com
Orange County Parks	Susan Brodeur	13042 Old Myford Rd. Irvine, CA 92602	949-585-6448	susan.brodeur@ocparks.com
Orange County Parks	Kory McCain	13042 Old Myford Rd. Irvine, CA 92602	714-856-5772	kory.mccain@ocparks.com

Government Agencies

Organization	Attention	Address	Phone Number	Email
Orange County Parks	Eric E. Hull	13043 Old Myford Rd. Irvine, CA 92602		
Orange County Parks	Tom Townsend	13042 Old Myford Rd. Irvine, CA 92602	949-923-3747	tom.townsend@ocparks.com
Orange County Water District	CEQA Notice	18700 Ward St. Fountain Valley, CA 92708	714-378-3200	chris.uzodiribe@ocpw.ocgov.com
San Diego Gas and Electric		662 Camino de Los Mares, San Clemente, CA		
San Juan Basin Authority	Daniel Ferons-c/o Santa Margarita Water District	26111 Antonio Parkway Rancho Santa Margarita, CA 92688	949-459-6400	
	Norris Brandt			
Santa Margarita Water District	Dan Ferons, Don Bunts	26111 Antonio Parkway, Rancho Santa Margarita, CA 92688	949-459-6400	danf@smwd.com; donb@smwd.com
Serrano Water District	CEQA Notice	18021 East Lincoln Street Villa Park, CA 92861	714-538-0079	
South Orange County Wastewater Authority	Amber Baylor	34156 Del Obispo Street Dana Point, CA 92629	949-234-5400	abaylor@socwa.com
South Orange County Wastewater Authority	Jim Burror	34156 Del Obispo Street Dana Point, CA 92629	949-234-5400	jburror@socwa.com
South Coast Air Quality Management District	Jillian Wong, Program Supervisor	21865 East Copley Drive Diamond Bar, CA 91765-4178	909-396-2000	jcheng@aqmd.gov
	Lijin Sun			
South Orange County Wastewater Authority	Betty Burnett, General Manager	34156 Del Obispo St. Dana Point, CA 92629	949-234-5400	bburnett@socwa.com
Southern California Association of Governments	Attn: Intergovernmental Review	900 Wilshire Blvd, 17th floor, Los Angeles, California 90017	(213) 236-1800	sunl@scag.ca.gov
Trabuco Canyon Water District	CEQA Notice	32003 Dove Canyon Drive Trabuco Canyon, CA 92679	949-858-0277	
Yorba Linda Water District	CEQA Notice	1717 East Miraloma Ave. Placentia, CA 92870	714-777-3018	
TRIBES				
Juaneno Band of Mission Indians Acjachemen Natior	Matias Belardes, Chairperson	32161 Avenida Los Amigos, San Juan Capistrano, CA 92675	949-293-8522 949-444-4340	
Juaneno Band of Mission Indians Acjachemen Natior	Joyce Perry, Tribal Manager	4955 Paseo Segovia, Irvine, CA 92612	949-293-8522	kaamalam@gmail.com
Juaneno Band of Mission Indians Acjachemen Natior	Teresa Romero, Chairwoman	31411-A La Matanza Street, San Juan Capistrano, CA 92675	949-488-3484 530-354-5876	
Juaneno Band of Mission Indians	Sonia Johnston, Tribal Chairperson	P.O. Box 25628, Santa Ana, CA 92799		sonia.johnston@sbcglobal.net
San Gabriel Band of Mission Indians	Anthony Morales, Chief	P.O. Box 693 San Gabriel, CA 91778	626-483-3564	GTTribalcouncil@aol.com

Non-Government Agencies

NAME	CONTACT	ADDRESS	PHONE	EMAIL
California Coastal Protection Network	Susan Jordan	Po Box 30290, Santa Barbara, CA 93130	(805) 637-3037	left a message
California Coastkeeper Alliance	Sara Aminzadeh	156 Second Street, San Francisco, CA 94105	(415) 794-8422	sara@cacoastkeeper.org
California Union for Reliable Energy (CURE)	Sheila Sannadan	601 Gateway Boulevard, Suite 1000, South San Francisco CA 94080	650-589-1660	ssannadan@adambroadwell.com
California Union for Reliable Energy (CURE)	Alisha C. Pember	601 Gateway Boulevard, Suite 1000, South San Francisco CA 94080	650-589-1660	apember@adamsbroadwell.com
California Union for Reliable Energy (CURE)	Linda T. Sobczynski	601 Gateway Boulevard, Suite 1000, South San Francisco CA 94080	650-589-1660	lsobczynski@adamsbroadwell.com
Capo Bay CSD	Don Russell	35000 Beach Rd, Capistrano Beach, CA 92624		drussell@capobay.org
Capo Cares				capocares@gmail.com
Center for Biological Diversity	Ilene Anderson	660 S. Figueroa St., Suite 1000, Los Angeles, CA 90017	(323) 654-5943	ianderson@biologicaldiversity.org
Clean Water Now!	Roger Von Butow	P.O. Box 4711, Laguna Beach CA 92652 2796 Victoria Drive "B", Laguna Beach, CA 92651	(949) 280-2225	info@clean-water-now.org; rogerbutow@me.com
Coastal Environmental Rights Foundation	Monika Whisenhunt	1140 South Coast Hwy 101, Encinitas, CA 92024	(760) 942-8505	monika@coastlawgroup.com
Dana Point Headlands Conservancy		34681 Calle Paso Robles, Capo Beach, CA 92624	(949) 248-3527	left a message
Dana Point Library		33841 Niguel Road, Dana Point, CA 92629		
Doheny Longboard Surfing Association		P.O. Box 664, Dana Point, CA 92629	949-413-6250	no email, for inquiries only
Earth Resource Foundation	Stephanie Barger, Executive Dir.	1706 Newport Blvd. Ste. B, Costa Mesa, CA 95627	(949) 645-5163	stephanie.barger@earthresource.org
Ecology Center	Meg Hiesinger	32701 Alipaz St., San Juan Capistrano, CA 92675	(949) 443-4223	meg@theecologycenter.org
Endangered Habitats League	Dan Silver	8424 Santa Monica Blvd., Ste. A 592, Los Angeles, CA 90069	(213) 804-2750	dsilverla@me.com
Heal the Bay	Rita Kampalath	1444 9th St. Santa Monica, CA 90401	(310) 451-1500	rkampalath@healthebay.org
Laguna Ocean Foundation		P.O. Box 5247 Laguna Beach, CA 92652	no phone number	lagunaoceanfoundation@gmail.com
Marlborough Seaside Villas, Homeowners Association	Lazar Skundric, Vice President	910 Calle Negocio, Suite 200 San Clemente, CA 92763	949-661-7767	
MetroLink	Christos Sourmelis	One Gateway Plaza , 12th Floor, Los Angeles, CA 90012		
Mi Ocean	Patrick Fuscoe	16795 Von Karman Ave, Ste 100, Irvine, CA 92606	(949) 271-4386	left a message
Natural Resources Defense Council	Joe Geever	1314 Second St., Santa Monica, CA 90401	(310) 434-2300	nrdcinfo@nrdc.org
Orange County Coastkeeper	Colin Kelly, Senior Staff Attorney	3151 Airway Ave. Suite F-110, Costa Mesa, CA 92626	(714) 850-1965	colin@coastkeeper.org
Pacific Marine Mammal Center		20612 Laguna Canyon Rd, Laguna Beach, CA 92651	(949) 494-3050	info@pacificmmc.org
Planning and Conservation League	Jonas Minton	1107 9th St., Ste. 901, Sacramento, CA 95814	(916) 822-5631	pclmail@pcl.org
Residents for Responsible Desalination	Dave Hamilton	P.O. Box 5422, Huntington Beach, CA 92615-5422		de.hamilton@verizon.net
Residents for Responsible Desalination - Huntington Beach	Don Shultz	21352 Yarmouth Ln, Huntington Beach, CA 92646	(714) 840-8901	info@r4rd.org
San Clemente Green	Bill Hart	2837 Penasco, San Clemente, CA 92673	emailed for number	bill@sanclementegreen.org
Sea and Sage Audubon Society	Dr. Victor Leipzig	Audubon House, 5 Riparian View, Irvine, CA 92612	(714) 848-5394	vicleipzig@aol.com
Sierra Club	Penny Elia	3435 Wilshire Blvd. Ste. 660, Los Angeles, CA 90010	(213) 387-4287	greenp1@cox.net
Soto Resources	Me Joey Soto			joey@sotoresources.com
South Laguna Civic Association	Greg O'Loughlin	31558 Eagle Rock Way, Laguna Beach, CA 92651	(949) 415-1312	GregO@SouthLaguna.org
South Orange County Economic Coalition		27758 Santa Margarita Parkway #378 Mission Viejo, CA 92691	949.600.5470	brian@communicationslab.com
Southern California Coastal Water Research Project	Dr. Stephen Weisberg, Executive Dir.	3535 Harbor Blvd., Costa Mesa, CA 92626	(714) 755-3200	christinas@sccwrp.org
Southern California Watershed Alliance, Desal Response Group	Aubrey Bettencourt	PO Box 1267, Hanford, CA 93232	(559) 816-8691	aubrey@californiawateralliance.org
Surfrider Foundation	Rick Wilson Katie Day Mandy Sackett	P.O. Box 6010, San Clemente, CA 92674	(949) 492-8170	rwilson@surfrider.org kday@surfrider.org Mandy Sackett <msackett@surfrider.org>
Transition Laguna Beach		1215 Bluebird Canyon Dr. Laguna Beach, CA 92561	emailed for number	ecolagunabeach@gmail.com
Trout Unlimited	Robert Blankenship	P.O. Box 1977, Costa Mesa, CA 92628	(703) 522-0500	bob@hremcleanup.com; SouthCoastTU@gmail.com
Trout Unlimited	George Sutherland	419 Via Presa, San Clemente, CA 92672	(703)522-0500	scgsland@gmail.com
Wyland Foundation	Greg Stone	6b Macon, Irvine, CA 92620	(949) 643-7070	info@wylandfoundation.org

Interested Parties

NAME	ADDRESS	EMAIL	PHONE
Robert Campbell	33231 Mesa Vista Drive, Dana Point, CA 92629	rsbobcamp@aol.com	
Richard Ciampa	25582 Mainsail Way, Dana Point, CA 92629	rciampa@cox.net	
Pam Enqille	33701 Surfside Dana Point, CA 92629	pabenqelke@gmail.com	
Richard Gardner		capopalm@hotmail.com	
Catherine Gick	27045 Mill Pond Road, Capistrano Beach, CA 92624	geoplex@earthlink.net	
Dennis Heider	34112 Bedford Lane, Dana Point, CA 92629	dheider@heiderinspection.com	909-673-0292
Jim Mahaney		Mahaney.jim@sbcglobal.net	
Jan Mestion		jan@citysun.com	
George Miller	24005 Atun, Dana Point, CA 92629	papageo13@aol.com	
Bobby Young		by4golden@yahoo.com	
Matt Allaire	24911 Sea Aire, Dana Point, CA 92629	mallaire2112@gmail.com	
Jonelle Malloy	33112 Palo Alto Street, Dana Point 92629	jonelle1malloy@gmail.com	
Irene Bowie	31582 Wildwood Rd, Laguna Beach CA 92651	huladog1@earthlink.net	
Ray Hiemstra		ray@coastkeeper.org	
Kaye Romo	24351 La Cresta Drive, Dana Point, CA		
Bob Oakley		bob_oakley@msn.com	

Proof of Publication

- Orange County Register

- Dana Point Times

AFFIDAVIT OF PUBLICATION

STATE OF CALIFORNIA,)
) ss.
County of Orange)

I am a citizen of the United States and a resident of the County aforesaid; I am over the age of eighteen years, and not a party to or interested in the above entitled matter. I am the principal clerk of **The Orange County Register**, a newspaper of general circulation, published in the city of Santa Ana, County of Orange, and which newspaper has been adjudged to be a newspaper of general circulation by the Superior Court of the County of Orange, State of California, under the date of November 19, 1905, Case No. A-21046, that the notice, of which the annexed is a true printed copy, has been published in each regular and entire issue of said newspaper and not in any supplement thereof on the following dates, to wit:

May 24, 2018

"I certify (or declare) under the penalty of perjury under the laws of the State of California that the foregoing is true and correct":

Executed at Santa Ana, Orange County, California, on

Date: May 24, 2018

Sandra Campos

Signature: **Sandra Campos**

The Orange County Register
2190 S. Towne Centre Place
Anaheim, CA 92806
(714) 796-2209

PROOF OF PUBLICATION

Notice of Availability & Public Meeting Notice

Doheny Ocean Desalination Project Environmental Impact Report

The South Coast Water District (District) has prepared an Environmental Impact Report (EIR) pursuant to the California Public Resources Code and the California Environmental Quality Act (CEQA) to evaluate the environmental effects associated with the proposed Doheny Ocean Desalination Project (project). This Notice of Availability (NOA) has been issued to notify interested parties that a Draft EIR is publicly available for review and comment. The District is requesting comments on the Draft EIR from Responsible and Trustee agencies, interested public agencies, organizations, and the general public (pursuant to CEQA Guidelines §15087). The Draft EIR assesses the potential environmental effects of implementing a proposed ocean water desalination facility of up to 15 million gallons per day (MGD) of potable drinking water, with an initial phase of up to 5 MGD. The proposed facilities are located in Dana Point, including subsurface intake wells proposed at Doheny State Beach and Capistrano Beach, and various conveyance lines connecting the intake and discharge facilities to existing District property located approximately ½ mile inland, adjacent to San Juan Creek.

The Notice of Availability and Draft EIR are available for review at the Orange County Public Library located in the City of Dana Point (Dana Point Library, 33841 Niguel Road, Dana Point, CA 92629). The documents may also be reviewed online at the District's website: www.scwd.org/desal

The Draft EIR addresses Aesthetics, Air Quality, Biological Resources, Cultural Resources, Geology and Soils, Greenhouse Gas Emissions, Hazards and Hazardous Materials, Hydrology and Water Quality, Land Use and Planning, Noise, Public Services, Recreation, Transportation and Traffic, Tribal Cultural Resources and Utilities and Service Systems, as well as energy conservation, alternatives, potential growth-inducing impacts, and cumulative impacts. With implementation of various Project Design Features and EIR mitigation measures, the EIR has concluded that the Phase I Project (up to 5 MGD) would not have any "unavoidable significant impacts." The Regional Project, if pursued at a later date, could result in unavoidable significant impacts, although this is speculative at this time due to lack of Regional Project details.

CEQA also requires this NOA to specify if the Project site contains any listed toxic sites. The Project site does not contain sites identified as meeting the "Cortese List" requirement (Government Code Section 65962.5).

Public Meeting: The South Coast Water District will conduct a public meeting to receive public comments on the Draft EIR. The meeting will be held at the following location, date and time:

Tuesday, June 26, 2018
6:00 p.m. (ending no later than 8:00 p.m. or when discussion concludes)
Capistrano Unified School District, CUSD Education Center
33122 Valle Road, San Juan Capistrano, CA 92675
Phone: (949) 234-9200

Public Review Period: The Draft EIR is available for public review for a period of 60 days. In accordance with CEQA, should you have any comments, please provide written comments on the Draft EIR within the 60-day period between **May 23, 2018 to July 23, 2018.**

Public Comments: The District requests your careful review and consideration of the Draft EIR, and invites **written comments** from interested agencies, persons, and organizations regarding environmental issues identified in the Draft EIR. Please indicate a contact person for your agency or organization. You may also provide oral or written comments in person at the **Public Meeting** noted above. Comments in response to this notice must be submitted to the District through close of business (5:00 PM) on **July 23, 2018**.

Lead Agency Contact: All comments should be submitted in writing to:

South Coast Water District
Attn: Mr. Rick Shintaku, PE - Acting General Manager, Chief Engineer
31592 West Street, Laguna Beach, CA 92651
(949) 499-4555

Publish: Orange County Register May 24, 2018 11125261

DOUG APPLEGATE, Democrat
• Has vowed to work with federal legislators to remove spent nuclear waste from San Onofre Nuclear Generating Station
• Wants to bring more power back to Congress to approve U.S. military engaging in combat missions and wars
• Is in favor of having 100 percent renewable energy in the U.S. in the next 10 years
• Advocates for a single-payer health care system

JOSHUA SCHOONOVER, Republican
• Vows to help keep the ocean and environment clean by providing incentives to businesses to use sustainable materials and waste removal practices
• Wants to decrease the cost of health care
• Advocates to fix illegal immigration problems
• Wants to remove cannabis from the Drug Enforcement Administration's schedule of illegal drugs

MIKE SCHMITT, Republican
• Advocates for health care reform to repeal the Affordable Care Act, return control to patients and doctors
• Opposes offshore drilling by 2024 off U.S. coasts
• Supports gun rights and is a "strong" proponent of the Second Amendment
• Advocates immigration policies to accept based on merit and skill sets

DANIELLE ST. JOHN, Green
• Is in favor of bringing harmonizing discussions to Congress in an effort to make collaborative legislation
• Wants to drive a "cooperative economy" that is accountable for all members, eliminating any kind of greed in the marketplace
• Advocates for a complete reform of the U.S. education system, increasing teachers' salaries and better physical health and creative curriculum
• Advocates for gun laws that allow responsible ownership

ROCKY J. CHÁVEZ, Republican
• Wants to scale back federal regulations and intervention into business to create an easier market on small businesses and industry
• Advocates creating programs to crack down on human trafficking and other violent felony-level crimes
• Has stayed neutral on federal and state gun-reform policies
• Favors creating a stronger national defense

DIANE L. HARKEY, Republican
• Wants to make changes to the federal tax code
• Advocates for strengthening California's resources of water, power and transportation
• Is in favor of reducing the national deficit by cutting federal spending
• Supports stronger national defense

JOSHUA L. HANCOCK, Libertarian
• Seeks to address the growing homeless population in California and U.S.
• Supports increasing funding for U.S. border security
• Wants to dramatically cut federal spending to decrease the national debt
• Supports responsible gun ownership, would not ban assault rifles

BRIAN MARYOTT, Republican
• Wants more national security measures in place; would vote to authorize more authority to the president to engage in foreign conflicts, such as with North Korea
• Does not support off-shore oil drilling, advocates for environmental protection, but wants a balance of responsibility
• Favors repealing the Affordable Care Act
• Opposes California's "Sanctuary State" laws

JORDAN P. MILLS, Peace and Freedom
• Opposes the Trump administration's proposed wall along the Mexican border as well as deportations
• Is in favor of outlawing high-caliber rifles for public purchase; opposes all war
• Supports pro-choice policies for women
• Wants to strengthen Social Security and social services

DAVID MEDWAY, Republican
• Aims to lower the national debt
• Favors affordable health care across the spectrum
• Is a pro-choice candidate and is pro women's rights to access health care services
• Wants to find "innovative" solutions to stop gun violence

CRAIG A. NORDAL, Republican
• Supports the Trump administration's plan to increase offshore oil drilling by 2024 in an effort to boost U.S. trade status
• Advocates for a more Christian-based Legislature, religious freedom and liberty
• Is in favor of overturning Roe V. Wade
• Supports building and completing a wall along the Mexican border

DOHENY OCEAN DESALINATION PROJECT

AMENDED

Notice of Availability and Public Meeting Notice

Affidavit of Distribution

Kimley-Horn and Associates, Inc.
3880 Lemon Street, Suite 420
Riverside, CA 92501

Table of Contents

Affidavit of Distribution

AFFIDAVIT OF MAILING

Date: June 6, 2018

Subject: Doheny Ocean Desalination Project Amended NOA and Public Meeting Notice

***Note:** Due to a digital file error which occurred on CDs distributed on May 23rd, this amended version was distributed to ensure all parties were properly notified and received all materials related to the project and details.*

AFFIDAVIT OF POSTING

I, Amanda McCallum, do hereby certify that a copy of the attached Notice of Availability and Public Meeting Notice was posted at the City of Dana Point Public Library, 33841 Niguel Road, Dana Point, CA 92629, the South Coast Water District, 31592 West Street, Laguna Beach, CA 92651, and with the County Clerks for Orange, Los Angeles, San Diego, San Bernardino, and Riverside Counties, on June 6, 2018. I declare under penalty of perjury that the foregoing is true and correct.

Amanda McCallum
Kimley-Horn and Associates

AFFIDAVIT OF MAILING

I, Amanda McCallum, do hereby certify that a copy of the attached Notice of Availability and Public Meeting Notice was mailed via USPS or sent via FedEx to each and every person on the attached distribution lists on June 6, 2018. Copies of the NOA distribution lists are attached. FedEx delivery receipts are on file. I declare under penalty of perjury that the foregoing is true and correct.

Amanda McCallum
Kimley-Horn and Associates

AFFIDAVIT OF NEWSPAPER PUBLICATION

I, Amanda McCallum, do hereby certify that a copy of the attached Notice of Availability and Public Meeting Notice was published in the OC Register on June 6, 2018 and the Dana Point Times on June 8, 2018. I declare under penalty of perjury that the foregoing is true and correct.

Amanda McCallum
Kimley-Horn and Associates

AMENDED NOA and Public Meeting Notice Posting

- Orange County Clerk Filing Copy

- San Diego County Clerk Filing Copy

- Los Angeles County Clerk Filing Copy

- San Bernardino County Clerk Filing Copy

- Riverside County Clerk Filing Copy

AMENDED NOTICE OF AVAILABILITY OF A DRAFT EIR & PUBLIC MEETING NOTICE

SCH# 2016031038

POSTED
JUN 05 2018
ORANGE COUNTY CLERK-RECORDER DEPARTMENT
BY: ________ DEPUTY

Date: June 4, 2018

To: Reviewing Agencies, Organizations, and Interested Parties

Lead Agency: South Coast Water District

Subject: ***Amended Notice of Availability & Public Meeting Notice***

DOHENY OCEAN DESALINATION PROJECT DRAFT ENVIRONMENTAL IMPACT REPORT
State Clearinghouse No. 2016031038

The South Coast Water District (District) has prepared an Environmental Impact Report (EIR) pursuant to the California Public Resources Code and the California Environmental Quality Act (CEQA) to evaluate the environmental effects associated with the proposed Doheny Ocean Desalination Project (Project). This ***Amended*** Notice of Availability (***Amended*** NOA) has been issued to notify interested parties that the District is re-releasing the Draft EIR for public review and comment. The District initially released the Draft EIR on May 23, 2018. The printed copies of the Draft EIR, including those available at the library and District office, were complete. However, the District since discovered that at least some of the electronic copies of the Draft EIR did not contain certain exhibits due to a reprographic error. Therefore the District is re-releasing the Draft EIR, containing all exhibits. The Project itself is unchanged. The date of the public meeting (see details below) is unchanged. To ensure full opportunity for public review and comment, the District is extending the public comment period accordingly. ***The District is extending the deadline for public comment from July 23, 2018 to August 6, 2018 to allow for a full 60-day review period of the complete Draft EIR.*** The District is requesting comments on the Draft EIR from Responsible and Trustee agencies, interested public agencies, organizations, and the general public (pursuant to CEQA Guidelines §15087). The original Notice of Availability, dated May 23, 2018, is attached to this ***Amended*** NOA.

PROJECT SUMMARY:

The Draft EIR assesses the potential environmental effects of implementing a proposed ocean water desalination facility located in Dana Point, including subsurface intake wells proposed at Doheny State Beach and Capistrano Beach, and various conveyance lines connecting the intake and discharge facilities to existing District property located approximately ½ mile inland, adjacent to San Juan Creek. The District is only proposing to pursue permits and approvals for the initial Phase I Project, which would provide up to 5 million-gallons-per-day (MGD) of potable water, and is therefore addressed at a "project level" of CEQA review. The EIR also evaluates a potential future Regional Project of up to 15 MGD, at a programmatic level, since specific Regional Project partners, financing, and facilities have yet to be defined. Specific Project component descriptions are provided below and on the District's website at: www.scwd.org/desal.

- **A subsurface water intake system** consisting of subsurface slant wells that draw ocean water from offshore subsurface alluvial material (located below the ocean floor), while providing natural sand bed filtration and eliminating the entrainment and impingement of marine biota. This subsurface intake system is the recommended approach by state and federal regulators, and is consistent with the State Water Resource Control Board's (State Board or SWRCB) recently adopted Ocean Plan Amendment. The slant wells would be located and fully buried near the beach, in a study area encompassing Doheny State Beach and Capistrano Beach Park.
- **A raw (ocean) water conveyance pipeline** that would deliver the subsurface intake system's ocean water to the desalination facility site.
- **A desalination facility** that would receive ocean feedwater at approximately 10 to 30 MGD, with a recovery rate of ~50% resulting in up to 5 to 15 MGD of potable drinking water (for the Phase I and Regional Project, respectively). The proposed desalination facility is located on the District's existing San Juan Creek Property site, on an industrial site located away from the beach but in close proximity to the subsurface intake wells. This facility siting is also consistent with state and federal regulator preference to minimize desalination facilities on the coast while being close enough to avoid lengthy raw water and brine conveyance pipelines. The desalination facility includes a variety of typical desalination process equipment and appurtenant facilities, such as pretreatment, seawater reverse osmosis (SWRO) membranes, an energy recovery system, post-treatment conditioning, solids handling and disposal, product water storage, electrical equipment, staff facilities, and connections to off-site brine disposal, sanitary sewer, and product water conveyance facilities. It is assumed there will be a utility power connection required; however, the District is also evaluating the feasibility of supplementing or replacing that supply with an alternative energy source. The desalination facility will include solar photovoltaic panels on flat rooftops where feasible. Other alternative energy sources being evaluated include natural-gas turbines and fuel cells to maximize efficiency and minimize energy costs.

POSTED
JUN 05 2018
ORANGE COUNTY CLERK-RECORDER DEPARTMENT
BY: DEPUTY

- **A concentrate (brine) disposal system** that would utilize the existing San Juan Creek Ocean Outfall (SJCOO), to return brine and treated process waste streams to the ocean with negligible impact on coastal and marine water quality. This would be achieved in part through blending in the outfall pipe with the existing wastewater stream from the J.B. Latham Wastewater Treatment Plant, and other regional treatment plants. Mixing desalination brine with existing wastewater treatment plant flow (a "comingled discharge") is the preferred method by state and federal regulators and is consistent with the State Board's Ocean Plan Amendment.
- **A product water storage tank and distribution system** that would feed into the District's local distribution system and, depending on plant capacity and District demands, other adjacent local and regional transmission pipelines that are located adjacent to the site. Desalinated product water from the Phase I Project could be conveyed entirely using existing District and local infrastructure with no off-site improvements other than a short connection to the District's existing local transmission lines.
- **All appurtenant facilities** (e.g. pump stations, valves and metering) as well as all construction, operation and maintenance activities associated with all Project facilities.

- **Offsite Electrical Transmission Facilities** provided by San Diego Gas & Electric Company (SDG&E). At this time, SDG&E has indicated that electrical service can be provided to the Phase I Project using existing facilities, with a short connection from the desalination site to underground electrical lines in Stonehill Drive.

SIGNIFICANT ENVIRONMENTAL IMPACTS:

The Draft EIR addresses Aesthetics, Air Quality, Biological Resources, Cultural Resources, Geology and Soils, Greenhouse Gas Emissions, Hazards and Hazardous Materials, Hydrology and Water Quality, Land Use and Planning, Noise, Public Services, Recreation, Transportation and Traffic, Tribal Cultural Resources and Utilities and Service Systems, as well as energy conservation, alternatives, potential growth-inducing impacts, and cumulative impacts. With implementation of various Project Design Features and EIR mitigation measures, the EIR has concluded that the Phase I Project (up to 5 MGD) would not have any "unavoidable significant impacts." The Regional Project, if pursued at a later date, could result in unavoidable significant impacts, although this is speculative at this time due to lack of Regional Project details.

CEQA also requires this NOA to specify if the Project site contains any listed toxic sites. The Project site does not contain sites identified as meeting the "Cortese List" requirement (Government Code Section 65962.5).

AGENCIES:

The District requests each Responsible and Trustee agency review the Draft EIR relevant to the agency's statutory responsibilities in connection with the proposed Project, in a manner consistent with California Code of Regulations, Title 14, Section 15087. Each agency may use the EIR prepared by the District when considering any permits that the agency must issue, or other approvals for the Project.

PUBLIC REVIEW PERIOD:

The Draft EIR is available for public review for a period of 60 days. In accordance with CEQA, should you have any comments, please provide written comments on the Draft EIR within the 60-day period between **June 6, 2018 to August 6, 2018.**

LOCATIONS WHERE DRAFT EIR IS AVAILABLE FOR PUBLIC REVIEW

An electronic PDF of the Draft EIR is available for download on the District's Project website at www.scwd.org/desal. In addition, during the 60-day public review period, hard copies of the Draft EIR and the documents referenced in the EIR will be available at the following locations:

- South Coast Water District Offices, address noted below
- Orange County Public Library, Dana Point Branch, 33841 Niguel Rd. Dana Point, CA 92629

POSTED
JUN 05 2018
ORANGE COUNTY CLERK-RECORDER DEPARTMENT
BY: ____ DEPUTY

PUBLIC COMMENTS:

The District requests your careful review and consideration of the Draft EIR, and invites ***written comments*** from interested agencies, persons, and organizations regarding environmental issues identified in the Draft EIR. Please indicate a contact person for your agency or organization. You may also provide oral or written comments in person at the ***Public Meeting*** noted below. Comments in response to this notice must be submitted to the District through close of business (5:00 PM) on **August 6, 2018.**

LEAD AGENCY CONTACT:

All comments should be submitted in writing to:

South Coast Water District
Attn: Mr. Rick Shintaku, PE – Acting General Manager, District Engineer
31592 West Street, Laguna Beach, CA 92651
(949) 499-4555

PUBLIC MEETING:

The District will conduct a public meeting in order to receive public comments on the Draft EIR. The meeting will be held at the following location, date and time:

Tuesday, June 26, 2018
6:30 p.m.
(ending no later than 8:00 p.m. or when discussion concludes)
Dana Hills High School - Gym
33333 Golden Lantern St, Dana Point, CA 92629
Phone: (949) 496-6666

Special Accommodations. Should you require special accommodations at the public meeting, such as for the hearing impaired or an English translator, please contact South Coast Water District no later than **June 19, 2018** (see contact information above).

POSTED

JUN 05 2018

ORANGE COUNTY CLERK-RECORDER DEPARTMENT

BY:______________________DEPUTY

NOTICE OF AVAILABILITY OF A DRAFT EIR & PUBLIC MEETING NOTICE

SCH# 2016031038

POSTED
JUN 05 2018
ORANGE COUNTY CLERK-RECORDER DEPARTMENT
BY: ______ DEPUTY

Date: May 23, 2018

To: Reviewing Agencies, Organizations, and Interested Parties

Lead Agency: South Coast Water District

Subject: ***Notice of Availability & Public Meeting Notice***

DOHENY OCEAN DESALINATION PROJECT DRAFT ENVIRONMENTAL IMPACT REPORT
State Clearinghouse No. 2016031038

The South Coast Water District (District) has prepared an Environmental Impact Report (EIR) pursuant to the California Public Resources Code and the California Environmental Quality Act (CEQA) to evaluate the environmental effects associated with the proposed Doheny Ocean Desalination Project (Project). This Notice of Availability (NOA) has been issued to notify interested parties that a Draft EIR is publicly available for review and comment. The District is requesting comments on the Draft EIR from Responsible and Trustee agencies, interested public agencies, organizations, and the general public (pursuant to CEQA Guidelines §15087).

PROJECT SUMMARY:

The Draft EIR assesses the potential environmental effects of implementing a proposed ocean water desalination facility located in Dana Point, including subsurface intake wells proposed at Doheny State Beach and Capistrano Beach, and various conveyance lines connecting the intake and discharge facilities to existing District property located approximately ½ mile inland, adjacent to San Juan Creek. The District is only proposing to pursue permits and approvals for the initial Phase I Project, which would provide up to 5 million-gallons-per-day (MGD) of potable water, and is therefore addressed at a "project level" of CEQA review. The EIR also evaluates a potential future Regional Project of up to 15 MGD, at a programmatic level, since specific Regional Project partners, financing, and facilities have yet to be defined. Specific Project component descriptions are provided below and on the District's website at: www.scwd.org/desal.

- **A subsurface water intake system** consisting of subsurface slant wells that draw ocean water from offshore subsurface alluvial material (located below the ocean floor), while providing natural sand bed filtration and eliminating the entrainment and impingement of marine biota. This subsurface intake system is the recommended approach by state and federal regulators, and is consistent with the State Water Resource Control Board's (State Board or SWRCB) recently adopted Ocean Plan Amendment. The slant wells would be located and fully buried near the beach, in a study area encompassing Doheny State Beach and Capistrano Beach Park.

- **A raw (ocean) water conveyance pipeline** that would deliver the subsurface intake system's ocean water to the desalination facility site.
- **A desalination facility** that would receive ocean feedwater at approximately 10 to 30 MGD, with a recovery rate of ~50% resulting in up to 5 to 15 MGD of potable drinking water (for the Phase I and Regional Project, respectively). The proposed desalination facility is located on the District's existing San Juan Creek Property site, on an industrial site located away from the beach but in close proximity to the subsurface intake wells. This facility siting is also consistent with state and federal regulator preference to minimize desalination facilities on the coast while being close enough to avoid lengthy raw water and brine conveyance pipelines. The desalination facility includes a variety of typical desalination process equipment and appurtenant facilities, such as pretreatment, seawater reverse osmosis (SWRO) membranes, an energy recovery system, post-treatment conditioning, solids handling and disposal, product water storage, electrical equipment, staff facilities, and connections to off-site brine disposal, sanitary sewer, and product water conveyance facilities. It is assumed there will be a utility power connection required; however, the District is also evaluating the feasibility of supplementing or replacing that supply with an alternative energy source. The desalination facility will include solar photovoltaic panels on flat rooftops where feasible. Other alternative energy sources being evaluated include natural-gas turbines and fuel cells to maximize efficiency and minimize energy costs.
- **A concentrate (brine) disposal system** that would utilize the existing San Juan Creek Ocean Outfall (SJCOO), to return brine and treated process waste streams to the ocean with negligible impact on coastal and marine water quality. This would be achieved in part through blending in the outfall pipe with the existing wastewater stream from the J.B. Latham Wastewater Treatment Plant, and other regional treatment plants. Mixing desalination brine with existing wastewater treatment plant flow (a "comingled discharge") is the preferred method by state and federal regulators and is consistent with the State Board's Ocean Plan Amendment.
- **A product water storage tank and distribution system** that would feed into the District's local distribution system and, depending on plant capacity and District demands, other adjacent local and regional transmission pipelines that are located adjacent to the site. Desalinated product water from the Phase I Project could be conveyed entirely using existing District and local infrastructure with no off-site improvements other than a short connection to the District's existing local transmission lines.
- **All appurtenant facilities** (e.g. pump stations, valves and metering) as well as all construction, operation and maintenance activities associated with all Project facilities.
- **Offsite Electrical Transmission Facilities** provided by San Diego Gas & Electric Company (SDG&E). At this time, SDG&E has indicated that electrical service can be provided to the Phase I Project using existing facilities, with a short connection from the desalination site to underground electrical lines in Stonehill Drive.

SIGNIFICANT ENVIRONMENTAL IMPACTS:

The Draft EIR addresses Aesthetics, Air Quality, Biological Resources, Cultural Resources, Geology and Soils, Greenhouse Gas Emissions, Hazards and Hazardous Materials, Hydrology and Water Quality, Land Use and Planning, Noise, Public Services, Recreation, Transportation and Traffic, Tribal Cultural Resources and Utilities and Service Systems, as well as energy conservation, alternatives, potential growth-inducing impacts, and cumulative impacts. With implementation of various Project Design Features and EIR mitigation measures, the EIR has concluded that the Phase I Project (up to 5 MGD) would not have any "unavoidable significant impacts." The Regional Project, if pursued at a later date, could result in unavoidable significant impacts, although this is speculative at this time due to lack of Regional Project details.

CEQA also requires this NOA to specify if the Project site contains any listed toxic sites. The Project site does not contain sites identified as meeting the "Cortese List" requirement (Government Code Section 65962.5).

AGENCIES:

The District requests each Responsible and Trustee agency review the Draft EIR relevant to the agency's statutory responsibilities in connection with the proposed Project, in a manner consistent with California Code of Regulations, Title 14, Section 15087. Each agency may use the EIR prepared by the District when considering any permits that the agency must issue, or other approvals for the Project.

PUBLIC REVIEW PERIOD:

The Draft EIR is available for public review for a period of 60 days. In accordance with CEQA, should you have any comments, please provide written comments on the Draft EIR within the 60-day period between **May 23, 2018 to July 23, 2018.**

LOCATIONS WHERE DRAFT EIR IS AVAILABLE FOR PUBLIC REVIEW

An electronic PDF of the Draft EIR is available for download on the District's Project website at www.scwd.org/desal. In addition, during the 60-day public review period, hard copies of the Draft EIR and the documents referenced in the EIR will be available at the following locations:

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- Orange County Public Library, Dana Point Branch, 33841 Niguel Rd. Dana Point, CA 92629

PUBLIC COMMENTS:

The District requests your careful review and consideration of the Draft EIR, and invites ***written comments*** from interested agencies, persons, and organizations regarding environmental issues identified in the Draft EIR. Please indicate a contact person for your agency or organization. You may also provide oral or written comments in person at the ***Public Meeting*** noted below. Comments in response to this notice must be submitted to the District through close of business (5:00 PM) on **July 23, 2018.**

LEAD AGENCY CONTACT:

All comments should be submitted in writing to:

South Coast Water District
Attn: Mr. Rick Shintaku, PE – Acting General Manager, Chief Engineer
31592 West Street, Laguna Beach, CA 92651
(949) 499-4555

PUBLIC MEETING:

The District will conduct a public meeting in order to receive public comments on the Draft EIR. The meeting will be held at the following location, date and time:

Tuesday, June 26, 2018
6:00 p.m.
(ending no later than 8:00 p.m. or when discussion concludes)
Capistrano Unified School District
CUSD Education Center
33122 Valle Road, San Juan Capistrano, CA 92675
Phone: (949) 234-9200

Special Accommodations. Should you require special accommodations at the public meeting, such as for the hearing impaired or an English translator, please contact South Coast Water District no later than **June 19, 2018** (see contact information above).

AMENDED NOTICE OF AVAILABILITY OF A DRAFT EIR & PUBLIC MEETING NOTICE

SCH# 2016031038

FILED
Ernest J Dronenburg, Jr. Recorder County Clerk
JUN 05 2018
BY [signature] DEPUTY

Date: June 4, 2018

To: Reviewing Agencies, Organizations, and Interested Parties

Lead Agency: South Coast Water District

Subject: ***Amended Notice of Availability & Public Meeting Notice***

DOHENY OCEAN DESALINATION PROJECT DRAFT ENVIRONMENTAL IMPACT REPORT
State Clearinghouse No. 2016031038

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PROJECT SUMMARY:

The Draft EIR assesses the potential environmental effects of implementing a proposed ocean water desalination facility located in Dana Point, including subsurface intake wells proposed at Doheny State Beach and Capistrano Beach, and various conveyance lines connecting the intake and discharge facilities to existing District property located approximately ½ mile inland, adjacent to San Juan Creek. The District is only proposing to pursue permits and approvals for the initial Phase I Project, which would provide up to 5 million-gallons-per-day (MGD) of potable water, and is therefore addressed at a "project level" of CEQA review. The EIR also evaluates a potential future Regional Project of up to 15 MGD, at a programmatic level, since specific Regional Project partners, financing, and facilities have yet to be defined. Specific Project component descriptions are provided below and on the District's website at: **www.scwd.org/desal.**

AMENDED NOTICE OF AVAILABILITY OF A DRAFT EIR & PUBLIC MEETING NOTICE

SCH# 2016031038

ORIGINAL FILED

JUN 05 2018

LOS ANGELES, COUNTY CLERK

Date: June 4, 2018

To: Reviewing Agencies, Organizations, and Interested Parties

Lead Agency: South Coast Water District

Subject: ***Amended Notice of Availability & Public Meeting Notice***

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AMENDED NOTICE OF AVAILABILITY OF A DRAFT EIR & PUBLIC MEETING NOTICE

SCH# 2016031038

Date: June 4, 2018

To: Reviewing Agencies, Organizations, and Interested Parties

Lead Agency: South Coast Water District

Subject: ***Amended Notice of Availability & Public Meeting Notice***

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State Clearinghouse No. 2016031038

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NOTICE OF AVAILABILITY OF A DRAFT EIR & PUBLIC MEETING NOTICE

SCH# 2016031038

Date: May 23, 2018

To: Reviewing Agencies, Organizations, and Interested Parties

Lead Agency: South Coast Water District

Subject: ***Notice of Availability & Public Meeting Notice***

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- **A raw (ocean) water conveyance pipeline** that would deliver the subsurface intake system's ocean water to the desalination facility site.
- **A desalination facility** that would receive ocean feedwater at approximately 10 to 30 MGD, with a recovery rate of ~50% resulting in up to 5 to 15 MGD of potable drinking water (for the Phase I and Regional Project, respectively). The proposed desalination facility is located on the District's existing San Juan Creek Property site, on an industrial site located away from the beach but in close proximity to the subsurface intake wells. This facility siting is also consistent with state and federal regulator preference to minimize desalination facilities on the coast while being close enough to avoid lengthy raw water and brine conveyance pipelines. The desalination facility includes a variety of typical desalination process equipment and appurtenant facilities, such as pretreatment, seawater reverse osmosis (SWRO) membranes, an energy recovery system, post-treatment conditioning, solids handling and disposal, product water storage, electrical equipment, staff facilities, and connections to off-site brine disposal, sanitary sewer, and product water conveyance facilities. It is assumed there will be a utility power connection required; however, the District is also evaluating the feasibility of supplementing or replacing that supply with an alternative energy source. The desalination facility will include solar photovoltaic panels on flat rooftops where feasible. Other alternative energy sources being evaluated include natural-gas turbines and fuel cells to maximize efficiency and minimize energy costs.
- **A concentrate (brine) disposal system** that would utilize the existing San Juan Creek Ocean Outfall (SJCOO), to return brine and treated process waste streams to the ocean with negligible impact on coastal and marine water quality. This would be achieved in part through blending in the outfall pipe with the existing wastewater stream from the J.B. Latham Wastewater Treatment Plant, and other regional treatment plants. Mixing desalination brine with existing wastewater treatment plant flow (a "comingled discharge") is the preferred method by state and federal regulators and is consistent with the State Board's Ocean Plan Amendment.
- **A product water storage tank and distribution system** that would feed into the District's local distribution system and, depending on plant capacity and District demands, other adjacent local and regional transmission pipelines that are located adjacent to the site. Desalinated product water from the Phase I Project could be conveyed entirely using existing District and local infrastructure with no off-site improvements other than a short connection to the District's existing local transmission lines.
- **All appurtenant facilities** (e.g. pump stations, valves and metering) as well as all construction, operation and maintenance activities associated with all Project facilities.
- **Offsite Electrical Transmission Facilities** provided by San Diego Gas & Electric Company (SDG&E). At this time, SDG&E has indicated that electrical service can be provided to the Phase I Project using existing facilities, with a short connection from the desalination site to underground electrical lines in Stonehill Drive.

SIGNIFICANT ENVIRONMENTAL IMPACTS:

The Draft EIR addresses Aesthetics, Air Quality, Biological Resources, Cultural Resources, Geology and Soils, Greenhouse Gas Emissions, Hazards and Hazardous Materials, Hydrology and Water Quality, Land Use and Planning, Noise, Public Services, Recreation, Transportation and Traffic, Tribal Cultural Resources and Utilities and Service Systems, as well as energy conservation, alternatives, potential growth-inducing impacts, and cumulative impacts. With implementation of various Project Design Features and EIR mitigation measures, the EIR has concluded that the Phase I Project (up to 5 MGD) would not have any "unavoidable significant impacts." The Regional Project, if pursued at a later date, could result in unavoidable significant impacts, although this is speculative at this time due to lack of Regional Project details.

CEQA also requires this NOA to specify if the Project site contains any listed toxic sites. The Project site does not contain sites identified as meeting the "Cortese List" requirement (Government Code Section 65962.5).

AGENCIES:

The District requests each Responsible and Trustee agency review the Draft EIR relevant to the agency's statutory responsibilities in connection with the proposed Project, in a manner consistent with California Code of Regulations, Title 14, Section 15087. Each agency may use the EIR prepared by the District when considering any permits that the agency must issue, or other approvals for the Project.

PUBLIC REVIEW PERIOD:

The Draft EIR is available for public review for a period of 60 days. In accordance with CEQA, should you have any comments, please provide written comments on the Draft EIR within the 60-day period between **May 23, 2018 to July 23, 2018.**

LOCATIONS WHERE DRAFT EIR IS AVAILABLE FOR PUBLIC REVIEW

An electronic PDF of the Draft EIR is available for download on the District's Project website at www.scwd.org/desal. In addition, during the 60-day public review period, hard copies of the Draft EIR and the documents referenced in the EIR will be available at the following locations:

- South Coast Water District Offices, address noted below
- Orange County Public Library, Dana Point Branch, 33841 Niguel Rd. Dana Point, CA 92629

PUBLIC COMMENTS:

The District requests your careful review and consideration of the Draft EIR, and invites ***written comments*** from interested agencies, persons, and organizations regarding environmental issues identified in the Draft EIR. Please indicate a contact person for your agency or organization. You may also provide oral or written comments in person at the ***Public Meeting*** noted below. Comments in response to this notice must be submitted to the District through close of business (5:00 PM) on **July 23, 2018.**

LEAD AGENCY CONTACT:

All comments should be submitted in writing to:

South Coast Water District
Attn: Mr. Rick Shintaku, PE – Acting General Manager, Chief Engineer
31592 West Street, Laguna Beach, CA 92651
(949) 499-4555

PUBLIC MEETING:

The District will conduct a public meeting in order to receive public comments on the Draft EIR. The meeting will be held at the following location, date and time:

Tuesday, June 26, 2018
6:00 p.m.
(ending no later than 8:00 p.m. or when discussion concludes)
Capistrano Unified School District
CUSD Education Center
33122 Valle Road, San Juan Capistrano, CA 92675
Phone: (949) 234-9200

Special Accommodations. Should you require special accommodations at the public meeting, such as for the hearing impaired or an English translator, please contact South Coast Water District no later than **June 19, 2018** (see contact information above).

AMENDED NOTICE OF AVAILABILITY PUBLIC MEETING NO

SCH# 2016031038

Date: June 4, 2018

To: Reviewing Agencies, Organizations, and Interested Parties

Lead Agency: South Coast Water District

Subject: ***Amended Notice of Availability & Public Meeting Notice***

DOHENY OCEAN DESALINATION PROJECT DRAFT ENVIRONMENTAL IMPACT REPORT
State Clearinghouse No. 2016031038

The South Coast Water District (District) has prepared an Environmental Impact Report (EIR) pursuant to the California Public Resources Code and the California Environmental Quality Act (CEQA) to evaluate the environmental effects associated with the proposed Doheny Ocean Desalination Project (Project). This ***Amended*** Notice of Availability (***Amended*** NOA) has been issued to notify interested parties that the District is re-releasing the Draft EIR for public review and comment. The District initially released the Draft EIR on May 23, 2018. The printed copies of the Draft EIR, including those available at the library and District office, were complete. However, the District since discovered that at least some of the electronic copies of the Draft EIR did not contain certain exhibits due to a reprographic error. Therefore the District is re-releasing the Draft EIR, containing all exhibits. The Project itself is unchanged. The date of the public meeting (see details below) is unchanged. To ensure full opportunity for public review and comment, the District is extending the public comment period accordingly. ***The District is extending the deadline for public comment from July 23, 2018 to August 6, 2018 to allow for a full 60-day review period of the complete Draft EIR.*** The District is requesting comments on the Draft EIR from Responsible and Trustee agencies, interested public agencies, organizations, and the general public (pursuant to CEQA Guidelines §15087). The original Notice of Availability, dated May 23, 2018, is attached to this ***Amended*** NOA.

PROJECT SUMMARY:

The Draft EIR assesses the potential environmental effects of implementing a proposed ocean water desalination facility located in Dana Point, including subsurface intake wells proposed at Doheny State Beach and Capistrano Beach, and various conveyance lines connecting the intake and discharge facilities to existing District property located approximately ½ mile inland, adjacent to San Juan Creek. The District is only proposing to pursue permits and approvals for the initial Phase I Project, which would provide up to 5 million-gallons-per-day (MGD) of potable water, and is therefore addressed at a "project level" of CEQA review. The EIR also evaluates a potential future Regional Project of up to 15 MGD, at a programmatic level, since specific Regional Project partners, financing, and facilities have yet to be defined. Specific Project component descriptions are provided below and on the District's website at: www.scwd.org/desal.

- **A subsurface water intake system** consisting of subsurface slant wells that draw ocean water from offshore subsurface alluvial material (located below the ocean floor), while providing natural sand bed filtration and eliminating the entrainment and impingement of marine biota. This subsurface intake system is the recommended approach by state and federal regulators, and is consistent with the State Water Resource Control Board's (State Board or SWRCB) recently adopted Ocean Plan Amendment. The slant wells would be located and fully buried near the beach, in a study area encompassing Doheny State Beach and Capistrano Beach Park.
- **A raw (ocean) water conveyance pipeline** that would deliver the subsurface intake system's ocean water to the desalination facility site.
- **A desalination facility** that would receive ocean feedwater at approximately 10 to 30 MGD, with a recovery rate of ~50% resulting in up to 5 to 15 MGD of potable drinking water (for the Phase I and Regional Project, respectively). The proposed desalination facility is located on the District's existing San Juan Creek Property site, on an industrial site located away from the beach but in close proximity to the subsurface intake wells. This facility siting is also consistent with state and federal regulator preference to minimize desalination facilities on the coast while being close enough to avoid lengthy raw water and brine conveyance pipelines. The desalination facility includes a variety of typical desalination process equipment and appurtenant facilities, such as pretreatment, seawater reverse osmosis (SWRO) membranes, an energy recovery system, post-treatment conditioning, solids handling and disposal, product water storage, electrical equipment, staff facilities, and connections to off-site brine disposal, sanitary sewer, and product water conveyance facilities. It is assumed there will be a utility power connection required; however, the District is also evaluating the feasibility of supplementing or replacing that supply with an alternative energy source. The desalination facility will include solar photovoltaic panels on flat rooftops where feasible. Other alternative energy sources being evaluated include natural-gas turbines and fuel cells to maximize efficiency and minimize energy costs.
- **A concentrate (brine) disposal system** that would utilize the existing San Juan Creek Ocean Outfall (SJCOO), to return brine and treated process waste streams to the ocean with negligible impact on coastal and marine water quality. This would be achieved in part through blending in the outfall pipe with the existing wastewater stream from the J.B. Latham Wastewater Treatment Plant, and other regional treatment plants. Mixing desalination brine with existing wastewater treatment plant flow (a "comingled discharge") is the preferred method by state and federal regulators and is consistent with the State Board's Ocean Plan Amendment.
- **A product water storage tank and distribution system** that would feed into the District's local distribution system and, depending on plant capacity and District demands, other adjacent local and regional transmission pipelines that are located adjacent to the site. Desalinated product water from the Phase I Project could be conveyed entirely using existing District and local infrastructure with no off-site improvements other than a short connection to the District's existing local transmission lines.
- **All appurtenant facilities** (e.g. pump stations, valves and metering) as well as all construction, operation and maintenance activities associated with all Project facilities.

- **Offsite Electrical Transmission Facilities** provided by San Diego Gas & Electric Company (SDG&E). At this time, SDG&E has indicated that electrical service can be provided to the Phase I Project using existing facilities, with a short connection from the desalination site to underground electrical lines in Stonehill Drive.

SIGNIFICANT ENVIRONMENTAL IMPACTS:

The Draft EIR addresses Aesthetics, Air Quality, Biological Resources, Cultural Resources, Geology and Soils, Greenhouse Gas Emissions, Hazards and Hazardous Materials, Hydrology and Water Quality, Land Use and Planning, Noise, Public Services, Recreation, Transportation and Traffic, Tribal Cultural Resources and Utilities and Service Systems, as well as energy conservation, alternatives, potential growth-inducing impacts, and cumulative impacts. With implementation of various Project Design Features and EIR mitigation measures, the EIR has concluded that the Phase I Project (up to 5 MGD) would not have any "unavoidable significant impacts." The Regional Project, if pursued at a later date, could result in unavoidable significant impacts, although this is speculative at this time due to lack of Regional Project details.

CEQA also requires this NOA to specify if the Project site contains any listed toxic sites. The Project site does not contain sites identified as meeting the "Cortese List" requirement (Government Code Section 65962.5).

AGENCIES:

The District requests each Responsible and Trustee agency review the Draft EIR relevant to the agency's statutory responsibilities in connection with the proposed Project, in a manner consistent with California Code of Regulations, Title 14, Section 15087. Each agency may use the EIR prepared by the District when considering any permits that the agency must issue, or other approvals for the Project.

PUBLIC REVIEW PERIOD:

The Draft EIR is available for public review for a period of 60 days. In accordance with CEQA, should you have any comments, please provide written comments on the Draft EIR within the 60-day period between **June 6, 2018 to August 6, 2018.**

LOCATIONS WHERE DRAFT EIR IS AVAILABLE FOR PUBLIC REVIEW

An electronic PDF of the Draft EIR is available for download on the District's Project website at www.scwd.org/desal. In addition, during the 60-day public review period, hard copies of the Draft EIR and the documents referenced in the EIR will be available at the following locations:

- South Coast Water District Offices, address noted below
- Orange County Public Library, Dana Point Branch, 33841 Niguel Rd. Dana Point, CA 92629

PUBLIC COMMENTS:

The District requests your careful review and consideration of the Draft EIR, and invites ***written comments*** from interested agencies, persons, and organizations regarding environmental issues identified in the Draft EIR. Please indicate a contact person for your agency or organization. You may also provide oral or written comments in person at the ***Public Meeting*** noted below. Comments in response to this notice must be submitted to the District through close of business (5:00 PM) on **August 6, 2018**.

LEAD AGENCY CONTACT:

All comments should be submitted in writing to:

South Coast Water District
Attn: Mr. Rick Shintaku, PE – Acting General Manager, District Engineer
31592 West Street, Laguna Beach, CA 92651
(949) 499-4555

PUBLIC MEETING:

The District will conduct a public meeting in order to receive public comments on the Draft EIR. The meeting will be held at the following location, date and time:

Tuesday, June 26, 2018
6:30 p.m.
(ending no later than 8:00 p.m. or when discussion concludes)
Dana Hills High School - Gym
33333 Golden Lantern St, Dana Point, CA 92629
Phone: (949) 496-6666

Special Accommodations. Should you require special accommodations at the public meeting, such as for the hearing impaired or an English translator, please contact South Coast Water District no later than **June 19, 2018** (see contact information above).

NOTICE OF AVAILABILITY OF A DRAFT EIR & PUBLIC MEETING NOTICE

SCH# 2016031038

Date: May 23, 2018

To: Reviewing Agencies, Organizations, and Interested Parties

Lead Agency: South Coast Water District

Subject: ***Notice of Availability & Public Meeting Notice***

DOHENY OCEAN DESALINATION PROJECT DRAFT ENVIRONMENTAL IMPACT REPORT
State Clearinghouse No. 2016031038

The South Coast Water District (District) has prepared an Environmental Impact Report (EIR) pursuant to the California Public Resources Code and the California Environmental Quality Act (CEQA) to evaluate the environmental effects associated with the proposed Doheny Ocean Desalination Project (Project). This Notice of Availability (NOA) has been issued to notify interested parties that a Draft EIR is publicly available for review and comment. The District is requesting comments on the Draft EIR from Responsible and Trustee agencies, interested public agencies, organizations, and the general public (pursuant to CEQA Guidelines §15087).

PROJECT SUMMARY:

The Draft EIR assesses the potential environmental effects of implementing a proposed ocean water desalination facility located in Dana Point, including subsurface intake wells proposed at Doheny State Beach and Capistrano Beach, and various conveyance lines connecting the intake and discharge facilities to existing District property located approximately ½ mile inland, adjacent to San Juan Creek. The District is only proposing to pursue permits and approvals for the initial Phase I Project, which would provide up to 5 million-gallons-per-day (MGD) of potable water, and is therefore addressed at a "project level" of CEQA review. The EIR also evaluates a potential future Regional Project of up to 15 MGD, at a programmatic level, since specific Regional Project partners, financing, and facilities have yet to be defined. Specific Project component descriptions are provided below and on the District's website at: www.scwd.org/desal.

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- **All appurtenant facilities** (e.g. pump stations, valves and metering) as well as all construction, operation and maintenance activities associated with all Project facilities.
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SIGNIFICANT ENVIRONMENTAL IMPACTS:

The Draft EIR addresses Aesthetics, Air Quality, Biological Resources, Cultural Resources, Geology and Soils, Greenhouse Gas Emissions, Hazards and Hazardous Materials, Hydrology and Water Quality, Land Use and Planning, Noise, Public Services, Recreation, Transportation and Traffic, Tribal Cultural Resources and Utilities and Service Systems, as well as energy conservation, alternatives, potential growth-inducing impacts, and cumulative impacts. With implementation of various Project Design Features and EIR mitigation measures, the EIR has concluded that the Phase I Project (up to 5 MGD) would not have any "unavoidable significant impacts." The Regional Project, if pursued at a later date, could result in unavoidable significant impacts, although this is speculative at this time due to lack of Regional Project details.

CEQA also requires this NOA to specify if the Project site contains any listed toxic sites. The Project site does not contain sites identified as meeting the "Cortese List" requirement (Government Code Section 65962.5).

AGENCIES:

The District requests each Responsible and Trustee agency review the Draft EIR relevant to the agency's statutory responsibilities in connection with the proposed Project, in a manner consistent with California Code of Regulations, Title 14, Section 15087. Each agency may use the EIR prepared by the District when considering any permits that the agency must issue, or other approvals for the Project.

PUBLIC REVIEW PERIOD:

The Draft EIR is available for public review for a period of 60 days. In accordance with CEQA, should you have any comments, please provide written comments on the Draft EIR within the 60-day period between **May 23, 2018 to July 23, 2018.**

LOCATIONS WHERE DRAFT EIR IS AVAILABLE FOR PUBLIC REVIEW

An electronic PDF of the Draft EIR is available for download on the District's Project website at www.scwd.org/desal. In addition, during the 60-day public review period, hard copies of the Draft EIR and the documents referenced in the EIR will be available at the following locations:

- South Coast Water District Offices, address noted below
- Orange County Public Library, Dana Point Branch, 33841 Niguel Rd. Dana Point, CA 92629

PUBLIC COMMENTS:

The District requests your careful review and consideration of the Draft EIR, and invites ***written comments*** from interested agencies, persons, and organizations regarding environmental issues identified in the Draft EIR. Please indicate a contact person for your agency or organization. You may also provide oral or written comments in person at the ***Public Meeting*** noted below. Comments in response to this notice must be submitted to the District through close of business (5:00 PM) on **July 23, 2018.**

LEAD AGENCY CONTACT:

All comments should be submitted in writing to:

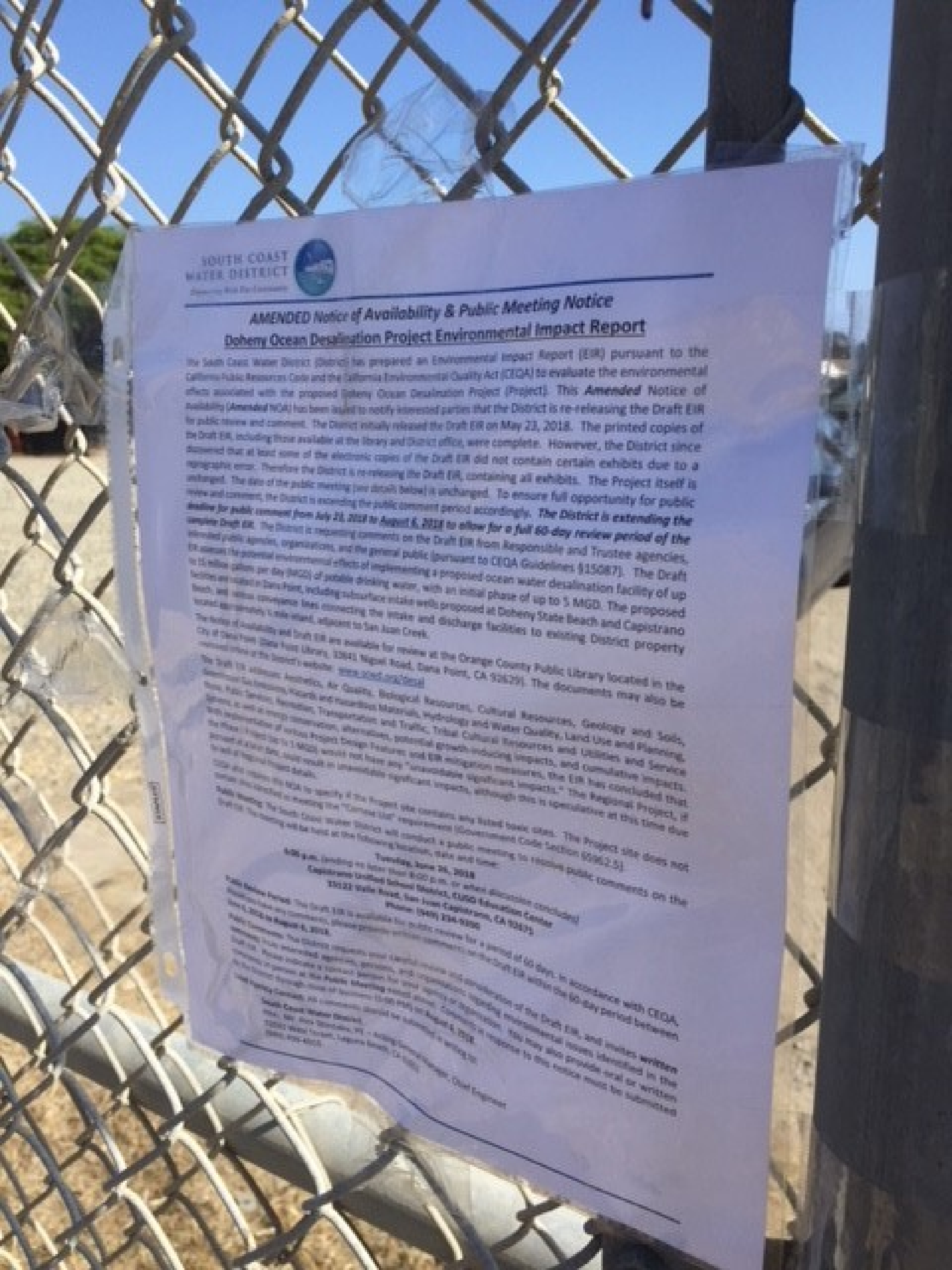
South Coast Water District
Attn: Mr. Rick Shintaku, PE – Acting General Manager, Chief Engineer
31592 West Street, Laguna Beach, CA 92651
(949) 499-4555

PUBLIC MEETING:

The District will conduct a public meeting in order to receive public comments on the Draft EIR. The meeting will be held at the following location, date and time:

Tuesday, June 26, 2018
6:00 p.m.
(ending no later than 8:00 p.m. or when discussion concludes)
Capistrano Unified School District
CUSD Education Center
33122 Valle Road, San Juan Capistrano, CA 92675
Phone: (949) 234-9200

Special Accommodations. Should you require special accommodations at the public meeting, such as for the hearing impaired or an English translator, please contact South Coast Water District no later than **June 19, 2018** (see contact information above).


SOUTH COAST
WATER DISTRICT
AMENDED Notice of Availability & Public Meeting Notice
Doheny Ocean Desalination Project Environmental Impact Report
The District initially released the Draft EIR on May 23, 2018. The printed copies of
However, the District since
did not contain certain exhibits due to a
containing all exhibits. The Project itself is
To ensure full opportunity for public
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to allow for a full 60-day review period of the
Responsible and Trustee agencies,
a proposed ocean water desalination facility of up
with an initial phase of up to 5 MGD. The proposed
Doheny State Beach and Capistrano
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listed toxic sites. The Project site does not
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will conduct a public meeting to receive public comments on the
Tuesday, June 26, 2018
Capistrano Unified School District, CUSD Education Center
33122 Valle Road, San Juan Capistrano, CA 92675
for a period of 60 days, in accordance with CEQA,
within the 60-day period between
of the Draft EIR, and invites written
environmental issues identified in the
may also provide oral or written
in response to this notice must be submitted
Chief Engineer

Transmittal Letters

- General

- State Clearinghouse

SOUTH COAST **WATER DISTRICT**

June 4, 2018

RE: *Amended* Notice of Availability for Doheny Ocean Desalination Project Draft Environmental Impact Report (Draft EIR) (SCH # 2016031038)

To Whom It May Concern:

On behalf of the South Coast Water District (District), please see the enclosed *Amended* Notice of Availability (*Amended* NOA) for the Draft EIR for the Doheny Ocean Desalination Project. The District has discovered that at least some of the electronic copies of the Draft EIR, released May 23, 2018, did not contain certain exhibits. Therefore, the enclosed CD, labeled "Updated Release June 4, 2018," contains the complete Draft EIR and associated appendices, including all exhibits. Please utilize the enclosed CD for your review of the Draft EIR. As explained in the *Amended* NOA, the District has extended the deadline for public comment to ***August 6, 2018***. Thank you.

Very truly yours,

Jody Brennan

Jody Brennan
Clerk of the Board

Mailing Address: *P.O. Box 30205, Laguna Niguel, CA 92607-0205*
Street Address: *31592 West Street, Laguna Beach, CA 92651*
Fax: *(949) 499-4256* ***Phone:*** *(949) 499-4555*

SOUTH COAST **WATER DISTRICT**

June 4, 2018

State Clearinghouse
1400 Tenth Street
Sacramento, California 95814

RE: ***Amended*** **Notice of Availability for Doheny Ocean Desalination Project Draft Environmental Impact Report (Draft EIR) (SCH # 2016031038)**

To Whom It May Concern:

On behalf of the South Coast Water District (District), please see the enclosed Amended Notice of Availability (Amended NOA) for the Draft EIR for the Doheny Ocean Desalination Project. The District has discovered that at least some of the electronic copies of the Draft EIR, released May 23, 2018, did not contain certain exhibits. Therefore, the enclosed CDs, labeled "Updated Release June 4, 2018," contain the complete Draft EIR and associated appendices, including all exhibits. Please distribute the enclosed CDs for state agency review of the Draft EIR.

Further, as explained in the Amended NOA, the District has extended the deadline for public comment to **August 6, 2018.** The District requests that the State Clearinghouse also extend the deadline for state agency review to the same date.

Very truly yours,

Jody Brennan

Jody Brennan
Clerk of the Board

Mailing Address: *P.O. Box 30205, Laguna Niguel, CA 92607-0205*
Street Address: *31592 West Street, Laguna Beach, CA 92651*
Fax: *(949) 499-4256* ***Phone:*** *(949) 499-4555*

Distribution Lists

- Government Agencies

- Non-Government Organizations

- Interested Parties

Government Agencies

Organization	Attention	Address	Phone Number	Email
FEDERAL				
NOAA National Marine Fisheries Services	Bryant Chesney	501 West Ocean Blvd. Suite 4200 Long Beach, CA 90802	562-980-4000 562-980-4197	bryant.chesney@noaa.gov Eric.Chavez@noaa.gov
NOAA National Marine Fisheries Services	Anthony P. Spina	501 West Ocean Blvd. Suite 4200 Long Beach, CA 90802		
U.S. Army Corps of Engineers	Regulatory Permitting-Orange County	U.S. Army Corps of Engineers 915 Wilshire Boulevard Suite 930 LOS ANGELES, CA 90017	213-452-3417	
U.S. Army Corps of Engineers	Therese O'Rourke-Bradford ACOE, Carlsbad Field Office	5900 La Place Court, Carlsbad, California, 92008		therese.o.bradford@usace.army.mil
U.S. Army Corps of Engineers	Cori Farrar ACOE Carlsbad Field Office	5900 La Place Court, Carlsbad, California, 92008		Corice.J.Farrar@usace.army.mil
U.S. Fish & Wildlife Service, Region 8	Environmental Services, April Evenas	Federal Building 2800 Cottage Way, Room W-2606 Sacramento, CA 95825	916-414-6464	april_evans@usace.army.mil stephen.M.Estes@usace.army.mil
Federal Emergency Mangement Agency	CEQA Review	1111 Broadway, Suite 1200, Oakland, CA 94607-4052		
STATE				
California Coastal Commission	Tom Luster	45 Fremont St #1900, San Francisco, CA 94105	(415) 904-5248	tluster@coastal.ca.gov
California Coastal Commission, South Coast District Office	Karl Schwing, South Coast District Manager	South Coast Area Office 200 Oceangate, Suite 1000 Long Beach, California 90802-4302	562-590-5071	Karl.Schwing@coastal.ca.gov
California Coastal Commission, South Coast District Office	Deborah Lee	South Coast Area Office 200 Oceangate, Suite 1000 Long Beach, California 90802-4302		DLee@coastal.ca.gov
California Department of Fish & Wildlife	Bill Paznokas	4949 Viewridge Avenue San Diego, CA 92123	(858) 467-4218	William.Paznokas@wildlife.ca.gov WPaznokas@dfg.ca.gov
California Department of Fish & Wildlife	Jennifer Edwards	3883 Ruffin Rd. San Diego, CA 92123	858-467-2717	jennifer.edwards@wildlife.ca.gov
California Department of Fish & Wildlife	Jennifer Turner	3883 Ruffin Rd. San Diego, CA 92123		Jennifer.Turner@wildlife.ca.gov
California Department of Fish & Wildlife	Loni Adams, Marine Environmental Scientist	3883 Ruffin Rd. San Diego, CA 92123	858-627-3985	loni.adams@wildlife.ca.gov
California Department of Fish & Wildlife, South Coast Region	Ed Pert, Regional Manager	3883 Ruffin Rd. San Diego, CA 92123	858-467-4201	epert@dfg.ca.gov
California Department of Parks and Recreation, Orange Coast District	Steve Scott	3030 Avenida del Presidente San Clemente, CA 92672	949-492-0802	sscott@parks.ca.gov
California Department of Transportation (Caltrans), Division 12	Maureen El Harake, Branch Chief	3347 Michelson Dr. Suite 100 Irvine, CA 92612	949-724-2000	Maureen.el.harake@dot.ca.gov
	Yatman Kwan			
California Department of Water Resources	Richard Mills, Section Chief, Water Recycling and Desalination	901 P Street, Room 313A , Third Floor. Sacramento, CA 94236-0001	(916) 651-0715	richard.mills@water.ca.gov
California Natural Resources Agency	Amy Vierra, Deputy Director, Ocean Protection Council	1416 Ninth Street, Suite 1311 Sacramento, CA 95814		Amy.Vierra@resources.ca.gov
California Public Utilities Commission	Chi Cheung To, P.E. Utilities Engineer	505 Van Ness Ave San Francisco, CA 94102	213.576.5766	cct@cpuc.ca.gov
California State Parks	Rich Haydon, State Park Superintendent	3030 Avenida del Presidente San Clemente, CA 92672	949-366-4895	rhaydon@parks.ca.gov; rich.haydon@parks.ca.gov
California State Parks, Orange County District	James Newland, Park and Recreation Specialist	3030 Avenida del Presidente San Clemente, CA 92672	949-607-9510	james.newlandd@parks.ca.gov
California Toxic Substances Control Department		5796 Corporate Ave, Cypress, CA 90630		
Native American Heritage Commission	Gayle Totton, Associate Analyst	1550 Harbor Blvd, Suite 100 West Sacramento, CA 95691	916-373-3710	nahc@nahc.ca.gov
Regional Water Quality Control Board, San Diego	David Gibson, Executive Officer	2375 Northside Drive, Suite 100 San Diego, CA 92108-2700	619-516-1990	David.Gibson@waterboards.ca.gov
Regional Water Quality Control Board, San Diego	Ben Neill, Brandi Outwin-Beals	2375 Northside Drive, Suite 100 San Diego, CA 92108-2700	(619) 521-3376	Ben.Neill@waterboards.ca.gov; Brandi.Outwin-Beals@waterboards.ca.gov
Southern California Regional Rail Authority		2703 Melbourne Ave, Pomona, CA 91767		
State Clearinghouse	Scott Morgan, Director	1400 Tenth Street, PO Box 3044 Sacramento, CA 95812-3044	916-445-0613	Scott.Morgan@opr.ca.gov
State Lands Commission	Cy Oggins, Chief, Environmental Planning and Management	100 Howe Avenue, Suite 100 South Sacramento, CA 95825	916-574-1900	cy.oggins@slc.ca.gov
State Lands Commission	Eric Gillies	100 Howe Avenue, Suite 100 South Sacramento, CA 95825		eric.gillies@slc.ca.gov
State Lands Commission	Alexandra Borack	100 Howe Avenue, Suite 100 South Sacramento, CA 95825		Alexandra.Borack@slc.ca.gov
State Office of Historic Preservation	CEQA Notice	1725 23rd Street, Suite 100 Sacramento, CA 95816	916-445-7000	
State Water Resources Control Board	Daniel Ellis	1001 I Street - P.O. Box 2815 Sacramento, CA 95812-2815		Daniel.Ellis@waterboards.ca.gov
State Water Resources Control Board	Scott Seyfried	1001 I Street - P.O. Box 2815 Sacramento, CA 95812-2815		Scott.Seyfried@waterboards.ca.gov
State Water Resources Control Board	Claire Waggoner and Kimberly Tenggardjaja	1001 I Street - P.O. Box 2815 Sacramento, CA 95812-2815	(916) 341-5858	Claire.Waggoner@waterboards.ca.gov Kimberly.Tenggardjaja@waterboards.ca.gov
State Water Resources Control Board	Mariela Carpio Obeso, Chief, Ocean Standards Unit	1001 I Street - P.O. Box 2815 Sacramento, CA 95812-2815		MarielaPaz.Carpio-Obeso@waterboards.ca.gov
State Water Resources Control Board - Division of Drinking Water	Oliver Pacifico	605 West Santa Ana Blvd, Building #28, Room 325 Santa Ana CA 92701	(714) 558-4410	oliver.pacifico@waterboards.ca.gov
State Water Resources Control Board - Drinking Water Revolving Fund	James Garrett	1001 I Street - P.O. Box 2815 Sacramento, CA 95812-2815		James.Garrett@waterboards.ca.gov
State Water Resources Control Board	Carol Atkins	1001 I Street - P.O. Box 2815 Sacramento, CA 95812-2815		Carol.Atkins@Waterboards.ca.gov
LOCAL				
California Air Resources Board	CEQA Notice	P.O. Box 2815 Sacramento, CA 95812		
Capistrano Bay District		35000 Beach Road, Capistrano Beach, CA 92624	9149-496-6576	drussell@capobay.org

Government Agencies

Organization	Attention	Address	Phone Number	Email
City of Brea, Water Division	CEQA Notice	One Civic Center Circle Brea, CA 92821	714-990-7687	
City of Buena Park, Water Services	CEQA Notice	6650 Beach Blvd. Buena Park, CA 90620	714-562-3721	
City of Dana Point, Community Development	CEQA Notice c/o Ursula Luna, Community Development Director	33282 Golden Lantern Dana Point, CA 92629	949-248-3567	mschneider@danapoint.org uluna@danapoint.org msinacori@danapoint.org
City of Dana Point, City Council	Richard Viczorek, Mayor	33282 Golden Lantern, Dana Point, CA 92629	(949) 248-3500	rviczorek@danapoint.org
City of Dana Point	Matt Schneider, Planning Manager	33282 Golden Lantern Dana Point, CA 92629	949 248 3560	
City of Dana Point	Matt Sinacori, Public Works Director	33282 Golden Lantern Dana Point, CA 92629	949 248 3574	
City of Fountain Valley, Water Department	CEQA Notice	18240 Ward Street Fountain Valley, CA 92708	714-593-4420	
City of Garden Grove, Water Services	CEQA Notice	13802 Newhope Street Garden Grove, CA 92843	714-741-5395	
City of Huntington Beach, Water Division	CEQA Notice	19001 Huntington St. Huntington Beach, CA 92648	714-536-5431	
City of La Habra, Water/Sewer Division	CEQA Notice	621 W Lambert Road La Habra, CA 90633	562-383-4170	
City of La Palma, Water Division	CEQA Notice	7822 Walker Street La Palma, CA 90623	714-690-3310	
City of Laguna Beach	IGR/CEQA Review	505 Forest Avenue, Laguna Beach, CA 92651		
City of Laguna Niguel	IGR/CEQA Review	30111 Crown Valley Parkway, Laguna Niguel, CA 92677		
City of Newport Beach, Public Works	CEQA Notice	100 Civic Center Drive, Bay 2D Newport Beach, CA 92660	949-644-3330	
City of Orange, Public Works	Water Service, CEQA Notice	189 S. Water Street Orange, CA 92866	714-288-2475	
City of San Clemente	IGR/CEQA Review	910 Calle Negocio # 100, San Clemente, CA 92673		
City of San Clemente, Utilities Services	CEQA Notice	100 Avenida Presidio 3, San Clemente, CA 92672	949-361-8200	
City of San Juan Capistrano	IGR/CEQA Review	32400 Paseo Adelanto, San Juan Capistrano, CA 92675		lmaravilla@planning.lacounty.gov
City of San Juan Capistrano, Utilities Department	Steve May, Public Works and Utilities Director	32400 Paseo Adelanto, San Juan Capistrano, CA 92675	949-234-4400	
City of Seal Beach, Public Works Department	Administrative & Engineering Division, CEQA Notice	211 8th Street Seal Beach, CA 90740	562-631-2527	
City of Tustin, Water Operations	CEQA Notice	300 Centennial Way, Tustin, CA 92780	714-573-3000	
City of Westminster, Water Division	CEQA Notice	8200 Westminster Blvd. Westminster, CA 92683	714-895-2876	
County of Los Angeles	IGR/CEQA Review	320 W Temple St, Los Angeles, CA 90012		kristi lovelady
County of Orange Planning	CEQA Notice	300 N. Flower St. Santa Ana, CA 92702	(714) 667-8845	chris.uzodiribe@ocpw.ocgov.com
County of Riverside	IGR/CEQA Review, Wendel Bugtai	4080 Lemon St. Riverside, CA 92501		
County of San Bernardino	IGR/CEQA Review, Tom Hudson	385 N Arrowhead Ave, San Bernardino, CA 92415		Tom.Hudson@lus.sbcounty.gov
County of San Diego	IGR/CEQA Review, Marc Cass	5510 Overland Ave, San Diego, CA 92123		Marc.Cass@sdcounty.ca.gov
East Orange County Water District	Lisa Ohlund, General Manager	185 N. McPherson Road Orange, CA 92869 2451	714-538-5815	lohlund@eocwd.com
El Toro Water District	CEQA Notice-Dennis Cafferty	Los Alisos Blvd. Lake Forest, CA 92630	949-837-7050	dcafferty@etwd.com
Emerald Bay Service District	Michael Dunbar, General Manager	600 Emerald Bay Laguna Beach, CA 92651	949-494-8571	mdunbar@ebservicedistrict.com
Golden State Water Company, West Orange County District	Dino Orbiso	1920 West Corporate Way Anaheim, CA 92801	714-535-8010	Dino.orbiso@gswater.com
Irvine Ranch Water District	Jo Ann Corey	15600 Sand Canyon Ave. Irvine, CA 92619-7000	949-453-5300	corey@irwd.com
Laguna Beach County Water District	CEQA Notice - David Youngblood	306 Third Street Laguna Beach, CA 92651	949-494-1041	dyoungblood@lbcwd.org
Mesa Water District	CEQA Notice	1965 Placentia Ave. Costa Mesa, CA 92627	949-631-1200	
MetroLink	Christos Sourmelis	One Gateway Plaza, 12th Floor, Los Angeles, CA 90012	909.392.8463	sourmelisc@scrra.net
MetroLink	Ron Mathieu	One Gateway Plaza, 12th Floor, Los Angeles, CA 90012		
Metropolitan Water District	Dee Bradshaw	700 North Alameda Street Los Angeles, CA 90012	(213) 217-6028	VBradshaw@mwdh2o.com
Metropolitan Water District	Warren Teitz	700 North Alameda Street Los Angeles, CA 90012	(213) 217-7418	wteitz@mwdh2o.com
Moulton Niguel Water District	Matt Collings, Asst. General Manager	27500 La Paz Road Laguna Niguel, CA 92677	949-448-4032	mcollings@mnwd.com
Municipal Water District of Orange County	Karl Seckel, Assistant Manager	18700 Ward Street Fountain Valley, CA 92708	714-963-3058	kseckel@mwdoc.com
Orange County Board of Supervisors	Supervisor Lisa Bartlett	34145 Pacific Coast Highway, Suite 710 Dana Point, Ca 92629	949-232-8882	Info@lisaforsupervisor.com
Orange County Board of Supervisors		333 W. Santa Ana Blvd., Santa Ana, CA 92701		
Orange County Transportation Authority	CEQA Notice	550 S. Main Street Orange, CA 92868		dphu@octa.net
Orange County Flood Control District	Ariel Corpuz	300 N. Flower Street, Suite 716 Santa Ana, CA 92703		ariel.corpuz@ocpw.ocgov.com
Orange County Health Care Agency	Anna Peters	405 W. Fifth Street Santa Ana, CA 92701	714-834-5150	apeters@ochca.com
Orange County Public Works	Robert McLean	300 N. Flower Street, Suite 716 Santa Ana, CA 92703		Robert.McLean@ocpw.ocgov.com
Orange County Public Works	Penny Lew	300 N. Flower Street, Suite 716 Santa Ana, CA 92703	714-647-3990	Penny.Lew@ocpw.ocgov.com
Orange County Public Works	James Tyler	300 N. Flower Street, Suite 716 Santa Ana, CA 92703	714-667-3210	James.Tyler@ocpw.ocgov.com
Orange County Public Works	Jeff Dickman	300 N. Flower Street, Suite 716 Santa Ana, CA 92703		jeff.dickman@ocpw.ocgov.com
Orange County Public Works	William Fegley	300 N. Flower Street, Suite 716 Santa Ana, CA 92703	949-923-2289	william.fegley@ocparks.com
Orange County Public Works	James Volz	300 N. Flower Street, Suite 716 Santa Ana, CA 92703	714-647-3904	james.volz@ocpw.ocgov.com
Orange County Public Works	Andy Ngo	300 N. Flower Street, Suite 716 Santa Ana, CA 92703	714-726-4297	andy.ngo@ocpw.ocgov.com
Orange County Public Works	Duc Nguyen	300 N. Flower Street, Suite 716 Santa Ana, CA 92703	714-955-0676	duc.nguyen@ocpw.ocgov.com
Orange County Public Works	Richard Vuong	300 N. Flower Street, Suite 716 Santa Ana, CA 92703		
Orange County Public Works	Laree Alonso	300 N. Flower Street, Suite 716 Santa Ana, CA 92703	714-647-9649	Laree.alonso@ocpw.ocgov.com
Orange County Public Works	Nardy Khan	300 N. Flower Street, Suite 716 Santa Ana, CA 92703	714-647-3906	nardy.khan@ocpw.ocgov.com
Orange County Parks	Susan Brodeur	13042 Old Myford Rd. Irvine, CA 92602	949-585-6448	susan.brodeur@ocparks.com
Orange County Parks	Kory McCain	13042 Old Myford Rd. Irvine, CA 92602	714-856-5772	kory.mccain@ocparks.com

Government Agencies

Organization	Attention	Address	Phone Number	Email
Orange County Parks	Eric E. Hull	13043 Old Myford Rd. Irvine, CA 92602		
Orange County Parks	Tom Townsend	13042 Old Myford Rd. Irvine, CA 92602	949-923-3747	tom.townsend@ocparks.com
Orange County Water District	CEQA Notice	18700 Ward St. Fountain Valley, CA 92708	714-378-3200	chris.uzodiribe@ocpw.ocgov.com
San Diego Gas and Electric		662 Camino de Los Mares, San Clemente, CA		
San Juan Basin Authority	Daniel Ferons-c/o Santa Margarita Water District	26111 Antonio Parkway Rancho Santa Margarita, CA 92688	949-459-6400	
	Norris Brandt			
Santa Margarita Water District	Dan Ferons, Don Bunts	26111 Antonio Parkway, Rancho Santa Margarita, CA 92688	949-459-6400	danf@smwd.com; donb@smwd.com
Serrano Water District	CEQA Notice	18021 East Lincoln Street Villa Park, CA 92861	714-538-0079	
South Orange County Wastewater Authority	Amber Baylor	34156 Del Obispo Street Dana Point, CA 92629	949-234-5400	abaylor@socwa.com
South Orange County Wastewater Authority	Jim Burror	34156 Del Obispo Street Dana Point, CA 92629	949-234-5400	jburror@socwa.com
South Coast Air Quality Management District	Jillian Wong, Program Supervisor	21865 East Copley Drive Diamond Bar, CA 91765-4178	909-396-2000	jcheng@aqmd.gov
	Lijin Sun			
South Orange County Wastewater Authority	Betty Burnett, General Manager	34156 Del Obispo St. Dana Point, CA 92629	949-234-5400	bburnett@socwa.com
Southern California Association of Governments	Attn: Intergovernmental Review	900 Wilshire Blvd, 17th floor, Los Angeles, California 90017	(213) 236-1800	sunl@scag.ca.gov
Trabuco Canyon Water District	CEQA Notice	32003 Dove Canyon Drive Trabuco Canyon, CA 92679	949-858-0277	
Yorba Linda Water District	CEQA Notice	1717 East Miraloma Ave. Placentia, CA 92870	714-777-3018	
TRIBES				
Juaneno Band of Mission Indians Acjachemen Natior	Matias Belardes, Chairperson	32161 Avenida Los Amigos, San Juan Capistrano, CA 92675	949-293-8522 949-444-4340	
Juaneno Band of Mission Indians Acjachemen Natior	Joyce Perry, Tribal Manager	4955 Paseo Segovia, Irvine, CA 92612	949-293-8522	kaamalam@gmail.com
Juaneno Band of Mission Indians Acjachemen Natior	Teresa Romero, Chairwoman	31411-A La Matanza Street, San Juan Capistrano, CA 92675	949-488-3484 530-354-5876	
Juaneno Band of Mission Indians	Sonia Johnston, Tribal Chairperson	P.O. Box 25628, Santa Ana, CA 92799		sonia.johnston@sbcglobal.net
San Gabriel Band of Mission Indians	Anthony Morales, Chief	P.O. Box 693 San Gabriel, CA 91778	626-483-3564	GTTribalcouncil@aol.com

Non-Government Agencies

NAME	CONTACT	ADDRESS	PHONE	EMAIL
California Coastal Protection Network	Susan Jordan	Po Box 30290, Santa Barbara, CA 93130	(805) 637-3037	left a message
California Coastkeeper Alliance	Sara Aminzadeh	156 Second Street, San Francisco, CA 94105	(415) 794-8422	sara@cacoastkeeper.org
California Union for Reliable Energy (CURE)	Sheila Sannadan	601 Gateway Boulevard, Suite 1000, South San Francisco CA 94080	650-589-1660	ssannadan@adambroadwell.com
California Union for Reliable Energy (CURE)	Alisha C. Pember	601 Gateway Boulevard, Suite 1000, South San Francisco CA 94080	650-589-1660	apember@adamsbroadwell.com
California Union for Reliable Energy (CURE)	Linda T. Sobczynski	601 Gateway Boulevard, Suite 1000, South San Francisco CA 94080	650-589-1660	lsobczynski@adamsbroadwell.com
Capo Bay CSD	Don Russell	35000 Beach Rd, Capistrano Beach, CA 92624		drussell@capobay.org
Capo Cares				capocares@gmail.com
Center for Biological Diversity	Ilene Anderson	660 S. Figueroa St., Suite 1000, Los Angeles, CA 90017	(323) 654-5943	ianderson@biologicaldiversity.org
Clean Water Now!	Roger Von Butow	P.O. Box 4711, Laguna Beach CA 92652 2796 Victoria Drive "B", Laguna Beach, CA 92651	(949) 280-2225	info@clean-water-now.org; rogerbutow@me.com
Coastal Environmental Rights Foundation	Monika Whisenhunt	1140 South Coast Hwy 101, Encinitas, CA 92024	(760) 942-8505	monika@coastlawgroup.com
Dana Point Headlands Conservancy		34681 Calle Paso Robles, Capo Beach, CA 92624	(949) 248-3527	left a message
Dana Point Library		33841 Niguel Road, Dana Point, CA 92629		
Doheny Longboard Surfing Association		P.O. Box 664, Dana Point, CA 92629	949-413-6250	no email, for inquiries only
Earth Resource Foundation	Stephanie Barger, Executive Dir.	1706 Newport Blvd. Ste. B, Costa Mesa, CA 95627	(949) 645-5163	stephanie.barger@earthresource.org
Ecology Center	Meg Hiesinger	32701 Alipaz St., San Juan Capistrano, CA 92675	(949) 443-4223	meg@theecologycenter.org
Endangered Habitats League	Dan Silver	8424 Santa Monica Blvd., Ste. A 592, Los Angeles, CA 90069	(213) 804-2750	dsilverla@me.com
Heal the Bay	Rita Kampalath	1444 9th St. Santa Monica, CA 90401	(310) 451-1500	rkampalath@healthebay.org
Laguna Ocean Foundation		P.O. Box 5247 Laguna Beach, CA 92652	no phone number	lagunaoceanfoundation@gmail.com
Marlborough Seaside Villas, Homeowners Association	Lazar Skundric, Vice President	910 Calle Negocio, Suite 200 San Clemente, CA 92763	949-661-7767	
MetroLink	Christos Sourmelis	One Gateway Plaza , 12th Floor, Los Angeles, CA 90012		
Mi Ocean	Patrick Fuscoe	16795 Von Karman Ave, Ste 100, Irvine, CA 92606	(949) 271-4386	left a message
Natural Resources Defense Council	Joe Geever	1314 Second St., Santa Monica, CA 90401	(310) 434-2300	nrdcinfo@nrdc.org
Orange County Coastkeeper	Colin Kelly, Senior Staff Attorney	3151 Airway Ave. Suite F-110, Costa Mesa, CA 92626	(714) 850-1965	colin@coastkeeper.org
Pacific Marine Mammal Center		20612 Laguna Canyon Rd, Laguna Beach, CA 92651	(949) 494-3050	info@pacificmmc.org
Planning and Conservation League	Jonas Minton	1107 9th St., Ste. 901, Sacramento, CA 95814	(916) 822-5631	pclmail@pcl.org
Residents for Responsible Desalination	Dave Hamilton	P.O. Box 5422, Huntington Beach, CA 92615-5422		de.hamilton@verizon.net
Residents for Responsible Desalination - Huntington Beach	Don Shultz	21352 Yarmouth Ln, Huntington Beach, CA 92646	(714) 840-8901	info@r4rd.org
San Clemente Green	Bill Hart	2837 Penasco, San Clemente, CA 92673	emailed for number	bill@sanclementegreen.org
Sea and Sage Audubon Society	Dr. Victor Leipzig	Audubon House, 5 Riparian View, Irvine, CA 92612	(714) 848-5394	vicleipzig@aol.com
Sierra Club	Penny Elia	3435 Wilshire Blvd. Ste. 660, Los Angeles, CA 90010	(213) 387-4287	greenp1@cox.net
Soto Resources	Me Joey Soto			joey@sotoresources.com
South Laguna Civic Association	Greg O'Loughlin	31558 Eagle Rock Way, Laguna Beach, CA 92651	(949) 415-1312	GregO@SouthLaguna.org
South Orange County Economic Coalition		27758 Santa Margarita Parkway #378 Mission Viejo, CA 92691	949.600.5470	brian@communicationslab.com
Southern California Coastal Water Research Project	Dr. Stephen Weisberg, Executive Dir.	3535 Harbor Blvd., Costa Mesa, CA 92626	(714) 755-3200	christinas@sccwrp.org
Southern California Watershed Alliance, Desal Response Group	Aubrey Bettencourt	PO Box 1267, Hanford, CA 93232	(559) 816-8691	aubrey@californiawateralliance.org
Surfrider Foundation	Rick Wilson Katie Day Mandy Sackett	P.O. Box 6010, San Clemente, CA 92674	(949) 492-8170	rwilson@surfrider.org kday@surfrider.org Mandy Sackett <msackett@surfrider.org>
Transition Laguna Beach		1215 Bluebird Canyon Dr. Laguna Beach, CA 92561	emailed for number	ecolagunabeach@gmail.com
Trout Unlimited	Robert Blankenship	P.O. Box 1977, Costa Mesa, CA 92628	(703) 522-0500	bob@hremcleanup.com; SouthCoastTU@gmail.com
Trout Unlimited	George Sutherland	419 Via Presa, San Clemente, CA 92672	(703)522-0500	scgsland@gmail.com
Wyland Foundation	Greg Stone	6b Macon, Irvine, CA 92620	(949) 643-7070	info@wylandfoundation.org

Interested Parties

NAME	ADDRESS	EMAIL	PHONE
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Pam Enqille	33701 Surfside Dana Point, CA 92629	pabenqelke@gmail.com	
Richard Gardner		capopalm@hotmail.com	
Catherine Gick	27045 Mill Pond Road, Capistrano Beach, CA 92624	geoplex@earthlink.net	
Dennis Heider	34112 Bedford Lane, Dana Point, CA 92629	dheider@heiderinspection.com	909-673-0292
Jim Mahaney		Mahaney.jim@sbcglobal.net	
Jan Mestion		jan@citysun.com	
George Miller	24005 Atun, Dana Point, CA 92629	papageo13@aol.com	
Bobby Young		by4golden@yahoo.com	
Matt Allaire	24911 Sea Aire, Dana Point, CA 92629	mallaire2112@gmail.com	
Jonelle Malloy	33112 Palo Alto Street, Dana Point 92629	jonelle1malloy@gmail.com	
Irene Bowie	31582 Wildwood Rd, Laguna Beach CA 92651	huladog1@earthlink.net	
Ray Hiemstra		ray@coastkeeper.org	
Kaye Romo	24351 La Cresta Drive, Dana Point, CA		
Bob Oakley		bob_oakley@msn.com	

Proof of Publication

- Orange County Register

- Dana Point Times

AFFIDAVIT OF PUBLICATION

STATE OF CALIFORNIA,)
) ss.
County of Orange)

I am a citizen of the United States and a resident of the County aforesaid; I am over the age of eighteen years, and not a party to or interested in the above entitled matter. I am the principal clerk of **The Orange County Register**, a newspaper of general circulation, published in the city of Santa Ana, County of Orange, and which newspaper has been adjudged to be a newspaper of general circulation by the Superior Court of the County of Orange, State of California, under the date of November 19, 1905, Case No. A-21046, that the notice, of which the annexed is a true printed copy, has been published in each regular and entire issue of said newspaper and not in any supplement thereof on the following dates, to wit:

June 6, 2018

"I certify (or declare) under the penalty of perjury under the laws of the State of California that the foregoing is true and correct":

Executed at Santa Ana, Orange County, California, on

Date: June 6, 2018

Signature: **Sandra Campos**

The Orange County Register
2190 S. Towne Centre Place
Anaheim, CA 92806
(714) 796-2209

PROOF OF PUBLICATION

AMENDED Notice of Availability & Public Meeting Notice
Doheny Ocean Desalination Project Environmental Impact Report

The South Coast Water District (District) has prepared an Environmental Impact Report (EIR) pursuant to the California Public Resources Code and the California Environmental Quality Act (CEQA) to evaluate the environmental effects associated with the proposed Doheny Ocean Desalination Project (Project). This **Amended** Notice of Availability (**Amended** NOA) has been issued to notify interested parties that the District is re-releasing the Draft EIR for public review and comment. The District initially released the Draft EIR on May 23, 2018. The printed copies of the Draft EIR, including those available at the library and District office, were complete. However, the District since discovered that at least some of the electronic copies of the Draft EIR did not contain certain exhibits due to a reprographic error. Therefore the District is re-releasing the Draft EIR, containing all exhibits. The Project itself is unchanged. The date of the public meeting (see details below) is unchanged. To ensure full opportunity for public review and comment, the District is extending the public comment period accordingly. **The District is extending the deadline for public comment from July 23, 2018 to August 6, 2018 to allow for a full 60-day review period of the complete Draft EIR.** The District is requesting comments on the Draft EIR from Responsible and Trustee agencies, interested public agencies, organizations, and the general public (pursuant to CEQA Guidelines §15087). The Draft EIR assesses the potential environmental effects of implementing a proposed ocean water desalination facility of up to 15 million gallons per day (MGD) of potable drinking water, with an initial phase of up to 5 MGD. The proposed facilities are located in Dana Point, including subsurface intake wells proposed at Doheny State Beach and Capistrano Beach, and various conveyance lines connecting the intake and discharge facilities to existing District property located approximately ½ mile inland, adjacent to San Juan Creek.

The Notice of Availability and Draft EIR are available for review at the Orange County Public Library located in the City of Dana Point (Dana Point Library, 33841 Niguel Road, Dana Point, CA 92629). The documents may also be reviewed online at the District's website: www.scwd.org/desal

The Draft EIR addresses Aesthetics, Air Quality, Biological Resources, Cultural Resources, Geology and Soils, Greenhouse Gas Emissions, Hazards and Hazardous Materials, Hydrology and Water Quality, Land Use and Planning, Noise, Public Services, Recreation, Transportation and Traffic, Tribal Cultural Resources and Utilities and Service Systems, as well as energy conservation, alternatives, potential growth-inducing impacts, and cumulative impacts. With implementation of various Project Design Features and EIR mitigation measures, the EIR has concluded that the Phase I Project (up to 5 MGD) would not have any "unavoidable significant impacts." The Regional Project, if pursued at a later date, could result in unavoidable significant impacts, although this is speculative at this time due to lack of Regional Project details.

CEQA also requires this NOA to specify if the Project site contains any listed toxic sites. The Project site does not contain sites identified as meeting the "Cortese List" requirement (Government Code Section 65962.5).

Public Meeting: The South Coast Water District will conduct a public meeting to receive public comments on the Draft EIR. The meeting will be held at the following location, date and time:

Tuesday, June 26, 2018
6:00 p.m. (ending no later than 8:00 p.m. or when discussion concludes)
Capistrano Unified School District, CUSD Education Center
33122 Valle Road, San Juan Capistrano, CA 92675
Phone: (949) 234-9200

Public Review Period: The Draft EIR is available for public review for a period of 60 days. In accordance with CEQA, should you have any comments, please provide written comments on the Draft EIR within the 60-day period between **June 6, 2018 to August 6, 2018.**

Public Comments: The District requests your careful review and consideration of the Draft EIR, and invites **written comments** from interested agencies, persons, and organizations regarding environmental issues identified in the Draft EIR. Please indicate a contact person for your agency or organization. You may also provide oral or written comments in person at the **Public Meeting** noted above. Comments in response to this notice must be submitted to the District through close of business (5:00 PM) on **August 6, 2018.**

Lead Agency Contact: All comments should be submitted in writing to:

South Coast Water District
Attn: Mr. Rick Shintaku, PE - Acting General Manager, Chief Engineer
31592 West Street, Laguna Beach, CA 92651
(949) 499-4555

Publish: Orange County Register June 6, 2018 11131580

(Cont. from page 8)

Monday | 11

KUNDALINI YOGA AND MEDITATION
8-9 a.m. Enjoy this ancient uplifting blend of spiritual and physical practice with Sukhmani, E-RYT. Kundalini Yoga incorporates physical movement to strengthen the nervous system and balance the glandular system as well as breathing techniques, meditation and the chanting of mantras to help process emotions and feelings, release stress, and help develop concentration and discipline of the mind. All levels are welcome and encouraged. Donation based – suggested $15. MindBody Wellness Club. 34207 Pacific Coast Hwy., Dana Point. www.dharmayogahouse.com.

Tuesday | 12

OCEAN INSTITUTE DISTINGUISHED SPEAKER SERIES: DR. GEOFF SHESTER
6 p.m. Join the Ocean Institute in the Samueli Conference Center for The Nicholas Endowment Distinguished Speaker Series with Dr. Geoff Shester. Dr. Geoff Shester is the senior scientist and California campaign director for Oceana, where he advocated for seafloor protection, including leading an expedition to gather footage and data on never-before-seen seafloor habitats. Light appetizers and refreshments from Above All Catering is included in the ticket price. For persons over the age of 21, your ticket will include one adult beverage, and a cash bar will be available for additional beverages. Tickets are $10. The Ocean Institute. 24200 Dana Point Harbor Dr. www.ocean-institute.org.

VETERANS OF FOREIGN WARS MEETING
6 p.m. All veterans who served in a combat zone are invited to attend their meeting on the second Tuesday of each month. Cannons Restaurant. 34344 Green Lantern, Dana Point. 949.248.1419. www.vfwpost9934.org.

Wednesday | 13

DOHENY STATE BEACH VISITOR CENTER
10 a.m.-4 p.m. Join the volunteer team at the Doheny State Beach Visitor Center. It's fun and easy. Open every day. If you have a few hours a month to come and welcome Doheny's many visitors, contact Kathy at volunteer@dohenystatebeach.org or Vicki at vicki.wiker@parks.ca.gov.

Thursday | 14

YAPPY HOUR
5 -8 p.m. Bark your calendar to join other canines and their companions to sniff and schmooze at Yappy Hour. Haute hounds enjoy lapping up libations, thanks to water in his favorite flavor – bacon, chicken, beef or vegan – while human guests relax with a glass of Mutt Lynch Unleashed Chardonnay, Merlot Over and Play Dead, Chateau d'Og Cabernet Sauvignon or a refreshing cocktail featuring Tito's Vodka. Beer and a selection of barbecue items are also available for purchase. Proceeds support The Veterans Initiative of Canine Companions for Independence. The Ritz-Carlton, Laguna Niguel. One Ritz Carlton Dr., Dana Point. 949.240.2000. www.ritzcarlton.com.

On Stage at the Coach House: Shannon Rae

Photo: Courtesy of Shannon Rae

BY EMILY RASMUSSEN, DANA POINT TIMES

Local country musician Shannon Rae, of San Clemente, is coming to The Coach House on Saturday, June 9, after recently releasing her second album *Lucky 13*.

Although *Lucky 13* was released Friday, June 1, Rae's performance at The Coach House will also be the official album release party. Rae's music is described as country rock, "similar to Keith Urban, Shania Twain and Carrie Underwood," according to a press release.

"It is my debut at The Coach House and I will be sharing the stage with The PettyBreakers, the No. 1 tribute band to Tom Petty," Rae said. "I play for one hour with my band and will also perform 'Stop Draggin My Heart Around' with The PettyBreakers."

Rae is accompanied by Dave Polich, producer, songwriter and keyboardist; Rick Gagliano with guitar and backing vocals; Ted Mentry on guitar; Rick Thibodeau on bass and backing vocals; and Tony Scarbrough on drums.

The six-piece band brings a rockin' high energy performance with tunes including "Both Guns Blazin'" and "Get Lucky" with Rae on lead vocals.

Rae has been a San Clemente resident for 16 years and also performs with her band 100 Proof. Tickets for The Coach House performance can be purchased directly through her website at www.shannonrae.com.

The doors open for the show at 6 p.m. and it starts at 8 p.m. on Saturday, June 9 at The Coach House, located at 33157 Camino Capistrano. Tickets are $20 and can also be bought by calling 949.496.8930 or online at www.thecoachhouse.com. **DP**

Section 4.2

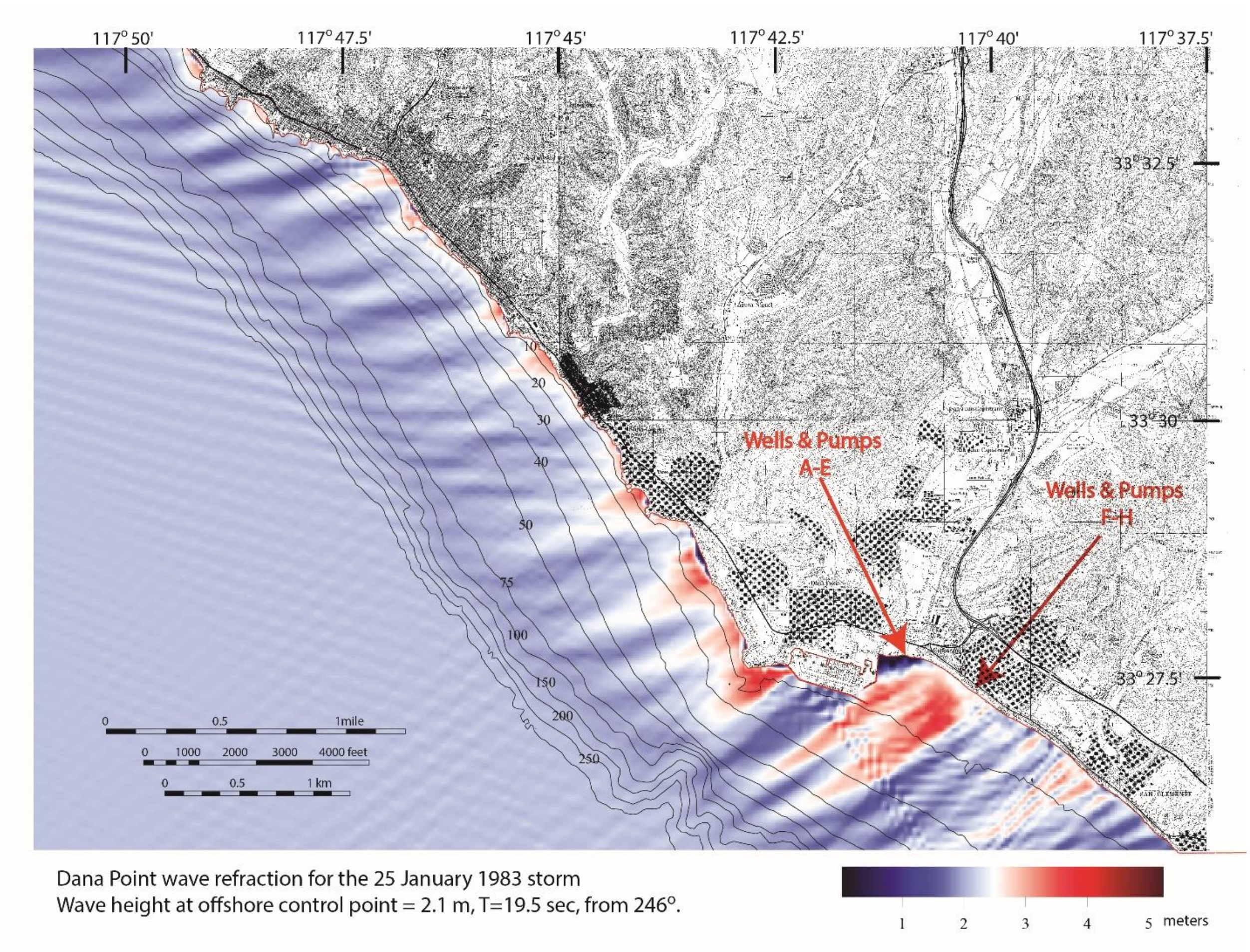
Final EIR Technical Analyses

APPENDIX 4.2.1

COASTAL HAZARDS ANALYSIS FOR THE FINAL EIR

Coastal Hazards Analysis for the Doheny Desalination Project for the Final EIR

By Scott A. Jenkins, Ph.D.



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°.

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

TABLE OF CONTENTS

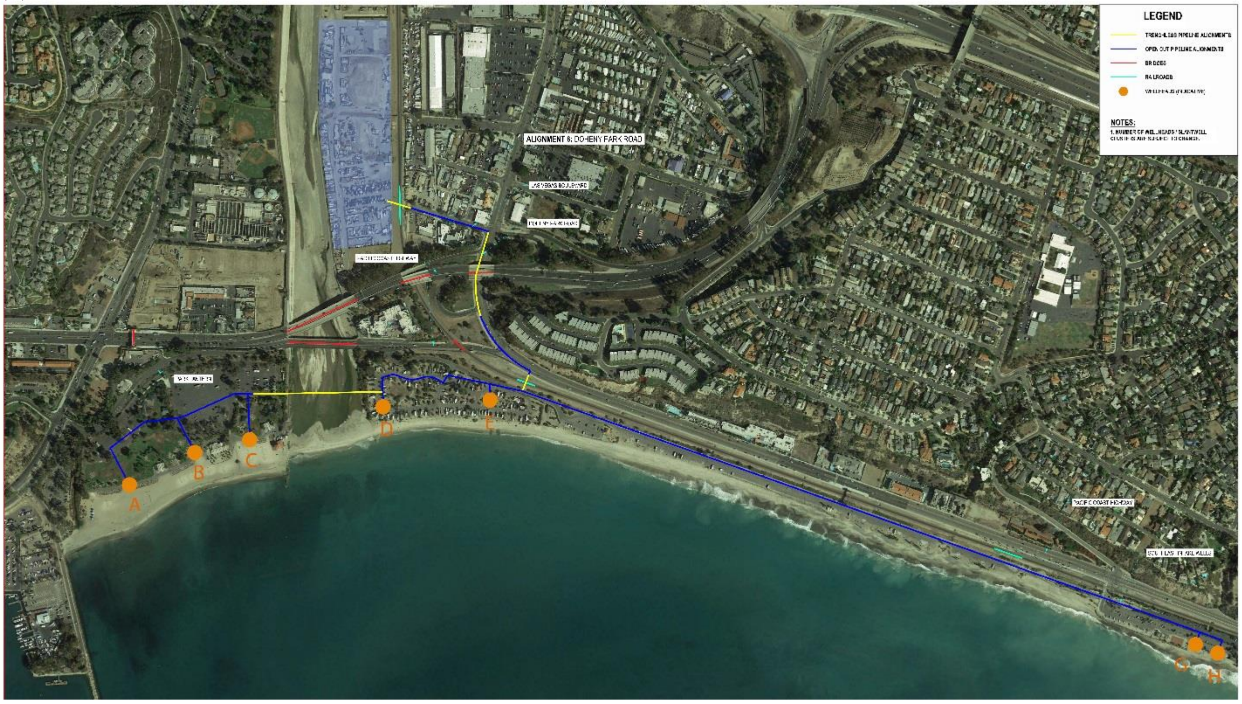
EXECUTIVE SUMMARY:

This 2019 study, prepared in response to comments for the Final EIR, provides further analysis to amplify the Coastal Hazards Analysis prepared in 2017 for the Draft EIR of the Doheny Desalination Project (DDP). That earlier work is being amplified herein in response to a revision of the *California Coastal Commission Sea Level Rise Policy Guidance* document that was originally released in August 2015, (CCC, 2015), but has been updated in July 2018 with new sea level rise projections. In addition, there have been minor adjustments in the locations of a number of the well heads and pump stations being proposed for the Doheny Desalination Project. The following study accounts for these intervening changes in policy guidance and minor modifications to the project description.

The primary analysis tool used in this study is the *Coastal Evolution Model* (CEM) developed at the Scripps Institution of Oceanography was used to evaluate Appendix-B requirements of the *California Coastal Commission Sea Level Rise Policy Guidance* document (CCC, 2015) for a sea level rise/coastal hazards analysis of the 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 grain 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 Oceanic and Atmospheric Administration (NOAA); and stream flow and sediment flux for the San Juan Creek from the United States Geological Survey (USGS) and the Federal Emergency Management Agency (FEMA). Sea level rise projections used in this study were based on the *best fit equation* from Appendix-B of the *California Coastal Commission 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 or tsunami concurrent with extreme ocean water levels and sea level rise are placed at two sites, namely Doheny State Beach and Capistrano Beach Park (cf. Figure ES-1a & b). At the Doheny Beach site, five potential locations are being evaluated for vaulted well heads with submersible pumps, including: ***Well Head A***, elevation 17 ft. NAVD, at 33°27'44.38"N, 117°41'16.32"W; ***Well Head B***, elevation 17 ft. NAVD, at 33°27'45.07"N, 117°41'10.30"W; ***Well Head C***, elevation 17 ft. NAVD at 33°27'45.12"N, 117°41'6.62"W; ***Well Head D***, at elevation 18 ft. NAVD at 33°27'44.48"N, 117°40'55.30"W; and ***Well Head E***, at elevation 18 ft. NAVD at 33°27'42.45"N, 117°40'47.33"W; (see Figure ES-1a). Two additional vaulted well heads with submersible pumps are being evaluated at the Capistrano Beach site (Figure ES-1b), which includes: ***Well Head G***, at elevation 18 ft. NAVD at 33°27'14.94"N, 117°39'59.91"W; and ***Well Head H***, at elevation 19 ft. NAVD at 33°27'13.17"N, 117°39'57.15"W.

This study is based on sea level rise projections appearing in Appendix-G, Table G-11, of the recently updated *California Coastal Commission Sea Level Rise Policy Guidance* document (CCC, 2018). This document provides no specific guidance on the redline frequency for flooding or inundation. In the absence of such guidance we have adopted FEMA standards for flooding

(a)



(b)

Figure ES-1: Critical shore-front infrastructure locations for the Doheny Desalination Project: a) Doheny State Beach site; and b) Capistrano Beach Park site 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 14 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 at present and at year 2100 sea levels, is summarized in Table ES-1a for structures at the Doheny Beach site and Table ES-1 b for the Capistrano Beach site. Inspection of Table ES-1a & b reveals that all the beach front well sites 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 for event return periods between 1 yr. and 100 yr. However, once we admit to 2100 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 low-range 2100 sea level projections, the three well sites on the north side of San Juan Creek (Well Heads A-C) and one of the wells at the Capistrano Beach site (Well Head G) will experience minor overtopping, even for a 1 year event if the beaches have been accreted by additional sands from water shed floods or still retain a built-out summer equilibrium beach profile, with overtopping rates of about Q'(1yr) = 0.038 cfs per lineal ft. of shoreline. However, if a 100-yr total water level event occurs during the low-range projection of 2100 sea levels, then all of the well sites will be overtopped to varying degrees if the beaches remain in an accreted condition with elevated berms and steep beach slopes. Under these beach conditions, overtopping rates will range from a high of Q'(100yr) = 0.094 cfs per lineal ft. of shoreline at Well Heads A- C, to a low of Q'(100yr) = 0.014 cfs/ft at Well Head H. Interestingly enough, none of the well heads would experience overtopping during a 100 year event when occurring during the low range 2100 sea levels if the beach were eroded, which would be the most likely condition during a 100-year event. 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-1a indicates the 100 year total water level events at the Doheny Beach site reach $TWL(100)$ = 21.9 ft. NAVD for the steeply sloping accreted beach conditions and $TWL(100)$ = 20.2 ft. NAVD for the eroded beach conditions. At the Capistrano Beach site, shoaling wave heights are higher and total water levels for a 100 year event superimposed on the high range projections for 2100 sea levels produce total water levels reaching $TWL(100)$ = 22.7 ft. NAVD for the steeply sloping accreted beach conditions and $TWL(100)$ = 21.1 ft. NAVD for the eroded beach conditions. Consequently, all beach front well head vaults for the Doheny Desalination Project will be overtopped when extreme waves happen concurrently with extreme ocean water levels that are superimposed on the high range of 2100 sea levels. The lowest lying well heads (Well Heads A-C) would experience the highest overtopping rates, ranging from Q'(100yr) = 0.216 cfs/ft. to 0.331 cfs/ft. depending on the eroded or accreted condition of Doheny State Beach. According Table VI-5-6 in the Coastal Engineering Manual (USACE, 2006) overtopping rates of this order of magnitude are very dangerous for pedestrian and vehicle traffic, and may cause structural damage to adjacent buildings, but the well heads and pumps for the Doheny Desalination project will be protected by steel vault enclosures. The smallest overtopping rates during the 100-year event at the high range

Table ES-1a: Doheny Beach Extremal Total Water Level ($*TWL$) and Overtopping Rates (Q')

	Well Head-A Elevation = 17 ft. NAVD	Well Head-B Elevation = 17 ft. NAVD	Well Head-C Elevation = 17 ft. NAVD	Well Head-D Elevation = 18 ft. NAVD	Well Head-E Elevation = 18 ft. NAVD
$*TWL(1)$ Present Sea Level (eroded/accreted)	8.7/10.5 ft. NAVD status = dry	8.7/10.5ft. NAVD status = dry	8.7/10.5ft. NAVD status = dry	8.7/10.5ft. NAVD status = dry	8.7/10.5ft. NAVD status = dry
$*Q'(1)$ 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.
$*TWL(1)$ 2100 Sea Level Low Range Projection (eroded/accreted)	12.3/14.1 ft. NAVD status = dry	12.3/14.1 ft. NAVD status = dry	12.3/14.1 ft. NAVD status = dry	12.3/14.1 ft. NAVD status = dry	12.3/14.1 ft. NAVD status = dry
$*Q'(1)$ 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.
$*TWL(1)$ 2100 Sea Level High Range Projection (eroded/accreted)	15.8/17.6 ft. NAVD status = flooded accreted beach	15.8/17.6 ft. NAVD status = flooded accreted beach	15.8/17.6 ft. NAVD status = flooded accreted beach	15.8/17.6 ft. NAVD status = dry	15.8/17.6 ft. NAVD status = dry
$*Q'(1)$ 2100 Sea Level High Range Projection (eroded/accreted)	0.0/0.038 cfs/ft.	0.0/0.038 cfs/ft.	0.0/0.038 cfs/ft.	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.
$**TWL(100)$ 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.
$**TWL(100)$ 2100 Sea Level Low Range Projection (eroded/accreted)	16.7/18.4 ft. NAVD status = flooded accreted beach	16.7/18.4 ft. NAVD status = flooded accreted beach	16.7/18.4 ft. NAVD status = flooded accreted beach	16.7/18.4 ft. NAVD status = flooded accreted beach	16.7/18.4 ft. NAVD status = flooded accreted beach
$**Q'(100)$ 2100 Sea Level Low Range Projection (eroded/accreted)	0.0/0.094 cfs/ft.	0.0/0.094 cfs/ft.	0.0/0.094 cfs/ft.	0.0/0.027 cfs/ft.	0.0/0.027 cfs/ft.
$**TWL(100)$ @ 2100 Sea Level High Range Projection (eroded/accreted)	20.2/21.9 ft. NAVD status = flooded	20.2/21.9 ft. NAVD status = flooded	20.2/21.9 ft. NAVD status = flooded	20.2/21.9 ft. NAVD status = flooded	20.2/21.9 ft. NAVD status = flooded
$**Q'(100)$ 2100 Sea Level High Range Projection (eroded/accreted)	0.216/0.331 cfs/ft.	0.216/0.331 cfs/ft.	0.216/0.331 cfs/ft.	0.149/0.263 cfs/ft.	0.149/0.263 cfs/ft.

*Evaluated for the 1-yr return period; ** Evaluated for the 100-yr return period

Table ES-1b: Capistrano Beach Extremal Total Water Level (TWL) and Overtopping Rates (Q')

	Well Head-G Elevation = 18 ft. NAVD	Well Head-H Elevation = 19 ft. NAVD
*$TWL(1)$ Present Sea Level (eroded/accreted)	9.7/11.5 ft. NAVD status = dry	9.7/11.5 ft. NAVD status = dry
*$Q'(1)$ Present Sea Level (eroded/accreted)	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.
*$TWL(1)$ 2100 Sea Level Low Range Projection (eroded/accreted)	13.3/15.1 ft. NAVD status = dry	13.3/15.1 ft. NAVD status = dry
*$Q'(1)$ 2100 Sea Level Low Range Projection (eroded/accreted)	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.
*$TWL(1)$ 2100 Sea Level High Range Projection (eroded/accreted)	16.8/18.6 ft. NAVD status = flooded accreted beach	16.8/18.6 ft. NAVD status = dry
*$Q'(1)$ 2100 Sea Level High Range Projection (eroded/accreted)	0.0/0.038 cfs/ft.	0.0/0.00 cfs/ft.
**$TWL(100)$ Present Sea Level (eroded/accreted)	14.0/15.6 ft. NAVD status = dry	14.0/15.6 ft. NAVD status = dry
**$Q'(100)$ Present Sea Level (eroded/accreted)	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.
**$TWL(100)$ 2100 Sea Level Low Range Projection (eroded/accreted)	17.6/19.2 ft. NAVD status = flooded accreted beach	17.6/19.2 ft. NAVD status = flooded accreted beach
**$Q'(100)$ 2100 Sea Level Low Range Projection (eroded/accreted)	0.0/0.081 cfs/ft.	0.0/0.014 cfs/ft.
**$TWL(100)$ @ 2100 Sea Level High Range Projection (eroded/accreted)	21.1/22.7 ft. NAVD status = flooded	21.1/22.7 ft. NAVD status = flooded
**$Q'(100)$ 2100 Sea Level High Range Projection (eroded/accreted)	0.209/0.318 cfs/ft.	0.142/0.250 cfs/ft.

*Evaluated for the 1-yr return period; ** Evaluated for the 100-yr return period

2100 sea level projections will occur at the highest located well head (Well Head H) at the Capistrano Beach site where overtopping rates will range from Q'(100yr) = 0.142 cfs/ft. to 0.250 cfs/ft. While these overtopping rates are still dangerous to pedestrian and vehicle traffic, they are easily managed by the steel vault enclosures of the well heads and pumps being placed at Capistrano Beach.

Tsunami induced erosion, runup, and inundation were analyzed for the Doheny and Capistrano Beaches 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 analysis was based on numerical refraction/diffraction codes for a shoaling solitary wave. 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 and Capistrano Beaches. 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 Table ES-2a for the Doheny Beach site, and Table ES-2b for the Capistrano Beach site. These tables indicate that all of the shore facilities of the Doheny Desalination Project are above tsunami inundation levels at present sea level. However, all of the well heads at both Doheny and Capistrano Beaches would suffer some degree of tsunami overtopping if concurrent with 2100 sea levels, and the overtopping rates could be quite severe, especially for the high 2100 sea level rise projections. At the low range of 2100 sea level projections, total water levels would reach *TWL* = 18.82 ft. NAVD at Doheny Beach and *TWL* = 18.83 ft. NAVD at Capistrano Beach. Well Heads A-C at Doheny Beach would experience the highest overtopping surges of Q' = 1.142 cfs/ft while Well Head G at Capistrano Beach would remain high and dry. However, if the tsunami occurred atop the high range sea level rise projections for year 2100, then total water levels would reach *TWL* = 22.31 ft. NAVD at Doheny Beach and *TWL* = 22.4 ft. NAVD at Capistrano Beach, sufficient to overtop all the well sites of the Doheny Desalination project. In this case the tsunami surge could produce very high, although short-lived, overtopping rates reaching a maximum of Q' = 5.691 cfs/ft at Well Heads A-C on Doheny Beach and a minimum of Q' = 2.916 cfs/ft at Well Head H on Capistrano Beach. Undoubtedly, the steel vault enclosures of the well heads can be designed to withstand these high surge rates, but particular attention should be given to the foundations of the vaults to assure those foundations have adequate depth to prevent undercutting by scour. 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 inundated by a shoaling tsunami solitary wave.

Table ES-2a: Tsunami Total Water Level (*TWL*) and Overtopping Rates (Q') Analysis at the Doheny Beach Site

	Well Head-A Elevation = 17 ft. NAVD	Well Head-B Elevation = 17 ft. NAVD	Well Head-C Elevation = 17 ft. NAVD	Well Head-D Elevation = 18 ft. NAVD	Well Head-E 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 cfs/ft.	0.0 cfs/ft.	0.0 cfs/ft.	0.0 cfs/ft.	0.0 cfs/ft.
TWL 2100 Sea Level Low Range Projection	18.82 ft. NAVD status = flooded	18.82 ft. NAVD status = flooded	18.82 ft. NAVD status = flooded	18.82 ft. NAVD status = flooded	18.82 ft. NAVD status = flooded
Q' 2100 Sea Level Low Range Projection	1.142 cfs/ft.	1.142 cfs/ft.	1.142 cfs/ft.	0.345 cfs/ft.	0.345 cfs/ft.
TWL @ 2100 Sea Level High Range Projection	22.31 ft. NAVD status = flooded	22.31 ft. NAVD status = flooded	22.31 ft. NAVD status = flooded	22.31 ft. NAVD status = flooded	22.31 ft. NAVD status = flooded
Q' 2100 Sea Level High Range Projection	5.691 cfs/ft.	5.691 cfs/ft.	5.691 cfs/ft.	4.162 cfs/ft.	4.162 cfs/ft.

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

Table ES-2b: Tsunami Total Water Level (*TWL*) and Overtopping Rates (Q') Analysis at the Capistrano Beach Site

	Well Head-G Elevation = 18 ft. NAVD eroded/accreted	Well Head-H Elevation = 19 ft. NAVD eroded/accreted
**TWL* Present Sea Level (eroded)	15.3 ft. NAVD status = dry	15.3 ft. NAVD status = dry
* Q' Present Sea Level (eroded)	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.
**TWL* 2100 Sea Level Low Range Projection (eroded)	18.83 ft. NAVD status = flooded	18.83 ft. NAVD status = dry
* Q' 2100 Sea Level Low Range Projection (eroded)	0.352 cfs/ft.	0.0/0.0 cfs/ft.
**TWL* 2100 Sea Level High Range Projection (eroded)	22.4 ft. NAVD status = flooded	22.4 ft. NAVD status = flooded
* Q' 2100 Sea Level High Range Projection (eroded)	4.293 cfs/ft.	2.916 cfs/ft.

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

Coastal Hazards Analysis for the Doheny Desalination Project for the Final EIR

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-channel 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-channel 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 sea levels; 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.

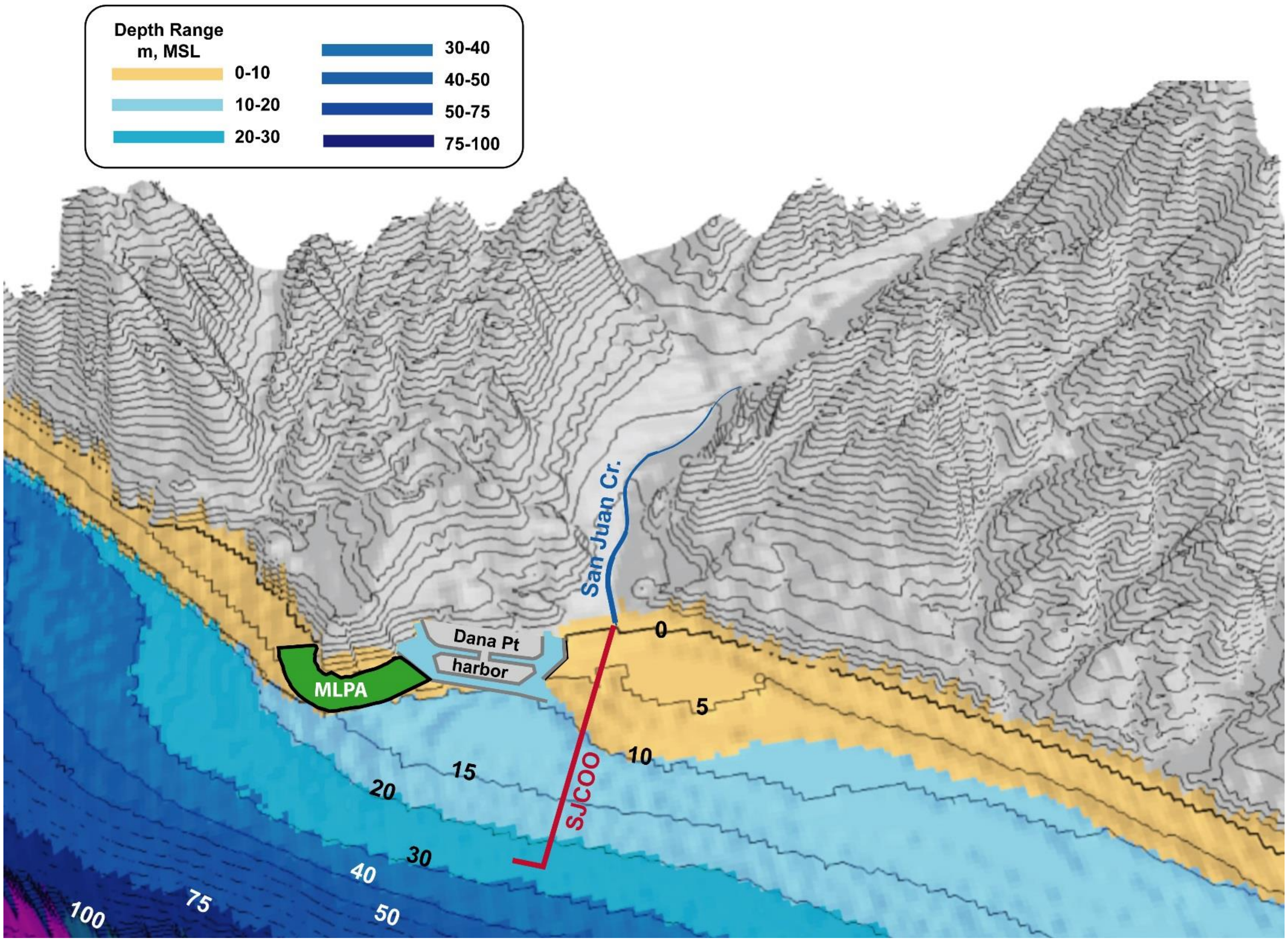


Figure 1.1: Project site map in GIS. Bathymetry contours in meters MSL. Data from GEODAS 3 arc-second database

2) Regulatory Requirements:

The *California Coastal Commision Sea Level Rise Guidance Policy Guidance* document (CCC, 2015) and CCC (2018) 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 interpolation[100], 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. 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 present analysis will use 2100 as the ultimate planning horizon for a critical infrastructure project.

Originally, 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). In our previous study, we selected 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), which appear as the cyan colored curve in Figure 3.1 for the low-range projection; and the magenta colored curve for the high range projection. For the 2100 planning horizon, sea level rise was originally projected to range from 1.37 ft to 5.50 ft. However, in the updated sea level rise policy guidance document, (CCC, 2018), equations (1) and (2) were abandoned in favor of a water level province tabulation centered around NOAA tide gage stations having long periods of record. The Doheny Desalination Projects lies in the La Jolla tide gage water level province, for which sea level rise projections are listed in Table G-11 in Appendix-G of CCC (2018). These new projections are plotted in Figure 3.1 as the blue curve for the low range projections and the red curve for the high range projections. Clearly the clarified sea level rise curves in Figure 3.1 project significantly higher future sea levels, particularly for the low range estimates. For the 2100 planning horizon sea level rise is projected range from 3.6 ft to 7.1 ft. The low range projection represents a 17% probability that sea level rise exceeds these values; while the high range projection represents a 0.5% probability that sea level rise exceeds these values. These values will be used in the calculations of extreme total water levels (TWL's) in the following sections.

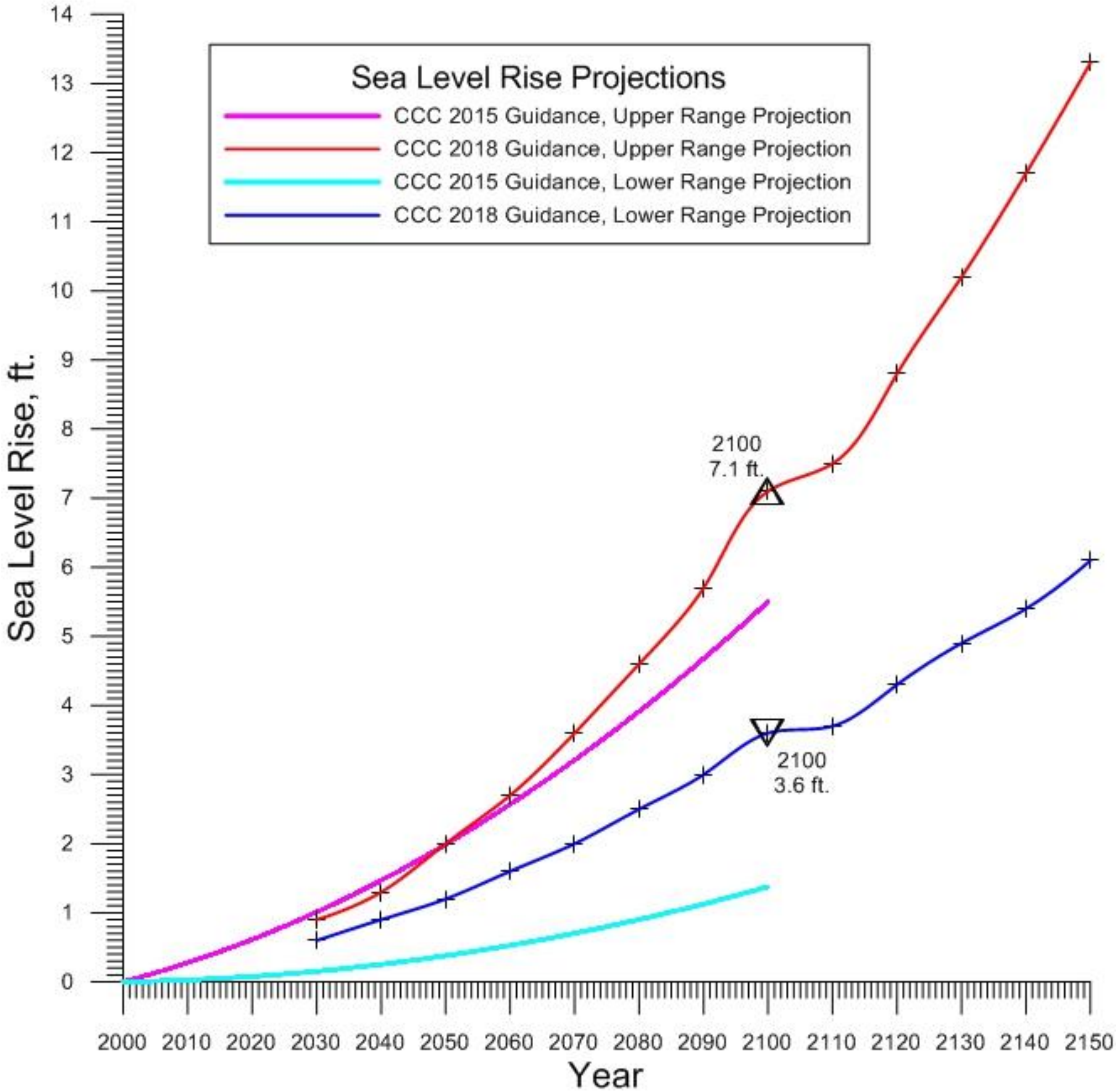


Figure 3.1: Range of sea level rise projections from the California Coastal Commission sea level rise guidance document, (CCC, 2018, Appendix-G). The 2100 planning horizon is indicated by symbols on the upper and lower range curves. Blue curve represents a 17% probability that sea level rise exceeds these values; red curve represents a 0.5% probability that sea level rise exceeds these values

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 CCC, (2015) and CCC (2018). 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 was not used as the basis for a water level province in Appendix-G of CCC (2018). 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:

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

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

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

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

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 :

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

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.

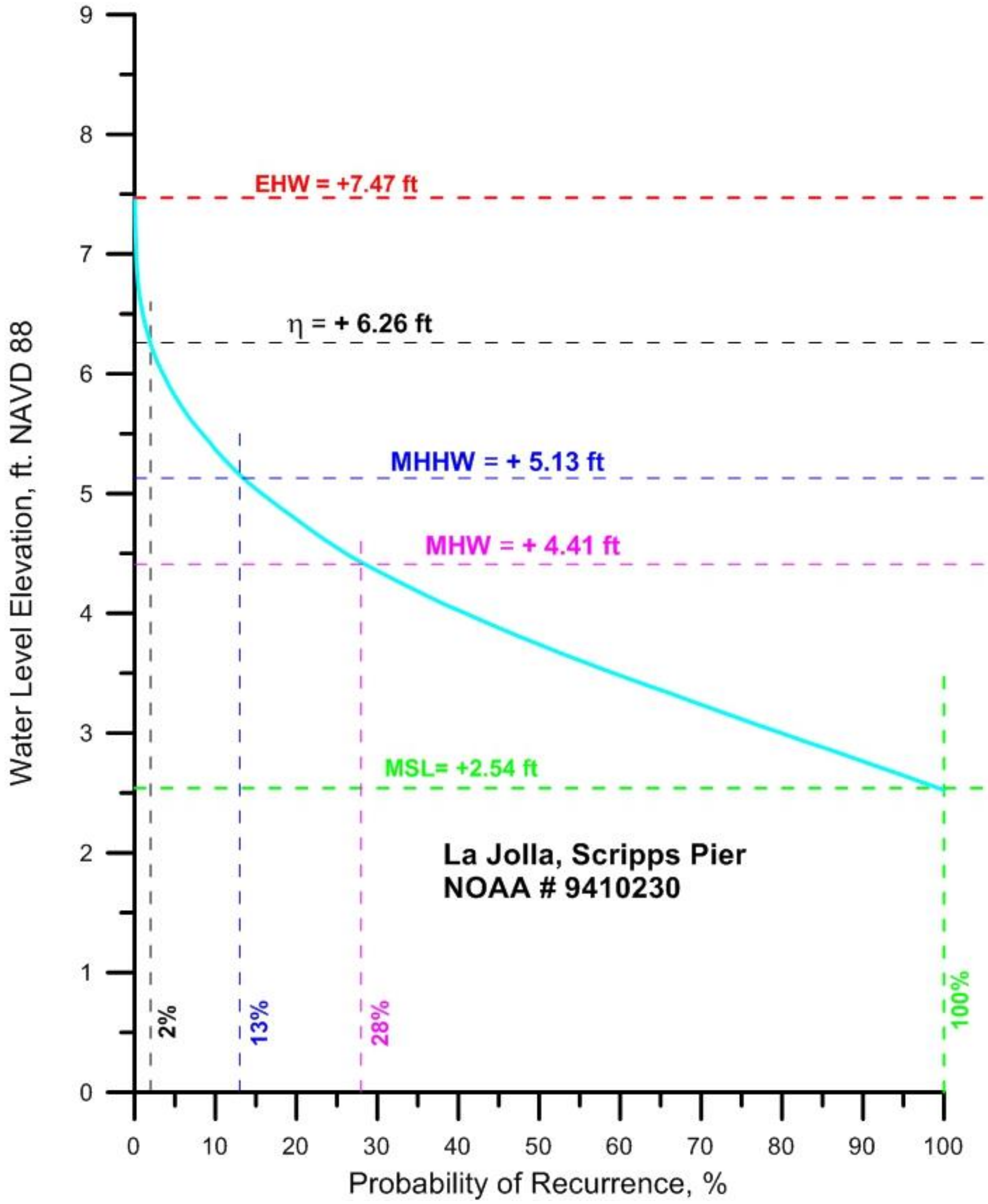


Figure 4.1: Hydroperiod function of still-water level elevations at present sea level, based on ocean water level measurements at the Scripps Pier tide gage station, NOAA #9410230, for the period of record 1924-2016. Tidal datums based on the 1983-2001 tidal epoch (latest datum analysis period).

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

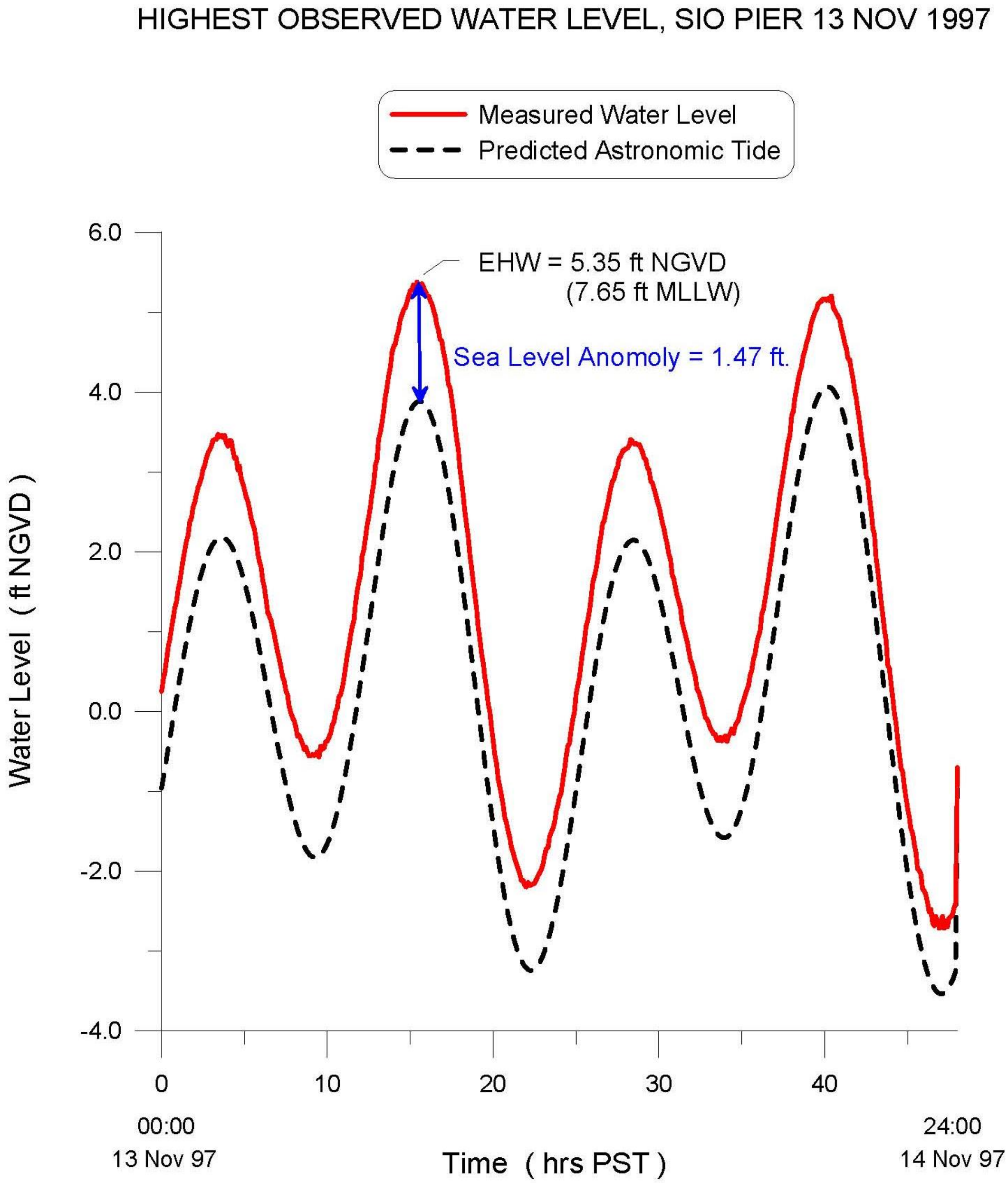


Figure 4.2: Comparison of measured ocean water level (red) at the Scripps Pier tide gage vs. predicted water level based on tidal constituents (black dashed) for the extreme high water event of 13 November 1997 (from Jenkins and Wasyl, 2005).

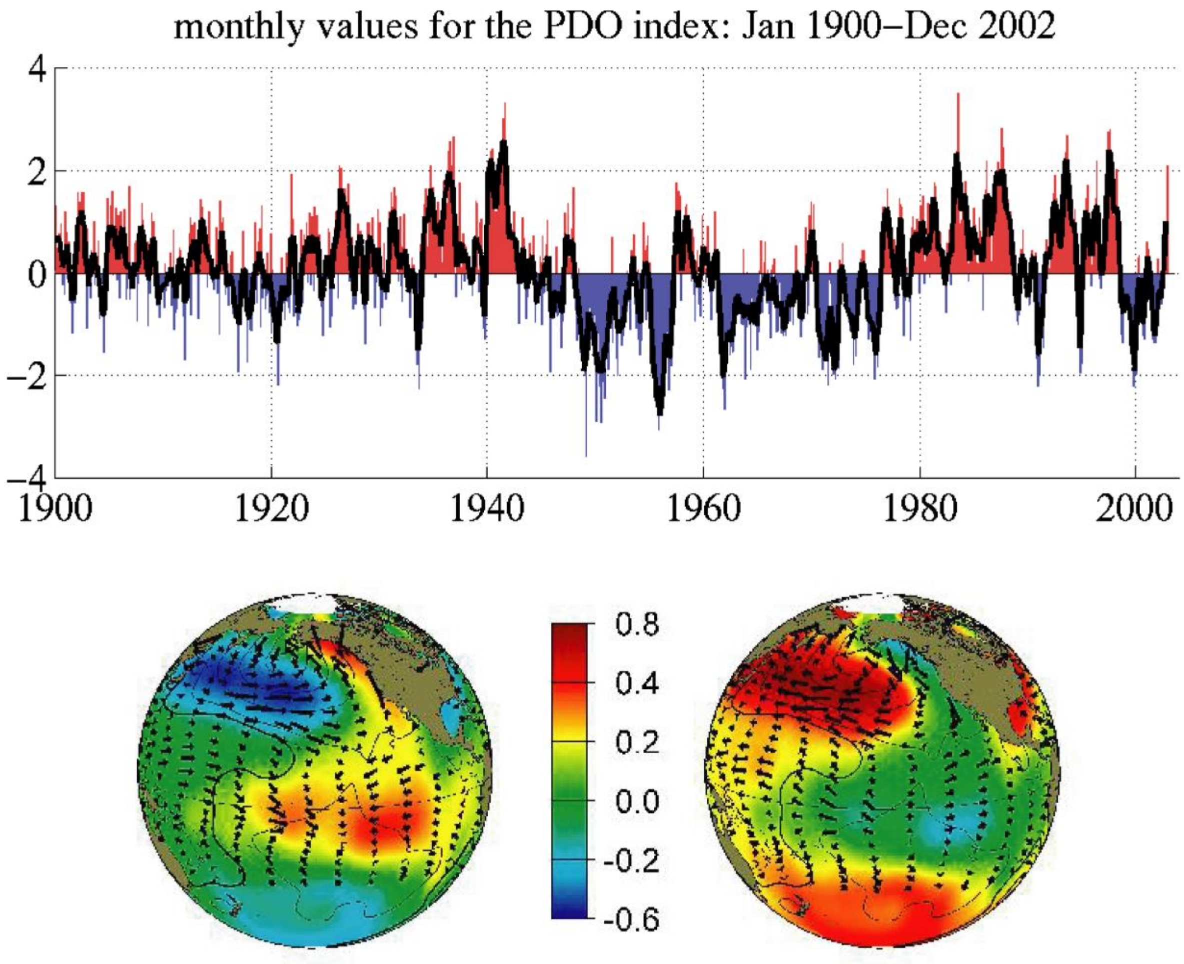


Figure 4.3. Typical wintertime Sea Surface Temperature (colors), Sea Level Pressure (contours) and surface wind stress (arrows) anomaly patterns during warm and cool phases of PDO.

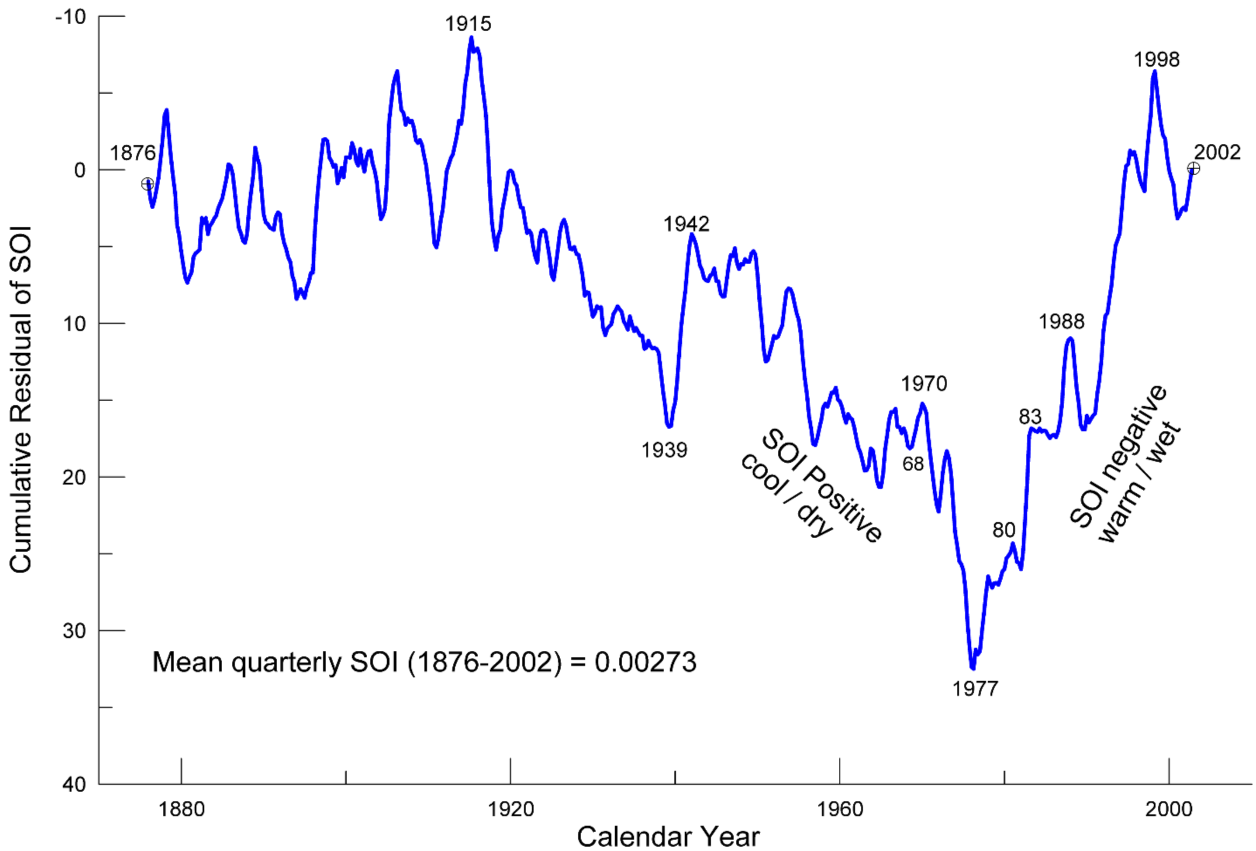


Figure 4.4. Cumulative residual of quarterly values of Southern Oscillation Index (SOI) [data from Australian Commonwealth Bureau of Meteorology].

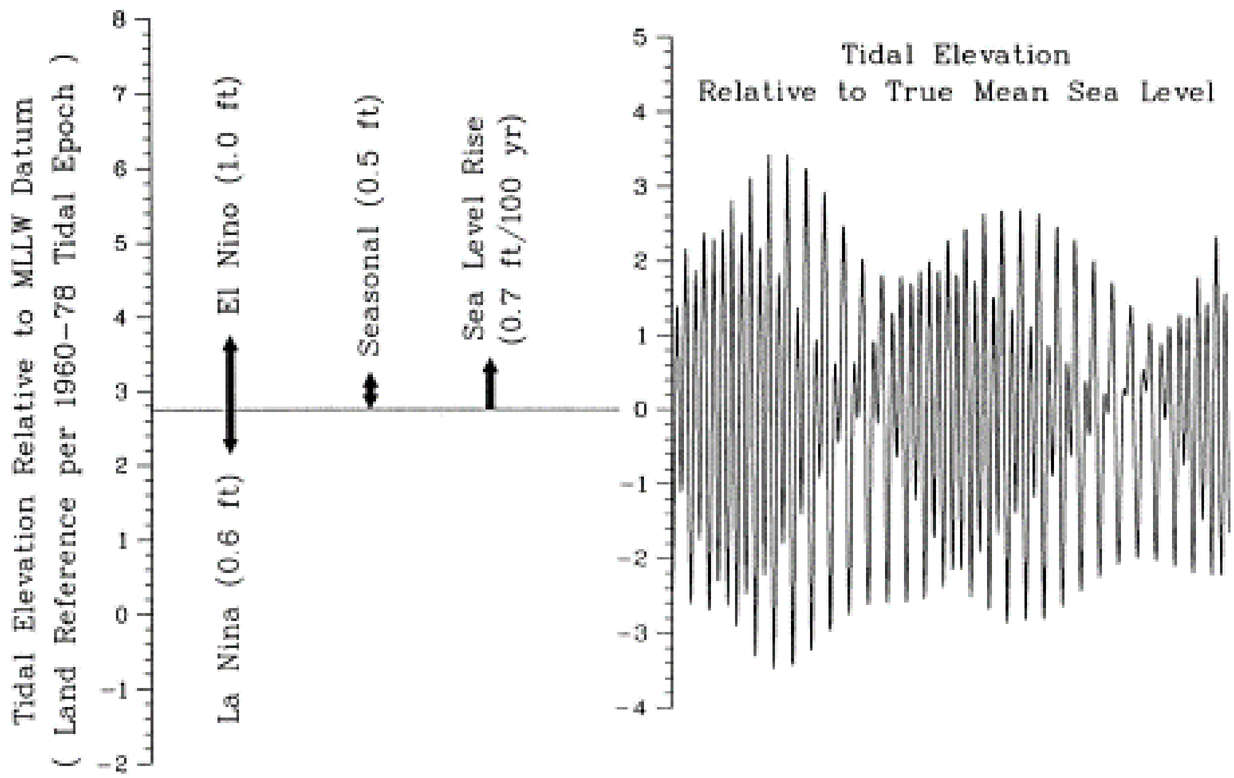


Figure 4.5 Schematic decomposition ocean water levels according to astronomic tides, and seasonal and ENSO/PDO cycle effects on steric sea levels.

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 2100 sea level in Figure 4.6.

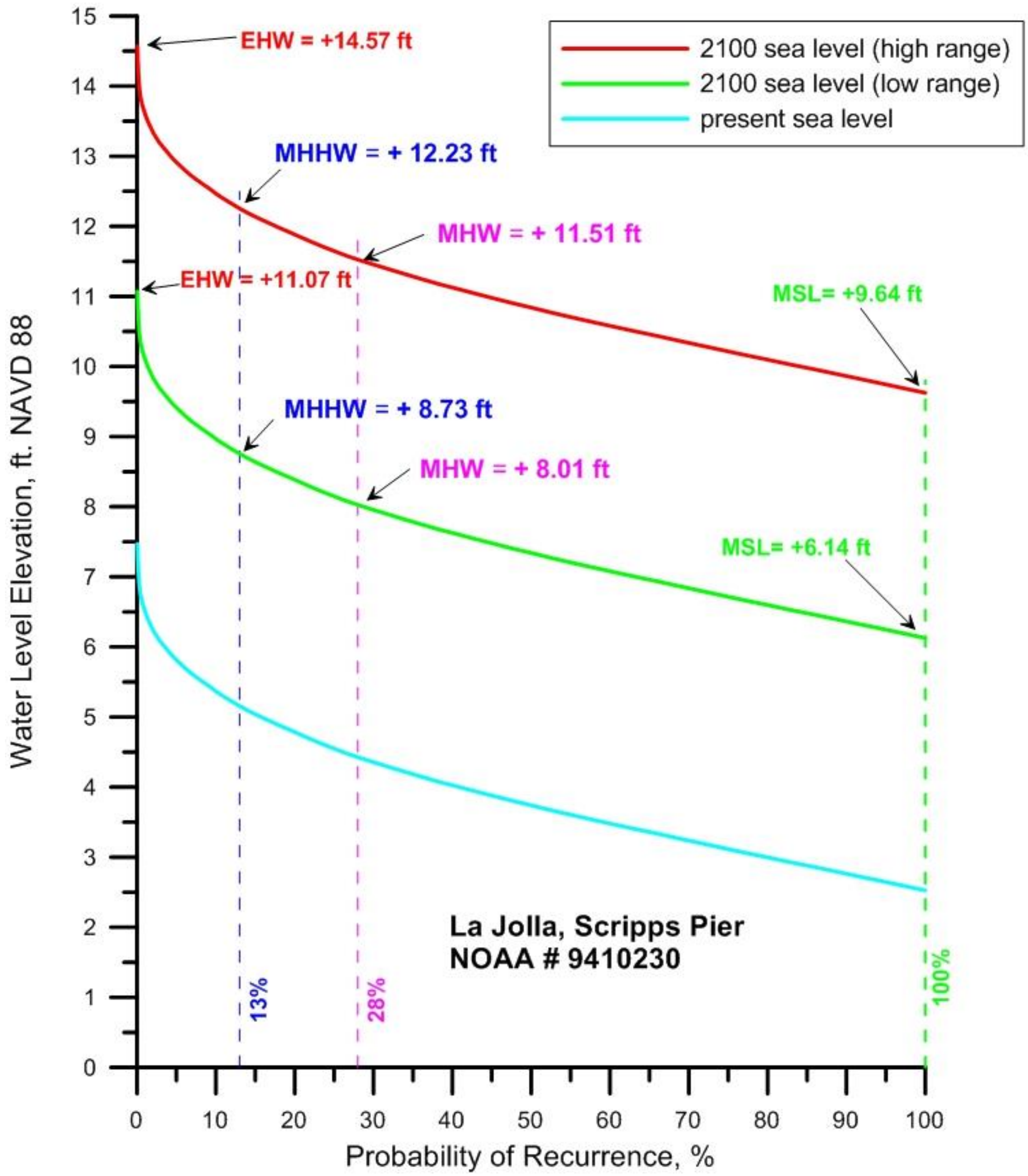


Figure 4.6: Hydroperiod function of still-water level elevations at 2100 sea level, based on ocean water level measurements at the Scripps Pier tide gage station, NOAA #9410230, for the period of record 1924-2016. Tidal datums based on the 1983-2001 tidal epoch (latest datum analysis period). Sea level rise component from Appendix-G, Table G-11 in CCC (2018)

At the year 2100 planning horizon for desalination projects, low range projections in Figure 4.6. indicate that mean sea level increases to MSL = +6.14 ft NAVD while extreme high water increases to EHW = +11.07 ft. NAVD, while mean higher high water increases to MHHW = + 8.73 ft. NAVD. At the high range 2100 projections, mean sea level increases to MSL = +9.64 ft. NAVD; extreme high water increases to an astonishing EHW = +14.57 ft. NAVD, and mean higher high water increases to MHHW = + 12.23 ft. NAVD. The still water elevations inferred at future sea levels from the linear superposition assumption are summarized in Table 4.2 below. It is interesting to note that under the updated policy guidance (CCC,2018) water levels for the high range sea level rise projections for 2070 are the same as water levels for the low range sea level rise projections for year 2100.

Table 4.2: Still Water elevations at present and future sea levels. Based on NOAA #941-0230 tide gage records and sea level rise from Appendix-G, Table G-11 in CCC (2018)

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 Low Range Projection (ft. NAVD 88)	*2100 Sea Level High Range Projection (ft. NAVD 88)
Mean Sea Level (MSL)	2.54	4.54	6.14	6.14	9.64
Mean High Water (MHW)	4.41	6.41	8.01	8.01	11.51
Mean Higher-HighWater (MHHW)	5.13	7.13	8.73	8.73	12.23
Extreme High Water (EHW)	7.47	9.47	11.07	11.07	14.57

*Planning horizon for the Doheny Desalination Project.

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

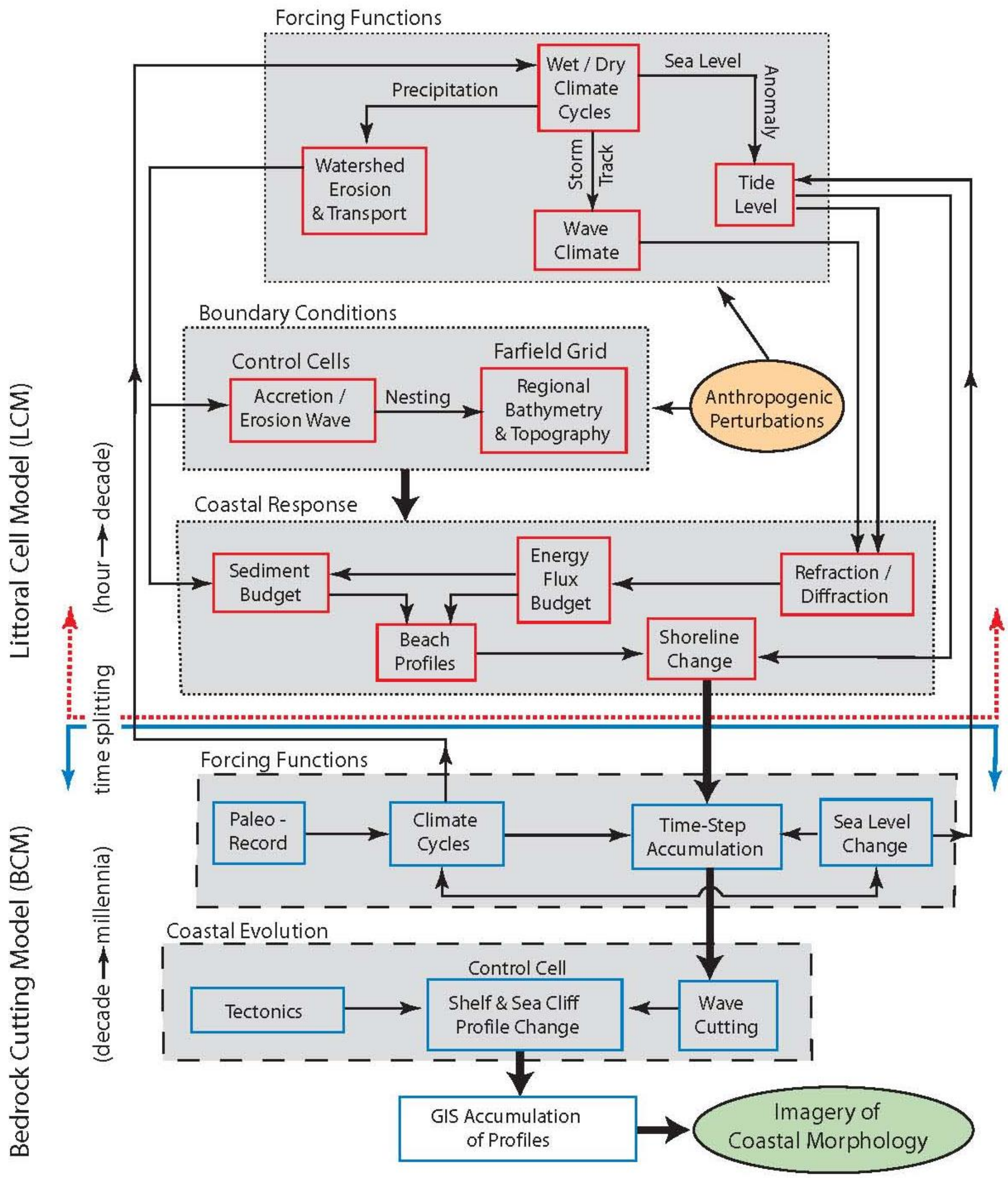


Figure 5.1: Architecture of the Coastal Evolution Model consisting of the Littoral Cell Model (above) and the Bedrock Cutting Model (below). Modules (shaded) are formed of coupled primitive process models. (Jenkins and Wasyl, 2005).

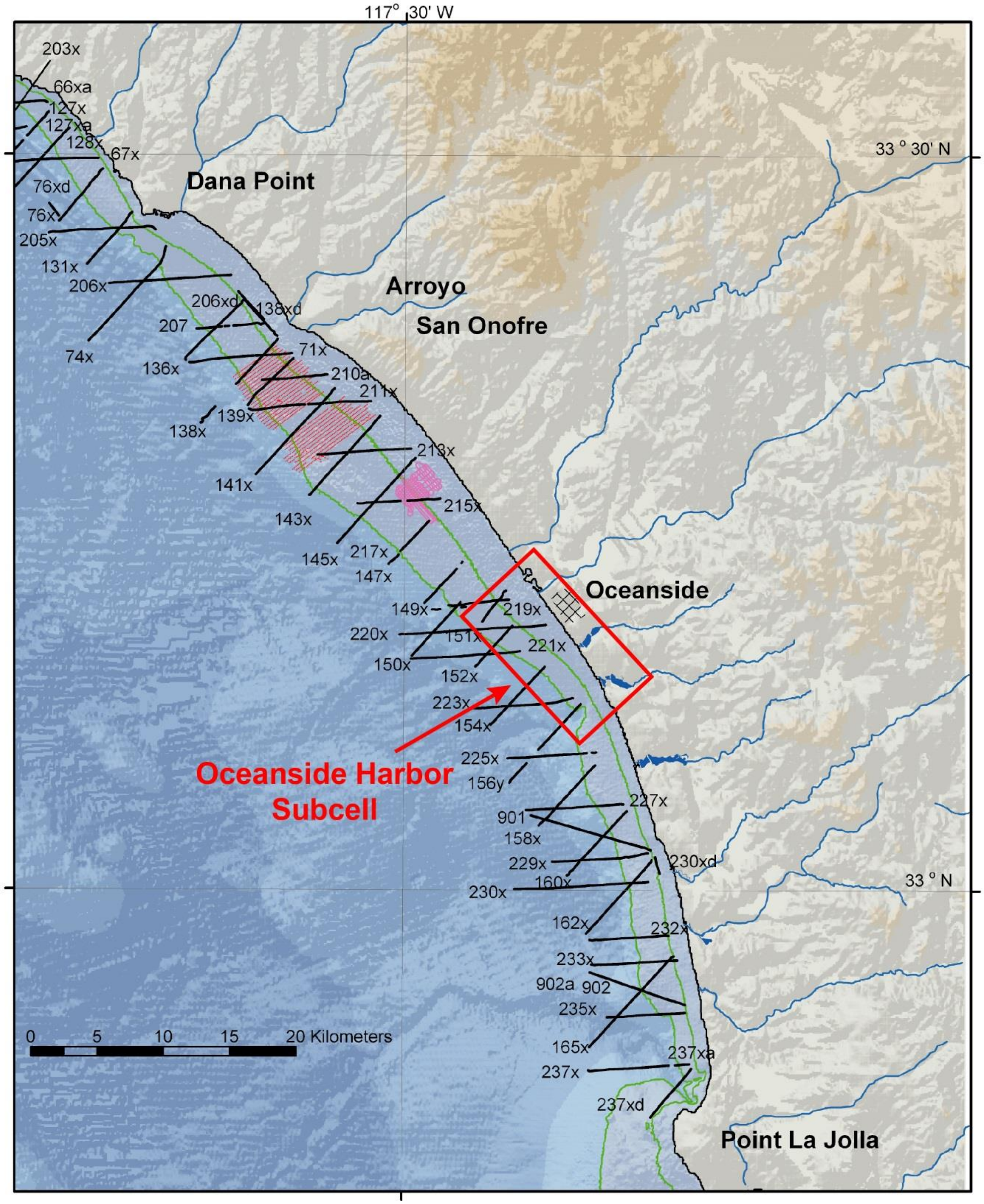


Figure 5.2: Oceanside Littoral Cell and Oceanside Harbor Sub-Cell. Composite bathymetry from NOS data base and equilibrium profiles after Jenkins and Inman (2006) for wave conditions of wet weather scenario. Depth contours shown in meters mean sea level. USGS cross-shelf survey tracks shown as numbered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

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

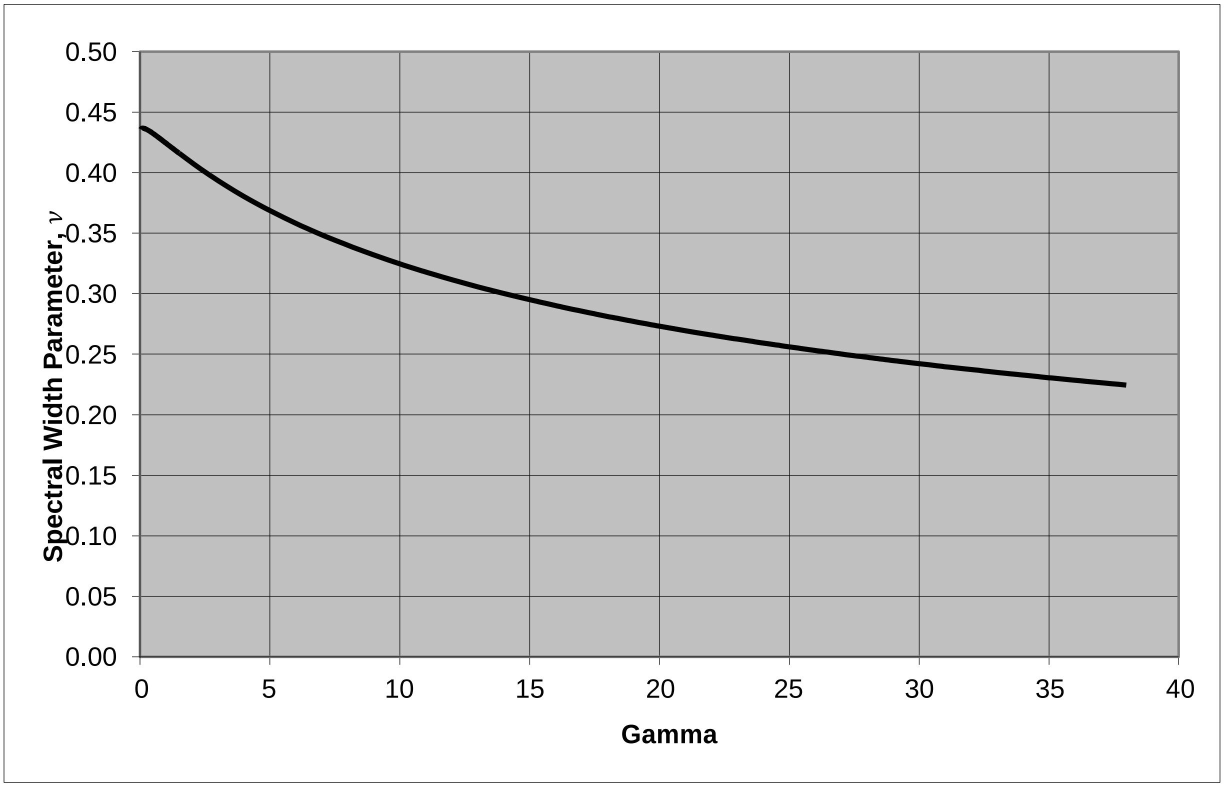


Figure 5.3 *Gamma* values are limited from 1 to 38, based on the range of wave data used (Section 4.4) to relate the spectral narrowness, ν, to the peak parameter, *Gamma*, as shown in Figure 3.3.

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 swash level, 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.

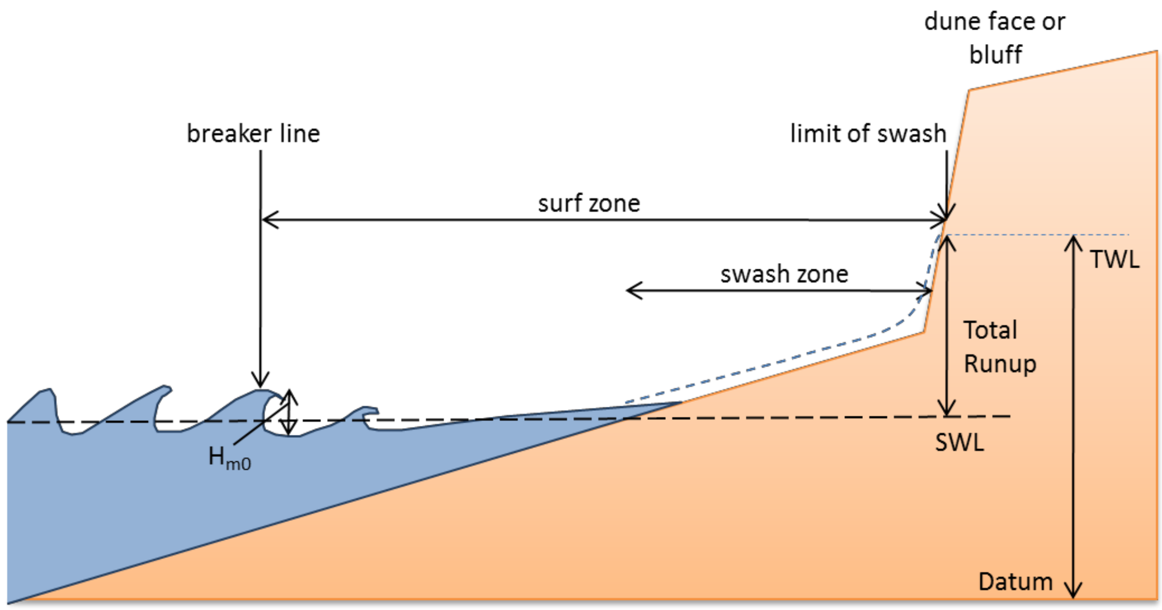


Figure 5.4: Conceptual Model Showing the Components of Wave Runup Associated with Incident Waves.

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 $\left(H_b\right)_{rms}$, and the depth at the toe of the barrier relative to SWL, h. The $\left(H_b\right)_{rms}$ is determined using the deepwater equivalent significant wave height, H'_0, and the peak wave period, T_P , according to:

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

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:

$$\overline{\eta}_0 = 0.189\left(H_b\right)_{rms} \qquad (14)$$

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

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

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:

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

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,

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

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

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

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.

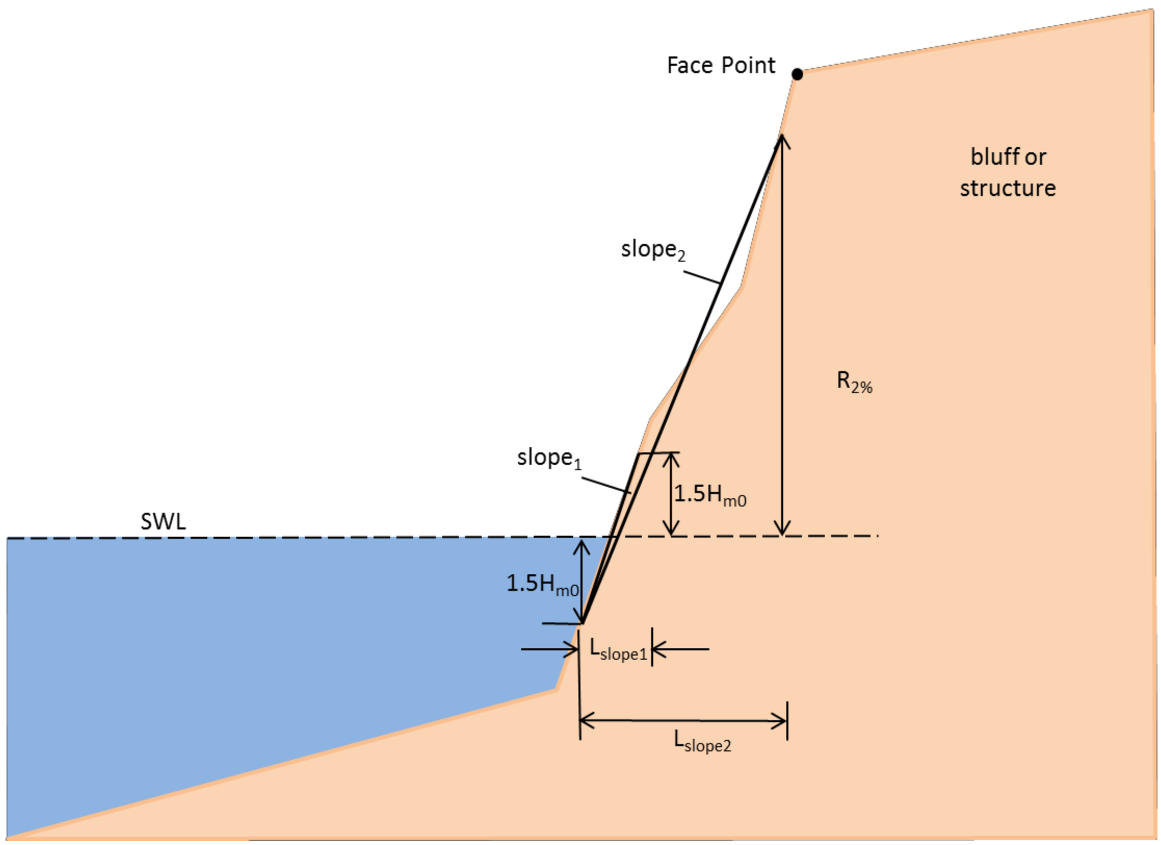


Figure 5.5: Determination of an Average Slope of Hard Back-Shore Formations (Bluff or Barriers) Based on an Iterative Approach, (Corrected from van der Meer, 2002)

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:

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

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 $\xi_{0m} = m_{TAW} / (H_{m0} / L_m)^{0.5}$ 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-5-8 in the Coastal Engineering Manual (USACE, 2006) which presents an overtopping formula for impermeable and permeable barriers and structures according to:

$$Q' = a\, gH_s T_{om} \exp\left(-\frac{bR_c}{H_s \gamma_r} \sqrt{\frac{s_{om}}{2\pi}} \right) \tag{17}$$

Where H_s, is the significant wave height at the structure, R_c is the freeboard, γ_r is the influence factor for surface roughness, T_{om} is the wave period associated with the spectral peak in deep water, s_{om} is the wave steepness associated with the spectral peak in deep water, and a and b are empirical constants based on beach slope and berm width as determined from measured beach profiles plotted in Section 6.4. 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).

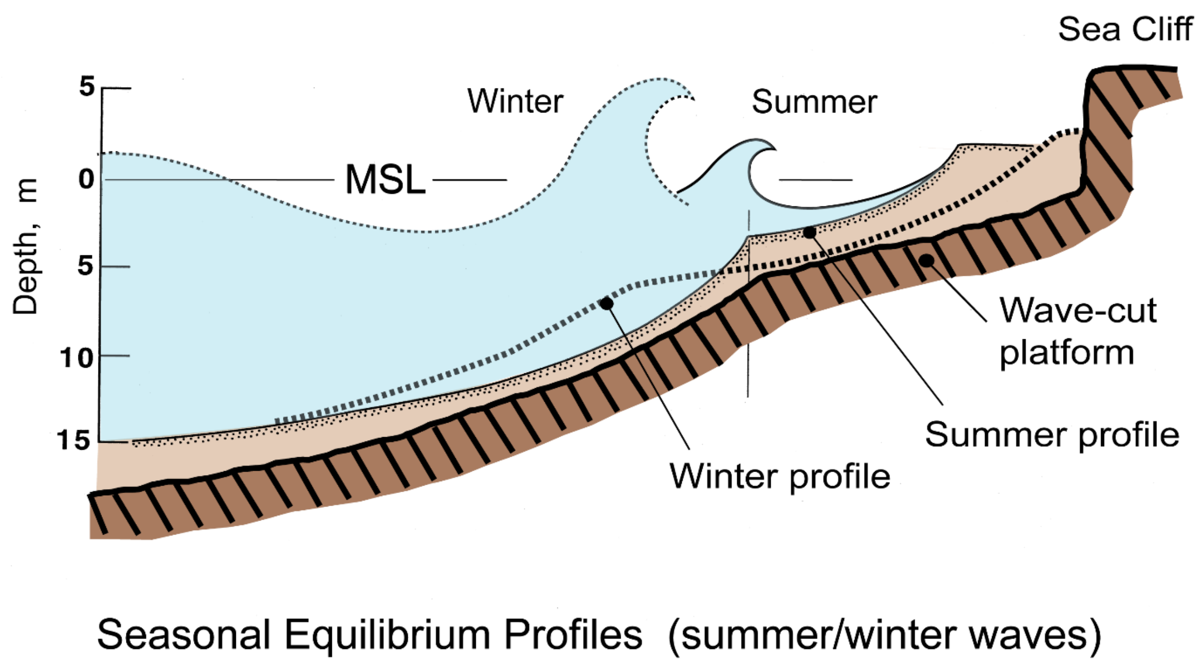


Figure 5.6: Schematic of summer and winter equilibrium beach profiles, from Inman, et al (1993).

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:

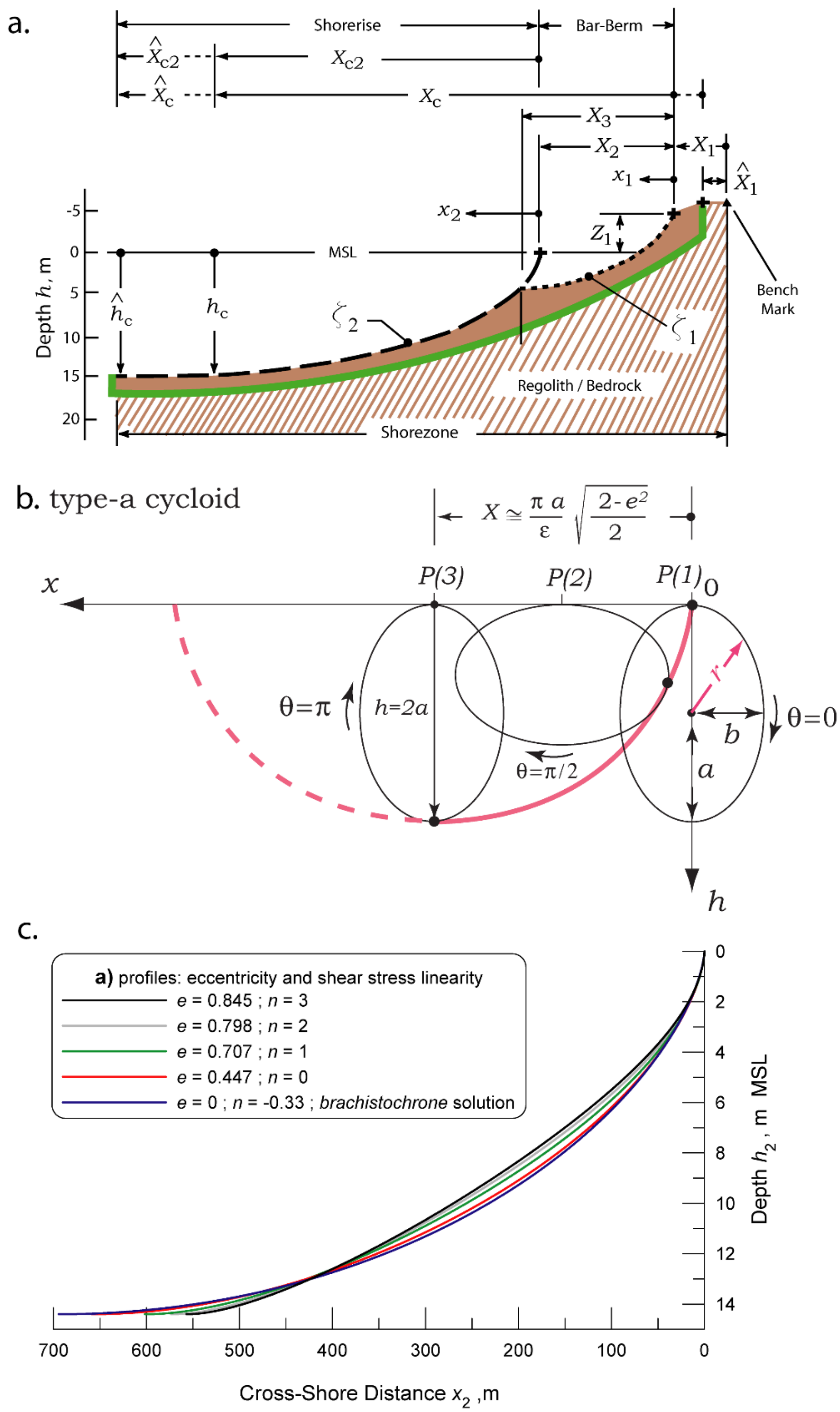


Figure 5.7. Equilibrium beach profile a) nomenclature, b) elliptic cycloid, c) Type-a cycloid solution.

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

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:

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

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:

$$h_{\mathbf{c}} = \frac{K_{\mathbf{e}} H_{\infty}}{\sinh k h_{\mathbf{c}}} \left(\frac{D_{\mathrm{o}}}{D_{\mathbf{2}}} \right)^{\psi} \tag{24}$$

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 $K_{\mathrm{e}} \sim 2.0, \psi \sim 0.33$ and $D_{\mathrm{o}} \sim 100\mu\mathrm{m}$, 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.

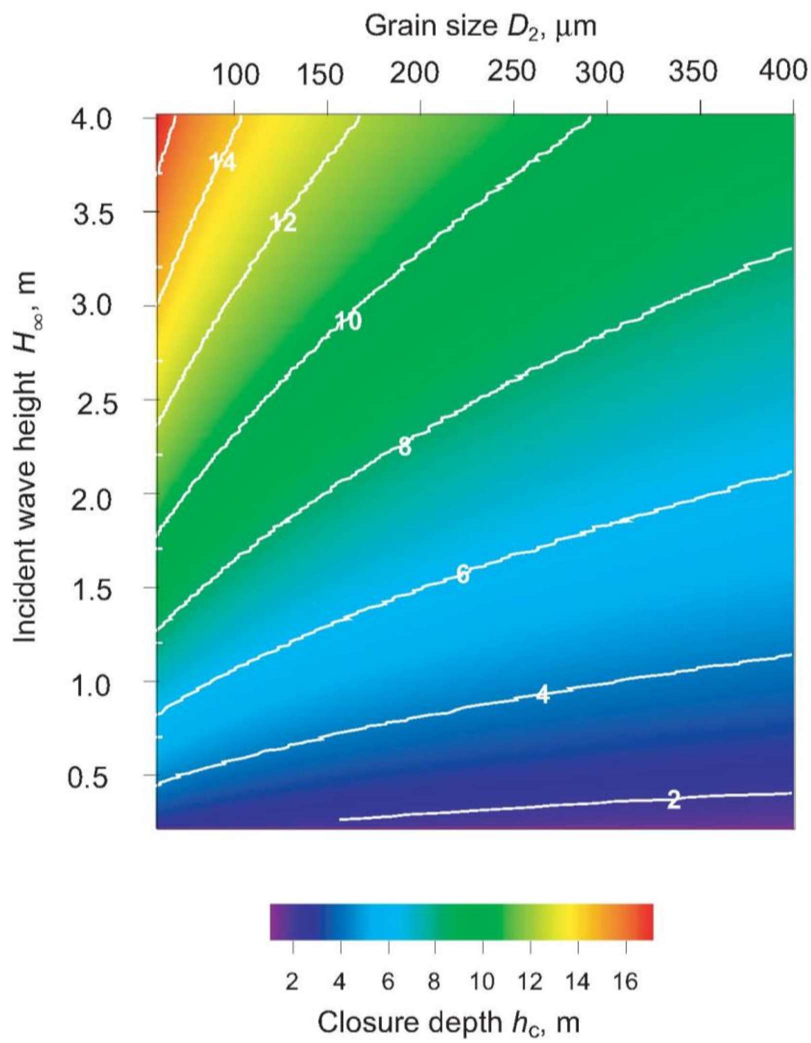


Figure 5.8: Closure depth contoured versus incident wave height and sediment grain size for waves of 15 second period, with $K_{\mathrm{e}} \sim 2.0, \psi \sim 0.33$ and $D_{\mathrm{o}} \sim 100\mu\mathrm{m}$. D_2 is the shorerise median grain size; and D_{o} is a reference grain size.

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: ***Well Head A***, elevation 17 ft.NAVD, at 33°27'44.38"N, 117°41'16.32"W; ***Well Head B***, elevation 17 ft. NAVD, at 33°27'45.07"N, 117°41'10.30"W; ***Well Head C***, elevation 17 ft. NAVD at 33°27'45.12"N, 117°41'6.62"W; ***Well Head D***, at elevation 18 ft. NAVD at 33°27'44.48"N, 117°40'55.30"W; and ***Well Head E***, at elevation 18 ft. NAVD at 33°27'42.45"N, 117°40'47.33"W; (see Figure 6.3a). Two additional vaulted well heads with submersible pumps will be placed at the Capistrano Beach site (Figure 6.3b), which includes: ***Well Head G***, at elevation 18 ft. NAVD at 33°27'14.94"N, 117°39'59.91"W; and ***Well Head H***, at elevation 19 ft. NAVD at 33°27'13.17"N, 117°39'57.15"W.

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.

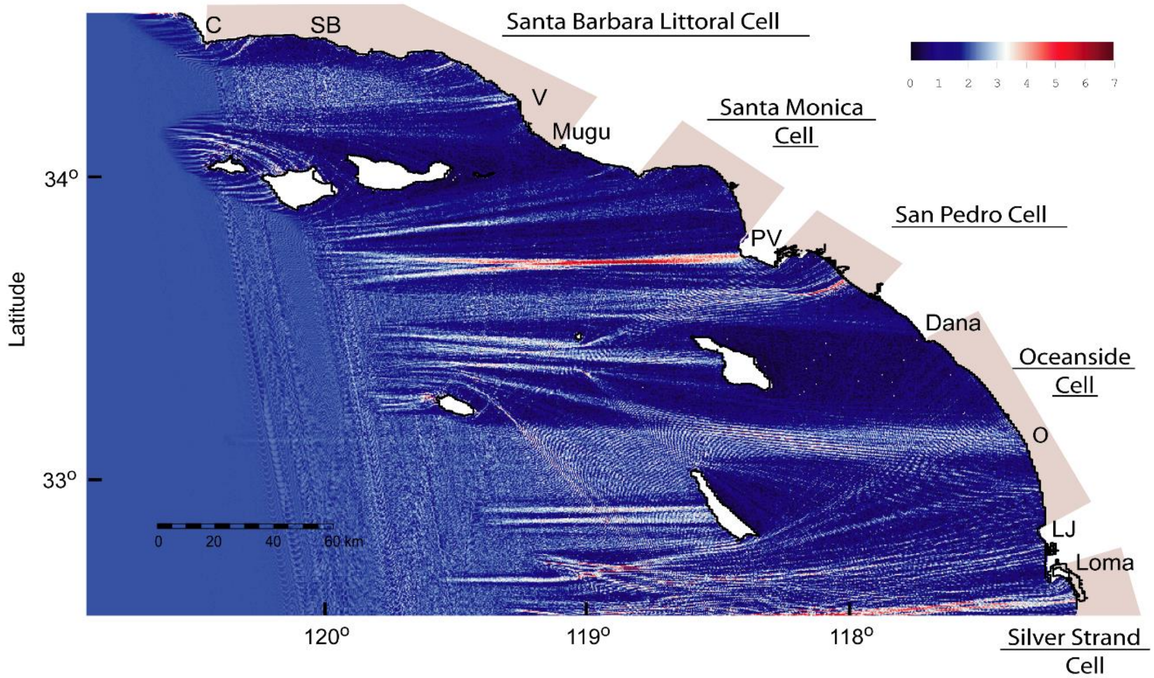
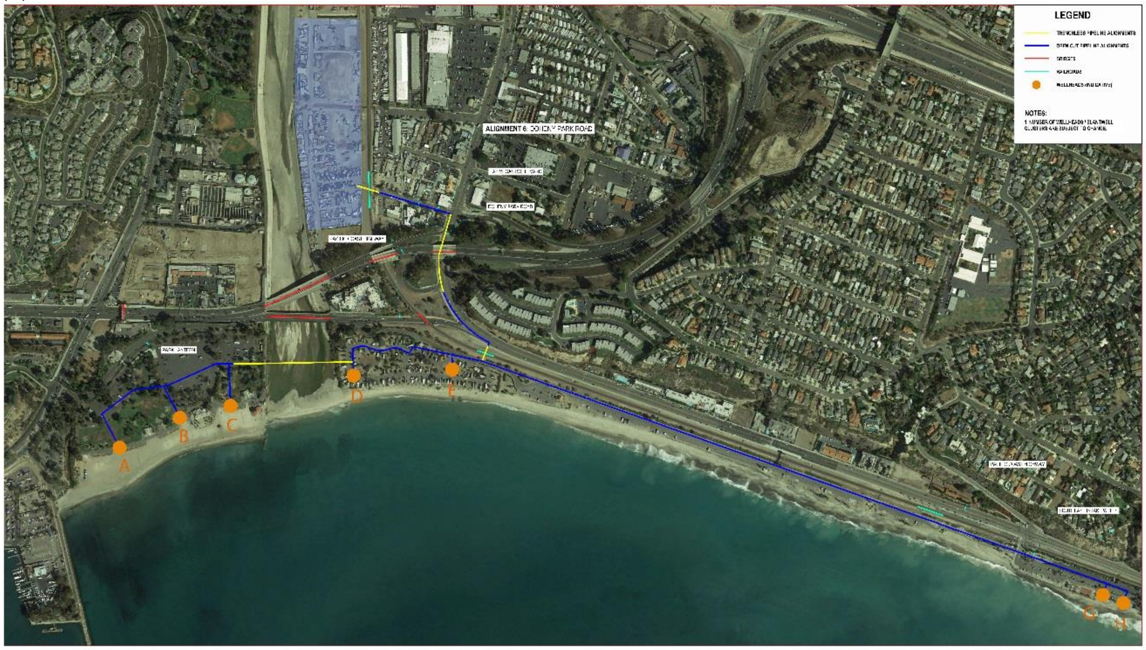


Figure 6.1: Far-field refraction/diffraction grid to simulate shoaling waves entering the Southern California Bight and Oceanside Littoral Cell. Results based on the 5 largest storms of the 1998 El Nino winter (from Jenkins and Wasyl, 2008b).

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

(a)



(b)

Figure 6.3: Critical shore-front infrastructure locations for the Doheny Desalination Project: a) Doheny Beach site; and b) Capistrano Beach site

Wave forcing to the Coastal Evolution Model (CEM) 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

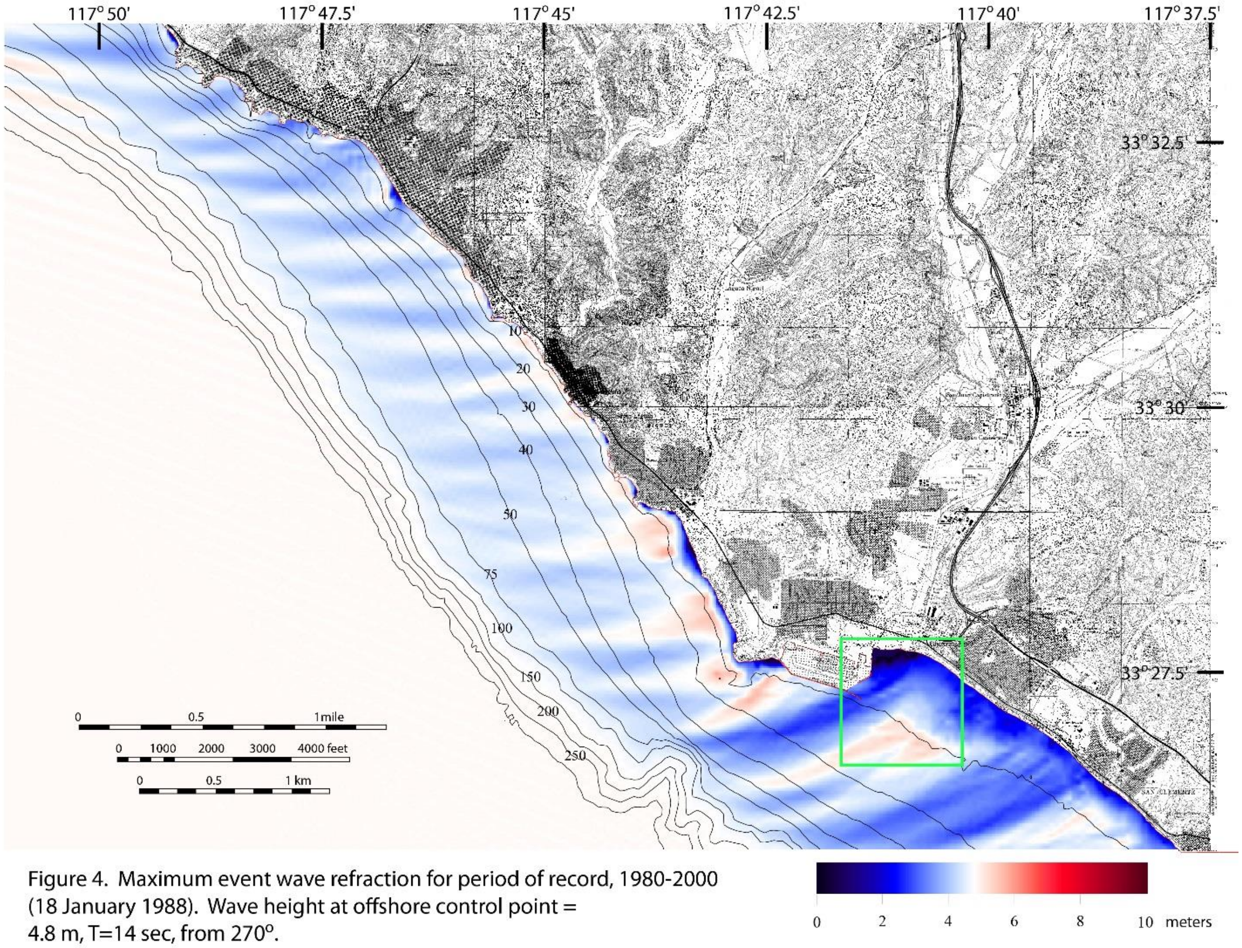


Figure 6.4: Forward wave refraction/diffraction for the 100-year storm-wave event of 17-18 January 1998. These local refraction results are used to provide the point-to-point initializations for the wave setup and runup inputs to the total water level problem. The nearfield domain of the SJCOO and the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

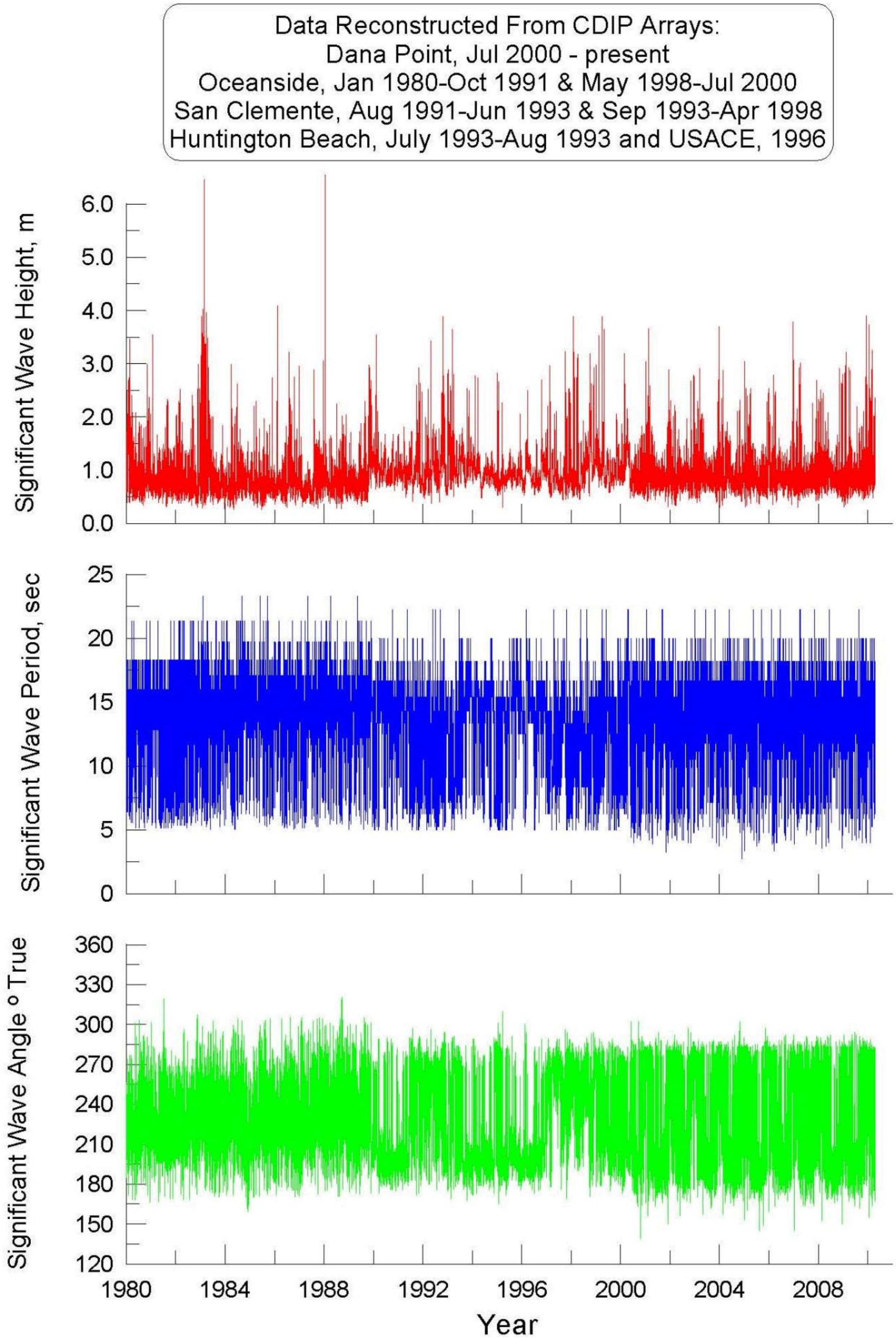


Figure 6.5: Archival wave forcing data 1980-2010 reconstructed for the SJCOO and Doheny Desalination Project modeling, from backward/forward refraction of regional CDIP wave monitoring data.

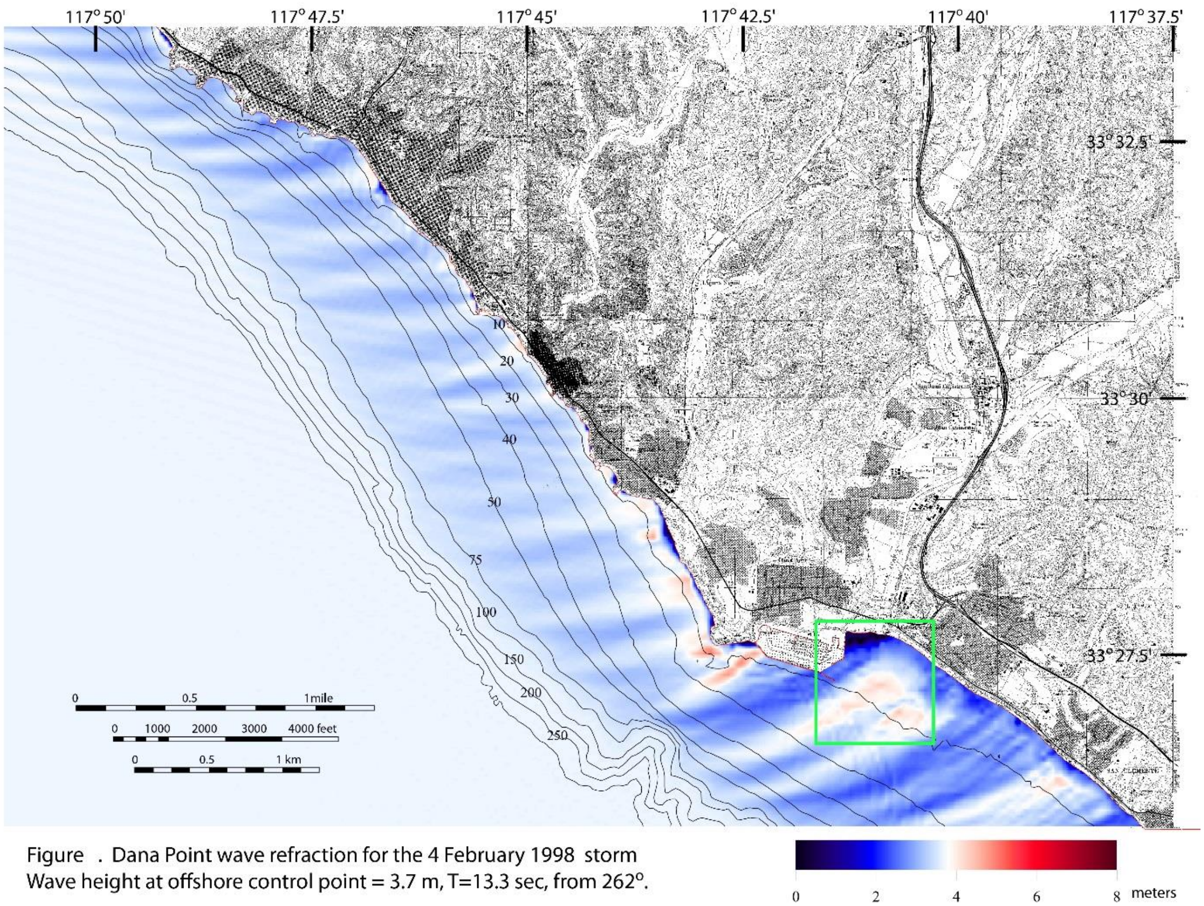


Figure 6.6: Wave refraction/diffraction field around the SJCOO site and the Doheny Desalination Project site derived from wave burst measurements from the Acoustic Doppler Current Profiler (ADCP) records taken under the MBC Applied Environmental Sciences (MBC) NPDES monitoring studies. Nearfield domain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

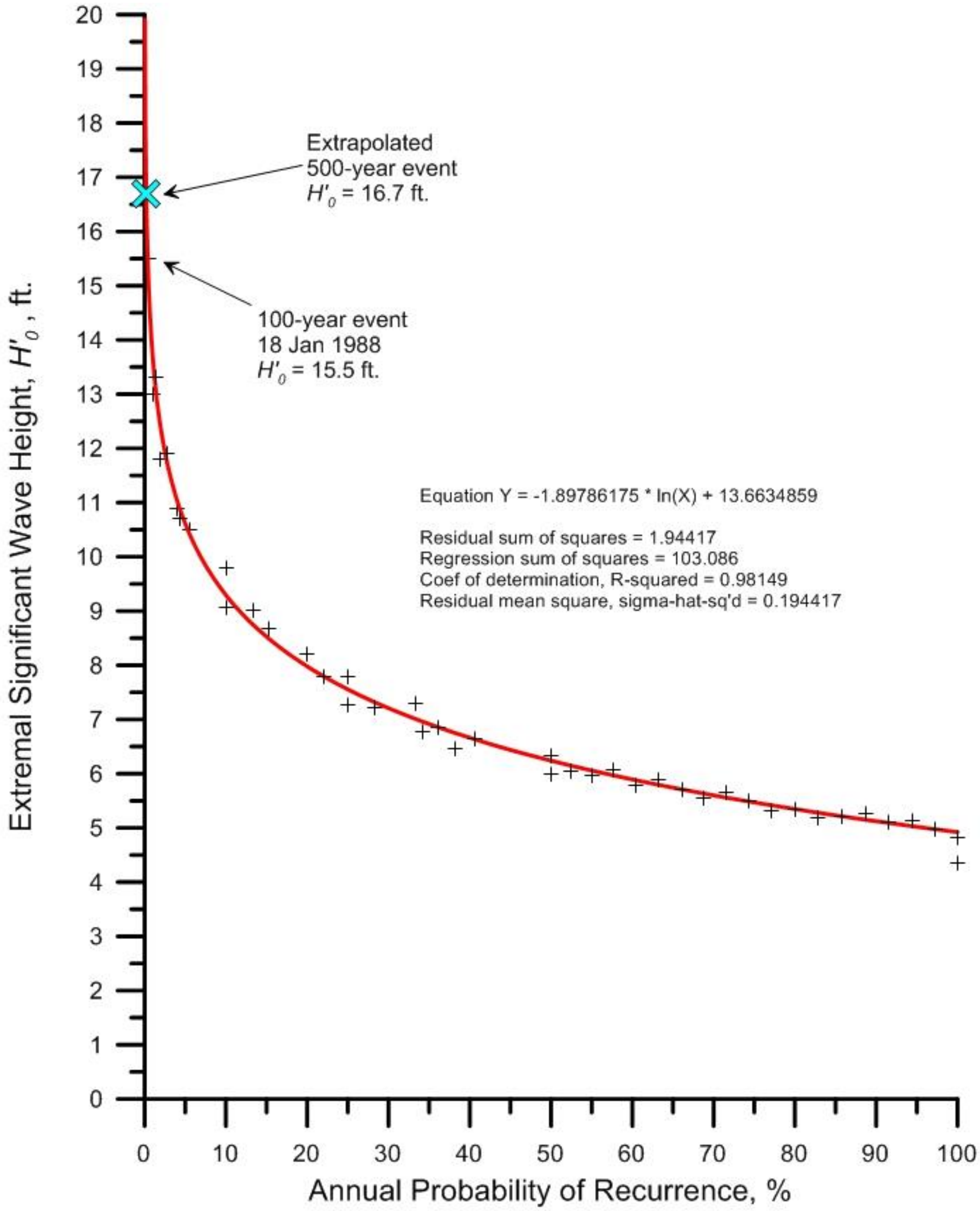


Figure 6.7 : Probability of recurrence of design wave heights based on Weibull extremal analysis of significant wave heights at Doheny & Capistrano Beaches. Analysisbased on Weibull Type III distribution applied to 12 m local water depth with K = 1.0. Recurrence Probability $P(H)$ = 100%/T, where T = 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, (proxy slope for an *eroded beach*). Figure 6.11 indicates that the average nearshore slope in summer steepens to m_{DIM} = 0.10, (proxy slope for an *accreted beach*). 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):

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

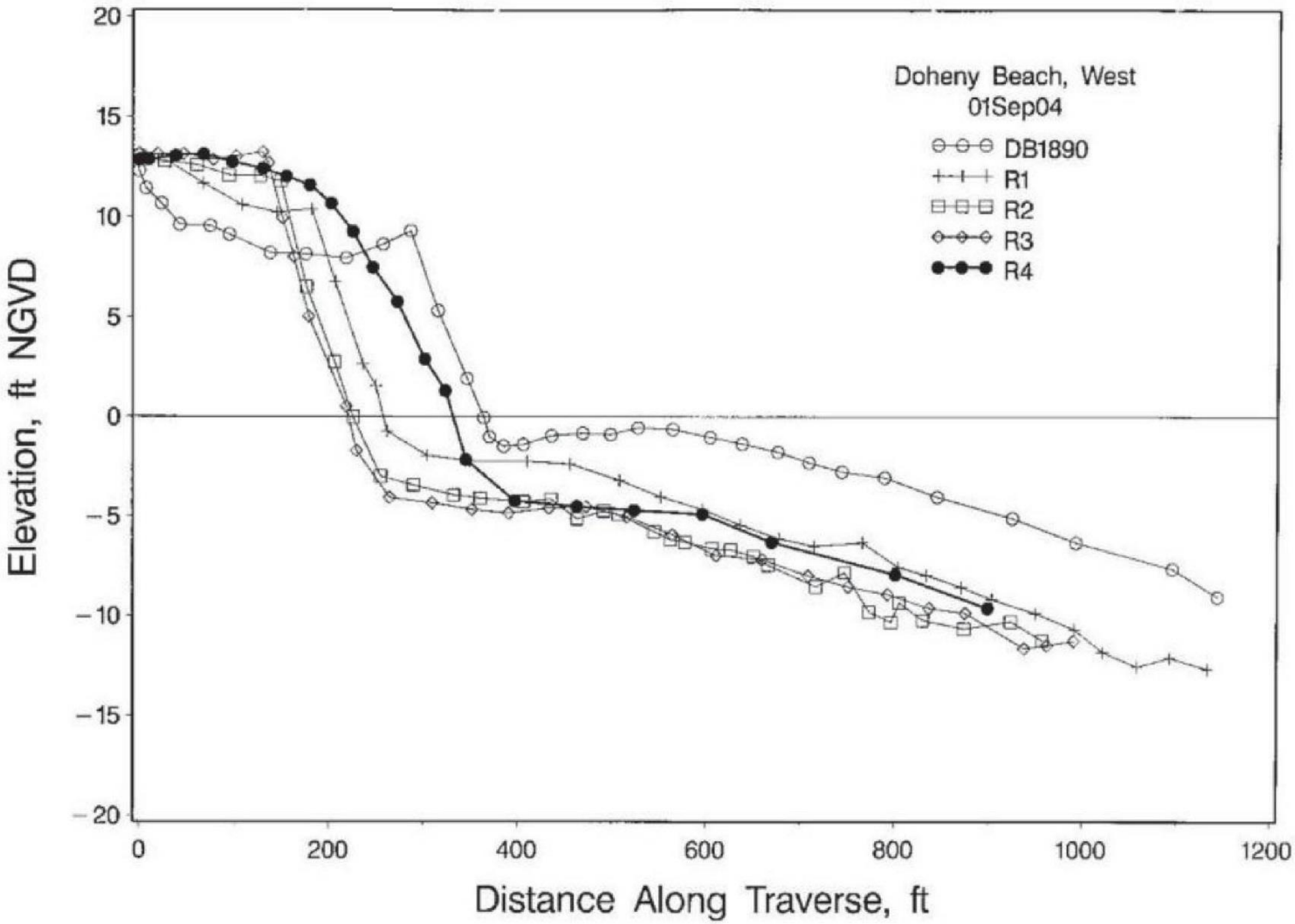


Figure 6.8: Beach profile surveys of Doheny Beach range lines adjacent to Well Heads A & B. Data from Coastal Environments, (2014).

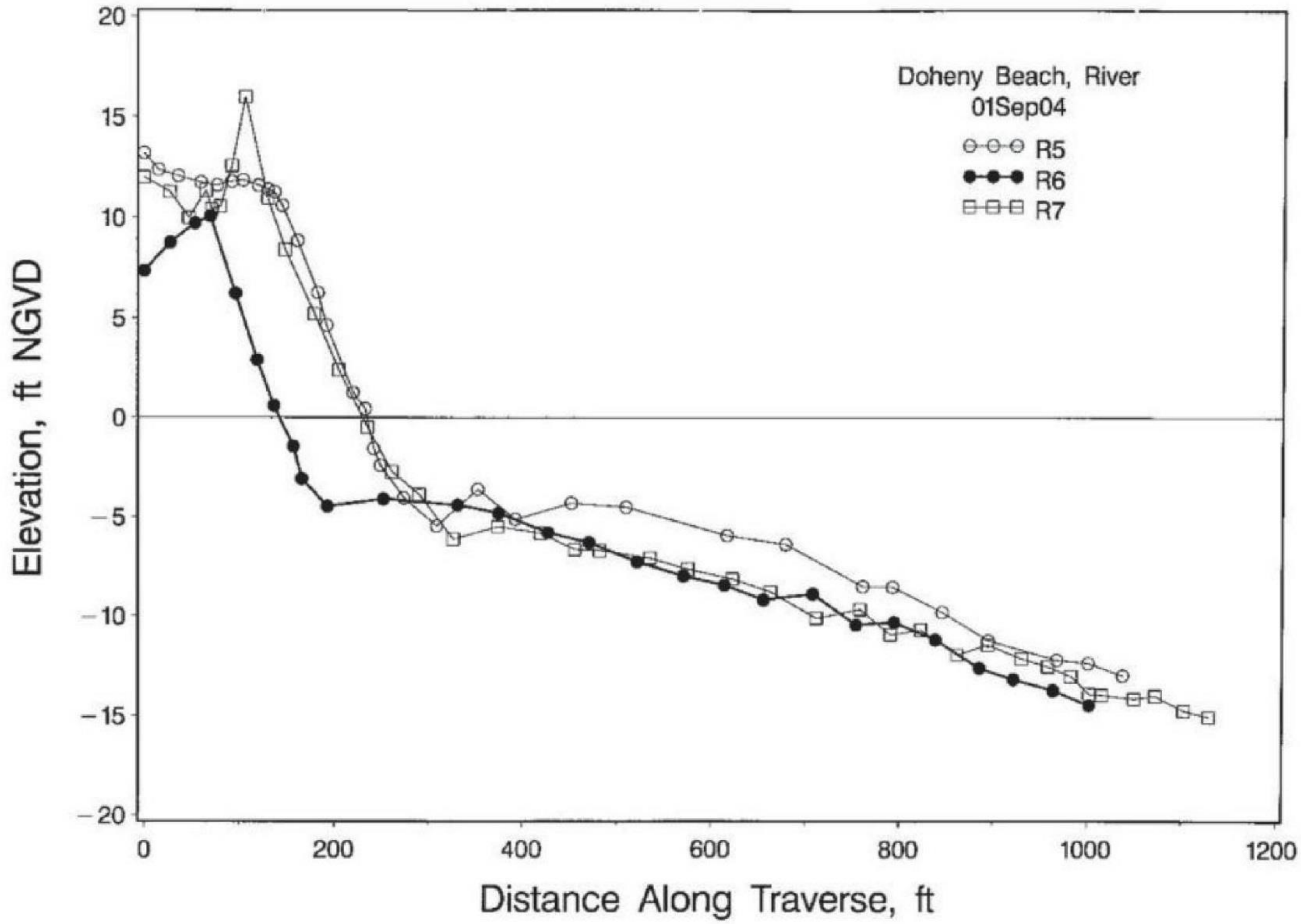


Figure 6.9: Beach profile surveys of Doheny Beach range lines adjacent to Well Heads C & D. Data from Coastal Environments, (2014).

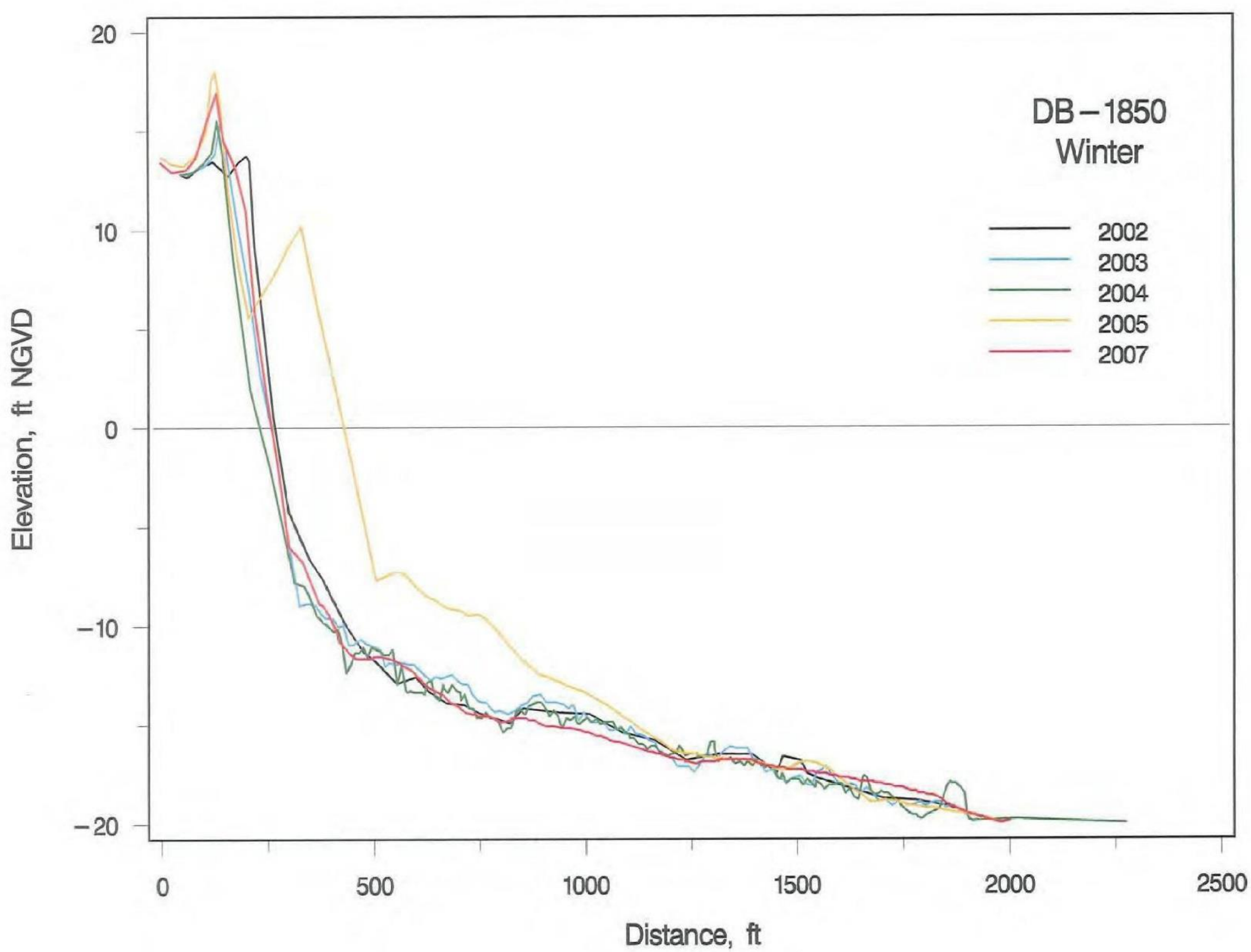


Figure 6.10: Winter beach profile surveys of Doheny Beach range lines adjacent to Well Heads D & E. Surveys due to the US Army Corps of Engineers. Data provided by Coastal Frontiers, (2014).

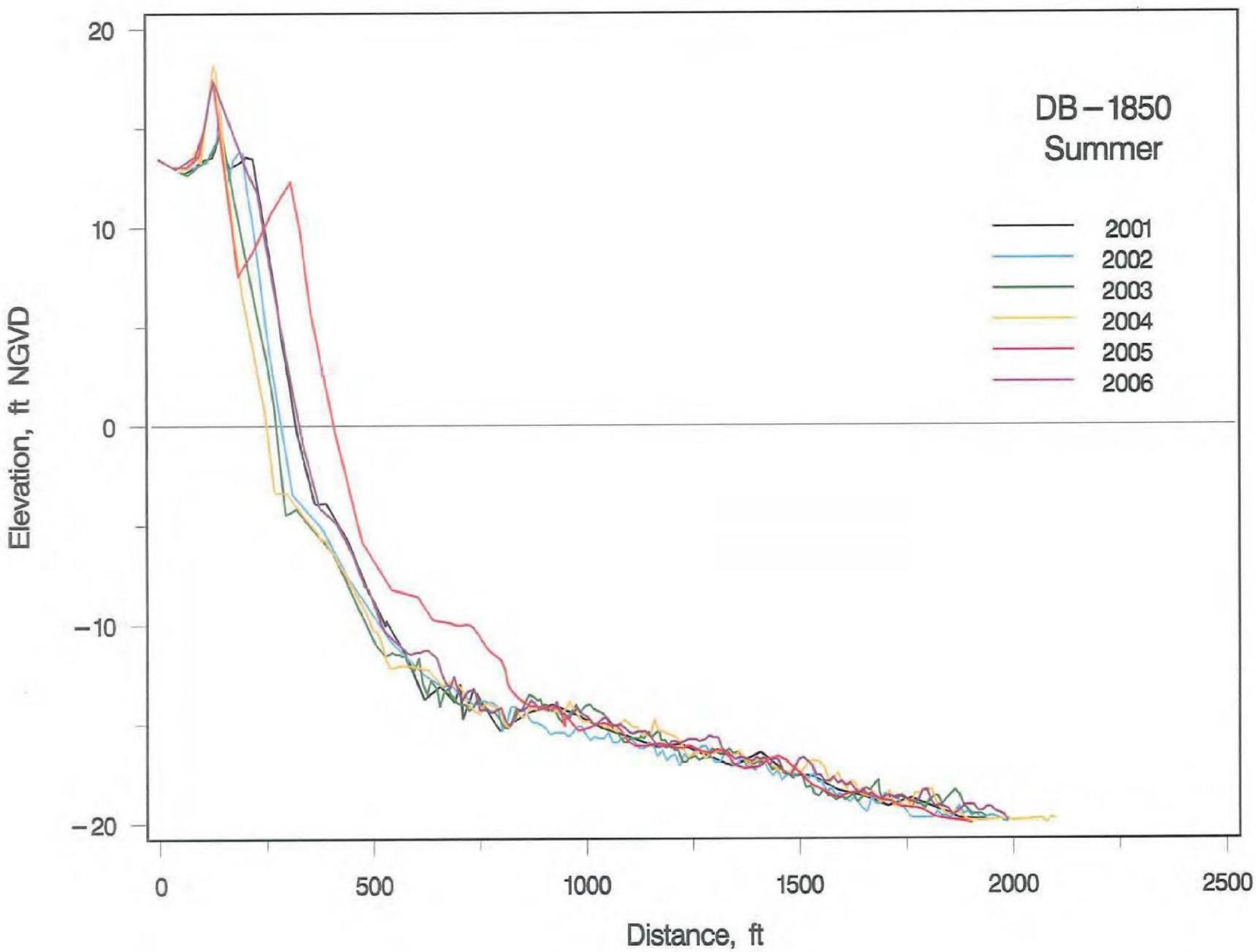


Figure 6.11: Summer beach profile surveys of Doheny Beach range lines adjacent to Well Heads D & E. Surveys due to the US Army Corps of Engineers. Data provided by Coastal Frontiers, (2014).

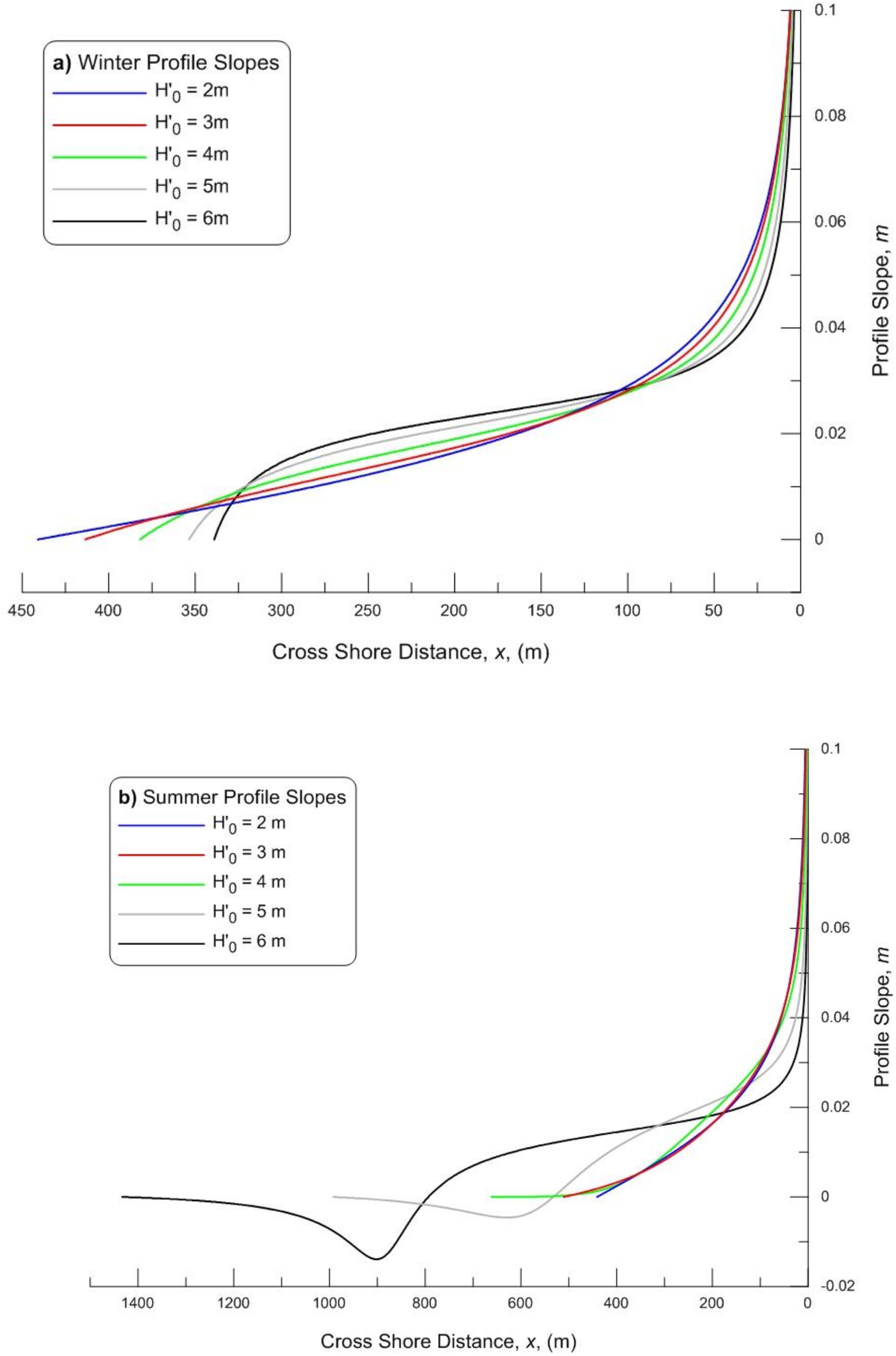


Figure 6.12. Family of elliptic cycloid slope solutions in the bar berm: a) type-a cycloids; b) type-b cycloids. Cycloids scaled for : H'_0 = 2 - 6 m; T = 15 sec; m_{DIM} = 0.06 (winter); m_{DIM} = 0.1 (summer); γ = 0.8; Γ = 0.76; Λ = 0.81

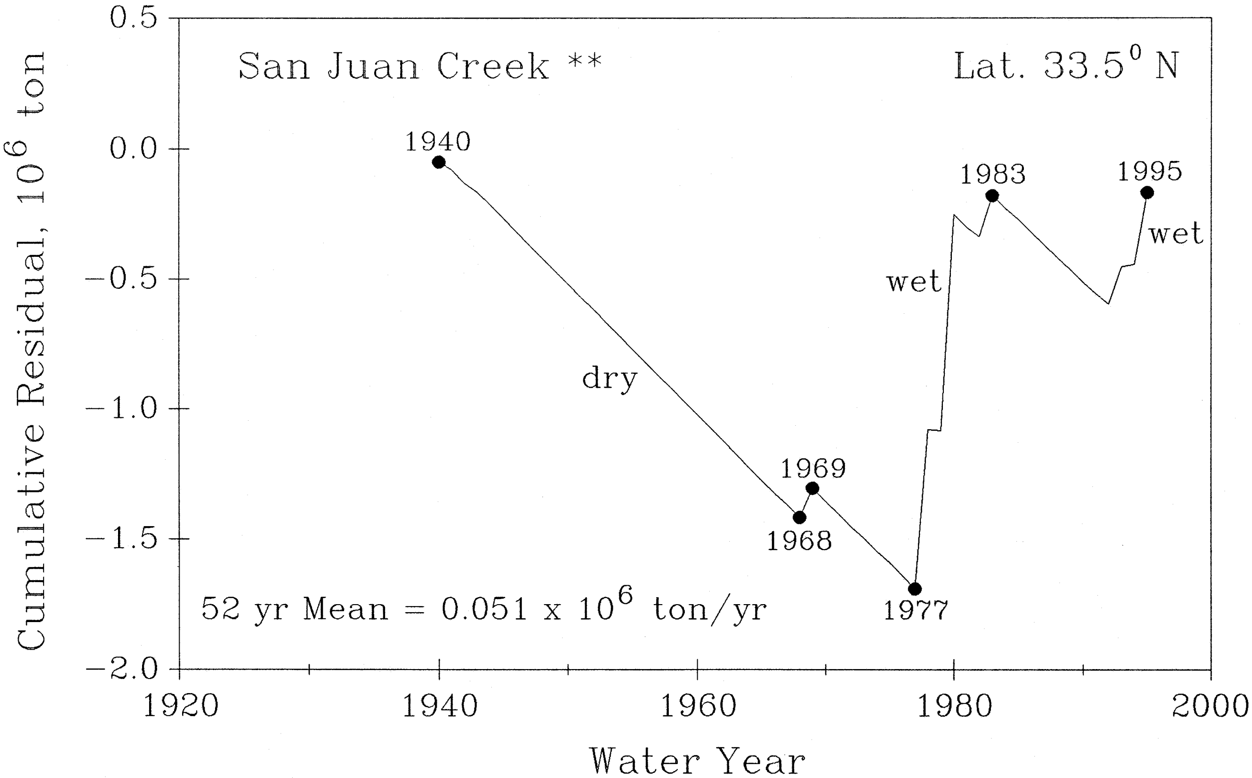


Figure 6.13. Cumulative residual time series of sediment flux for the San Juan Creek calculated using a 56-year mean (1940-1995), from Inman and Jenkins (1999).

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

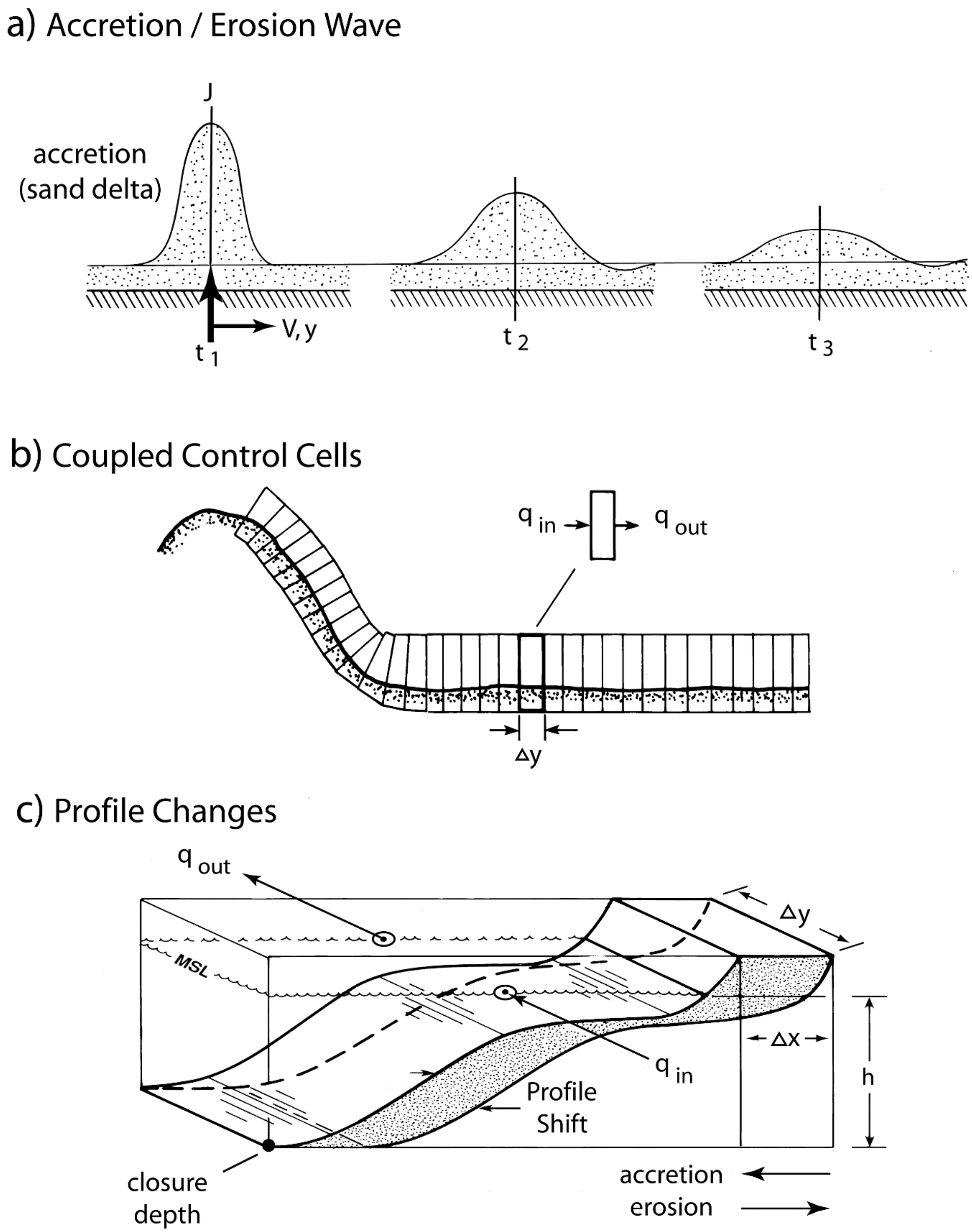


Figure 6.14: Computational approach for modeling changes in beach width and shoreline positon after Jenkins, et. al., (2007).

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:

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

where $$Z = Z_1 + h_c \qquad (28)$$

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:

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

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

$$J = \xi\, Q^{\omega} \qquad (30)$$

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 Doheny and Capistrano State Beaches. In 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 7.2 actually gives maximum total wave run-up of 11.88 ft. for the accreted beach conditions at Doheny Beach and 9.98 ft. for the eroded beach conditions. At Capistrano Beach, shoaling wave heights are greater and maximum total wave run-up is 12.73 ft. for the accreted beach conditions and 10.83 ft. for the eroded beach conditions

The annualized probability of recurrence of total water level is plotted in Figures 7.4 and 7.5 at Doheny and Capistrano Beaches, respectively, under the stationary hypothesis for still

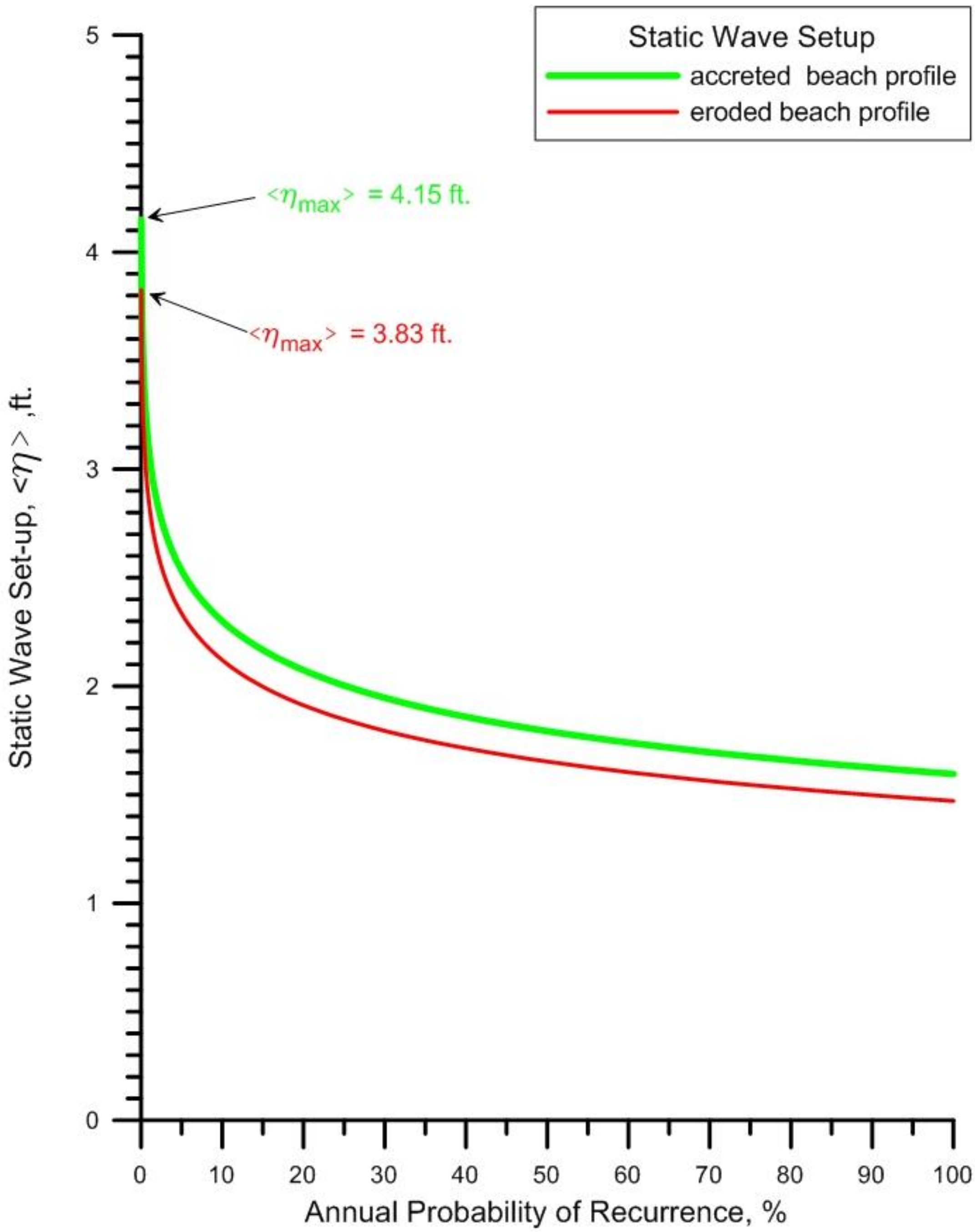


Figure 7.1: Probability of recurrence of static wave setup based on on extremal design wave heights from Weibull Type III distribution and beach profiles from Figures 6.8 - 6.11.

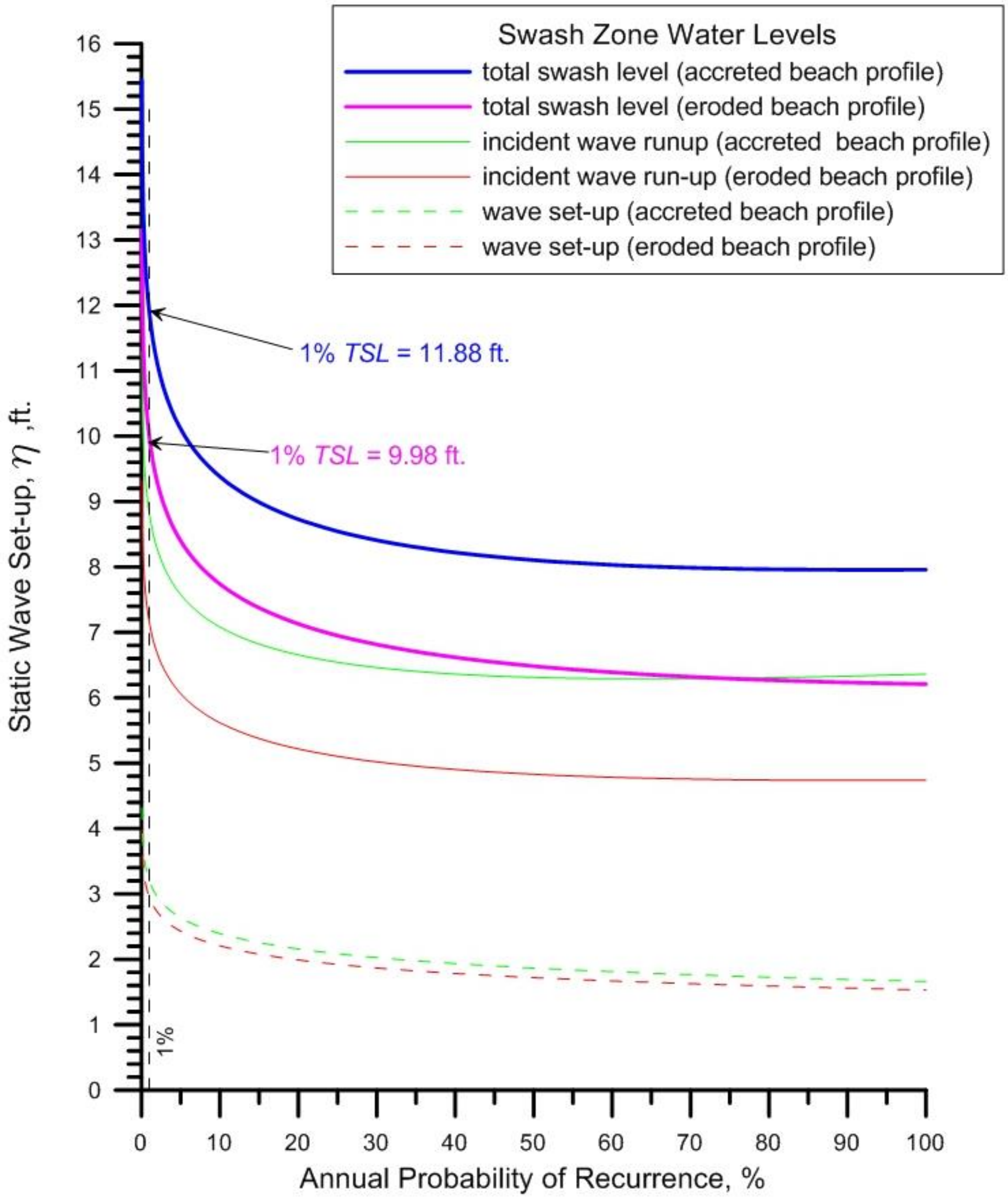


Figure 7.2 : Probability of recurrence of total swash level (*TSL*) at Doheny State Beach based on extremal design wave heights from Weibull Type III distribution

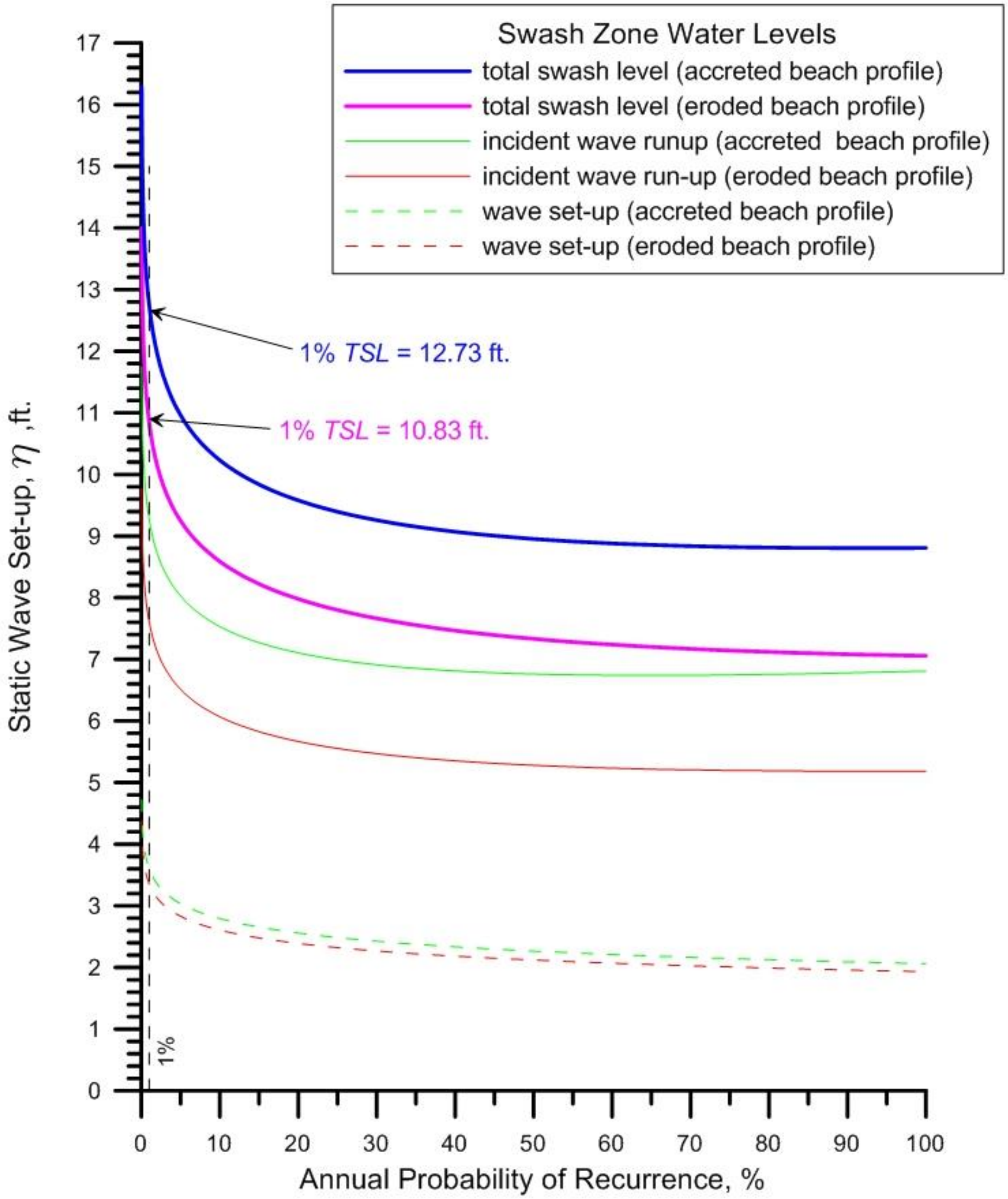


Figure 7.3 : Probability of recurrence of total swash level (*TSL*) at Capistrano State Beach based on extremal design wave heights from Weibull Type III distribution

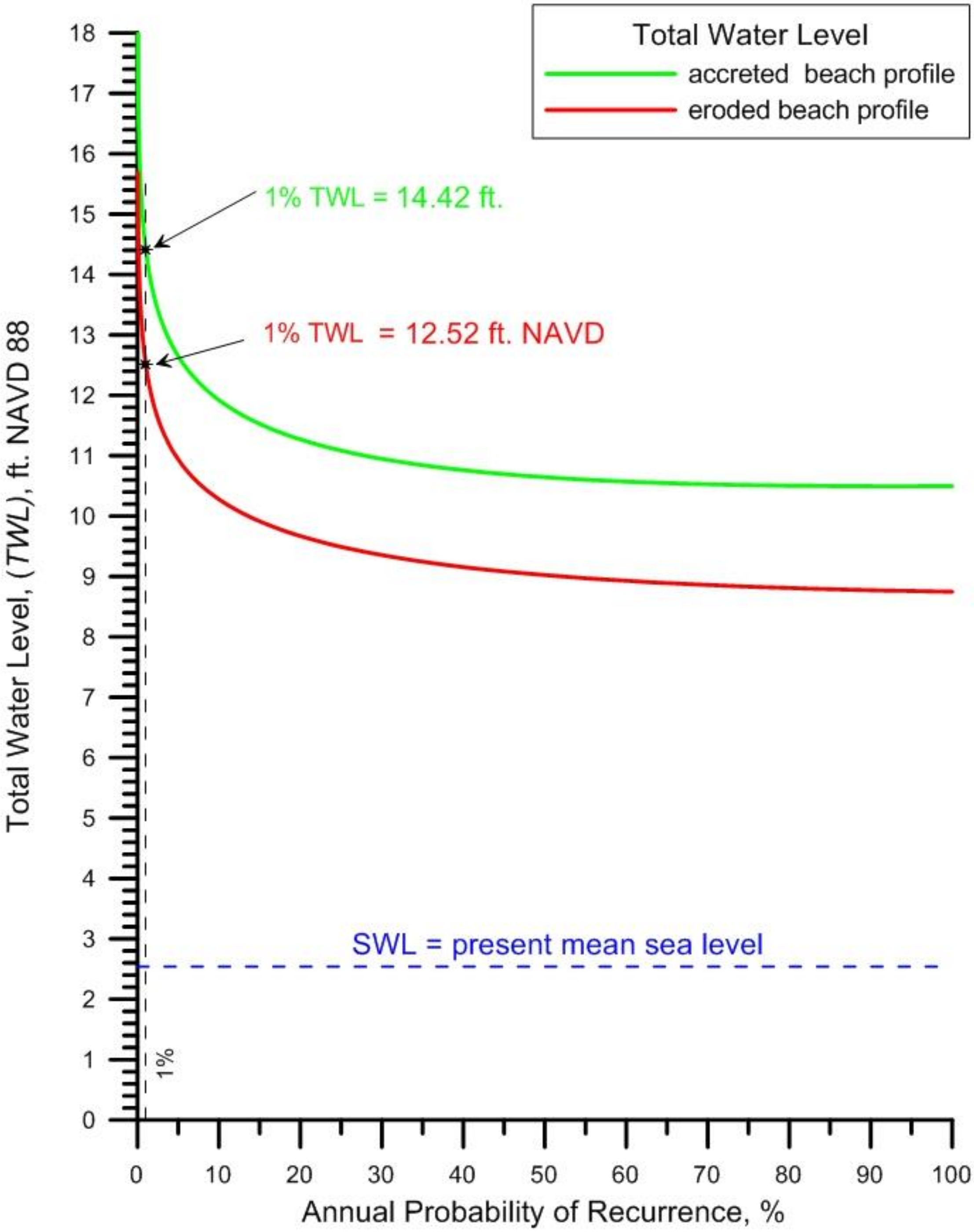


Figure 7.4: Annualized probability of recurrence of total water level at Doheny State Beach based on present sea level and extremal design wave heights from Weibull Type III distribution. SWL = MSL

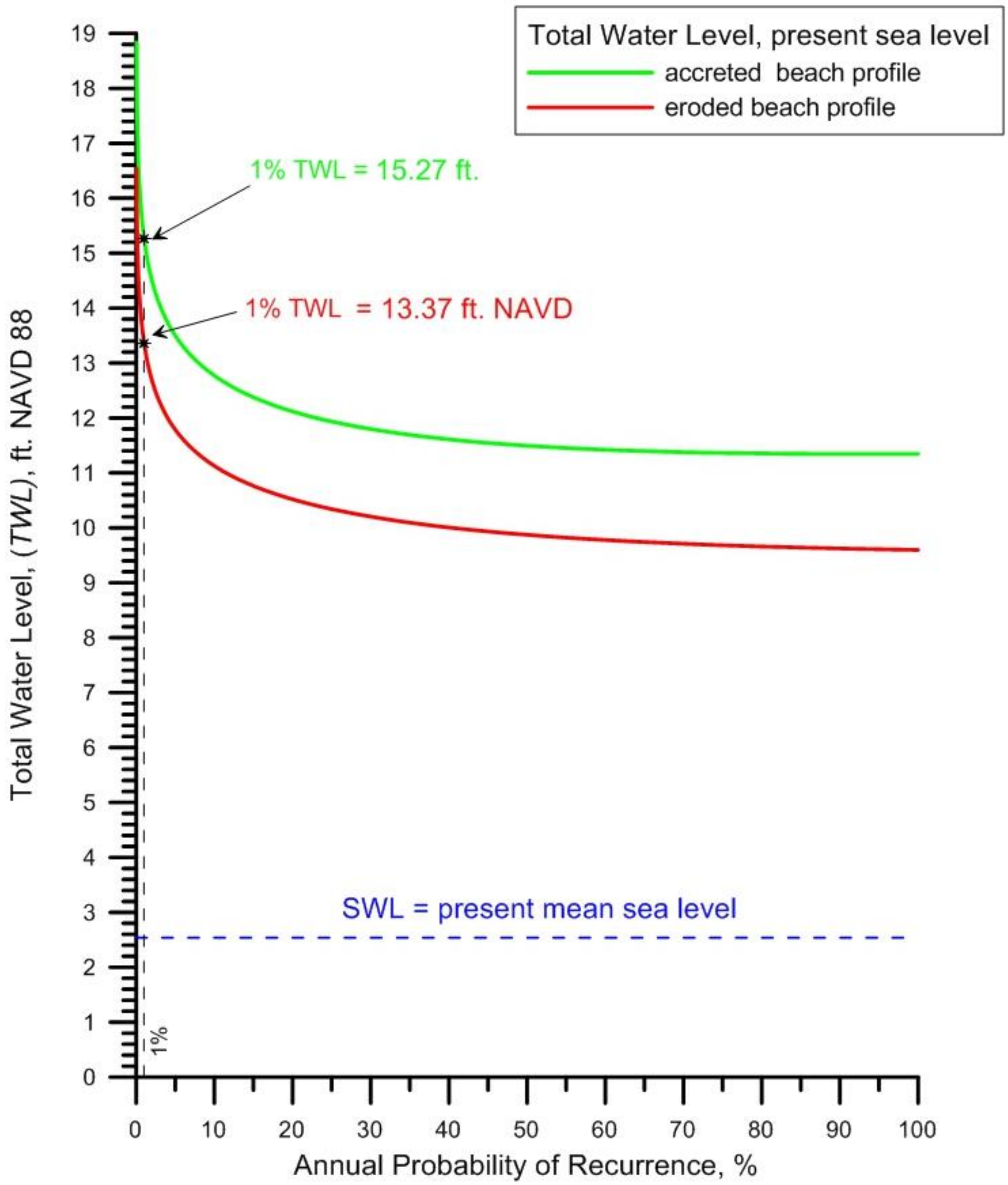


Figure 7.5 : Probability of recurrence of total water level at Capistrano State Beach for still water level at persent mean sea level based on extremal design wave heights from Weibull Type III distribution; SWL = MSL

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 at Doheny Beach is *TWL* = 17.98 ft. NAVD for the accreted beach conditions and *TWL* = 15.69 ft. NAVD for the eroded beach conditions. At Capistrano Beach (Figure 7.5), the maximum total water level is *TWL* = 18.83 ft. NAVD for the accreted beach conditions and *TWL* = 16.54 ft. NAVD for the eroded beach conditions. (Total water levels are higher at Capistrano Beach because shoaling waves during the 100-year event are higher at that location, cf. Figure 6.4). The total water level achieved under accreted beach conditions at present sea level exceeds the elevations of well heads A, B, C and G, which are located at Z_i == 17 ft. NAVD and Z_i == 17 ft. NAVD, respectively; 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, Figures 7.4 & 7.5 have 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 at Doheny Beach. At Capistrano Beach. The 1% probability (100-yr event) yields *TWL*(1%) = 15.27 ft. NAVD for the accreted beach conditions and *TWL*(1%) = 13.37 ft. NAVD for the eroded beach conditions.. Consequently we conclude that all the beach front facilities for the Doheny Desalination Project (Figure 6.3a & b) 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 7.6 and 7.7 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 at Doheny Beach, (Figure 7.6), the 1% total water level events reach *TWL*(1%) = 18.02 ft. NAVD for the accreted beach conditions and *TWL*(1%) = 16.12 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 if the beach is in an eroded winter condition. However, in the unlikely event that the 100 year storm occurs while the beach is still in a summer equilibrium condition (accreted beach), then Well Heads A-C will be overtopped by about 1 ft of excess runup, while Well Heads D and E would be partially wetted. At Capistrano Beach, the 1% total water level events at the low range projection for 2100 sea level, (Figure 7.7), reach *TWL*(1%) = 18.87 ft. NAVD for the accreted beach conditions and *TWL*(1%) = 16.97 ft. NAVD for the eroded beach conditions. While both well heads at Capistrano Beach would be safe from overtopping if Capistrano Beach were in an eroded winter state, Well Head G would be overtopped by about 0.87 ft. of runup if the beach remained in an accreted summer condition.

For the high-range 2100 sea level projections, (Figures 7.8 and 7.9) the 1% total water level events will overtop all of the well sites. At Doheny Beach, (Figure 7.8), the 1% total water level events reach *TWL*(1%) = 21.52 ft. NAVD for the accreted beach conditions and *TWL*(1%) = 19.62 ft. NAVD for the eroded beach conditions, exceeding the elevations of all well sites regardless of beach erosion or accretion. Similarly, at Capistrano Beach (Figure 7.9), the 1% total water level events reach *TWL*(1%) = 22.37 ft. NAVD for the accreted beach conditions and *TWL*(1%) = 20.47 ft. NAVD for the eroded beach conditions, again exceeding the elevations of all well sites regardless of beach erosion or accretion.

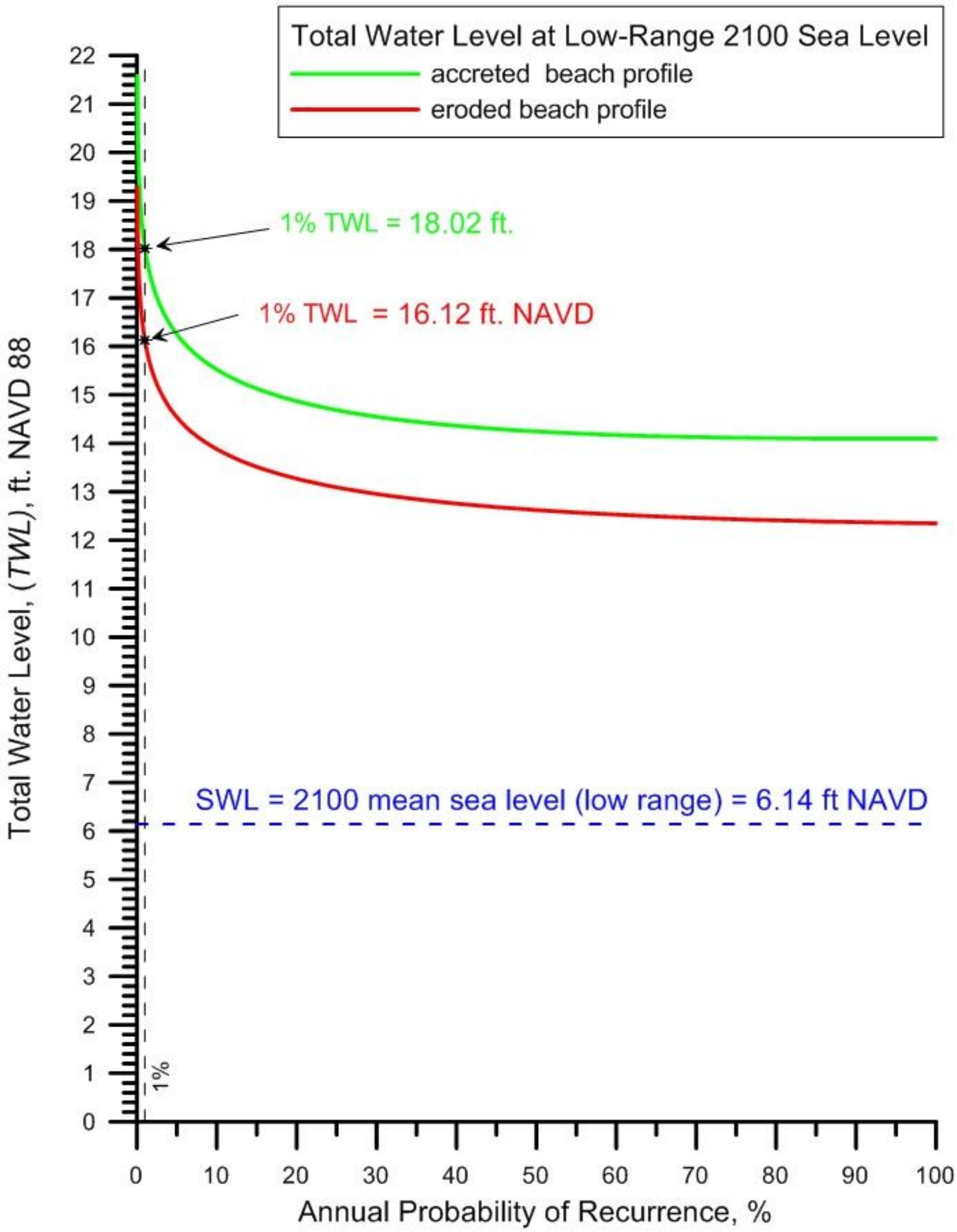


Figure 7.6: Annualized probability of recurrence of total water level at Doheny State Beach for still water level at 2100 (low range) mean sea level, based on extremal design wave heights from Weibull Type III distribution. Sea level based on CCC (2018), Appendix-G, Table G-11

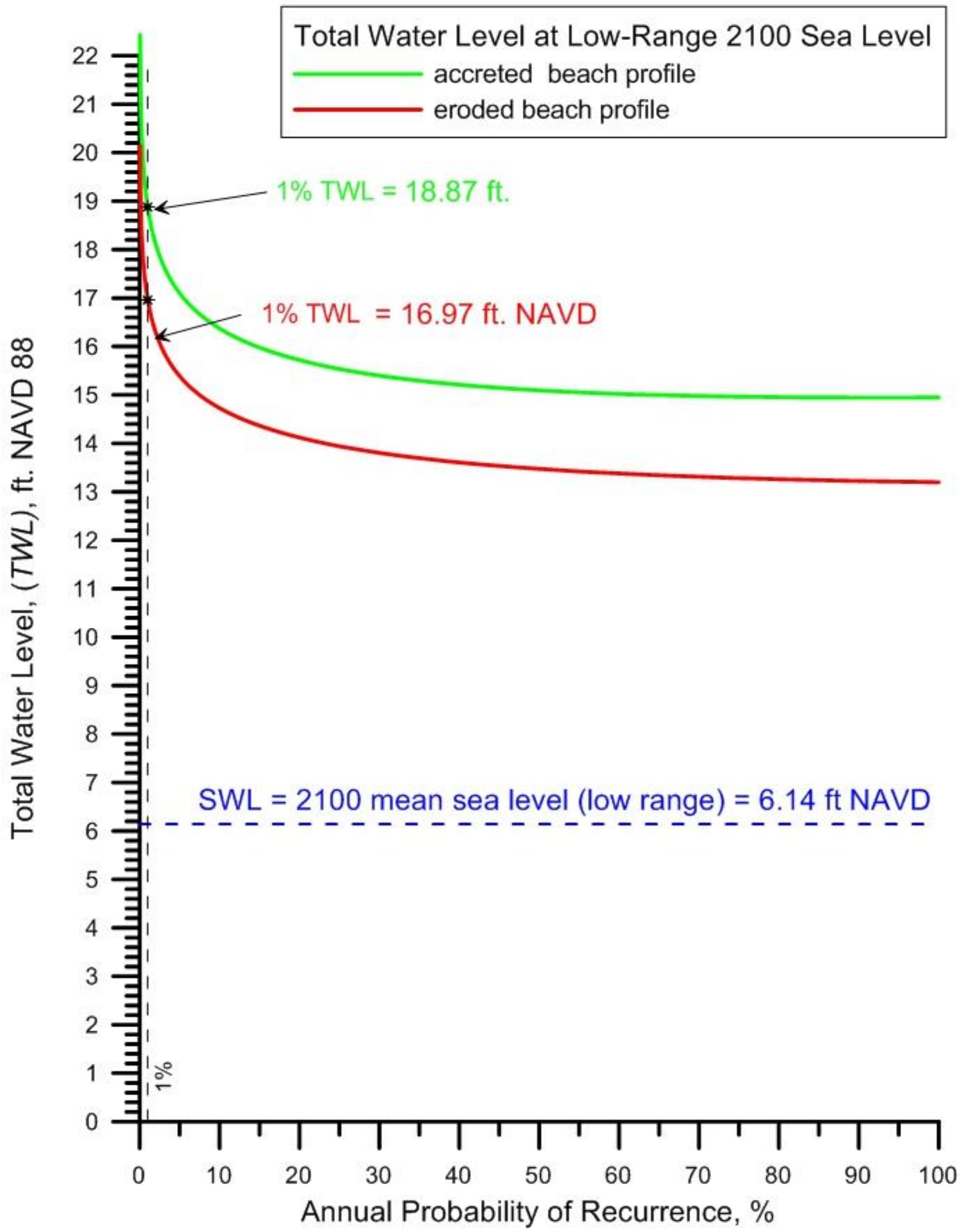


Figure 7.7: Annualized probability of recurrence of total water level at Capistrano State Beach for still water level at 2100 (low range) mean sea level, based on extremal design wave heights from Weibull Type III distribution. Sea level based on CCC (2018), Appendix-G, Table G-11

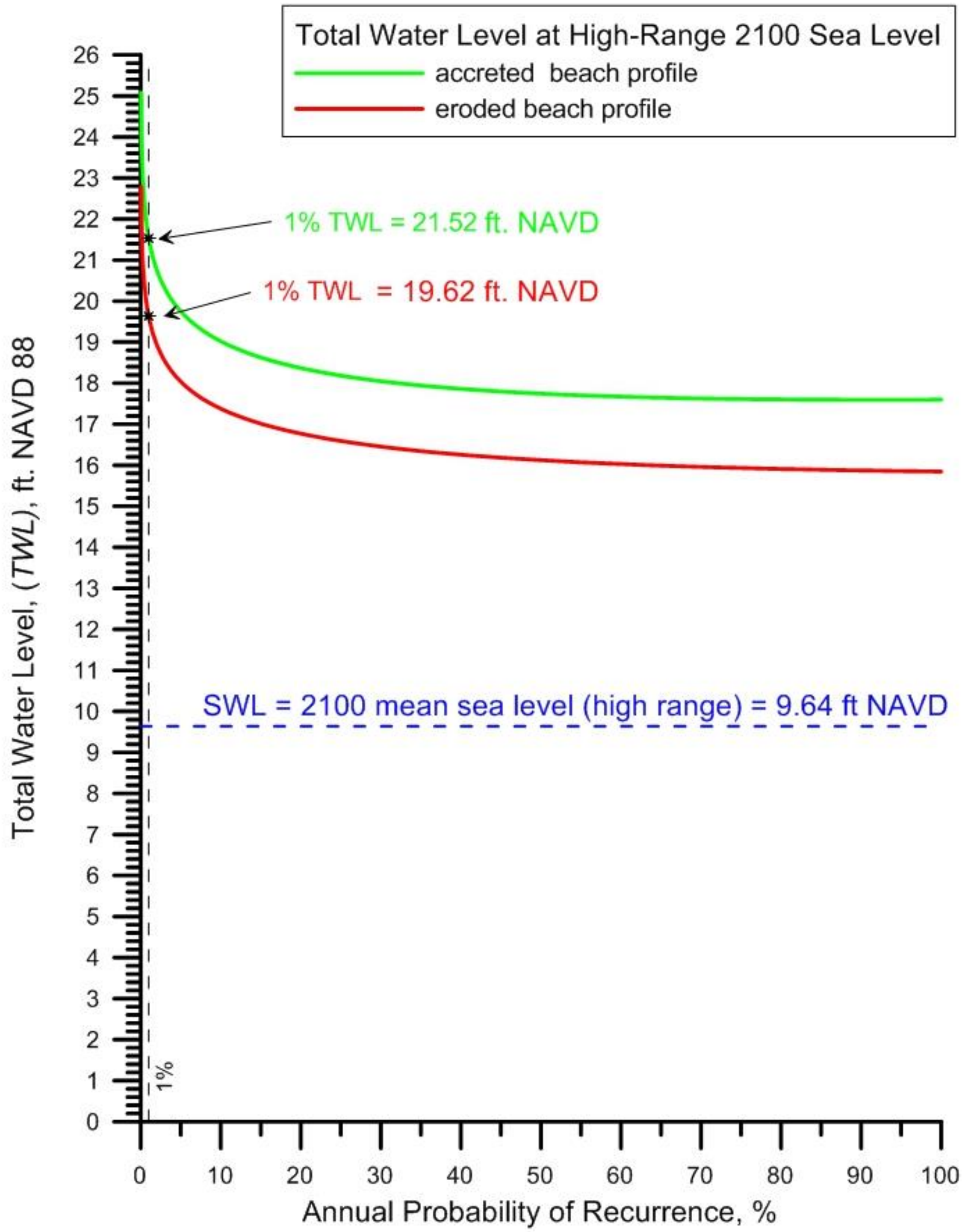


Figure 7.8: Annualized probability of recurrence of total water level at Doheny State Beach for still water level at 2100 (high range) mean sea level, based on extremal design wave heights from Weibull Type III distribution. Sea level based on CCC (2018), Appendix-G, Table G-11

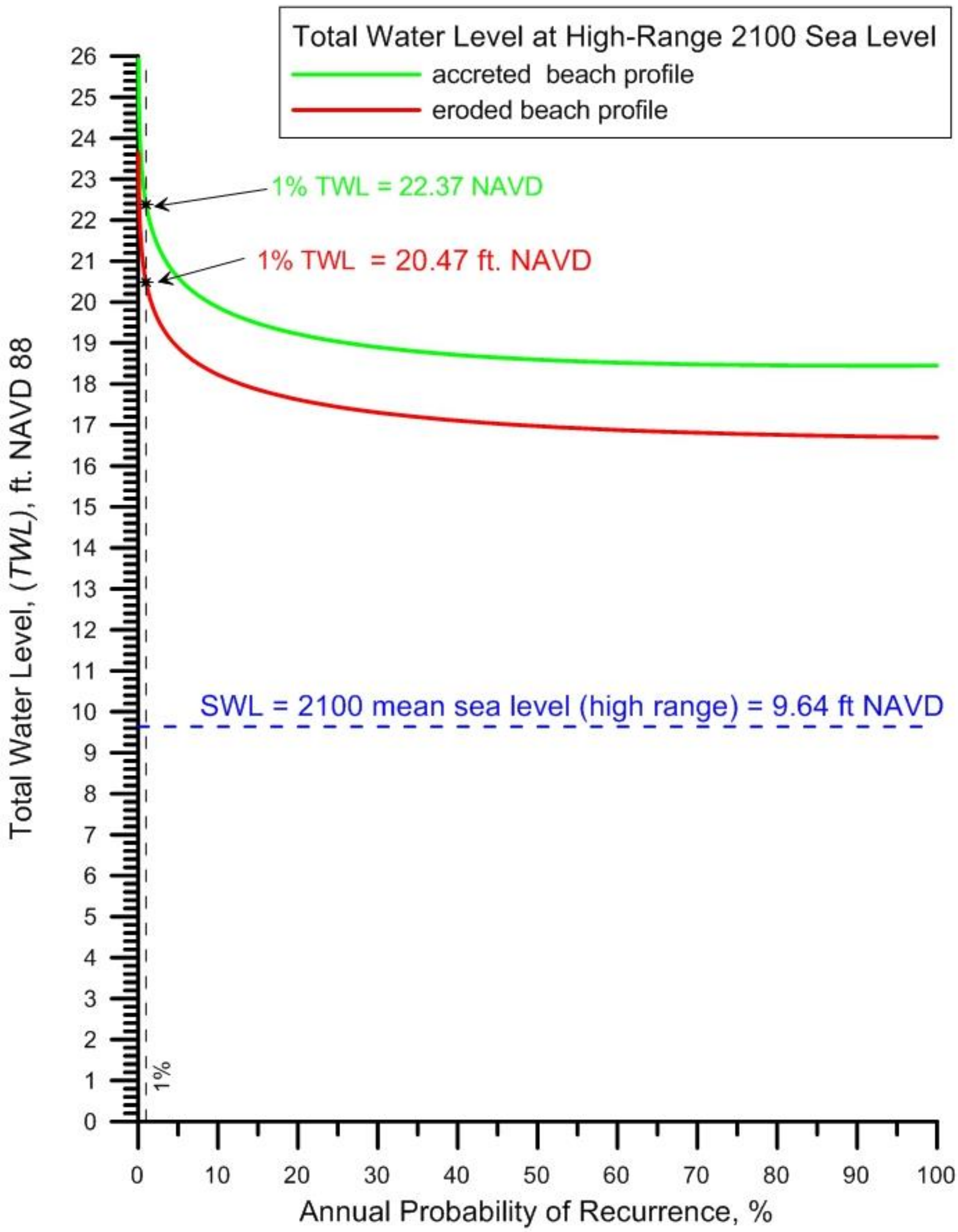


Figure 7.9: Annualized probability of recurrence of total water level at Capistrano State Beach for still water level at 2100 (high range) mean sea level, based on extremal design wave heights from Weibull Type III distribution. Sea level based on CCC (2018), Appendix-G, Table G-11

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 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 7.10 & 7.11 for Doheny and Capistrano Beaches, respectively, while those for 2100 sea levels are found in Figures 7.12 – 7.15. Comparing these results with the total water level results in Figures 7.4-7.9 (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 at present sea level at Doheny Beach in Figure 7.10 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 present 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 Figures 7.10 & 7.11 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. However, once we admit to 2100 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 low-range 2100 sea level projections, (Figures 7.12 &7.13) the three well sites at Doheny Beach on the north side of San Juan Creek (Well Heads A-C) and one of the wells at the Capistrano Beach site (Well Head G) will experience minor overtopping, even for a 1 year event if the beaches have been accreted by additional sands from water shed floods or still retain a built-out summer equilibrium beach profile, with overtopping rates of about Q'(1yr) = 0.038 cfs per lineal ft. of shoreline, per equation (17). If a 100-yr total water level event occurs during the low-range projection of 2100 sea levels, then Figures 7.12 & 7.13 indicate that all of the well sites will be overtopped to varying degrees if the beaches remain in an accreted condition (i.e.,with elevated berms and steep beach slopes). Under these beach conditions, overtopping rates will range from a high of Q'(100yr) = 0.094 cfs per lineal ft. of shoreline at Well Heads A- C on Doheny Beach, to a low of Q'(100yr) = 0.014 cfs/ft at Well Head H on Capistrano Beach, while overtopping rates at Well Heads D & E would be Q'(100yr) = 0.027 cfs/ft at Doheny Beach and Q'(100yr) = 0.081 cfs/ft at Well Head G on Capistrano Beach. Interestingly enough, none of the well heads would experience overtopping during a 100 year event when occurring during the low range 2100 sea levels if the beach were eroded, which would be the most likely condition during a 100-

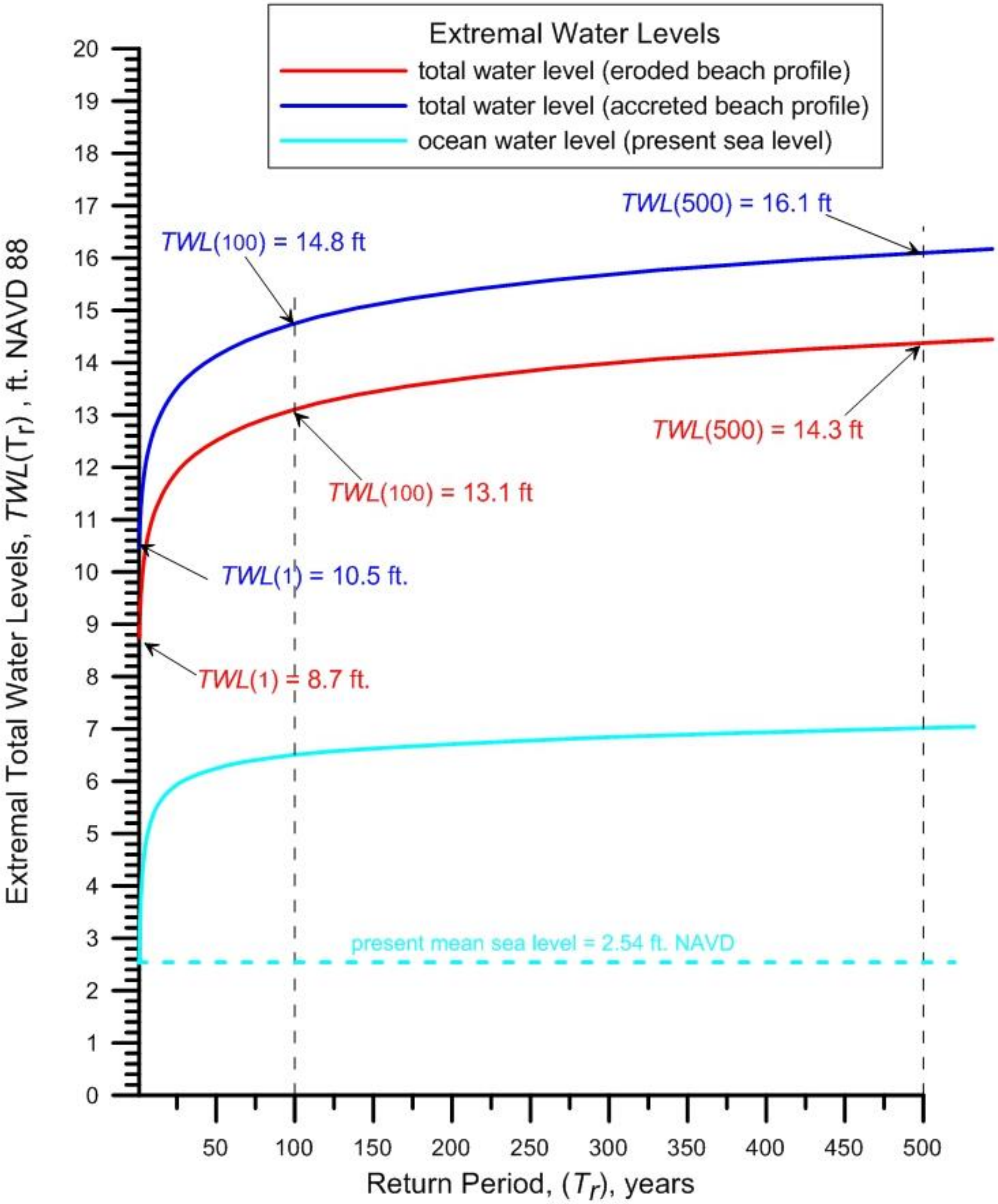


Figure 7.10: Return period of extremal total water level (TWL) at Doheny State Beach based on extremal design wave heights from Weibull Type III distribution and still water levels set at extremal ocean water levels for present sea level, per NOAA tide gage #941-0230

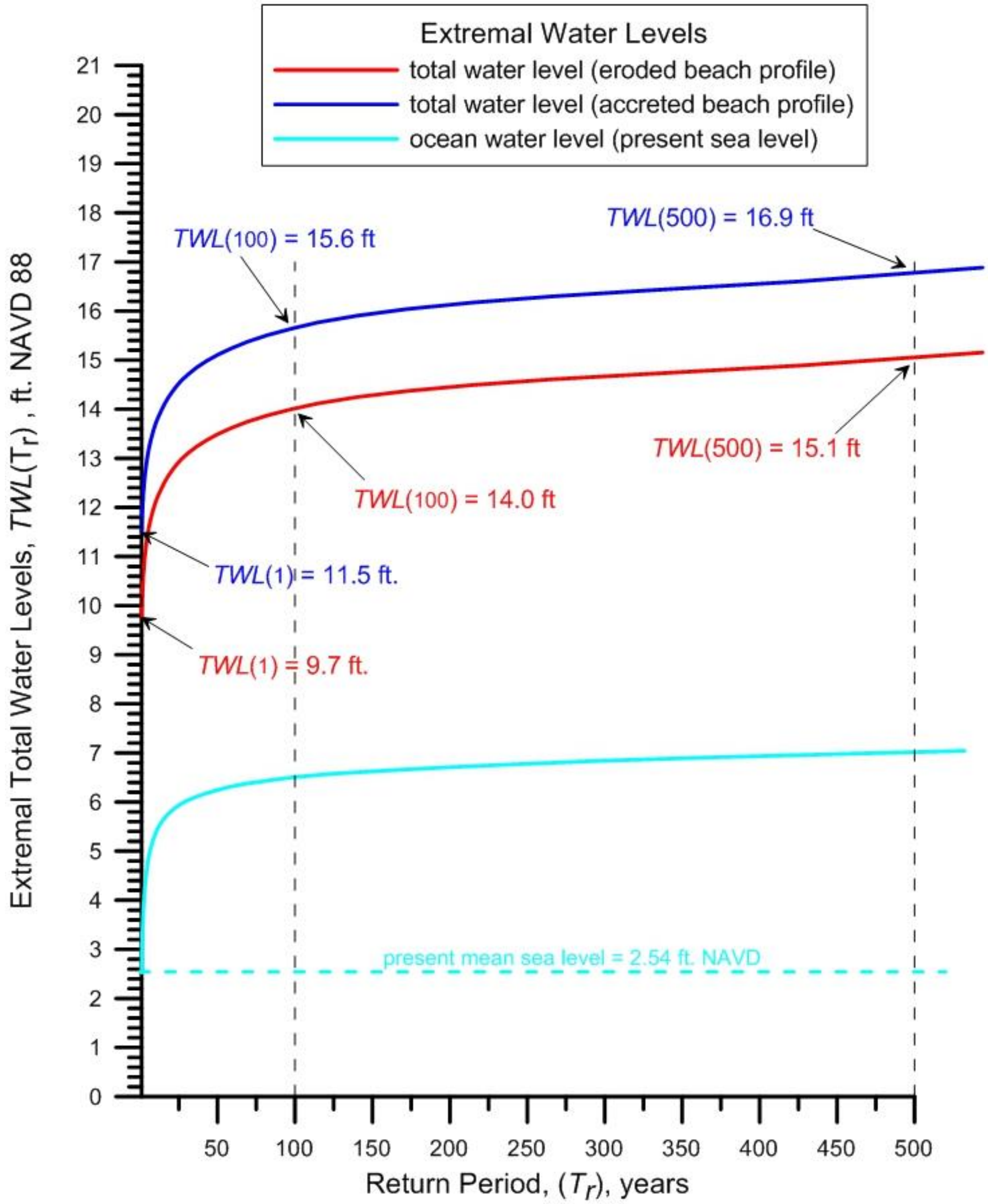


Figure 7.11: Return period of extremal total water level (TWL) at Capistrano State Beach based on extremal design wave heights from Weibull Type III distribution and still water levels set at extremal ocean water levels for present sea level, per NOAA tide gage #941-0230.

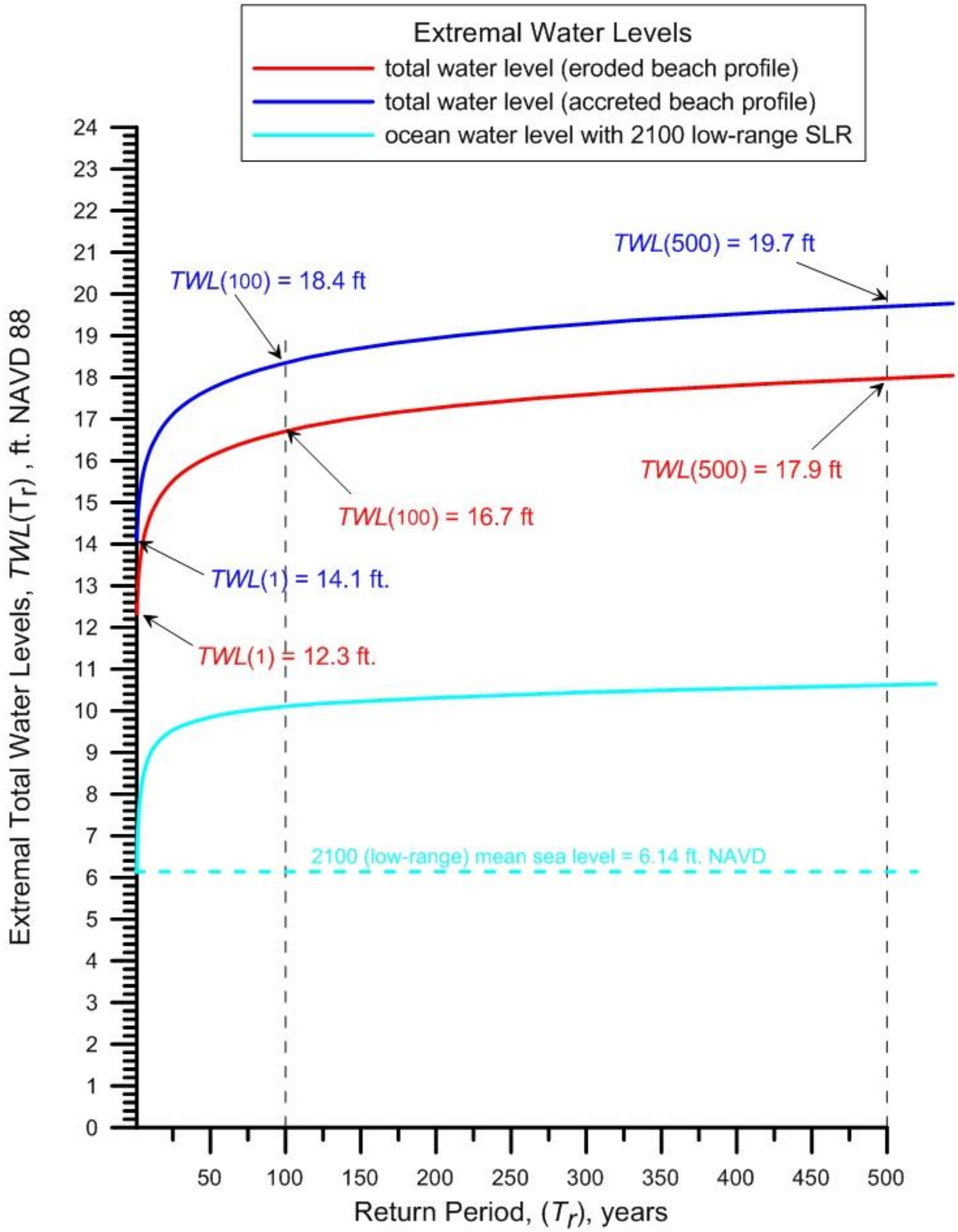


Figure 7.12: Return period of extremal total water level (TWL) at Doheny State Beach based on extremal design wave heights from Weibull Type III distribution and still water levels set at extremal ocean water levels for the low-range 2100 sea level rise, per CCC (2018), Appendix-G, Table G-11

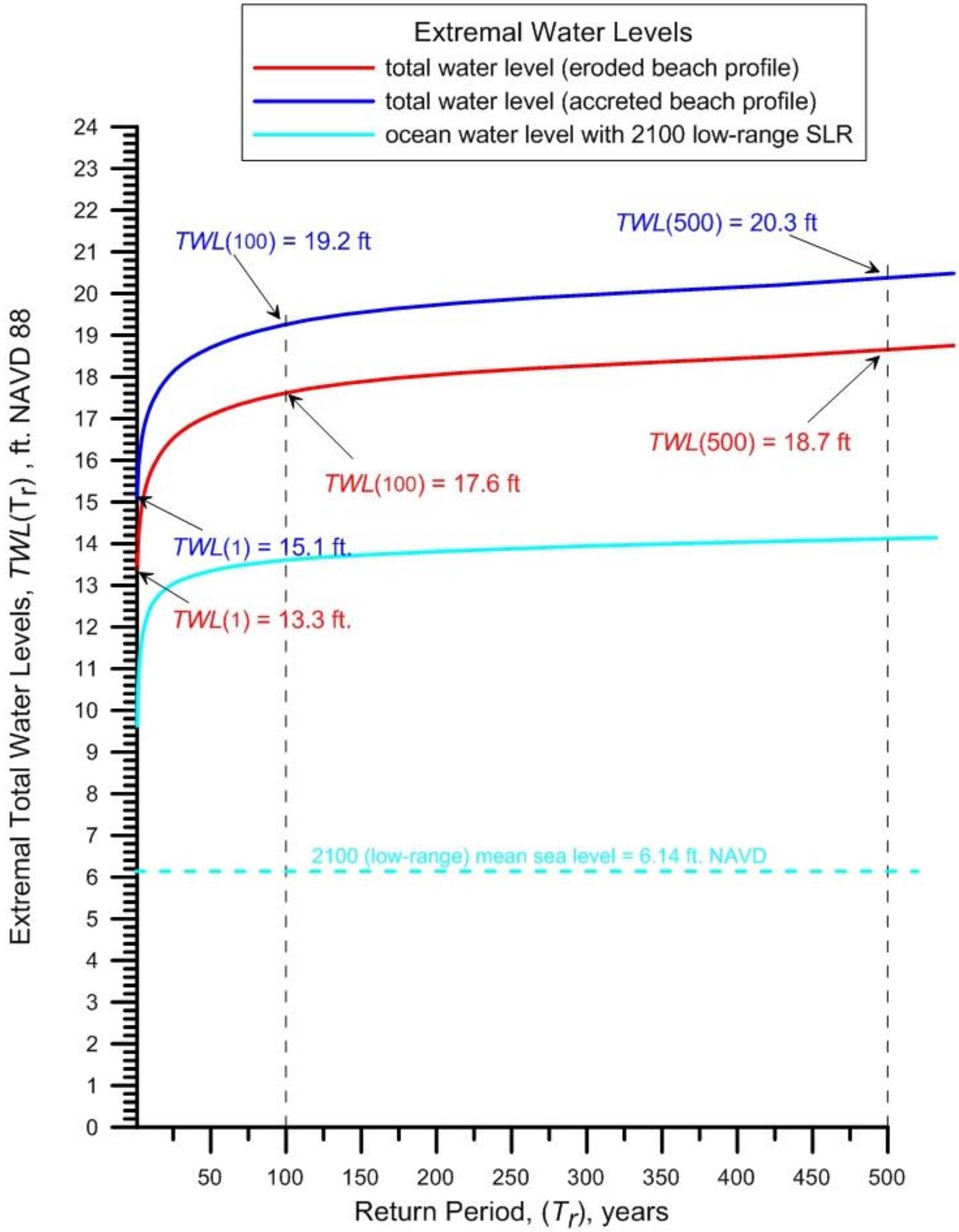


Figure 7.13: Return period of extremal total water level (TWL) at Capistrano State Beach based on extremal design wave heights from Weibull Type III distribution and still water levels set at extremal ocean water levels for the low-range 2100 sea level rise, per CCC (2018), Appendix-G, Table G-11

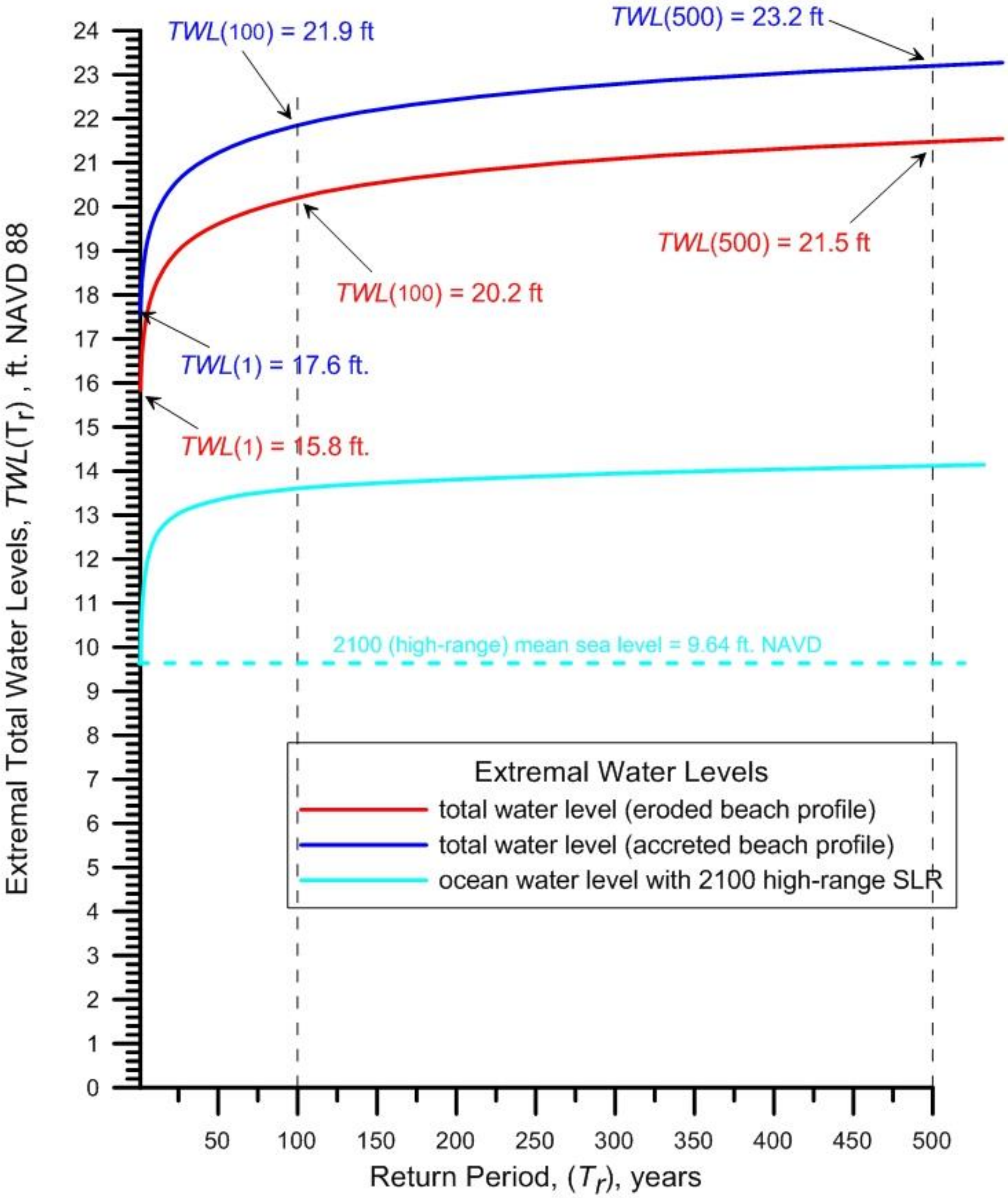


Figure 7.14: Return period of extremal total water level (TWL) at Doheny State Beach based on extremal design wave heights from Weibull Type III distribution and still water levels set at extremal ocean water levels for the high-range 2100 sea level rise, per CCC (2018), Appendix-G, Table G-11

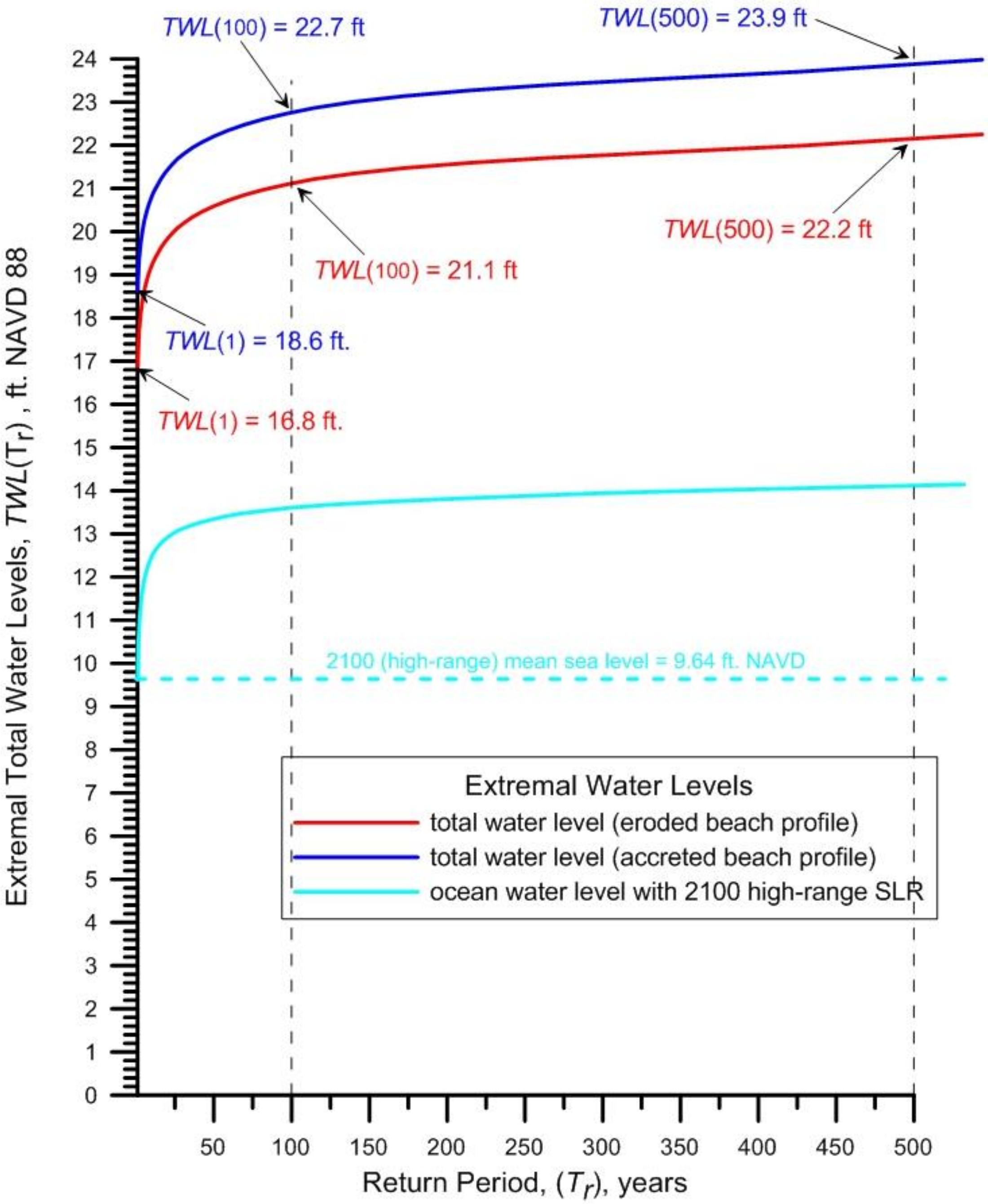


Figure 7.15: Return period of extremal total water level (TWL) at Capistrano State Beach based on extremal design wave heights from Weibull Type III distribution and still water levels set at extremal ocean water levels for the high-range 2100 sea level rise, per CCC (2018), Appendix-G, Table G-11

year event. 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 at Doheny Beach (Figure 7.14), the 100 year total water level events reach $TWL(100)$ = 21.9 ft. NAVD for the accreted beach conditions and $TWL(100)$ = 20.2 ft. NAVD for the eroded beach conditions; while at Capistrano Beach, (Figure 7.15), $TWL(100)$ = 22.7 ft. NAVD for the accreted beach conditions and $TWL(100)$ = 21.1 ft. NAVD for the eroded beach conditions. Consequently all beach front facilities for the Doheny Desalination Project would be vulnerable to flooding by the 100-year event if it were occur during 2100 high range sea level projections. The lowest lying well heads (Well Heads A-C at Doheny Beach) would experience the highest overtopping rates, ranging from Q'(100yr) = 0.216 cfs/ft. to 0.331 cfs/ft. depending on the eroded or accreted condition of Doheny State Beach. According Table VI-5-6 in the Coastal Engineering Manual (USACE, 2006) overtopping rates of this order of magnitude are very dangerous for pedestrian and vehicle traffic, and may cause structural damage to adjacent buildings; but the well heads and pumps for the Doheny Desalination project will be protected by steel vault enclosures. The smallest overtopping rates during the 100-year event at the high range 2100 sea level projections will occur at the highest located well head (Well Head H) at the Capistrano Beach site where overtopping rates will range from Q' (100yr) = 0.142 cfs/ft. to 0.250 cfs/ft., with overtopping rates at Well Heads D & E on Doheny Beach ranging from Q'(100yr) = 0.149 cfs/ft to 0.263cfs/ft and Q'(100yr) = 0.209 cfs/ft to 0.318 cfs/ft at Well Head G on Capistrano Beach. While these overtopping rates are still dangerous to pedestrian and vehicle traffic, they are easily mitigated by the steel vault enclosures of the well heads and pumps being placed at Capistrano Beach. The results for total water levels and overtopping rates based on extremal still water levels analysis methods are summarized in Table 7.1 for the Doheny Beach well sites, and in Table 7.2 for the Capistrano Beach well sites.

Table 7.1:Doheny Beach Extremal Total Water Level (*TWL) and Overtopping Rates (Q')

	Well Head-A Elevation = 17 ft. NAVD	Well Head-B Elevation = 17 ft. NAVD	Well Head-C Elevation = 17 ft. NAVD	Well Head-D Elevation = 18 ft. NAVD	Well Head-E Elevation = 18 ft. NAVD
*$TWL(1)$ Present Sea Level (eroded/accreted)	8.7/10.5 ft. NAVD status = dry	8.7/10.5ft. NAVD status = dry	8.7/10.5ft. NAVD status = dry	8.7/10.5ft. NAVD status = dry	8.7/10.5ft. NAVD status = dry
*$Q'(1)$ 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.
*$TWL(1)$ 2100 Sea Level Low Range Projection (eroded/accreted)	12.3/14.1 ft. NAVD status = dry	12.3/14.1 ft. NAVD status = dry	12.3/14.1 ft. NAVD status = dry	12.3/14.1 ft. NAVD status = dry	12.3/14.1 ft. NAVD status = dry
*$Q'(1)$ 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.
*$TWL(1)$ 2100 Sea Level High Range Projection (eroded/accreted)	15.8/17.6 ft. NAVD status = flooded accreted beach	15.8/17.6 ft. NAVD status = flooded accreted beach	15.8/17.6 ft. NAVD status = flooded accreted beach	15.8/17.6 ft. NAVD status = dry	15.8/17.6 ft. NAVD status = dry
*$Q'(1)$ 2100 Sea Level High Range Projection (eroded/accreted)	0.0/0.038 cfs/ft.	0.0/0.038 cfs/ft.	0.0/0.038 cfs/ft.	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.
**$TWL(100)$ 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.
**$TWL(100)$ 2100 Sea Level Low Range Projection (eroded/accreted)	16.7/18.4 ft. NAVD status = flooded accreted beach	16.7/18.4 ft. NAVD status = flooded accreted beach	16.7/18.4 ft. NAVD status = flooded accreted beach	16.7/18.4 ft. NAVD status = flooded accreted beach	16.7/18.4 ft. NAVD status = flooded accreted beach
**$Q'(100)$ 2100 Sea Level Low Range Projection (eroded/accreted)	0.0/0.094 cfs/ft.	0.0/0.094 cfs/ft.	0.0/0.094 cfs/ft.	0.0/0.027 cfs/ft.	0.0/0.027 cfs/ft.
**$TWL(100)$ @ 2100 Sea Level High Range Projection (eroded/accreted)	20.2/21.9 ft. NAVD status = flooded	20.2/21.9 ft. NAVD status = flooded	20.2/21.9 ft. NAVD status = flooded	20.2/21.9 ft. NAVD status = flooded	20.2/21.9 ft. NAVD status = flooded
**$Q'(100)$ 2100 Sea Level High Range Projection (eroded/accreted)	0.216/0.331 cfs/ft.	0.216/0.331 cfs/ft.	0.216/0.331 cfs/ft.	0.149/0.263 cfs/ft.	0.149/0.263 cfs/ft.

*Evaluated for the 1-yr return period; ** Evaluated for the 100-yr return period

Table 7.2: Capistrano Beach Extremal Total Water Level (TWL) and Overtopping Rates (Q')

	Well Head-G Elevation = 18 ft. NAVD	Well Head-H Elevation = 19 ft. NAVD
$*TWL(1)$ Present Sea Level (eroded/accreted)	9.7/11.5 ft. NAVD status = dry	9.7/11.5 ft. NAVD status = dry
$*Q'(1)$ Present Sea Level (eroded/accreted)	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.
$*TWL(1)$ 2100 Sea Level Low Range Projection (eroded/accreted)	13.3/15.1 ft. NAVD status = dry	13.3/15.1 ft. NAVD status = dry
$*Q'(1)$ 2100 Sea Level Low Range Projection (eroded/accreted)	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.
$*TWL(1)$ 2100 Sea Level High Range Projection (eroded/accreted)	16.8/18.6 ft. NAVD status = flooded accreted beach	16.8/18.6 ft. NAVD status = dry
$*Q'(1)$ 2100 Sea Level High Range Projection (eroded/accreted)	0.0/0.038 cfs/ft.	0.0/0.00 cfs/ft.
$**TWL(100)$ Present Sea Level (eroded/accreted)	14.0/15.6 ft. NAVD status = dry	14.0/15.6 ft. NAVD status = dry
$**Q'(100)$ Present Sea Level (eroded/accreted)	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.
$**TWL(100)$ 2100 Sea Level Low Range Projection (eroded/accreted)	17.6/19.2 ft. NAVD status = flooded accreted beach	17.6/19.2 ft. NAVD status = flooded accreted beach
$**Q'(100)$ 2100 Sea Level Low Range Projection (eroded/accreted)	0.0/0.081 cfs/ft.	0.0/0.014 cfs/ft.
$**TWL(100)$ @ 2100 Sea Level High Range Projection (eroded/accreted)	21.1/22.7 ft. NAVD status = flooded	21.1/22.7 ft. NAVD status = flooded
$**Q'(100)$ 2100 Sea Level High Range Projection (eroded/accreted)	0.209/0.318 cfs/ft.	0.142/0.250 cfs/ft.

*Evaluated for the 1-yr return period

** Evaluated for the 100-yr return period

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 the *California Coastal Commision Sea Level Rise Guidance Policy Guidance* document (CCC, 2018), 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 & 8.2 also indicate that all of the shore facilities of the Doheny Desalination Project are above tsunami inundation levels at present sea level. However, all of the well heads at both Doheny and Capistrano Beaches would suffer some degree of tsunami overtopping if concurrent with 2100 sea levels, and the overtopping rates could be quite severe, especially for the high 210 sea level rise projections. At the low range of 2100 sea level projections, total water levels would reach *TWL* = 18.82 ft. NAVD at Doheny Beach and *TWL* = 18.83 ft. NAVD at Capistrano Beach. Well Heads A-C at Doheny Beach would experience the highest overtopping surges of Q' = 1.142 cfs/ft while Well Head G at Capistrano Beach would remain high and dry. However, if the tsunami occurred atop the high range sea level rise projections for year 2100, then total water levels would reach *TWL* = 22.31 ft. NAVD at Doheny Beach and *TWL* = 22.4 ft. NAVD at Capistrano Beach, sufficient to overtop all the well sites of the Doheny Desalination Project. In this case the tsunami surge could produce very high, although short-lived, overtopping rates reaching a maximum of Q' = 5.691 cfs/ft at Well Heads A-C on Doheny Beach and a minimum of Q' = 2.916 cfs/ft at Well Head H on Capistrano Beach. Undoubtedly, the steel vault enclosures of the well heads can be designed to withstand these high surge rates, but particular attention should be given to the foundations of the vaults to assure the foundations have adequate depth to prevent undercutting by scour. 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 inundated by a shoaling tsunami solitary wave.

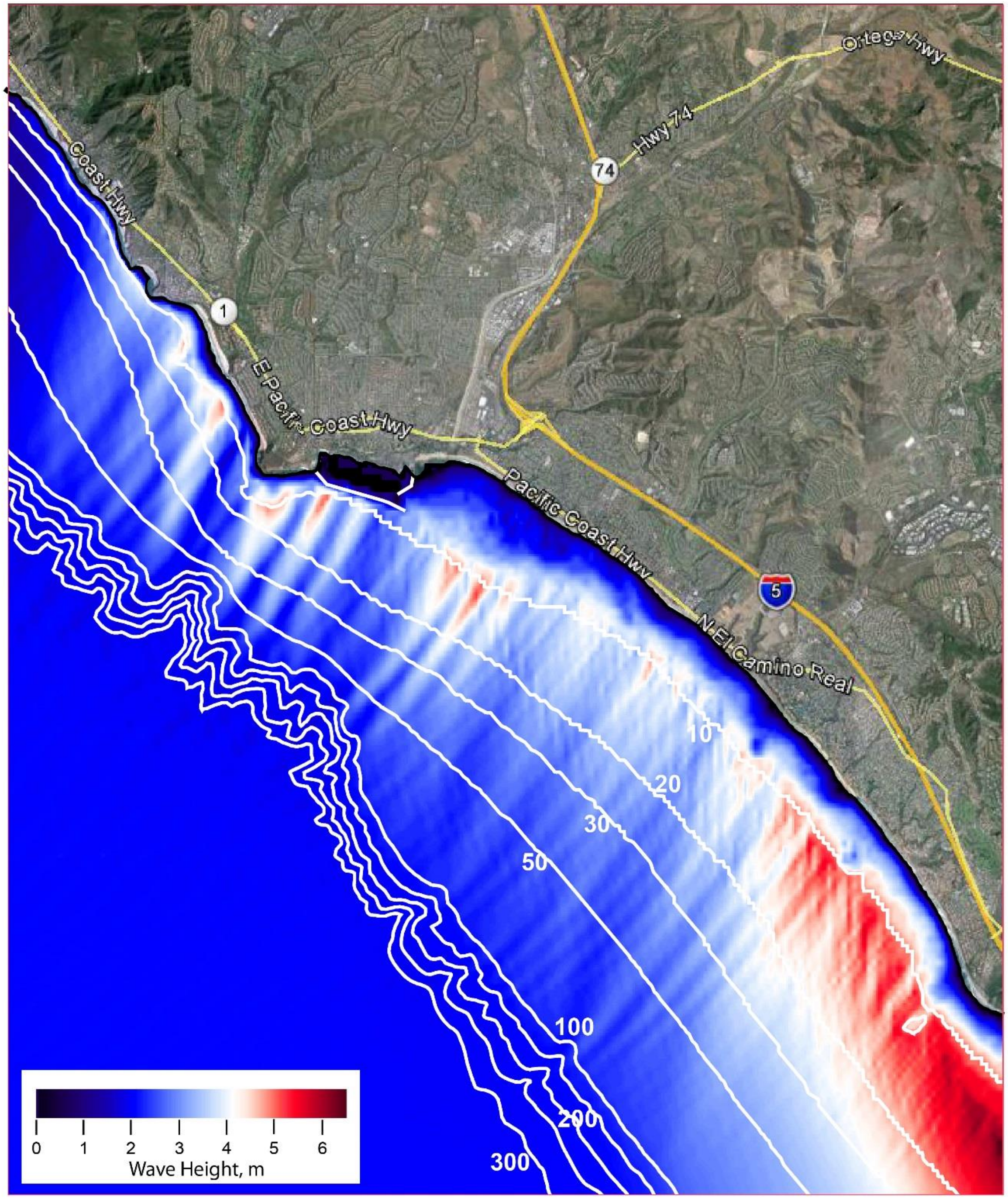


Figure 8.1: High resolution refraction/diffraction computation for a 2m high solitary tsunami wave approaching Doheny Beach from 165 degrees true.

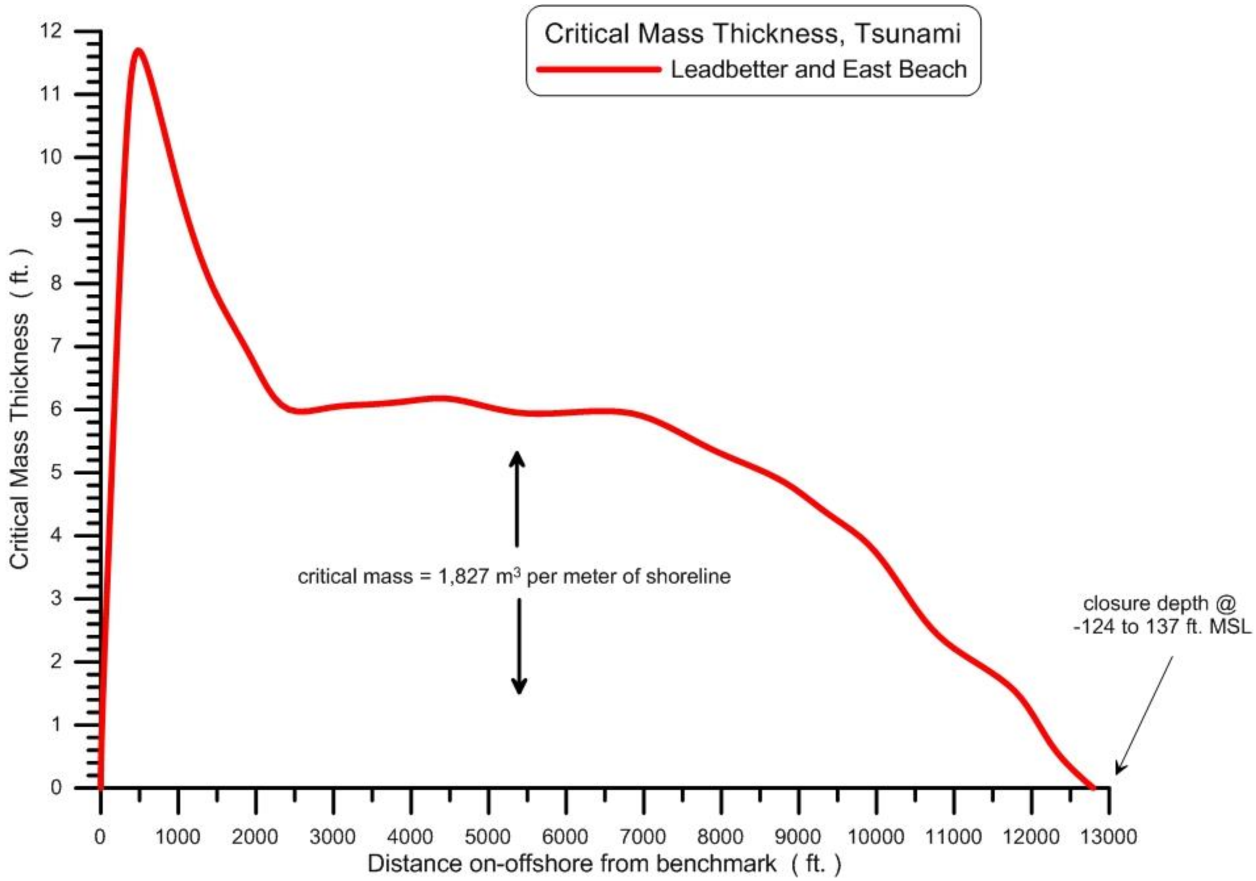


Figure 8.2: Thickness of critical mass envelope at historic survey ranges Doheny Beach, calculated by the calibrated CEM sediment budget based a 2m high solitary tsunami wave approaching Doheny Beach from 165 degrees true. Closure depth = -124 to -137 ft. MSL; critical mass volume = 1,827 m^3 per meter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

Table 8.1: Tsunami Total Water Level (*TWL*) and Overtopping Rates (Q') Analysis at the Doheny Beach Site

	Well Head-A Elevation = 17 ft. NAVD	Well Head-B Elevation = 17 ft. NAVD	Well Head-C Elevation = 17 ft. NAVD	Well Head-D Elevation = 18 ft. NAVD	Well Head-E 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 cfs/ft.	0.0 cfs/ft.	0.0 cfs/ft.	0.0 cfs/ft.	0.0 cfs/ft.
TWL 2100 Sea Level Low Range Projection	18.82 ft. NAVD status = flooded	18.82 ft. NAVD status = flooded	18.82 ft. NAVD status = flooded	18.82 ft. NAVD status = flooded	18.82 ft. NAVD status = flooded
Q' 2100 Sea Level Low Range Projection	1.142 cfs/ft.	1.142 cfs/ft.	1.142 cfs/ft.	0.345 cfs/ft.	0.345 cfs/ft.
TWL @ 2100 Sea Level High Range Projection	22.31 ft. NAVD status = flooded	22.31 ft. NAVD status = flooded	22.31 ft. NAVD status = flooded	22.31 ft. NAVD status = flooded	22.31 ft. NAVD status = flooded
Q' 2100 Sea Level High Range Projection	5.691 cfs/ft.	5.691 cfs/ft.	5.691 cfs/ft.	4.162 cfs/ft.	4.162 cfs/ft.

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

Table ES-2b: Tsunami Total Water Level (TWL) and Overtopping Rates (Q') Analysis at the Capistrano Beach Site

	Well Head-G Elevation = 18 ft. NAVD eroded/accreted	Well Head-H Elevation = 19 ft. NAVD eroded/accreted
*TWL Present Sea Level (eroded)	15.3 ft. NAVD status = dry	15.3 ft. NAVD status = dry
*Q' Present Sea Level (eroded)	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.
*TWL 2100 Sea Level Low Range Projection (eroded)	18.83 ft. NAVD status = flooded	18.83 ft. NAVD status = dry
*Q' 2100 Sea Level Low Range Projection (eroded)	0.352 cfs/ft.	0.0/0.0 cfs/ft.
*TWL 2100 Sea Level High Range Projection (eroded)	22.4 ft. NAVD status = flooded	22.4 ft. NAVD status = flooded
*Q' 2100 Sea Level High Range Projection (eroded)	4.293 cfs/ft.	2.916 cfs/ft.

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

9) Summary and Conclusions:

This 2019 study, prepared in response to comments for the Final EIR, provides further analysis to amplify the Coastal Hazards Analysis prepared in 2017 for the Draft EIR of the Doheny Desalination Project. That earlier work is being amplified herein in response to a revision of the *California Coastal Commission Sea Level Rise Policy Guidance* document that was originally released in August 2015, (CCC, 2015), but has been updated in July 2018 with new sea level rise projections. In addition, there have been minor adjustments in the locations of a number of the well heads and pump stations being proposed for the Doheny Desalination Project. The following study accounts for these intervening changes in policy guidance and minor modifications to the project description.

The primary analysis tool used in this study is the *Coastal Evolution Model* (CEM) developed at the Scripps Institution of Oceanography was used to evaluate Appendix-B requirements of the *California Coastal Commission 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 grain 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 Oceanic and Atmospheric 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 Commission 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 or tsunami concurrent with extreme ocean water levels and sea level rise are placed at two sites, namely Doheny State Beach and Capistrano State Beach. At the Doheny Beach site, five potential locations are being evaluated for vaulted well heads with submersible pumps, including : ***Well Head A***, elevation 17 ft. NAVD, at 33°27'44.38"N, 117°41'16.32"W; ***Well Head B***, elevation 17 ft. NAVD, at 33°27'45.07"N, 117°41'10.30"W; ***Well Head C***, elevation 17 ft. NAVD at 33°27'45.12"N, 117°41'6.62"W; ***Well Head D***, at elevation 18 ft. NAVD at 33°27'44.48"N, 117°40'55.30"W; and ***Well Head E***, at elevation 18 ft. NAVD at 33°27'42.45"N, 117°40'47.33"W. Two additional vaulted well heads with submersible pumps are being evaluated at the Capistrano Beach site, which includes: ***Well Head G***, at elevation 18 ft. NAVD at 33°27'14.94"N, 117°39'59.91"W; and ***Well Head H***, at elevation 19 ft. NAVD at 33°27'13.17"N, 117°39'57.15"W.

This study is based on sea level rise projections appearing in Appendix-G, Table G-11, of the recently updated *California Coastal Commission Sea Level Rise Policy Guidance* document (CCC, 2018). This document 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 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 14 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 at present and at year 2100 sea levels, is summarized in Table 7.1 for structures at the Doheny Beach site and Table 7.2 for the Capistrano Beach site. Inspection of Table 7.1 & 7.2 reveals that all the beach front well sites for the Doheny Desalination Project are safe from flooding or inundation at present sea levels by extreme event waves concurrent with extreme ocean water levels for event return periods between 1 yr. and 100 yr. However, once we admit to 2100 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 low-range 2100 sea level projections, the three well sites on the north side of San Juan Creek (Well Heads A-C) and one of the wells at the Capistrano Beach site (Well Head G) will experience minor overtopping, even for a 1 year event if the beaches have been accreted by additional sands from water shed floods or still retain a built-out summer equilibrium beach profile, with overtopping rates of about Q'(1yr) = 0.038 cfs per lineal ft. of shoreline. However, if a 100-yr total water level event occurs during the low-range projection of 2100 sea levels, then all of the well sites will be overtopped to varying degrees if the beaches remain in an accreted condition with elevated berms and steep beach slopes. Under these beach conditions, overtopping rates will range from a high of Q'(100yr) = 0.094 cfs per lineal ft. of shoreline at Well Heads A- C, to a low of Q'(100yr) = 0.014 cfs/ft at Well Head H. Interestingly enough, none of the well heads would experience overtopping during a 100 year event when occurring during the low range 2100 sea levels if the beach were eroded, which would be the most likely condition during a 100-year event. 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 at the Doheny Beach site reach *TWL*(100) = 21.9 ft. NAVD for the steeply sloping accreted beach conditions and *TWL*(100) = 20.2 ft. NAVD for the eroded beach conditions. At the Capistrano Beach site, shoaling wave heights are higher and total water levels for a 100 year event superimposed on the high range projections for 2100 sea levels produce total water levels reaching *TWL*(100) = 22.7 ft. NAVD for the steeply sloping accreted beach conditions and *TWL*(100) = 21.1 ft. NAVD for the eroded beach conditions. Consequently, all beach front well heads for the Doheny Desalination Project will be overtopped and flooded when extreme waves happen concurrently with extreme ocean water levels that are superimposed on the high range of 2100 sea levels. The lowest lying well heads (Well Heads A-C) would experience the highest overtopping rates, ranging from Q'(100yr) = 0.216 cfs/ft. to 0.331 cfs/ft. depending on the eroded or accreted condition of Doheny State Beach. According Table VI-5-6 in the Coastal Engineering Manual (USACE, 2006) overtopping rates of this order of magnitude are very dangerous for pedestrian and vehicle traffic, and may cause structural damage to adjacent buildings, but the well heads and pumps for the Doheny Desalination project will be protected by steel vault enclosures. The smallest overtopping rates during the 100-year event at the high range2100 sea level projections will occur at the highest located well head (Well Head H) at the Capistrano Beach site where overtopping rates will range from Q'(100yr) = 0.142 cfs/ft. to 0.250 cfs/ft. While these overtopping rates are still dangerous to pedestrian and vehicle traffic, they are easily mitigated by the steel vault enclosures of the well heads and pumps being placed at Capistrano Beach.

Tsunami induced erosion, runup, and inundation were analyzed for the Doheny and Capistrano State Beaches 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 analysis was based on numerical refraction/diffraction codes for a shoaling solitary wave. 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 and Capistrano Beaches. 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 Table 8.1 for the Doheny Beach site, and Table 8.2 for the Capistrano Beach site. These tables indicate that all of the shore facilities of the Doheny Desalination Project are above tsunami inundation levels at present sea level. However, all of the well heads at both Doheny and Capistrano Beaches would suffer some degree of tsunami overtopping if concurrent with 2100 sea levels, and the overtopping rates could be quite severe, especially for the high 210 sea level rise projections. At the low range of 2100 sea level projections, total water levels would reach TWL = 18.82 ft. NAVD at Doheny Beach and TWL = 18.83 ft. NAVD at Capistrano Beach. Well Heads A-C at Doheny Beach would experience the highest overtopping surges of Q' = 1.142 cfs/ft while Well Head G at Capistrano Beach would remain high and dry. However, if the tsunami occurred atop the high range sea level rise projections for year 2100, then total water levels would reach TWL = 22.31 ft. NAVD at Doheny Beach and TWL = 22.4 ft. NAVD at Capistrano Beach, sufficient to overtop all the well sites of the Doheny Desalination project. In this case the tsunami surge could produce very high, although short-lived, overtopping rates reaching a maximum of Q' = 5.691 cfs/ft at Well Heads A-C on Doheny Beach and a minimum of Q' = 2.916 cfs/ft at Well Head H on Capistrano Beach. Undoubtedly, the steel vault enclosures of the well heads can be designed to withstand these high surge rates, but particular attention should be given to the foundations of the vaults to assure those foundations have adequate depth to prevent undercutting by scour. 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 inundated by a shoaling tsunami solitary wave.

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

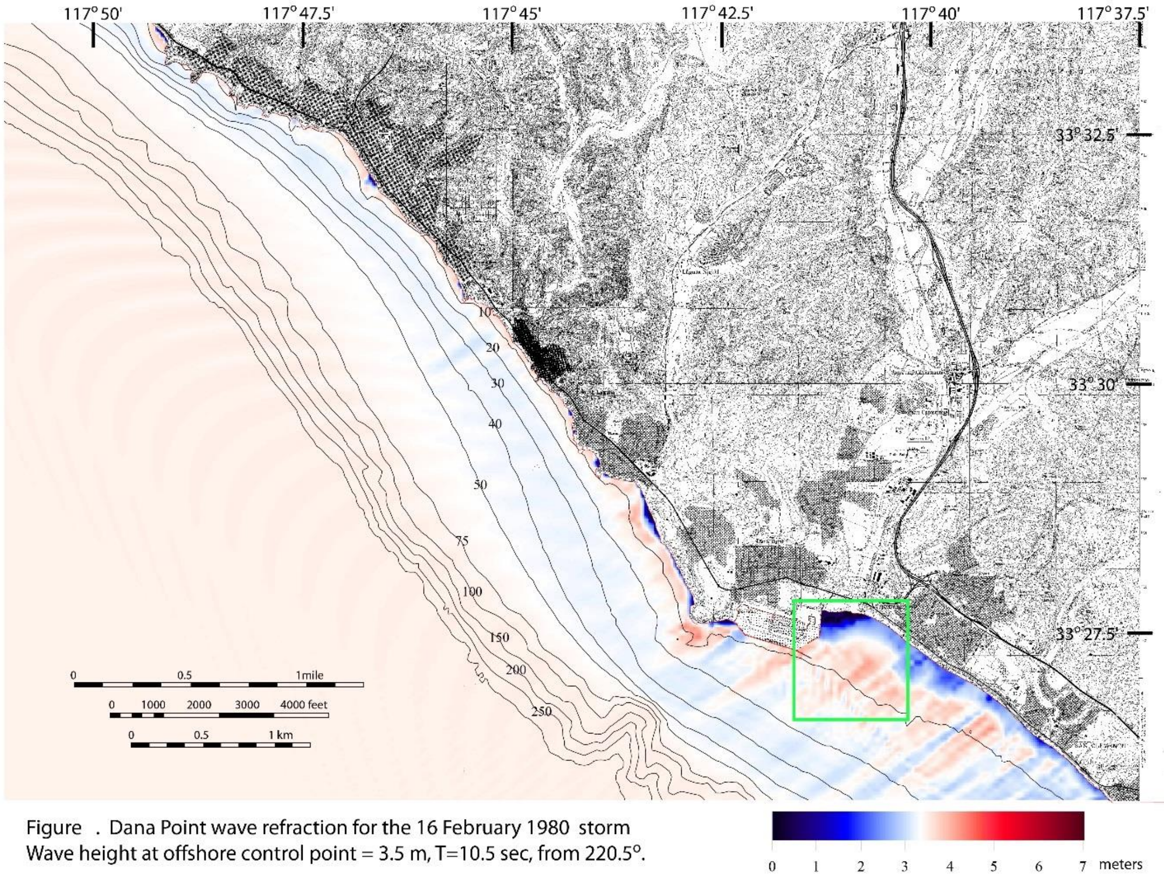


Figure . Dana Point wave refraction for the 16 February 1980 storm
Wave height at offshore control point = 3.5 m, T=10.5 sec, from 220.5°.

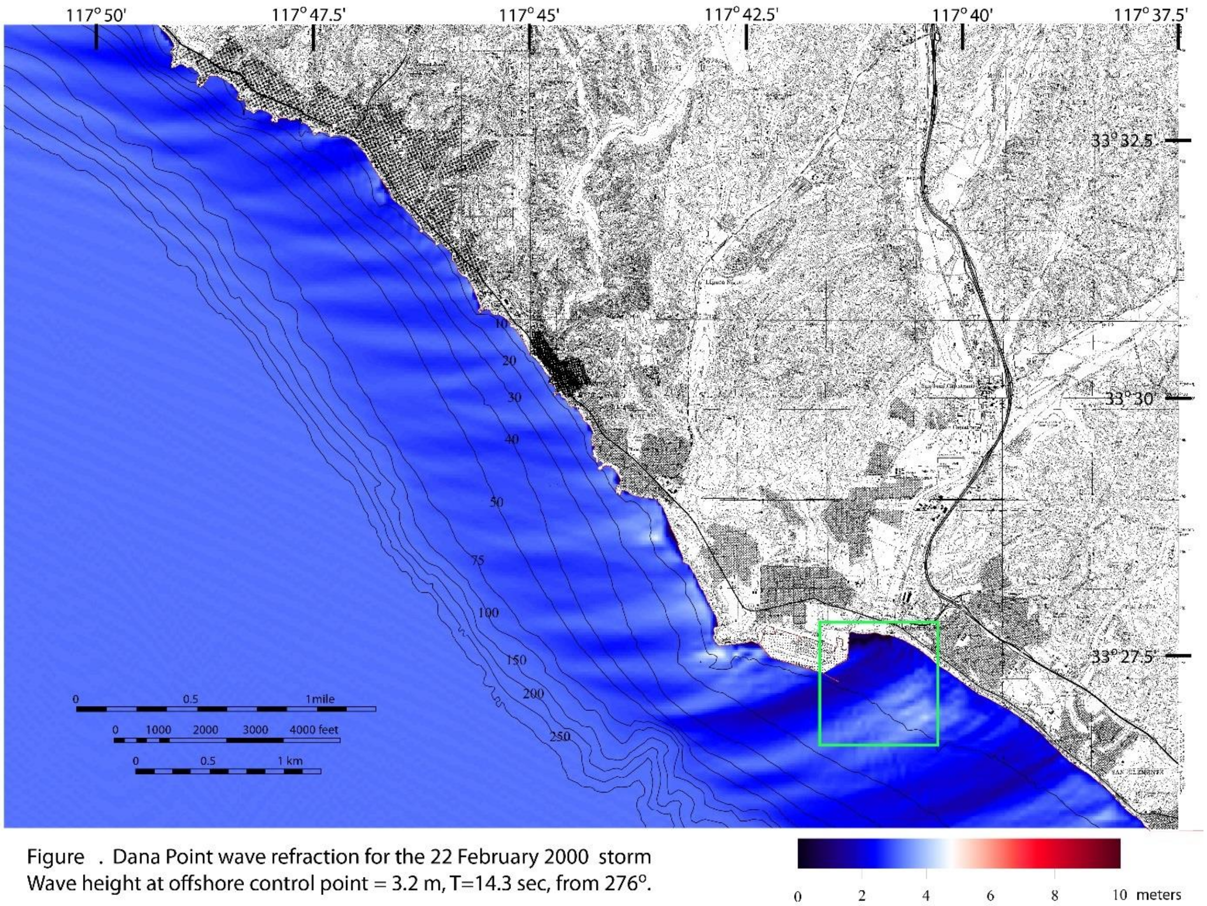


Figure . Dana Point wave refraction for the 22 February 2000 storm
Wave height at offshore control point = 3.2 m, T=14.3 sec, from 276°.

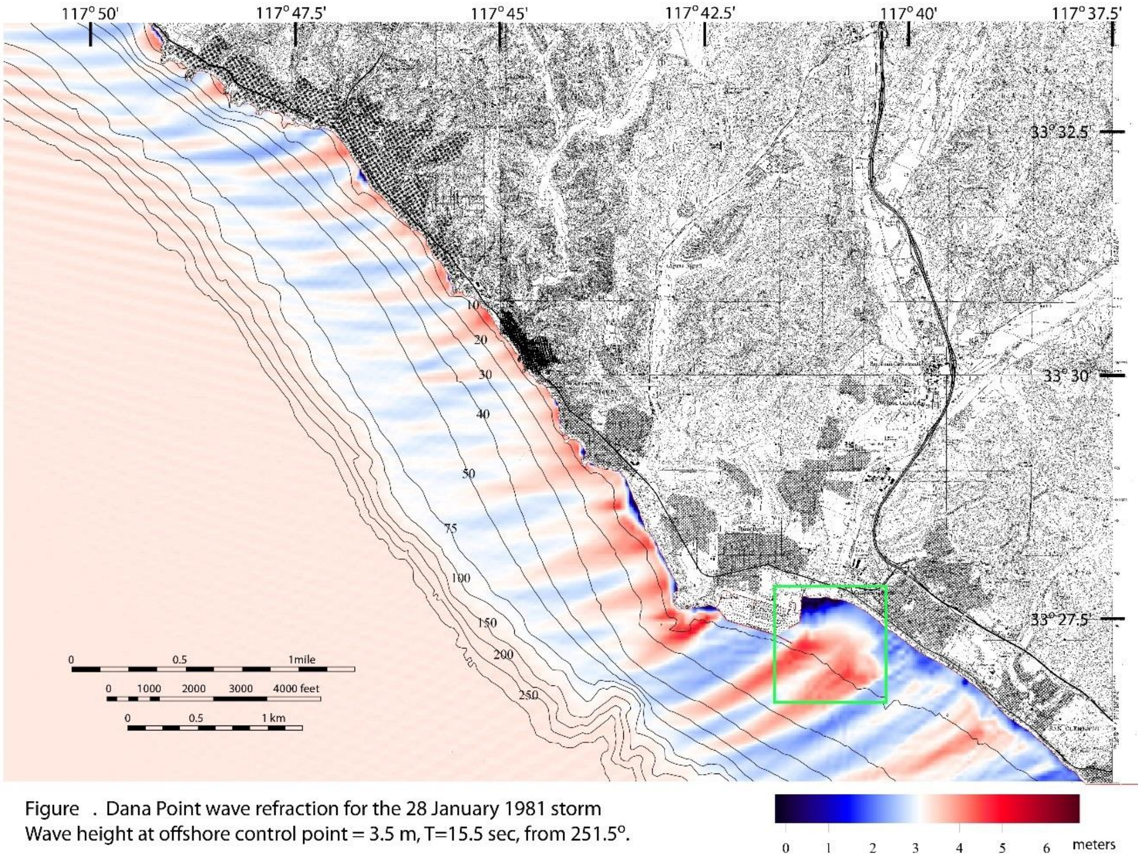


Figure . Dana Point wave refraction for the 28 January 1981 storm
Wave height at offshore control point = 3.5 m, T=15.5 sec, from 251.5°.

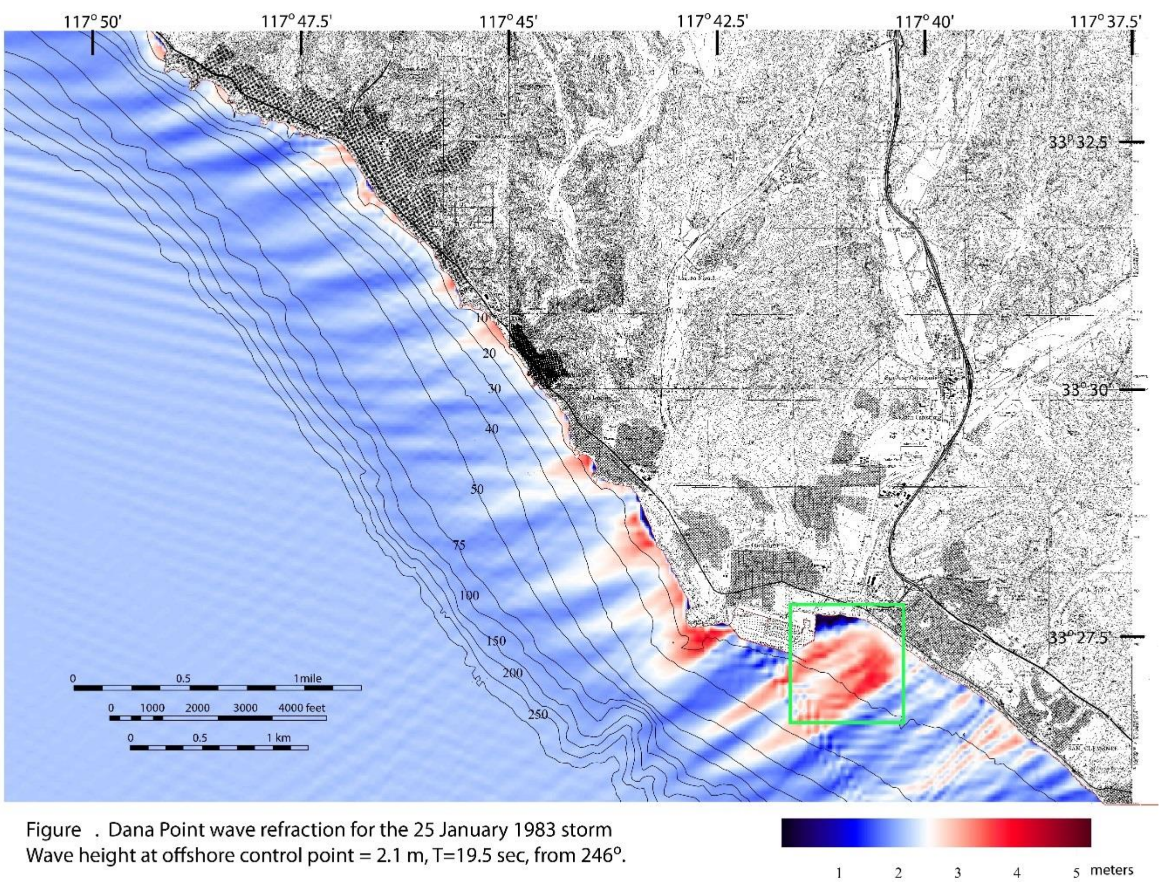


Figure . Dana Point wave refraction for the 25 January 1983 storm
Wave height at offshore control point = 2.1 m, T=19.5 sec, from 246°.

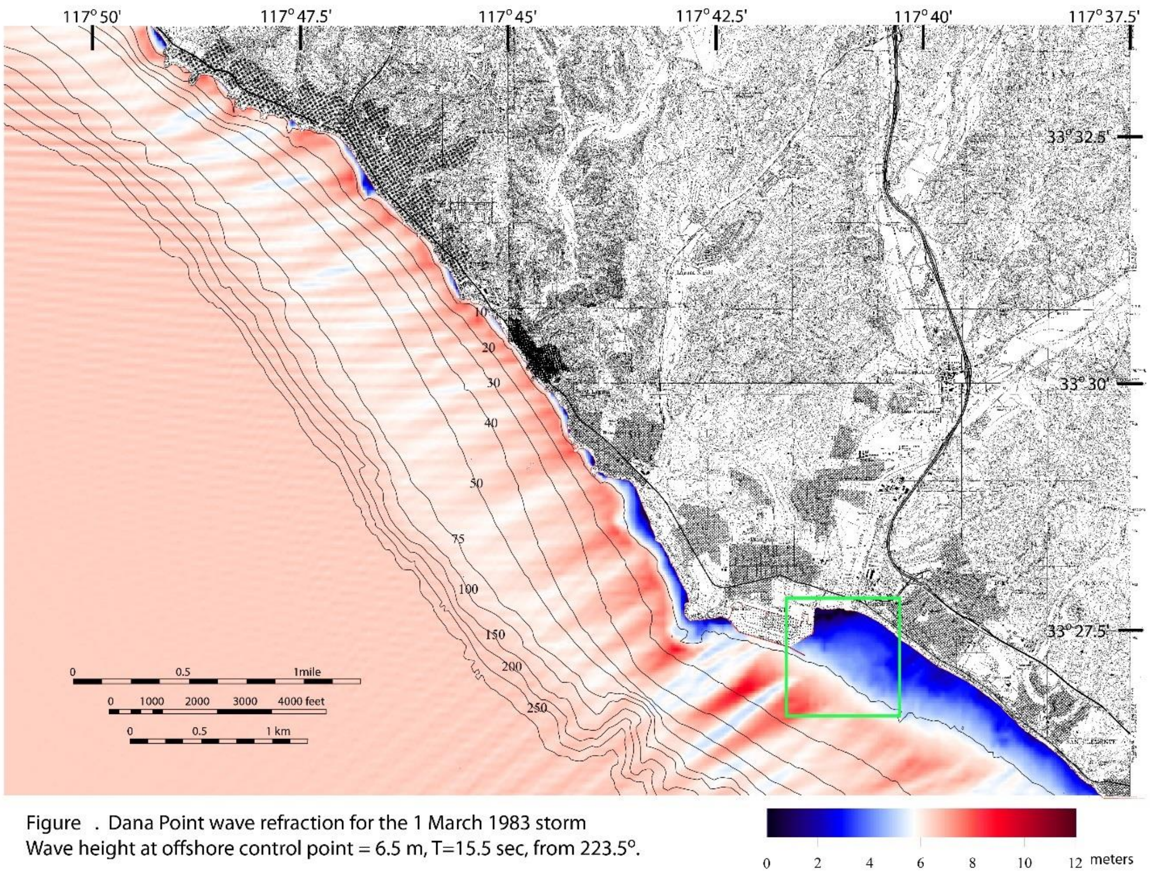


Figure . Dana Point wave refraction for the 1 March 1983 storm
Wave height at offshore control point = 6.5 m, T=15.5 sec, from 223.5°.

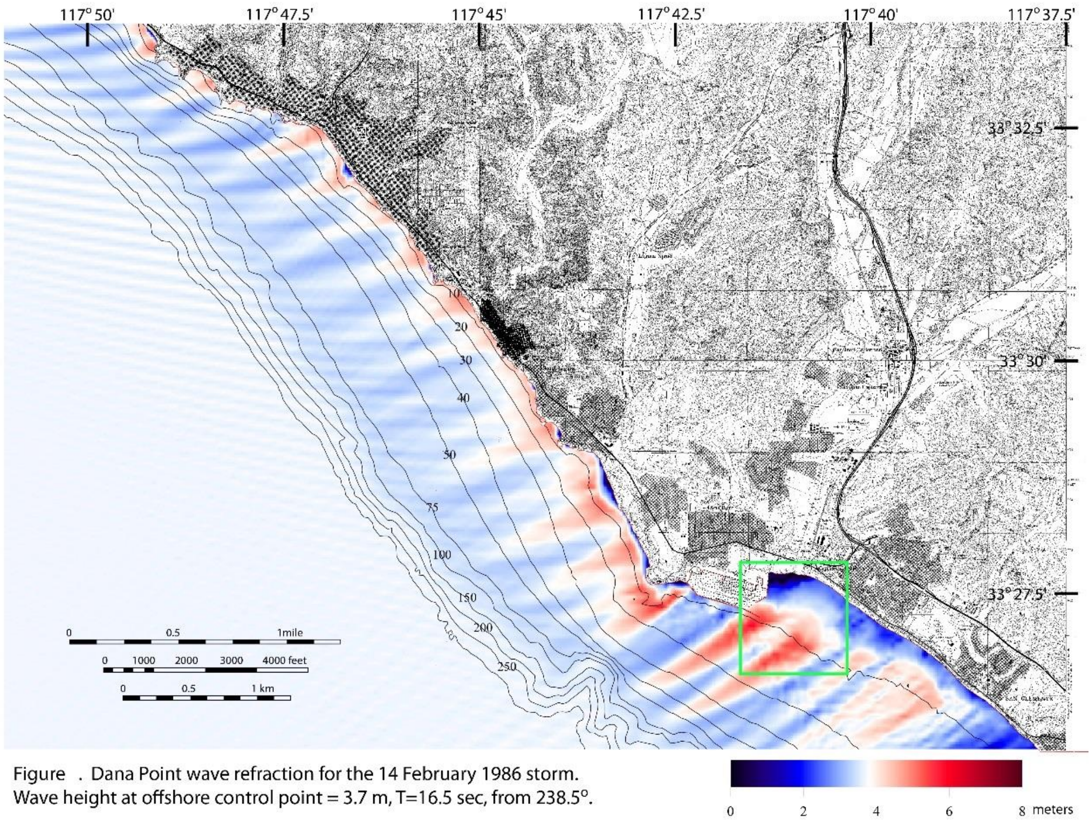
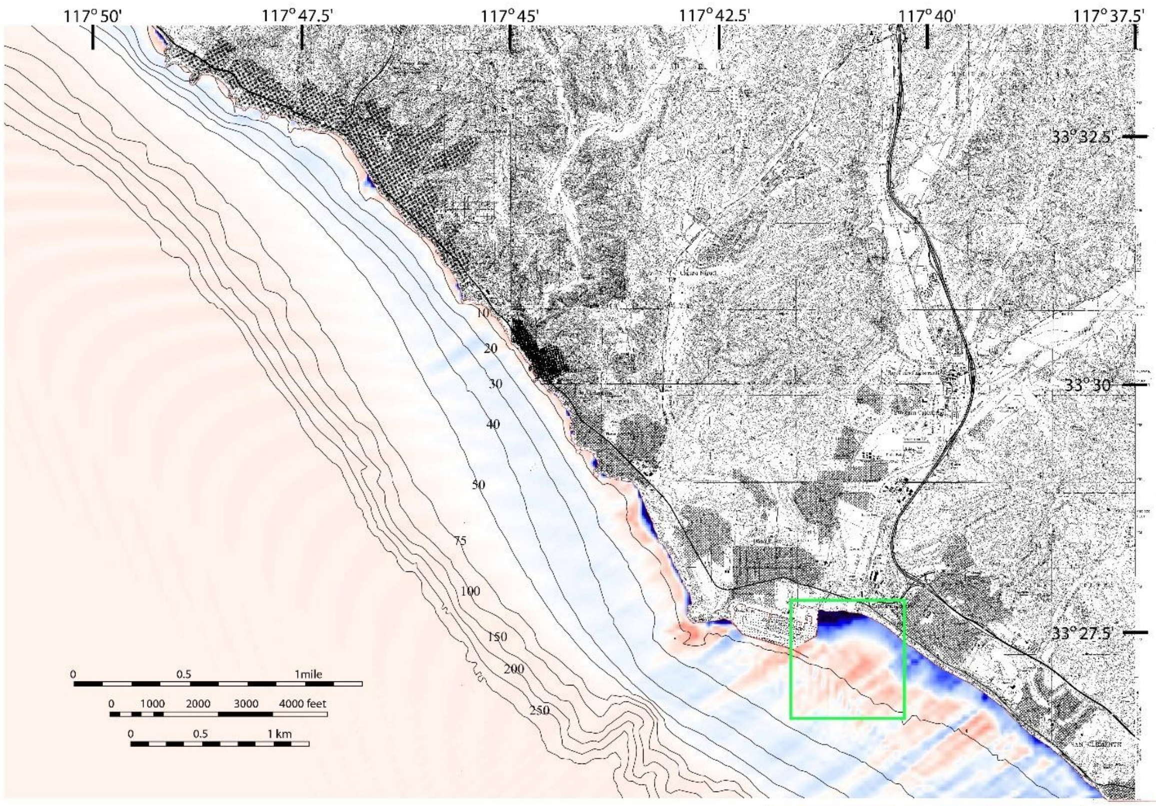


Figure . Dana Point wave refraction for the 14 February 1986 storm.
Wave height at offshore control point = 3.7 m, T=16.5 sec, from 238.5°.



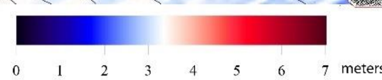


Figure . Dana Point wave refraction for the 16 February 1980 storm
Wave height at offshore control point = 3.5 m, T=10.5 sec, from 220.5°.

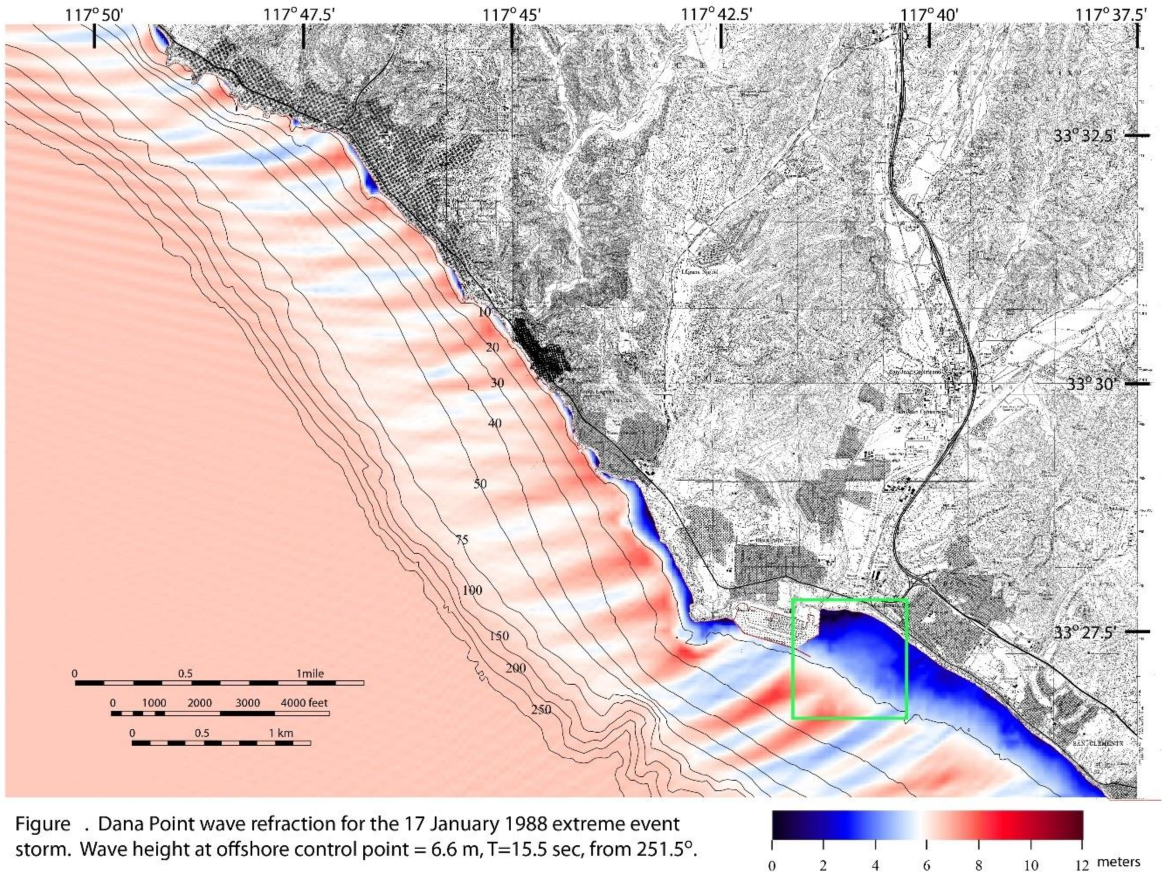


Figure . Dana Point wave refraction for the 17 January 1988 extreme event storm. Wave height at offshore control point = 6.6 m, T=15.5 sec, from 251.5°.

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

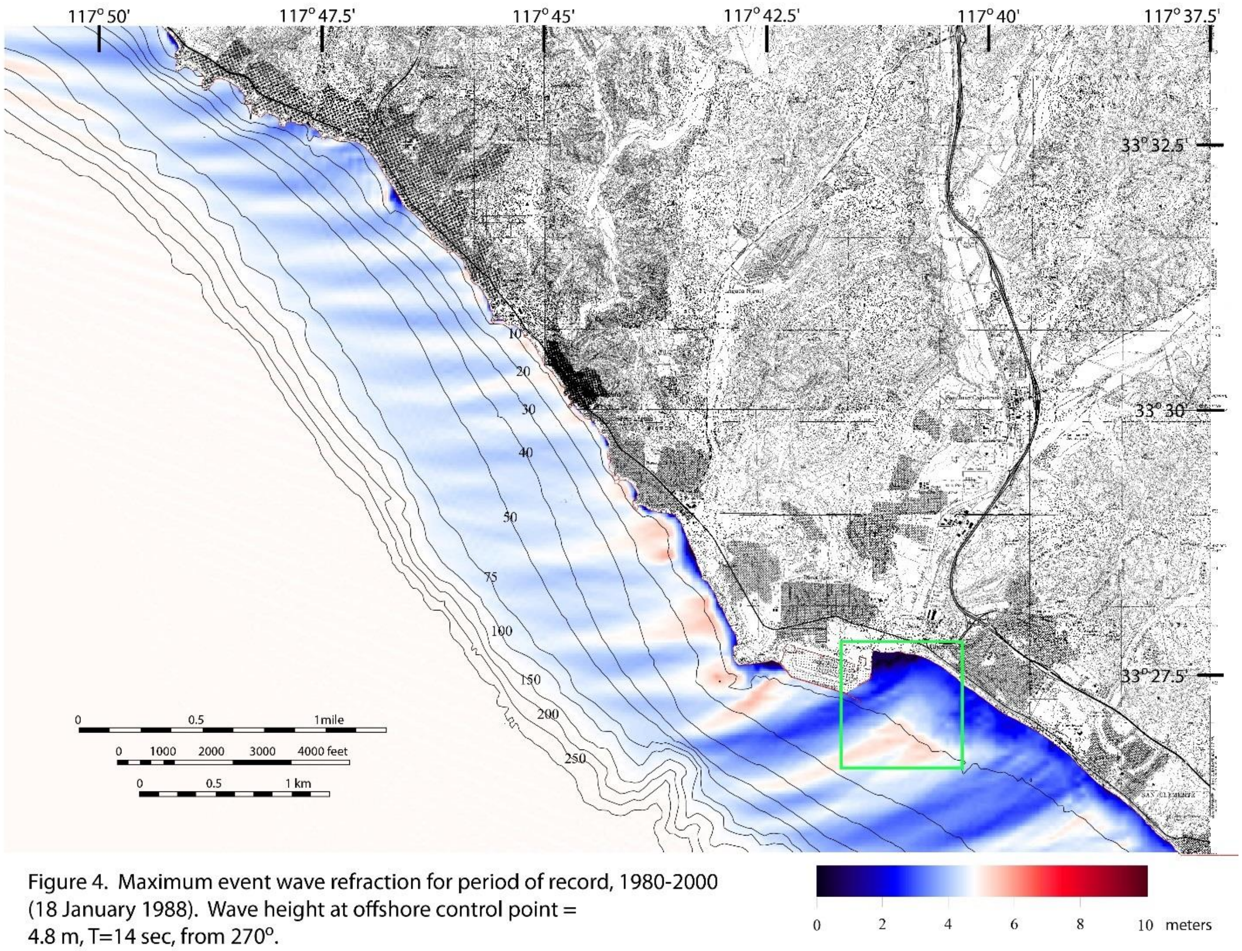


Figure 4. Maximum event wave refraction for period of record, 1980-2000 (18 January 1988). Wave height at offshore control point = 4.8 m, T=14 sec, from 270°.

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

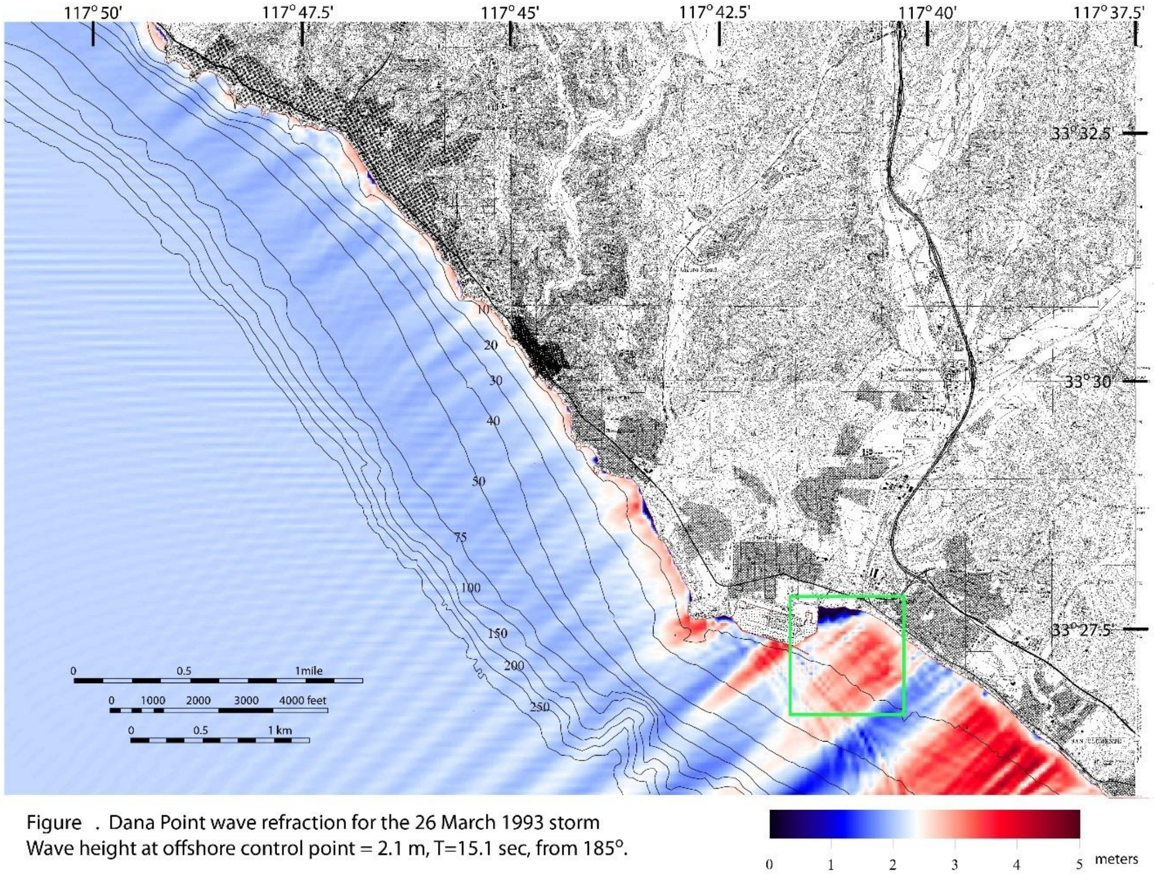


Figure . Dana Point wave refraction for the 26 March 1993 storm
Wave height at offshore control point = 2.1 m, T=15.1 sec, from 185°.

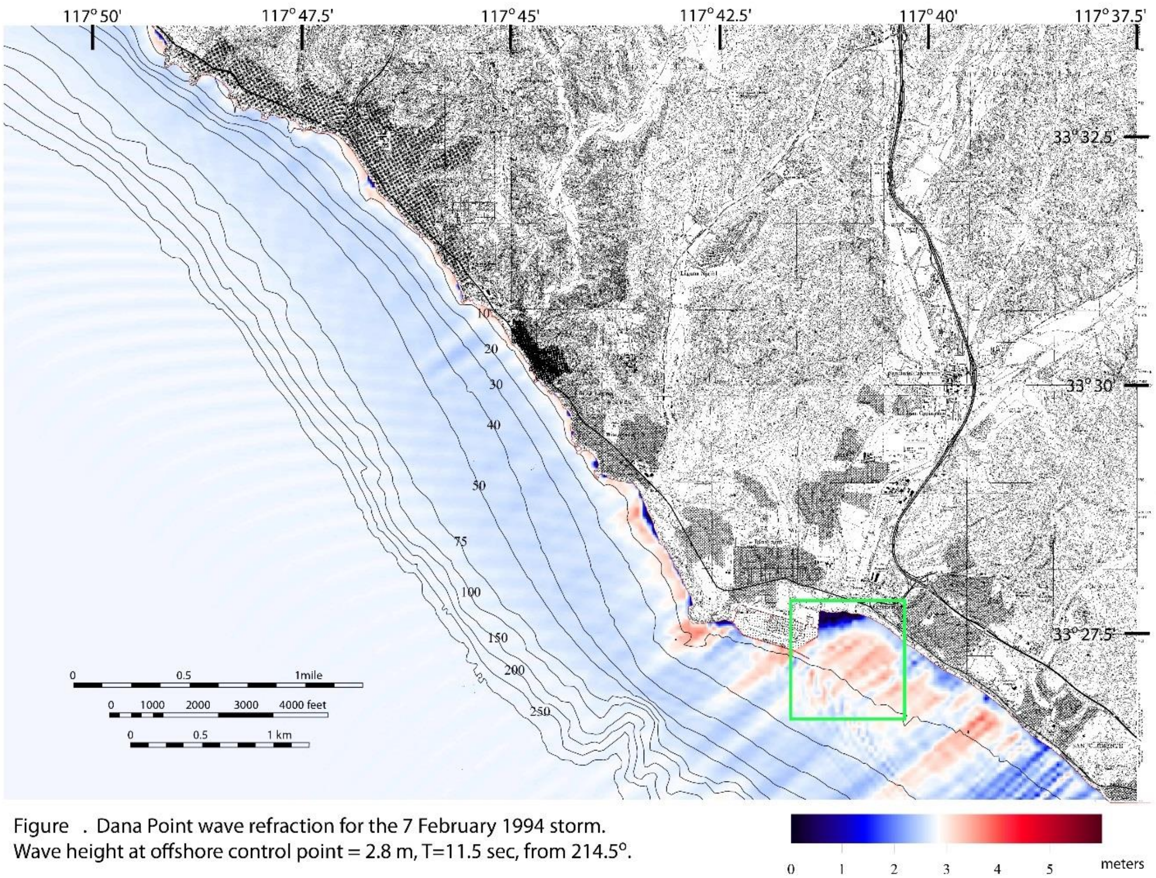


Figure . Dana Point wave refraction for the 7 February 1994 storm.
Wave height at offshore control point = 2.8 m, T=11.5 sec, from 214.5°.

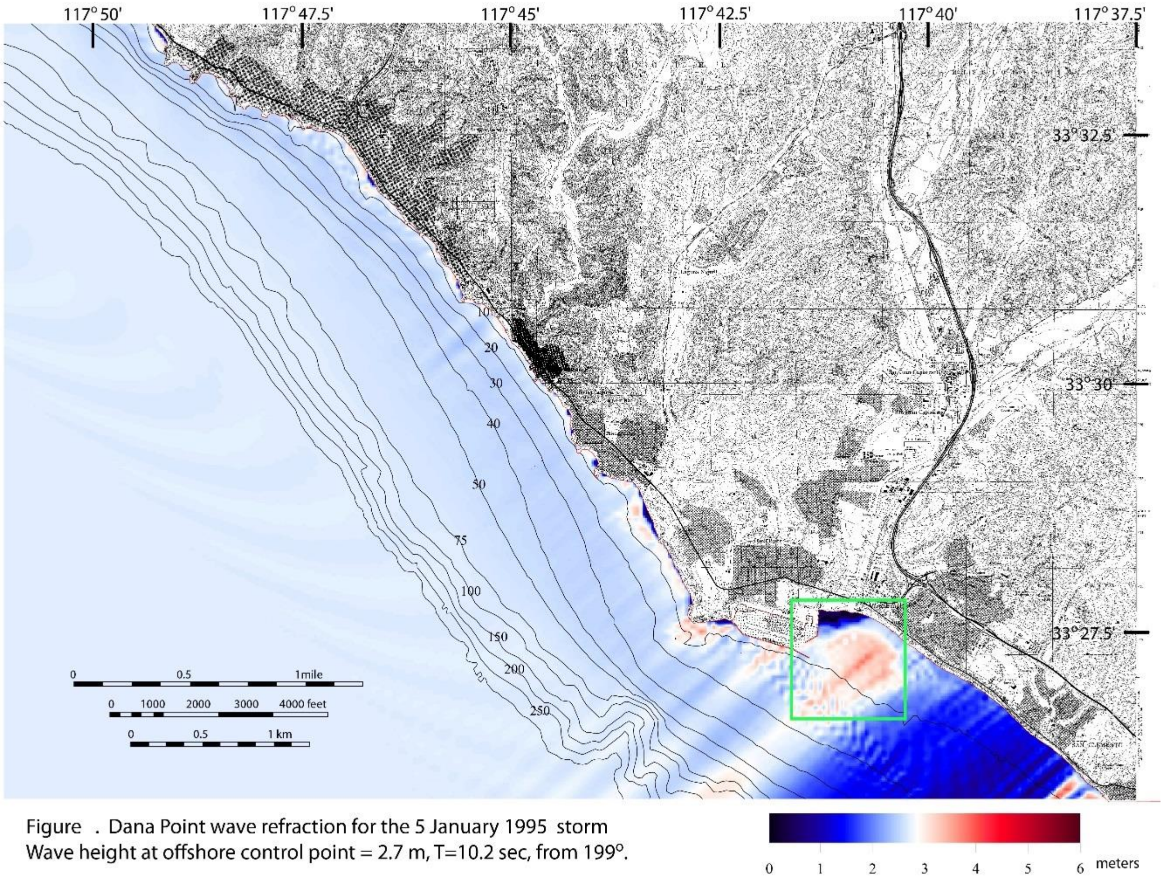


Figure . Dana Point wave refraction for the 5 January 1995 storm
Wave height at offshore control point = 2.7 m, T=10.2 sec, from 199°.

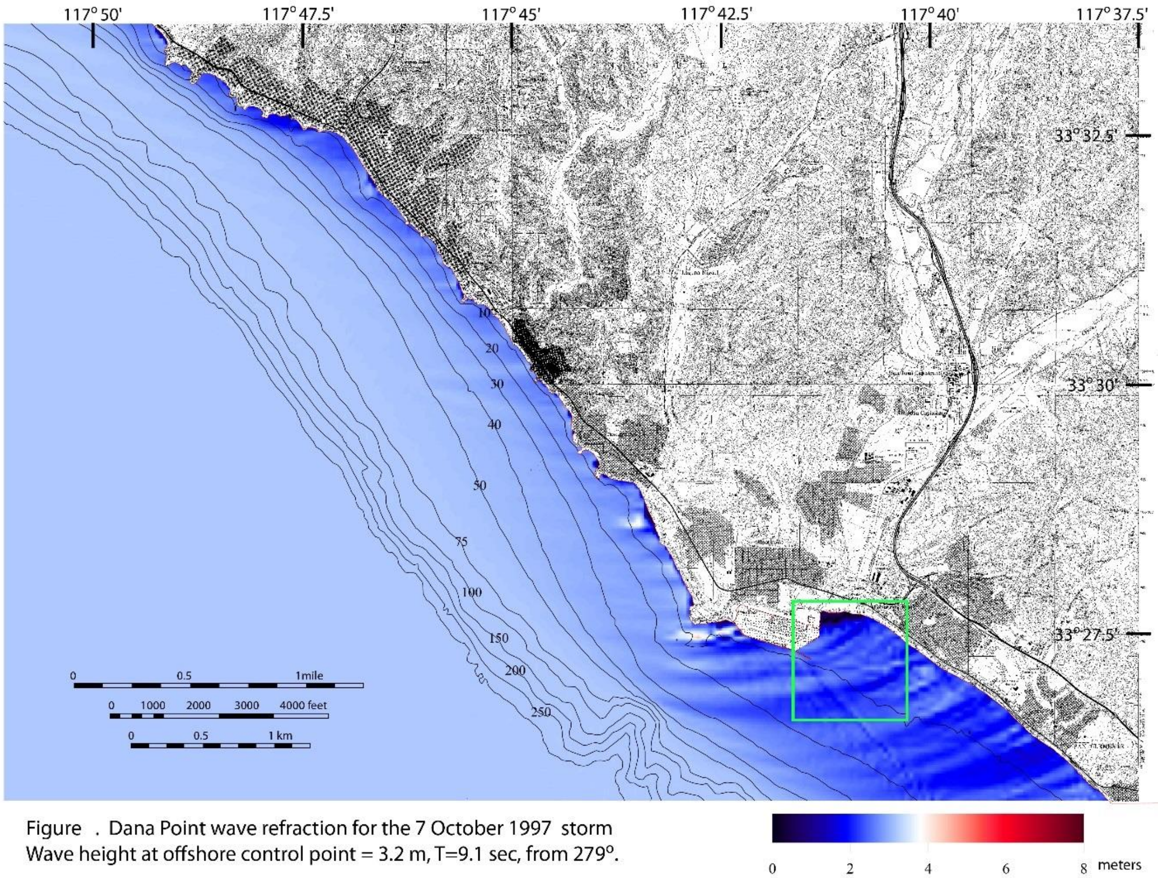


Figure . Dana Point wave refraction for the 7 October 1997 storm
Wave height at offshore control point = 3.2 m, T=9.1 sec, from 279°.

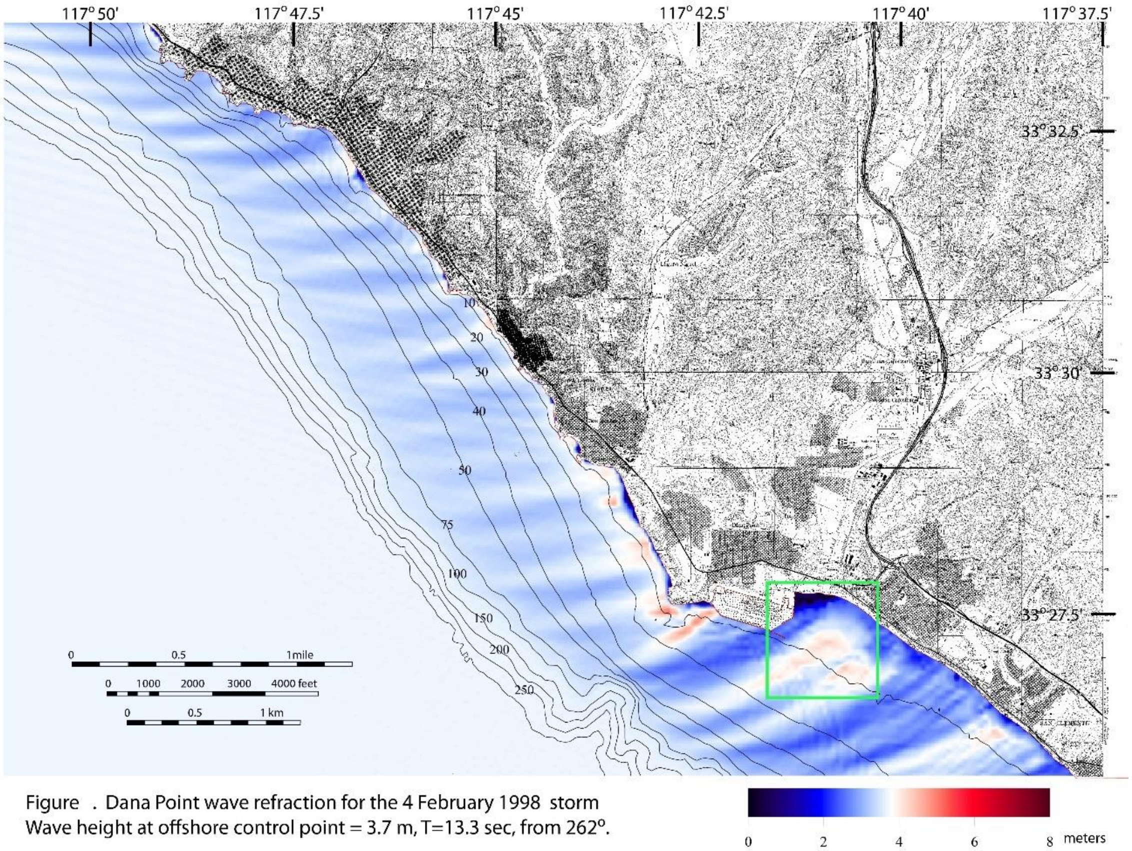


Figure . Dana Point wave refraction for the 4 February 1998 storm
Wave height at offshore control point = 3.7 m, T=13.3 sec, from 262°.

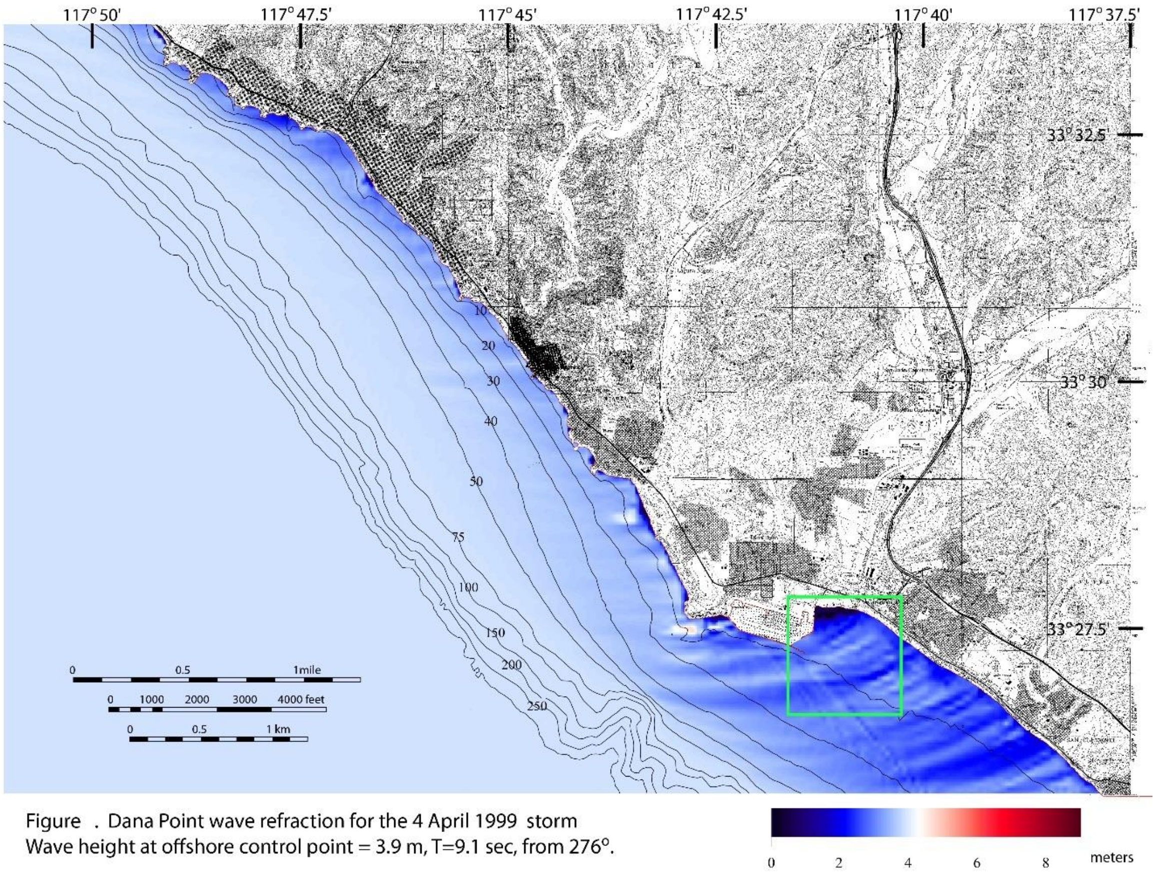


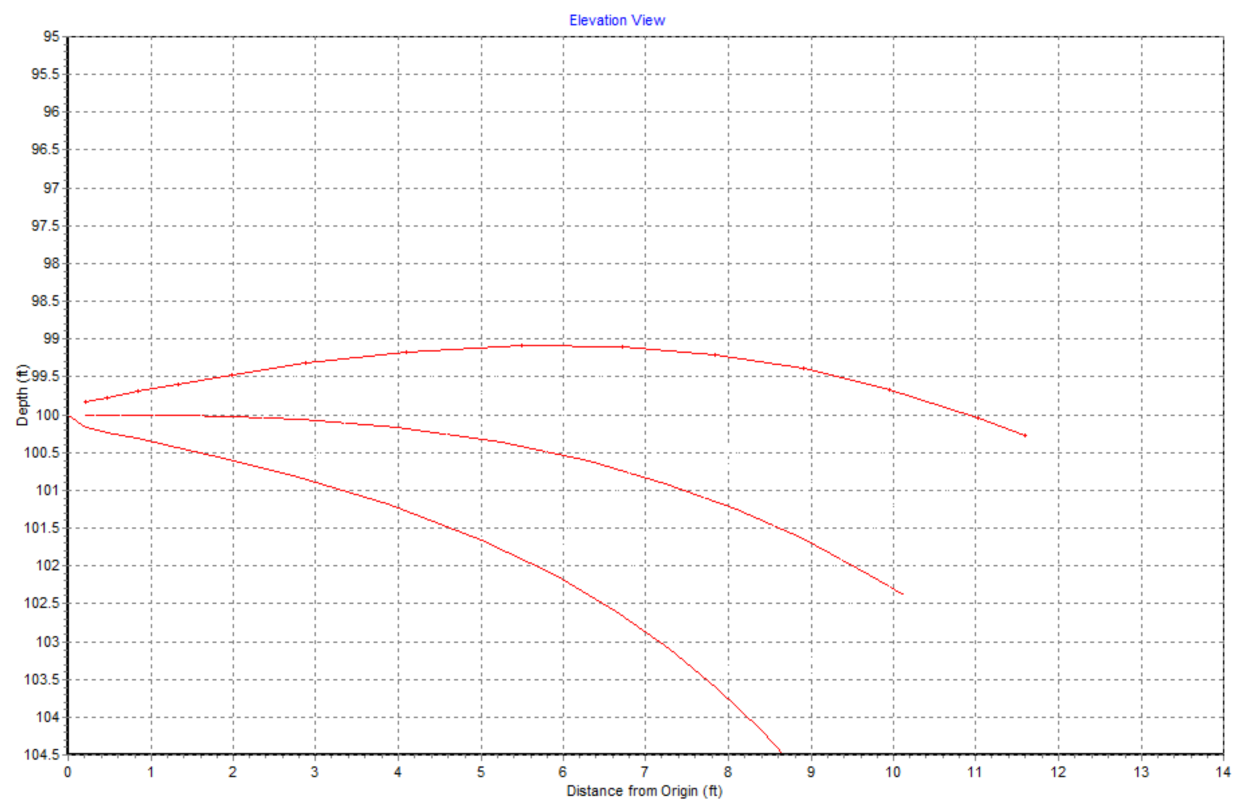
Figure . Dana Point wave refraction for the 4 April 1999 storm
Wave height at offshore control point = 3.9 m, T=9.1 sec, from 276°.

APPENDIX 4.2.2

BRINE DISCHARGE ANALYSIS FOR THE FINAL EIR

Plumes 18b Modeling Assessment of Deleterious Diffuser Entrainment for the Doheny Desalination Project

By Scott A. Jenkins, Ph.D.



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ABSTRACT: Diffusers intrinsically generate strong turbulent jets in order to produce mixing and rapid dilution of effluent, and the shearing action of those turbulent jets can potentially damage or kill small delicate organisms entrained into those jets (sometimes referred to as *diffuser turbulence mortality*). The implementation section of the brine amendment to the California Ocean Plan, Section III.M.2 (b), requires that brine diffusers must minimize and mitigate for such marine life impacts, and the California State Water Resources Control Board has released newly defined protocols that require the use of a specific hydrodynamic mixing model (referred to as *Plumes 18b*) to assess those impacts. Plumes 18b is not supported by US EPA, but the State Water Board has made executable files for this model publicly available on their web site, along with a technical guidance document on how to assess deleterious entrainment from brine diffusers. These protocols using the Plumes 18b model are implemented in this study to assess potential injury or mortality to small marine organisms entrained by discharges from the diffuser of the San Juan Creek Ocean Outfall (SJCOO) that is being proposed as the discharge structure for brine by-product from the Doheny Desalination Project (DDP).

In general, Plumes 18b predicted higher *Minimum Initial Dilution*, and smaller *Zones of Initial Dilution*, ZID at deeper depths than was reported previously by DDP dilution studies using the US EPA supported Visual Plumes (UM3). Using Plumes 18b, all of the buoyant DDP discharge scenarios are found to achieve the required 101 to 1 minimum initial dilution required under the current NPDES permit for the SJCOO, (No. CA 0107417, Order No. R9-2012-0012 as amended by Order No. R9-2014-0105). For any given amount of SOCWA wastewater, the addition of any amount of DDP brine reduces both the minimum initial dilution as well as the effective (average or bulk) dilution at the maximum rise of the plume, while reducing the size of the ZID. This is not altogether a bad result, so long as there remains adequate dilution to satisfy present or future NPDES permit requirements for minimum initial dilution; which indeed appears to be the case. The reduction of buoyant effluent dilution caused by adding brine to SOCWA wastewater has a favorable effect on potentially deleterious diffuser entrainment, even though buoyant discharges appear to be exempt from requirements to assess, minimize or mitigate for diffuser turbulence mortality impacts to entrained marine organisms under the present structure of the amended Ocean Plan (SWRCB,2015). The Plumes 18b results show that for any given amount of SOCWA wastewater, the addition of any amount of DDP brine reduces the deleterious diffuser entrainment rate, thus improving upon an existing condition that is not mitigatable under present implementation practices of the Ocean Plan. Therefore, no mitigation should be required for DDP operational scenarios that result in buoyant combined discharges with SOCWA wastewater. The net turbulence mortality benefit achieved by combining DDP brine with SOCWA wastewater increases with decreasing combined discharge rate, as smaller jet velocities with larger Kolmogorov turbulent eddies occur at lower combined discharge rates.

The Plumes 18b dilution modeling results for the combinations of SOCWA wastewater and DDP brine that produce dense (negatively buoyant) discharges involve either brine-only or high-brine ratio discharges, typical of conditions anticipated during dry-weather wastewater effluent streams or future water reclamation conditions. Again, Plumes 18b has predicted higher effective dilution and shorter distances to the 2 ppt over natural background compliance threshold than was reported previously in DDP dilution studies using the US EPA supported Visual Plumes (UM3). Based on long term averages of ambient salinity records, natural background salinity at the SJCOO is 33.52 ppt, so that the compliance threshold 35.52 ppt under Appendix-A brine amendment provisions of the California Ocean Plan (SWRCB, 2015). Plumes

18b results indicate this compliance threshold is met in less than 2.5 ft. from the point of discharge by all DDP dense discharge operating conditions; whereas the Ocean Plan requires this compliance threshold is reached within 100 m from the discharge point. Thus, the DDP would be fully compliant with Ocean Plan brine discharge limits by a wide margin of safety, according to Plumes 18b dilution simulations. The jets of the SJCOO diffuser discharge parallel to the bottom and the ports are only 4.5 ft. above the bottom. Consequently the trajectories of these dense DDP discharges travel relatively short distances before reaching maximum rise or bottom hit points. This behavior, in turn, causes the Kolmogorov eddy scales in the diffuser jets to remain small (less than 0.2 mm), and presumably injurious according to the injury hypothesis advanced in the State Water Boards turbulence mortality guidance document. But these short trajectories also limit the effective (bulk or average) dilution and therefore limit the deleterious diffuser entrainment. By the literal interpretation of the State Water Board’s guidance document mitigation scaling for brine diffuser turbulence mortality should only be based on the entrainment at the maximum rise of the plume, which range from 67 mgd to 729 mgd for dense DDP discharges.

Plumes 18b Modeling Assessment of Deleterious Diffuser Entrainment for the Doheny Desalination Project

By Scott A. Jenkins, Ph.D.

1) Introduction:

This is a hydrodynamic modeling analysis to assess potential injury or mortality to small marine organisms entrained by discharges from the diffuser of the San Juan Creek Ocean Outfall (SJCOO) that is being proposed as the discharge structure for brine by-product from the Doheny Desalination Project (DDP). Diffusers intrinsically generate strong turbulent jets in order to produce mixing and rapid dilution of effluent, and the shearing action of those turbulent jets can potentially damage or kill small delicate organisms entrained into those jets, phenomena referred to herein as *turbulence mortality*. The present analysis is based on newly defined protocols by the California State Water Resources Control Board as outlined in Roberts (2018a). These protocols require the use of a specific hydrodynamic mixing model referred to as *Plumes 18b*. Plumes 18b is not supported by the USEPA, but is a derivative of the *Visual Plumes (UM3)* model which USEPA does support, (cf. Frick, et al, 2003); and both models share the same principal developer, Dr. Walter Frick. The antecedent dilution modeling for the Doheny Desalination Project appearing in Jenkins (2016 & 2017) was performed using the Visual Plumes (UM3) model; but implementation of the methods in Roberts (2018) technical guidance document require that dilution be recalculated using Plumes 18b. Some differences were found between the dilution estimates originally calculated in Jenkins (2016 & 2017) using Visual Plumes (UM3) versus those calculated herein using Plumes 18b, but those differences did not change the fundamental conclusion that the 14 brine discharge scenarios in Table 1 that span the proposed operating range of the Doheny Desalination Project, are all compliant with requirements for both buoyant and dense discharges under the California Ocean Plan, (SWRCB, 2015).

A careful read of the *Appendix-A brine amendment* of the California Ocean Plan indicates that combining indicates that combining brine from desalination plants with wastewater from municipal wastewater treatment facilities, and utilizing existing treated wastewater outfalls is the preferred discharge technology, and that discharge strategy is exactly what is proposed for the Doheny Desalination Project. The implementation section of the brine amendment to the California Ocean Plan, Section III.M.2 (b), requires that:

> *Multiport diffusers shall be engineered to maximize dilution, minimize the size of the brine mixing zone, minimize suspension of benthic sediments, and mortality of all forms of marine life*

This requirement appears only in the *Appendix-A brine amendment* of the California Ocean Plan, and therefore, implicitly applies only to the dense (negatively buoyant) discharge scenarios that appear as the red or black entries in Table-1. Buoyant discharges, such as wastewater discharges are regulated under a completely different set of compliance standards found in *Appendix I* of the *California Ocean Plan*. There are no implementation provisions in Appendix-I that require diffusers discharging buoyant effluent to minimize turbulence mortality; and no wastewater

Table 1 Plumes 18b Modeling Scenarios for the Doheny Desalination Project

SOCWA Wastewater Flow Rates (MGD)	Brine Discharge Rate (MGD)	Combined Discharge Rate (MGD)	Combined Discharge Salinity (ppt)	Density Anomaly $\Delta\rho/\rho$
0	**3.0**	***3.0***	***67.0***	**-0.0268**
1.8*	***3.0***	***4.8***	***54.43***	***-0.0167***
0.0	**5.0**	**5.0**	**67.0**	**-0.0268**
0.35	**5**	**5.35**	**62.63**	**-0.0233**
0.0	**10.0**	**10.0**	**67.0**	**-0.0268**
0.0	**15.0**	**15.0**	**67.0**	**-0.0268**
8.0	**15.0**	**23.0**	**43.69**	**-0.00839**
13.0	**15.0**	**28.0**	**35.89**	**-0.00197**
8.0	**5.0**	**13.0**	**25.77**	**+0.00636**
13.0	**5.0**	**18.0**	**18.61**	**+0.01225**
18.9	**5.0**	**23.9**	**14.02**	**+0.0160**
18.9	**15.0**	**33.9**	**29.64**	**+0.0032**
31.0	**5.0**	**36.0**	**9.30**	**+0.0199**
31.0	**15.0**	**46.0**	**21.85**	**+0.0096**

Notes:
*well water from Doheny and Capistrano beaches substituted for SOCWA wastewater
Red & Black = dense (negatively buoyant) discharges
Blue = buoyant discharges

authority or sanitation district in California has been required to assess or mitigate for deleterious entrainment by the wastewater outfall diffusers. Consequently, many readers of the Ocean Plan have inferred that turbulence mortality assessment and mitigation would not be required for blended brine/wastewater operating conditions that result in a buoyant combined effluent (such as the blue entries in Table-1). Nonetheless we will apply herein the State Water Board turbulence mortality assessment criteria outlined in Roberts (2108a) to the 6 buoyant effluent operating scenarios in Table-1; and demonstrate net incremental changes over present SOCWA wastewater-only operating conditions would be reductions in potentially deleterious entrainment due to additions of brine, and consequently the Doheny Desalination Project would have no net mitigatable impact for those combined buoyant effluent cases. For the remaining 8 dense effluent discharge cases in Table 1 (red and black entries), turbulence mortality assessment criteria are applied as outlined in Roberts (2108a), leading to results for potentially deleterious entrainment, which will be throughput to a subsequent ETM/APF (Empirical Transport Model/Area of Production Foregone) analysis in a companion study to compute the mitigation scaling for diffuser turbulent shear impact.

2) Turbulence Mortality Technical Approach:

The calculus presented in Roberts, (2018a) to assess injury or mortality to organisms entrained by brine diffuser discharges (aka, turbulence mortality) has three components:

1) *Injury Hypothesis* based on the notion that injury or mortality occurs when entrained organisms are exposed to a specific type of diffuser-induced turbulent eddy that is smaller than what is found in ambient ocean turbulence yet comparable to the size of the organism. Therefore, the smallest naturally occurring eddies in ambient ocean turbulence establish the injury threshold, which is assumed to be 1 mm.

2*) Empirical Relations* that relate the size of that specific type of diffuser-induced eddy to the distance from the point of discharge.

3) *Entrainment Calculations* based on dilution-trajectory results from the Plumes 18b that yield the entrainment rate between the point of discharge and the point where a specific type of diffuser-induced eddy becomes comparable to or larger than what is found in ambient ocean turbulence. This entrainment rate is presumed to be deleterious and is throughput to the ETM/APF (Empirical Transport Model/Area of Production Foregone) calculus to compute the mitigation scaling for diffuser turbulent shear impact. (Note the ETM/APF is not a component of the Roberts calculus)

The injury hypothesis is based on the notion that only those entrained organisms which are comparable to, or smaller than, Kolmogorov turbulence scales will suffer injury or mortality. To isolate the incremental injury and mortality due to the diffuser from what occurs naturally in ambient ocean turbulence, diffuser entrainment impacts are assumed to occur only in those regions of the diffuser discharge where the Kolmogorov scales are smaller than the natural Kolmogorov scale in the ocean water mass around the diffuser. This limits the size of the entrained organism that are assumed to be impacted by the diffuser to only the smallest, most fragile, populations in the receiving waters. However, this assumption also makes implementation of this theory reliant on a highly site-specific parameter that is extremely difficult and costly to measure, namely Kolmogorov scale ocean turbulence. Walter, et al., (2014) measured Kolmogorov scale ocean turbulence in a massive field effort that deployed Doppler velocimeters and fast-response conductivity-temperature sensors mounted on an underwater turbulence flux tower located in the far southern end of Monterey Bay, (offshore of the Hopkins Marine Station in Pacific Grove). These measurements suggest the smallest naturally occurring turbulent eddies in Monterey Bay are about 1 mm in size. No such direct measurements of Kolmogorov scale ocean turbulence exist anywhere else in California, and collecting such data would be a significant research effort. We note turbulence measurements off Vancouver Island by Grant, et al. (1962) found that Kolmogorov scale ocean turbulence was on the order of 2 cm, 20 times greater than the Monterey Bay measurements. The uncertainty of how Kolmogorov scale ocean turbulence varies throughout the coastal waters of California will radically impact the final calculations of volume of entrained water that is considered to be deleterious, because it dictates the injury threshold of the entire turbulence mortality assessment.

Nonetheless, we are compelled herein to adopt the nearest neighbor assessment of ocean Kolmogorov scales, and base our turbulence mortality assessments on the Monterey Bay measurements.

The second component in the Roberts method for turbulent mortality assessment are *empirical relations* that relate the size of a specific type of diffuser-induced eddy (the Kolmogorov scale) to the distance from the point of discharge. The Kolmogorov eddy size is calculated with a simple empirical relation derived from laboratory measurements of turbulent jet:

$$\eta_c = 0.24\, x\, \mathrm{Re}^{-3/4} \qquad (1)$$

Where η_c is the Kolmogorov eddy size along the jet centerline, x is the distance from the discharge point, $\mathrm{Re} = u\,d / \nu$ is the Reynolds number based on the discharge velocity, *u*, the jet port diameter, *d*, and the kinematic viscosity, $\nu = 1.17 \times 10^{-6}$ m^2/s. However, in both Roberts (2018 a & b) the calculations of Kolmogorov eddy scales and the associated deleterious entrainment are stopped at the maximum rise height (apex) of the brine plume trajectory, even though the Kolmogorov scale eddies at the apex are still very much smaller than the injury threshold of 1 mm. The reason for this truncation of the calculation is because equation (1) is based on measurements of laboratory scale jets by Wygnanski and Fiedler (1969), which omitted buoyancy effects. Beyond the apex of a brine discharge trajectory buoyancy forces begin to exceed the inertial forces and the discharge transitions from being a jet to becoming a negatively buoyant plume. In the application to the 8 dense (negatively buoyant) discharge cases in Table 1 (red and black entries), we not only estimate deleterious entrainment from Kolmogorov eddy scales at the apex of the trajectory, but also carry the calculation all the way to the point where the Plumes 18b model finds the trajectory hits the bottom. We do so because the jets of the SJCOO diffuser discharge parallel to the bottom and the ports are only 4.5 ft. above the bottom, whence the discharge trajectory probably remains a jet at the point of contact with the bottom. The decisive issue with proceeding with the equation (1) is that the Kolmogorov scale remains less than 1 mm at either the apex of the trajectory or at the point where the trajectory makes contact with the bottom

The third component of the turbulence mortality assessment (as set forth in the Roberts (2018a) turbulent mortality guidance document) are *entrainment calculations*, most accurately made using hydrodynamic mixing models to determine dilution at the maximum rise or bottom hit points of the discharge trajectory. Until April 2018, the California State Water Resources Control Board had been following a procedure where dilution credits for ocean outfall diffusers, and other diffuser related issues were evaluated using only those models that had been fully vetted by US EPA. The last time US EPA went through this formal vetting processes was 2003 (cf. Frick et al., 2003), and only three mixing models emerged with EPA certifications: *PDSWIN*, *Visual Plumes* or *CORMIX*. However, Roberts (2018a) is recommending use of a model that US EPA has not formally vetted, namely *UM3 version 17b*, aka *Plumes 17b*. US EPA does not support Plumes 17b, and the executable files for that model were only made publicly available by the State Water Resources Control Board on their web site for a brief time in late April and early May 2018. When it was discovered that Plumes17b had programming bugs, it was replaced with Plumes 18b on the State Water Resources Control Board web site, circa 30 May 2018. There is no written documentation specific to the implementation of Plumes 18b and the “Help” buttons in the model do not work. However, Plumes 18b has been graciously supported by Dr. Walter Frick (US EPA retired), the Plumes 17b and Plumes 18b developer, who has answered many questions by e-mails and phone calls that has allowed us to become proficient in running this model.

The key outputs of the Plumes 18b are the trajectories of discharge and the dilution calculated along those trajectories. Plumes 18b is used to find the distance to the point of maximum rise of the discharge trajectory, X_a or the distance to the point where the plume contacts the bottom, X_b; and these values are inserted in equation (1) to determine if the Kolmogorov scale eddies remain less than the injury threshold of 1 mm. Once this condition has been verified, the deleterious entrainment by the diffuser is calculated by:

$$Q_c = S_{a(x=X_a)} Q_j \tag{2}$$

or:

$$Q_c = S_{a(x=X_b)} Q_j \tag{3}$$

where Q_c is the deleterious entrainment rate, Q_j is the total discharge rate of all 125 jets of the SJCOO diffuser; $S_{a(x=X_a)}$ is the effective (average or bulk) dilution at the maximum rise of the discharge trajectory, and $S_{a(x=X_b)}$ is the effective (average or bulk) dilution where the discharge trajectory makes contact with the bottom. The solutions to equations (2) and (3) are then passed on to the ETM/APF model to compute the mitigation scaling for diffuser turbulent shear impact.

3) Initialization of Plumes 18b:

Plumes 18b provides data entry with three main input tabs: 1) Diffuser, 2) Ambient, and 3) Special Settings. The input fields for these three tabs are listed at the top of the text output files of each of the modeling scenarios appearing in Appendices A-C. The input fields for are listed below with applicable explanations for the input into each field:

3.1) Diffuser Input Tab: Diffuser and effluent characteristics are necessary to determine the momentum of the effluent as it enters the receiving water and the density of the effluent (which will affect its buoyancy in the receiving water).

3.1.1. Port Diameter: Plumes 18b data entry limitations only allow a single input for "Port Diameter". Thus, a single port diameter must be determined. This

was done by taking an average port size of all the ports as summarized in Table 1. Using the information contained in Table 1, one may compute the average port area (7.30 in^2) and average port diameter (3.05 inches) for the SJCOO diffuser. A port diameter of 3.05 inches was input to Plumes 18b.

3.1.2. Vertical Angle: The vertical angle is defined in the Visual Plumes manual (http://www.epa.gov/ceampubl/swater/vplume/) as the discharge angle relative to the horizontal with zero being horizontal, 90 being vertical upward, and -90 being vertically downward. Appendix-A drawings indicate that the ports are located on the diffuser facing opposing directions, 180 degrees away from each other. A data entry limitation of Plumes 18b is that only one vertical angle may be entered. In cases where there is potential for two plumes emitted from different angles on the diffuser to merge within

the water column, the Visual Plumes manual suggests modeling the diffuser as if all ports are on one side of the diffuser and with half the spacing. In situations where the potential for plume merging is considered to be negligible, an alternative approach is to model one-half of the diffuser (i.e., one plume) and assume no cross-merging of plumes. Because the plumes from each side of the diffuser are assumed to have the potential to merge, both sides of the diffuser have been included in the simulation (i.e., all ports are treated as if they are on one side of the diffuser and with half the spacing). A single vertical angle of 0 degrees was used in all runs of the Plumes 18b model.

3.1.3. Horizontal Angle: Appendix-A drawings indicate that the ports are located on the diffuser with no horizontal deflection. Therefore a single horizontal angle of 0 degrees was used on each leg in the model.

3.1.4. Source Coordinates: these entries establish the origin of the Plumes 18b coordinate system. These were set at x-coord = 0, y-coord = 0, consistent with the values used in the Plumes 17b simulation examples appearing in Appendix-B of Roberts (2018a).

3.1.5. Number of Ports: The number of ports specified in the Appendix A drawings of the most recent NPDES permit for the SJCOO outfall (cf: RWQCB, 2014) is 125 ports. The outfall rehabilitation report indicated all obstructed ports were cleared in April and May of 2015. 125 ports was entered into the model.

3.1.6. Port Spacing: The Appendix A drawings in the most recent NPDES permit for the SJCOO outfall (cf: RWQCB, 2014) indicate that the ports were approximately 24 feet apart. Both sides of the diffuser are being modeled on one side of the diffuser; a value of 12 feet was entered into the model.

3.1.7. n/r: This entry defines the maximum run time allowed by the model. A value of 3600 s on the advice of Dr. Walter Frick, (Plumes 18b developer).

3.1.8. Mix Zone Distance: This value is not relevant to the final initial dilution calculations and has no impact on model output. The Plumes 18b software requires that a value be entered into these fields. Therefore, 1000ft was entered based on the size of the monitoring zone under the present NPDES permit (cf; RWQCB, 2014)

3.1.9. Isopleth Value: This value is not relevant to the final initial dilution calculations. A value of concent = 0 was entered, consistent with the values used in the Plumes 17b simulation examples appearing in Appendix-B of Roberts (2018a).

3.1.10. Port Depth: Appendix A and Figure 1 of the most recent NPDES permit for the SJCOO outfall (cf: RWQCB, 2014) indicate that the diffuser discharge depth is 100 feet at the inshore end of the diffuser. A value of 100 feet was used in the model.

3.1.11. Effluent Flow: These values were separately entered into the effluent flow field for each of the modeling scenarios listed in Table-1.

3.1.12. Effluent Salinity: These values were separately entered into the effluent saliity field for each of the modeling scenarios listed in Table-1.

3.1.13) Effluent Temperature: SOCWA provided average monthly temperature data from January 2014 through September 2016. The density of water is a function of temperature. Therefore, a smaller difference in temperature between the effluent and receiving waters will produce a relatively smaller difference between the densities of the effluent and receiving waters and less dilution is likely to occur. Effluent temperatures ranged from a maximum of 29.44^{0}C to a minimum of 21.66^{0}C, with a mean effluent temperature of 25.62^{0}C. Receiving water temperatures are significantly lower than the temperature of the effluent discharged from the SJCOO. Thus, a lower effluent temperature is likely to result in lower dilution. The lowest monthly average temperature of 71^{o}F (21.66 ^{0} C) was entered into the data field.

3.1.14) Effluent Concentration: This data field does not have an effect on the final initial dilution calculated. However a value must be entered into this field for the model to run, and a default value equivalent to the salinity field expressed in parts per million (ppm) was entered.

3.2 Ambient Input Tab: This tap specifies ambient profiles for neafield current speeds and directions, salinity, temperature, background concentrations, pollutant decay rates, the n/r run-time parameter, and the far-field diffusion coefficient. The tab only excepts nine depth increments to specify the ambient profiles. The current is always set to zero when running models for the Ocean Plan, and the background concentrations, pollutant decay rates, the n/r run-time parameter, and the far-field diffusion coefficients are irrelevant entries, and the values used for these entries are the same as those used in the Plumes 17b simulation examples appearing in Appendix-B of Roberts (2018a). However, the depth profiles for ambient salinity and temperature are most important as these entries define the natural stratification of the receiving waters.

The receiving water salinity/temperature profile from September 2008 was used to define worst case scenario for determination of "*the lowest average initial dilution within any single month of the year*" per Provision III.C.4.d of the Ocean Plan. This is the same profile used in the Appendix H dilution study of the most recent NPDES permit for the SJCOO outfall (cf: RWQCB, 2014). These profiles are plotted in Figure 1. While the salinity profile is fairly uniform with depth of water over the SJCOO, (with an average salinity of 33.37 ppt), the temperature is found to gradually decline with water depth, varying between 19.9 ^{0} C on the surface to 13.4^0 C at the seafloor around the outfall. Normally there is a very abrupt change in water temperature between the warm surface mixed layer and the cold bottom water; and this

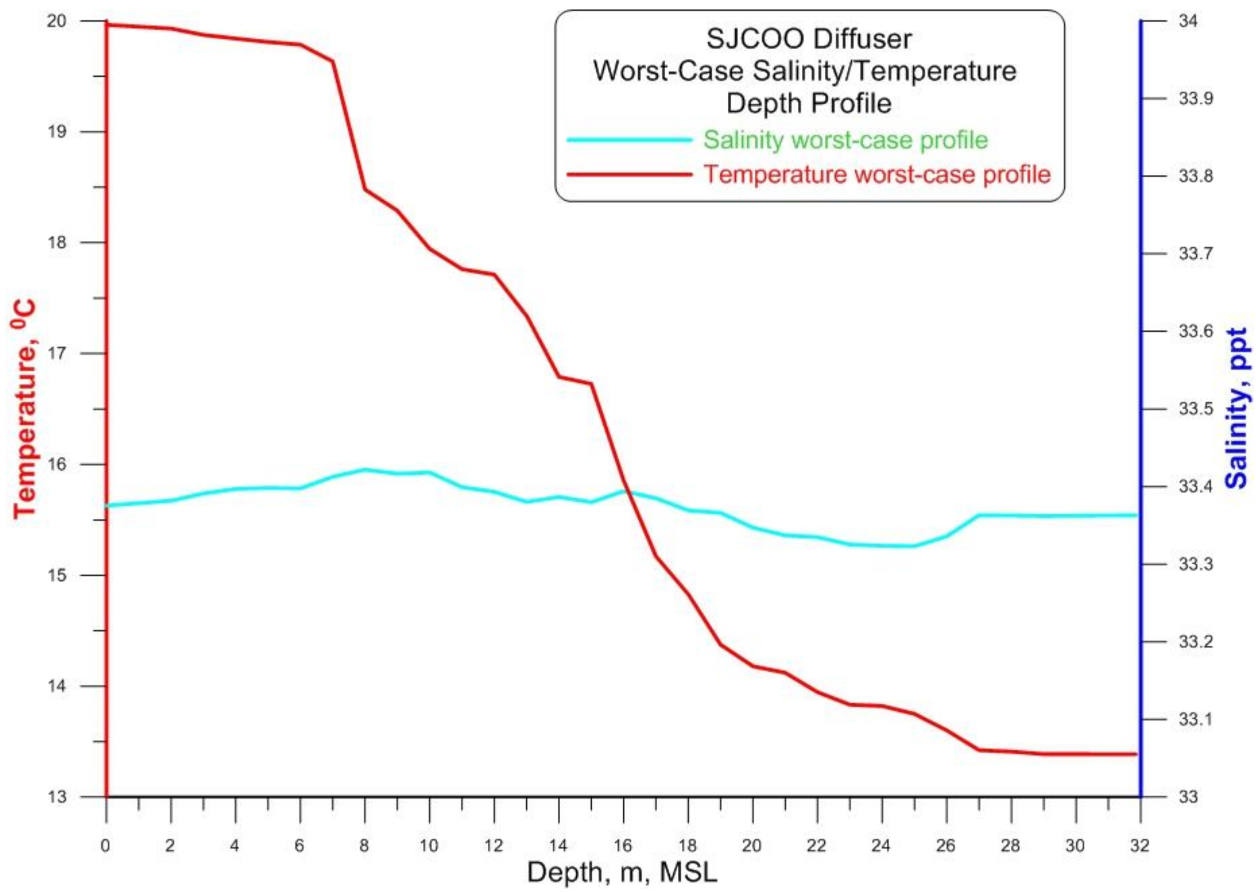

Figure 1: Worst-case temperature salinity profile as presented in Appendix-H of RWCQB (2014) for update of the diffuser performance and minimum dilution assessment of the SJCOO. Profiles based on 17 September 2008 upwelling and discharge conditions.

abrupt change referred to as a thermocline produces a trapping layer at the thermocline interface, where the partially diluted discharge plume no longer has sufficient positive buoyancy to penetrate the thermocline, and instead spreads out horizontally along the thermocline interface resulting in a trapping level beneath the sea surface. However, the temperature profile in Figure 1 varies so gradually that there is not a well-defined pycnocline and the trapping layer is poorly formed; whence the buoyant wastewater is able to rise to the sea surface, 29 m above the deepest sections of the SJCOO diffuser. As a result the ZID boundary becomes the sea surface and the distance from the point of discharge where minimum dilution is defined defaults to 29 m, in spite of the fact that the plume still has residual momentum and kinetic energy imparted to it by the discharge jets.

There is a minor difference in the particular portions of the Figure 1 ambient profile that were used for the Plumes 18b simulations of the buoyant discharge scenarios versus that used for the dense (negatively buoyant) simulations. Plumes 18b only allows 9 depth entries to specify the ambient profile, and yet there are 30 depth entries in the field data for the September 2008 ambient profile in Figure 1. Consequently, some selective judgement was used in deciding which of the 30 measured depth increments in Figure 1 ambient profile should be loaded into the ambient input tab of Plumes 18b. It is notable that there is little variability in both the salinity or temperature profiles in Figure-1 below a depth of 28 m MSL. For the buoyant discharge scenarios in Table-1, where the discharge trajectory is essentially vertically upward from the discharge point, the last depth entry in the ambient tab was set at a depth of 29 m, about 1.5 m above the depth of the discharge ports. However, when this same ambient tab profile was used on the dense (negatively buoyant) scenarios, The Plumes 18b simulations would continue down through the bottom and beyond. (This same problem is found in the Plumes 17b simulations of the Huntington Beach diffuser in Roberts, 2018b). The seabed at the SJCOO outfall is at a depth of 104.5 ft. MSL, or 31.85 m MSL, and yet some of the negatively buoyant simulations would continue on to depths of several hundred meters. This problem arises from the fact that Plumes 18b (unlike Visual Plumes), has no data entry for the elevation of the discharge ports above the seabed, and thus the model really doesn't know where the bottom is! After consulting with Dr. Walter Frick (Plumes 18b developer), the solution to this problem was achieved by setting the last depth entry in the Plumes 18b ambient tab at precisely the depth of the seabed, and selecting "stop at bottom hit" under special settings. This insures that Plumes 18b does not generate spurious solutions that extend below the seabed. Therefore, all of the dense (negatively buoyant) scenarios from Table-1 were run with the 29 m depth entry to the ambient profile replaced with a 31.85 m depth entry.

3.2.1) Far-field Diffusion Coefficient: The Visual Plumes manual recommends the use of 0.0003 $m^{0.67}/s^2$. This value was used in the data field as a constant (not extrapolated as the ambient temperature and density were).

3.3 Special Settings:

3.3.1) Tidal Pollutant Build-up, Channel Width : This data field does not have an effect on the final initial dilution calculated. A value of 100 was entered, consistent with the values used in the Plumes 17b simulation examples appearing in Appendix-B of Roberts (2018a).

3.3.2) Diffuser Port Contraction Coefficient: The shape of the diffuser ports is specified in the Appendix-A drawings in the most recent NPDES permit for the SJCOO outfall (cf: RWQCB, 2014). Accordingly, a diffuser port contraction coefficient of 1.0 was used, consistent with the values used in the Plumes 17b simulation examples appearing in Appendix-B of Roberts (2018a).

3.3.3) Standard Light Adsorption Coefficient: The value of 0.16 is recommended in the Visual Plumes manual as a conservative value. This is not relevant to final initial dilution, and is for the Mancini bacteria model applications of the model.

3.3.4) Far-field Increment: This value controls the number of lines output by the Brooks far-field algorithm. A small value produces more lines and graphic output than large values. A value between 100 to 1000 m is recommended by the Plumes 18b manual. This field has little effect on the final calculated initial dilution; a value of 100 m was used in the data field.

3.3.5) UM3 Aspiration Coefficient: This is the rate at which ambient fluid is entrained (diluted) into the plume. The default value of 0.1 is an average that is rarely changed. A larger value causes more rapid plume spreading and affects other characteristics, like plume rise. The default value of 0.1 was used in the data field.

3.3.6) Output Settings: Output settings were configured for “standard text output format” with a group of selected variables that included: “depth, Amb-cur, P-dia, Eff-sal, Polutnt, Dilutn, x-posn, y-posn, Iso dia”. The most relevant of these variables for proceeding with the turbulence mortality assessment are “depth”, “Eff-sal,” “Dilutn”, “x-posn” and “y-posn.” In particular, the “x-posn” and “y-posn” output variables are used to quantify the distances to the maximum rise and bottom hit points of the trajectories, while the “Dilutn” output variable quantifies the effective (average or bulk) dilution at those points.

3.3.5) UM3 Options and Controls: Under the vertical reversals options, “to max rise or fall” and “allow induced currents (multiport)” were selected. For the dense (negatively buoyant) discharge scenarios, the “stop on bottom hit” setting was selected in order to prevent the simulations from running through the seabed.

4) Results

Text file and graphical output from Plumes 18b for each of the Doheny Desalination Project (DDP) modeling scenarios in Table-1 are found in Appendices A-C. The results for the buoyant discharge scenarios are found in Appendix-B, while those for the dense (negatively buoyant) discharge scenarios are in Appendix-C. In order to resolve incremental turbulence mortality impacts of the buoyant discharge scenarios, it was necessary to run a separate set of wastewater-only baseline cases using the particular flow volumes for the each wastewater increment used in the buoyant discharge scenarios. The Plumes 18b text and graphics results for these wastewater-only baseline cases are Found in Appendix-A.

4.1) Results for Buoyant DDP Discharge Scenarios: The Plumes 18b dilution modeling results for the combinations of SOCWA wastewater and DDP brine that produce buoyant discharges are summarized in Table 2, and contrasted there with wastewater-only baseline results. In general, Plumes 18b has predicted higher *Minimum Initial Dilution*, Dm, and smaller *Zones of Initial Dilution*, ZID at deeper depths than was reported previously by Jenkins (2016 & 2017) using the US EPA supported Visual Plumes (UM3). All of the buoyant DDP discharge scenarios are found to achieve the required Dm = 101 to 1 minimum initial dilution required under the current NPDES permit (No. CA 0107417, Order No. R9-2012-0012 as amended by Order No. R9-2014-0105). For any given amount of SOCWA wastewater, the addition of any amount of DDP brine reduces both the minimum initial dilution as well as the effective (average or bulk) dilution at the maximum rise of the plume, while reducing the size of the ZID. This occurs because the addition of any amount of DDP brine to SOCWA wastewater reduces the buoyancy of the discharge plume, causing it to cease rising at a lower altitude (deeper depth) in the water column, thereby reducing the amount of lateral spreading of the plume with associated reductions in dilution and the size of the ZID. This is not altogether a bad result, so long as there remains adequate dilution to satisfy present or future NPDES permit requirements for minimum initial dilution; which indeed appears to be the case. The lowest minimum initial dilutions and smallest ZIDs occur for operating conditions when the combined discharge rate is high with high proportions of brine relative to wastewater, such as the combination of 15 mgd of brine and 18.9 mgd of wastewater that resulted in a minimum initial dilution of Dm = 107.6 to 1 with a ZID = 63 m. All the other DDP operating scenarios producing buoyant combined effluent result in minimum initial

Table 2: Plumes 18b Modeling of Doheny Buoyant Discharge Scenarios

Discharge Scenario Brine + Wastewater = Total Flow Rate (MGD)	Combined Discharge Salinity (ppt)	Discharge Velocity m/sec	Densimetric Froude Number $F_r = u / \sqrt{g' d}$	Depth of 101 to 1 dilution factor (ft)	Depth of maximum rise of plume (ft)	Distance to maximum rise of plume, Z_a (m)	effective dilution at maximum rise of plume, S_a	Minimum Initial Dilution, Dm	Diameter of ZID (m)
8 mgd wastewater-only baseline	***1.25***	***0.595***	***4.231***	***86.96***	***56.35***	***13.304***	***375.5***	***383.1***	***196***
5 + 8 = 13	***25.77***	***0.967***	***12.95***	***70.89***	***62.90***	***11.308***	***130.9***	***133.9***	***78***
13 mgd wastewater-only baseline	***1.25***	***0.967***	***6.875***	***84.51***	***52.82***	***14.380***	***315.4***	***321.9***	***165***
5 + 13 = 18.0	***18.61***	***1.338***	***13.62***	***71.72***	***53.98***	***14.026***	***175.2***	***178.7***	***160***
18.9 mgd wastewater-only baseline	***1.25***	***1.405***	***9.996***	***82.69***	***50.81***	***14.992***	***273.8***	***279.5***	***143***
5 + 18.9 = 23.9	***14.02***	***1.777***	***16.02***	***71.01***	***50.90***	***14.965***	***185.4***	***189.1***	***135***
15 + 18.9 = 33.9	***29.64***	***2.521***	***43.40***	***68.94***	***68.94***	***9.467***	***105.0***	***107.6***	***63***
31 mgd wastewater-only baseline	***1.25***	***2.305***	***16.39***	***79.853***	***48.50***	***15.696***	***226.6***	***231.3***	***123.0***
5 + 31 = 36.0	***9.30***	***2.677***	***21.83***	***78.03***	***50.05***	***15.224***	***192.7***	***196.6***	***110.0***
****15 + 31 = 46.0***	***21.85***	***3.420***	***38.72***	***67.04***	***55.57***	***13.542***	***152.0***	***155.1***	***109***

Red = dry-weather
Blue = average conditions
Green = wet-weather
*Exceeds maximum permitted combined discharge rate of 38.78 mgd under NPDES permit (No. CA 0107417, Order No. R9-2012-0012 as amended by Order No. R9-2014-0105)

dilutions that exceed present NPDES permit requirements by a factor of 1.5 to 3.2 with ZIDs well over 100 m in diameter.

The reduction of buoyant effluent dilution caused by adding brine to SOCWA wastewater has a favorable effect on potentially deleterious diffuser entrainment, even though buoyant discharges appear to be exempt from requirements to assess, minimize or mitigate for turbulence mortality impacts to entrained marine organisms under the present structure of the amended Ocean Plan (SWRCB,2015). Table-3 summarizes all the parameters and results for deleterious diffuser entrainment for both the DDP buoyant discharge operating scenarios as well as the SOCWA wastewater-only baseline simulations. The results show that for any given amount of SOCWA wastewater, the addition of any amount of DDP brine reduces the deleterious diffuser entrainment rate, thus improving upon an existing condition that is presently not mitigatable under present implementation practices of the Ocean Plan. Therefore, no mitigation should be required for DDP operational scenarios that result in buoyant combined discharges with SOCWA wastewater. Inspection of Table-3 indicates that the net turbulence mortality benefit achieved by combining DDP brine with SOCWA wastewater increases with decreasing combined discharge rate, as smaller jet velocities with larger Kolmogorov turbulent eddies occur at lower combined discharge rates.

4.2) Results for Dense (Negatively Buoyant) DDP Discharge Scenarios: The Plumes 18b dilution modeling results for the combinations of SOCWA wastewater and DDP brine that produce dense (negatively buoyant) discharges are summarized in Table 4. All of the dense discharge cases in Table-4 involve either brine-only or high-brine ratio discharges, typical of conditions anticipated dry-weather wastewater effluent streams or future water reclamation conditions. Again, Plumes 18b has predicted higher effective dilution, S_a, and shorter distances to the 2 ppt over natural background compliance threshold than was reported previously by Jenkins (2016 & 2017) using the US EPA supported Visual Plumes (UM3). Based on long term averages of ambient salinity records reported in Jenkins (2016), natural background salinity at the SJCOO is 33.52 ppt, so that the compliance threshold 35.52 ppt under Appendix-A brine amendment provisions of the California Ocean Plan (SWRCB, 2015). Plumes 18b results in Table 4 indicate this compliance threshold is met in less than 2.5 ft. from the point of discharge by all DDP dense discharge operating conditions; whereas the Ocean Plan requires this compliance threshold is reached within 100 m from the discharge point. Thus the DDP would be fully compliant with Ocean Plan brine discharge limits by a wide margin of safety, according to Plumes 18b dilution simulations. As mentioned in Section 2, the jets of the SJCOO diffuser discharge parallel to the bottom and the ports are only 4.5 ft. above the bottom. Consequently, dense discharges from the SJCOO diffuser only rise a couple of feet above the discharge point (due to vertical spreading of the discharge jets by the action of turbulent mixing), before the trajectory bends downward under the action of negative buoyancy and makes contact with the bottom. By the literal interpretation of the State Water Board’s guidance document (Roberts, 2018a) deleterious diffuser entrainment should only be based on the entrainment at the maximum rise of the plume. But, Table-5 indicates that the Kolmogorov eddy scales remain substantially less than the injury threshold of 1 mm all the way until the discharge trajectory makes contact with the bottom. Therefore, Table 4 includes trajectory analysis of the distances to both the

Table 3: Deleterious Diffuser Entrainment for Doheny Buoyant Discharge Scenarios

Discharge Scenario Brine + Wastewater = Total Flow Rate (MGD)	Discharge Velocity, u m/sec	*Jet Reynolds Number $\mathrm{Re} = ud/\nu$	Depth of maximum rise of plume (ft)	Distance to maximum rise of plume, X_a (m)	Kolmogorov scale at maximum rise of plume** (mm)	effective dilution at maximum rise of plume, S_a	Deleterious diffuser entrainment at maximum rise of plume (MGD)	Incremental Impact of Deleterious diffuser entrainment (MGD)	Diameter of ZID (m)	Incremental Impact on Diameter of ZID (m)
8 mgd wastewater-baseline	0.595	39,397.1	56.35	13.304	1.142	375.5	3,004	N/A	196	N/A
5 + 8 = 13	0.967	64,028.6	62.90	11.308	0.674	130.9	1,701.7	-1,302.3	78	-188
13 mgd wastewater-baseline	0.967	64,028.6	52.82	14.380	0.857	315.4	4,100.2	N/A	165	N/A
5 + 13 = 18.0	1.338	88,593.8	53.98	14.026	0.656	175.2	3,153.6	-946.6	160	-5
18.9 mgd wastewater-baseline	1.405	93,030.2	50.81	14.992	0.675	273.8	5,174.82	N/A	143	N/A
5 + 18.9 = 23.9	1.777	117,661.7	50.90	14.965	0.565	185.4	4,431.06	-743.76	135	-8
15 + 18.9 = 33.9	2.521	166,924.6	68.94	9.467	0.275	105.0	3,559.5	-1,615.32	63	-80
31 mgd wastewater-baseline	2.305	152,622.5	48.50	15.696	0.487	226.6	7,024.6	N/A	123.0	N/A
5 + 31 = 36.0	2.677	177,254.0	50.05	15.224	0.422	192.7	6,937.2	-87.4	110.0	-13
15 + 31 = 46.0	3.420	226,450.7	55.57	13.542	0.313	152.0	6,992	-32.6	109	-14

*****Based on jet diameter d = 3.05 in. and kinematic viscosity, ν = 1.17 X 10^{-6} m^2/s

******Based on Kolmogorov scale $\eta_c = 0.24 X_a \mathrm{Re}^{-3/4}$, per equation (22) in Roberts, (2018a)

Table 4: Plumes 18b Modeling of Doheny Dense (Negatively Buoyant) Discharge Scenarios

Discharge Scenario Brine + Wastewater = Total Flow Rate (MGD)	Combined Discharge Salinity (ppt)	Discharge Velocity m/sec	Densimetric Froude Number $F_r = u / \sqrt{g' d}$	Horizontal Distance to within 2 ppt of *Natural Background (ft)	Distance to bottom hit, X_b (ft)	Depth of maximum rise of plume (ft)	Distance to maximum rise of plume, X_a (ft)	effective dilution at maximum rise of plume, $S_a(x=X_a)$	effective dilution at bottom hit, $S_a(x=X_b)$
3 + 0 = 3	***67.0***	***0.223***	***1.678***	***0.566***	***0.876***	***99.8***	***0.750***	***26.03***	***40.26***
3 + 1.8* = 4.8	***54.44***	***0.357***	***3.468***	***0.260***	***1.423***	***99.0***	***0.735***	***17.19***	***36.66***
5 + 0 = 5	***67.0***	***0.372***	***2.796***	***0.653***	***1.252***	***99.3***	***0.800***	***19.08***	***37.94***
5 + 0.35 = 5.35	***62.63***	***0.398***	***3.226***	***1.095***	***1.348***	***99.1***	***0.462***	***12.53***	***37.18***
10 + 0 = 10	***67.0***	***0.744***	***5.593***	***1.346***	***2.555***	***99.2***	***1.704***	***20.92***	***34.96***
15 + 0 = 15	***67.0***	***1.115***	***8.389***	***2.466***	***4.076***	***99.2***	***2.803***	***19.42***	***31.83***
15 + 8 = 23	***43.69***	***1.710***	***25.36***	***2.176***	***10.14***	***99.1***	***6.038***	***12.09***	***21.26***
15 + 13 = 28.0	***35.89***	***2.082***	***165.0***	***0.116***	***19.76***	***96.8***	***20.10***	***26.04***	***26.04***

Black = dry-weather with well water substituted for SOCWA wastewater
Red = dry-weather or future water reclamation conditions
*Natural background salinity at the SJCOO is 33.5 ppt.
**Fails to dilute to within 2 ppt of natural background salinity within a horizontal distance of 100 m

Table 5: Deleterious Diffuser Entrainment for Doheny Dense (Negatively Buoyant) Discharge Scenarios

Discharge Scenario Brine + Wastewater = Total Flow Rate (MGD)	Discharge Velocity, u m/sec	*Jet Reynolds Number $\mathrm{Re} = u\,d/\nu$	Distance to maximum rise of plume, X_a (ft)	Distance to plume bottom hit, X_b (ft)	Kolmogorov scale at maximum rise of plume** (mm)	Kolmogorov scale at plume bottom hit*** (mm)	effective dilution at maximum rise of plume, $S_a(x=X_a)$	effective dilution at bottom hit, $S_a(x=X_b)$	Deleterious diffuser entrainment at maximum rise of plume (MGD)	Deleterious diffuser entrainment at bottom hit of plume (MGD)
3 + 0 = 3	0.223	14,765.6	0.750	0.876	0.041	0.048	26.03	40.26	78.09	120.78
3 + 1.8[a] = *4.8*	0.357	23,638.2	0.735	1.423	0.028	0.054	17.19	36.66	82.512	175.968
5 + 0 = 5	0.372	24,631.4	0.800	1.252	0.029	0.046	19.08	37.94	95.4	189.7
5 + 0.35 = 5.35	0.398	26,353.0	0.462	1.348	0.016	0.048	12.53	37.18	67.0355	198.913
10 + 0 = 10	0.744	49,262.9	1.704	2.555	0.038	0.057	20.92	34.96	209.2	349.6
15 + 0 = 15	1.115	73,828.2	2.803	4.076	0.046	0.066	19.42	31.83	291.3	477.45
15 + 8 = 23	1.710	113,225.3	6.038	10.14	0.072	0.120	12.09	21.26	278.07	488.98
15 + 13 = 28.0	2.082	137,856.8	20.10	19.76	0.21	0.202	26.04	26.04	729.12	729.12

***Based on jet diameter d = 3.05 in. and kinematic viscosity, ν = 1.17 X 10^{-6} m^2/s

****Based on Kolmogorov scale $\eta_c = 0.24\,X_a\,\mathrm{Re}^{-3/4}$, per equation (22) in Roberts, (2018a)

****Based on Kolmogorov scale $\eta_c = 0.24\,X_b\,\mathrm{Re}^{-3/4}$, per equation (22) in Roberts, (2018a)

[a] well water substituted for SOCWA wastewater

maximum rise and bottom hit points, and the associated effective (bulk or average) dilutions at both of those points. While no clear relationships appear to emerge from Table-4 entries for effective dilution at the maximum rise points of the trajectory; effective dilution at the bottom hit points increases with decreasing salinity and flow rates of the combined brine/wastewater effluent.

Table-5 summarizes all the parameters and results for deleterious diffuser entrainment for the dense (negatively buoyant) DDP discharge operating scenarios. Because of the relatively short distances traveled by the dense discharges before reaching maximum rise or bottom hit points, the Kolmogorov eddy scales remain small (less than 0.2 mm), and presumably injurious according to the injury hypothesis advanced in the State Water Boards turbulence mortality guidance document (Roberts, 2018a). But these short trajectories also limit the effective (bulk or average) dilution and therefore limit the deleterious diffuser entrainment. Again, by the literal interpretation of the State Water Board's guidance document (Roberts, 2018a) mitigation scaling for brine diffuser turbulence mortality should only be based on the entrainment at the maximum rise of the plume, as indicated by the deleterious entrainment numbers appearing in the second to last column in Table-5 that range from 67 mgd to 729 mgd. But, the short travel distances to bottom hit points and small Kolmogorov eddy scales may deviate the assessment to the entrainment numbers at the bottom hit points, appearing in the last column of Table-5. In either case, deleterious diffuser entrainment increases with increasing combined discharge rate, (as would be expected with associated increasing discharge velocities); while the deleterious entrainment numbers at the bottom hit points are about a factor of 1.7 to 2.1 larger at the bottom hit points than at the maximum rise point of the discharge trajectories. These deleterious entrainment rates are to be throughput to the ETM/APF (Empirical Transport Model/Area of Production Foregone) calculus to compute the mitigation scaling for DDP diffuser turbulent shear impact

5) Conclusions:

Diffusers intrinsically generate strong turbulent jets in order to produce mixing and rapid dilution of effluent, and the shearing action of those turbulent jets can potentially damage or kill small delicate organisms entrained into those jets (sometimes referred to as *diffuser turbulence mortality*). The implementation section of the brine amendment to the California Ocean Plan, Section III.M.2 (b), requires that brine diffusers must minimize and mitigate for such marine life impacts, and the California State Water Resources Control Board has released newly defined protocols that require the use of a specific hydrodynamic mixing model (referred to as *Plumes 18b*) to assess those impacts. Plumes 18b is not supported by US EPA, but the State Water Board has made executable files for this model publicly available on their web site, along with a technical guidance document on how to assess deleterious entrainment from brine diffusers. These protocols using the Plumes 18b model are implemented in this study to assess potential injury or mortality to small marine organisms entrained by discharges from the diffuser of the San Juan Creek Ocean Outfall (SJCOO) that is being proposed as the discharge structure for brine by-product from the Doheny Desalination Project (DDP).

In general, Plumes 18b predicted higher *Minimum Initial Dilution,* and smaller *Zones of Initial Dilution,* ZID at deeper depths than was reported previously by DDP dilution studies using the US EPA supported Visual Plumes (UM3). Using Plumes 18b, all of the buoyant DDP discharge scenarios are found to achieve the required 101 to 1 minimum initial dilution required under the current NPDES permit for the SJCOO, (No. CA 0107417, Order No. R9-2012-0012 as amended by Order No. R9-2014-0105). For any given amount of SOCWA wastewater, the addition of any amount of DDP brine reduces both the minimum initial dilution as well as the effective (average or bulk) dilution at the maximum rise of the plume, while reducing the size of the ZID. This is not altogether a bad result, so long as there remains adequate dilution to satisfy present or future NPDES permit requirements for minimum initial dilution; which indeed appears to be the case. The reduction of buoyant effluent dilution caused by adding brine to SOCWA wastewater has a favorable effect on potentially deleterious diffuser entrainment, even though buoyant discharges appear to be exempt from requirements to assess, minimize or mitigate for diffuser turbulence mortality impacts to entrained marine organisms under the present structure of the amended Ocean Plan (SWRCB,2015). The Plumes 18b results show that for any given amount of SOCWA wastewater, the addition of any amount of DDP brine reduces the deleterious diffuser entrainment rate, thus improving upon an existing condition that is not mitigatable under present implementation practices of the Ocean Plan. Therefore, no mitigation should be required for DDP operational scenarios that result in buoyant combined discharges with SOCWA wastewater. The net turbulence mortality benefit achieved by combining DDP brine with SOCWA wastewater increases with decreasing combined discharge rate, as smaller jet velocities with larger Kolmogorov turbulent eddies occur at lower combined discharge rates.

The Plumes 18b dilution modeling results for the combinations of SOCWA wastewater and DDP brine that produce dense (negatively buoyant) discharges involve either brine-only or high-brine ratio discharges, typical of conditions anticipated during dry-weather wastewater effluent streams or future water reclamation conditions. Again, Plumes 18b has predicted higher effective dilution and shorter distances to the 2 ppt over natural background compliance threshold than was reported previously in DDP dilution studies using the US EPA supported Visual Plumes (UM3). Based on long term averages of ambient salinity records, natural background salinity at the SJCOO is 33.52 ppt, so that the compliance threshold 35.52 ppt under Appendix-A brine amendment provisions of the California Ocean Plan (SWRCB, 2015). Plumes 18b results indicate this compliance threshold is met in less than 2.5 ft. from the point of discharge by all DDP dense discharge operating conditions; whereas the Ocean Plan requires this compliance threshold is reached within 100 m from the discharge point. Thus, the DDP would be fully compliant with Ocean Plan brine discharge limits by a wide margin of safety, according to Plumes 18b dilution simulations. The jets of the SJCOO diffuser discharge parallel to the bottom and the ports are only 4.5 ft. above the bottom. Consequently the trajectories of these dense DDP discharges travel relatively short distances before reaching maximum rise or bottom hit points. This behavior, in turn, causes the Kolmogorov eddy scales in the diffuser jets to remain small (less than 0.2 mm), and presumably injurious according to the injury hypothesis advanced in the State Water Boards turbulence mortality guidance document. But these short trajectories also limit the effective (bulk or average) dilution and therefore limit the deleterious diffuser entrainment. By the literal interpretation of the State Water Board’s guidance document mitigation scaling for brine diffuser turbulence mortality should only be based on the entrainment at the maximum rise of the plume, which range from 67 mgd to 729 mgd for dense DDP discharges.

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APPENDIX-A: Wastewater Only Baseline Results

A.1: Plumes 18b Results for SJCOO discharges of 31 mgd Wastewater Only:

SJCOO discharging 31 mgd of wastewater at TDS = 1.25
ppt

Model configuration items checked:
Channel width (m) 100
Start case for graphs 1
Max detailed graphs 10 (limits plots that can overflow memory)
Elevation Projection Plane (deg) 0
Shore vector (m,deg) not checked
Bacteria model : Mancini (1978) coliform model
PDS sfc. model heat transfer : Medium
Equation of State : S, T
Similarity Profile : Default profile (k=2.0, ...)
Diffuser port contraction coefficient 1
Light absorption coefficient 0.16
Farfield increment (m) 200
UM3 aspiration coefficient 0.1
Output file: text output tab
Output each ?? steps 10
Maximum dilution reported 1000
Text output format : Standard
Max vertical reversals : to max rise or fall

/ UM3. 1/9/2019 2:49:23 PM
Case 1; ambient file C:\Plumes18\SJCOO_WW31mgd_b0mgd_T-12.001.db; Diffuser table record 1: ------------------
----

Ambient Table:

Depth	Amb-cur	Amb-dir	Amb-sal	Amb-tem	Amb-pol	Decay	Far-spd	Far-dir	Disprsn	Density
m	m/s	deg	psu	C	kg/kg	s-1	m/s	deg	m0.67/s2	sigma-T
0.0	0.0	0.0	33.38	19.97	0.0	0.0	0.0	0.0	0.0003	23.55719
2.000	0.0	0.0	33.39	19.87	0.0	0.0	0.0	0.0	0.0003	23.59288
4.000	0.0	0.0	33.40	19.81	0.0	0.0	0.0	0.0	0.0003	23.61484
7.000	0.0	0.0	33.41	19.64	0.0	0.0	0.0	0.0	0.0003	23.67096
10.00	0.0	0.0	33.40	17.76	0.0	0.0	0.0	0.0	0.0003	24.12888
12.00	0.0	0.0	33.38	17.34	0.0	0.0	0.0	0.0	0.0003	24.21549
14.00	0.0	0.0	33.38	16.73	0.0	0.0	0.0	0.0	0.0003	24.35932
16.00	0.0	0.0	33.38	15.17	0.0	0.0	0.0	0.0	0.0003	24.71340
18.00	0.0	0.0	33.37	14.37	0.0	0.0	0.0	0.0	0.0003	24.87044
20.00	0.0	0.0	33.35	14.18	0.0	0.0	0.0	0.0	0.0003	24.89660
22.00	0.0	0.0	33.33	13.83	0.0	0.0	0.0	0.0	0.0003	24.95172
24.00	0.0	0.0	33.32	13.75	0.0	0.0	0.0	0.0	0.0003	24.96707
26.00	0.0	0.0	33.36	13.42	0.0	0.0	0.0	0.0	0.0003	25.06459
29.00	0.0	0.0	33.36	13.39	0.0	0.0	0.0	0.0	0.0003	25.07096

Diffuser table:

P-dia	Ver angl	H-Angle	SourceX	SourceY	Ports	Spacing	MZ-dis	Isoplth	P-depth	Ttl-flo	Eff-sal	Temp	Polutnt
(in)	(deg)	(deg)	(ft)	(ft)	()	(ft)	(ft)	(concent)	(ft)	(MGD)	(psu)	(C)	(ppm)
3.0500	0.0	0.0	0.0	0.0	125.00	12.000	1000.0	0.0	100.00	31.000	1.2500	20.660	1250.0

Simulation:
Froude No: 16.39; Strat No: 1.48E-4; Spcg No: 47.21; k: 2.31E+5; eff den (sigmaT) -0.921168; eff vel 2.305(m/s);
Current is very small, flow regime may be transient.

Step	Depth (ft)	Amb-cur (m/s)	P-dia (in)	Eff-sal (psu)	Polutnt (ppm)	Dilutn ()	x-posn (m)	y-posn (m)	Iso dia (m)
0	100.0	1.000E-5	3.050	1.250	1250.0	1.000	0.0	0.0	0.07747;
1	100.00	0.0	3.104	2.385	1206.9	1.036	0.0219	0.0	0.07884; bottom hit;
10	100.00	0.0	3.732	7.442	1014.0	1.233	0.112	0.0	0.09479;
20	100.00	0.0	4.541	12.10	835.0	1.497	0.230	0.0	0.1153;
30	100.00	0.0	5.527	15.92	687.1	1.819	0.375	0.0	0.1404;
40	100.00	0.0	6.728	19.05	565.0	2.212	0.551	0.0	0.1709;
50	99.99	0.0	8.193	21.62	464.5	2.691	0.766	0.0	0.2081;
60	99.99	0.0	9.977	23.73	381.7	3.275	1.028	0.0	0.2534;
70	99.97	0.0	12.15	25.46	313.5	3.987	1.347	0.0	0.3086;
80	99.95	0.0	14.79	26.88	257.5	4.854	1.733	0.0	0.3757;
90	99.90	0.0	17.99	28.05	211.4	5.912	2.198	0.0	0.4570;
100	99.83	0.0	21.64	28.95	175.6	7.120	2.714	0.0	0.5496;
110	99.75	0.0	25.13	29.58	150.7	8.294	3.166	0.0	0.6384;
120	99.65	0.0	28.51	30.04	132.2	9.453	3.567	0.0	0.7242;
130	99.54	0.0	31.82	30.41	117.7	10.62	3.931	0.0	0.8082;
140	99.42	0.0	35.07	30.70	106.0	11.80	4.267	0.0	0.8907;
150	99.29	0.0	38.27	30.95	96.18	13.00	4.583	0.0	0.9722;
160	99.15	0.0	41.43	31.16	87.88	14.22	4.883	0.0	1.0523;
170	99.00	0.0	44.53	31.34	80.73	15.48	5.172	0.0	1.1311;
180	98.83	0.0	47.57	31.49	74.51	16.78	5.453	0.0	1.2083;
190	98.65	0.0	50.53	31.63	69.03	18.11	5.729	0.0	1.2835;
200	98.45	0.0	53.41	31.75	64.15	19.48	6.004	0.0	1.3565;
210	98.22	0.0	56.19	31.86	59.78	20.91	6.280	0.0	1.4272;
220	97.97	0.0	58.88	31.96	55.81	22.40	6.560	0.0	1.4955;
230	97.68	0.0	61.47	32.05	52.17	23.96	6.847	0.0	1.5614;
240	97.35	0.0	63.99	32.14	48.79	25.62	7.145	0.0	1.6253;
250	96.97	0.0	66.45	32.22	45.61	27.41	7.456	0.0	1.6878;
260	96.53	0.0	68.90	32.30	42.56	29.37	7.786	0.0	1.7500;
270	96.00	0.0	71.40	32.37	39.59	31.57	8.139	0.0	1.8137;
280	95.36	0.0	74.06	32.44	36.65	34.11	8.521	0.0	1.8812;
290	94.57	0.0	77.02	32.52	33.67	37.13	8.940	0.0	1.9562;
300	93.58	0.0	80.48	32.60	30.60	40.85	9.403	0.0	2.0441;
310	92.32	0.0	84.76	32.68	27.39	45.63	9.923	0.0	2.1529;
320	90.66	0.0	90.35	32.76	24.01	52.06	10.52	0.0	2.2950;
330	88.40	0.0	98.05	32.85	20.43	61.18	11.20	0.0	2.4904;
340	85.34	0.0	108.7	32.94	16.83	74.29	11.97	0.0	2.7606;
350	81.84	0.0	121.7	33.02	13.80	90.55	12.70	0.0	3.0903;
360	77.85	0.0	138.3	33.07	11.32	110.4	13.41	0.0	3.5138;
363	76.54	0.0	144.2	33.09	10.67	117.1	13.62	0.0	3.6619; merging;
370	72.54	0.0	164.5	33.12	9.290	134.6	14.23	0.0	4.1793;
380	64.76	0.0	215.6	33.16	7.621	164.0	15.28	0.0	5.4760;
389	56.09	0.0	298.6	33.19	6.377	196.0	16.35	0.0	7.5853; trap level;
390	55.05	0.0	311.9	33.19	6.252	199.9	16.48	0.0	7.9222;
398	50.15	0.0	517.9	33.21	5.726	218.3	17.17	0.0	13.156; begin overlap;
400	49.88	0.0	565.6	33.21	5.702	219.2	17.22	0.0	14.366;
410	49.15	0.0	765.5	33.21	5.648	221.3	17.38	0.0	19.444;
420	48.83	0.0	952.5	33.21	5.617	222.6	17.46	0.0	24.193;
430	48.65	0.0	1135.6	33.21	5.592	223.5	17.52	0.0	28.845;
440	48.55	0.0	1315.7	33.21	5.571	224.4	17.56	0.0	33.418;

450	48.51	0.0	1439.4	33.21	5.556	225.0	17.58	0.0	36.562;
460	48.50	0.0	1460.1	33.21	5.542	225.5	17.58	0.0	37.087;
470	48.50	0.0	1467.7	33.21	5.529	226.1	17.58	0.0	37.279;
480	48.50	0.0	1474.7	33.21	5.516	226.6	17.58	0.0	37.456; local maximum rise or fall;
487	51.72	0.0	1493.0	33.22	5.403	231.3	18.25	0.0	37.921;

Horiz plane projections in effluent direction: radius(m): 0.0; CL(m): 5.5640
Lmz(m): 5.5640
forced entrain 1 0.0 14.72 37.92 0.409
Rate sec-1 0.0 dy-1 0.0 kt: 0.0 Amb Sal 33.3818
;
2:49:24 PM. amb fills: 4

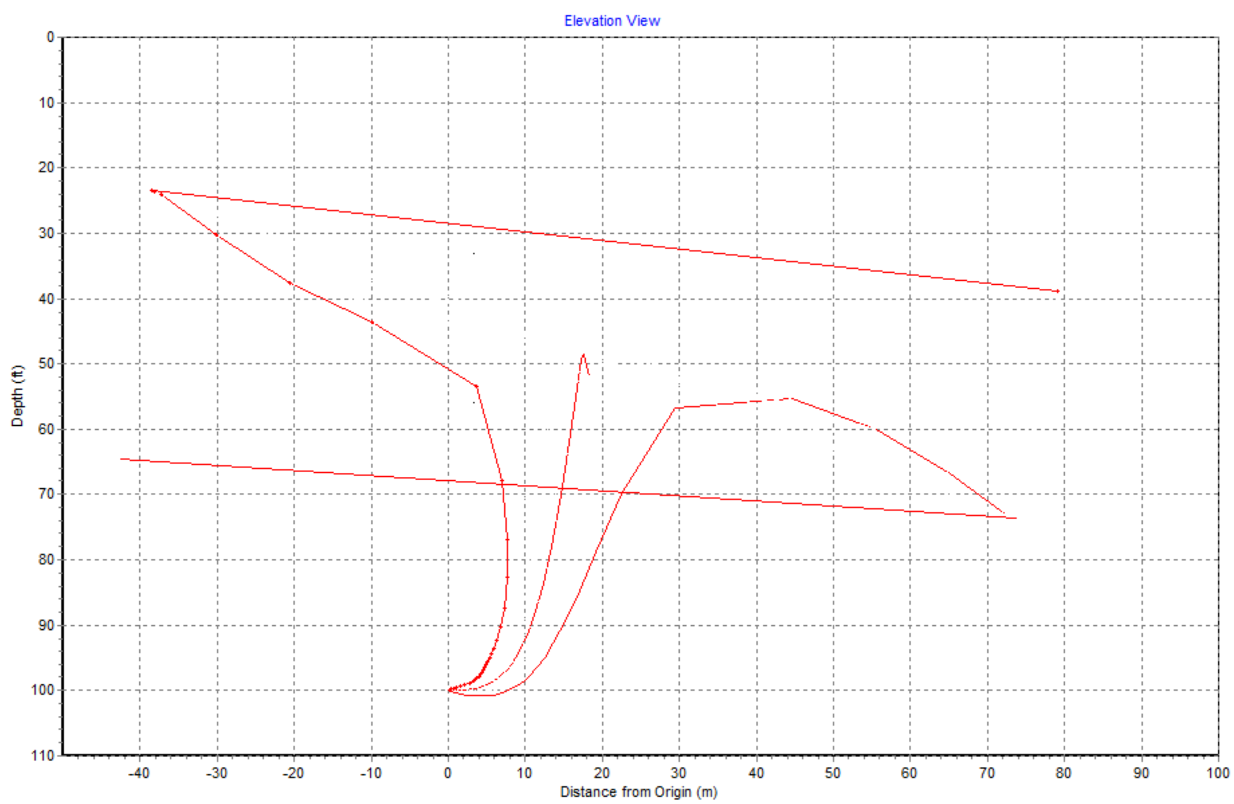

Figure A.1.1: Plumes 18b solution of discharge plume trajectories for discharges of 31 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt. ZID is defined by the maximum horizontal excursion of trajectories from the origin. From the maximum horizontal spreading of the plume, the ZID extends from X = -43 m to X = +80 m so that ZID = 123 m

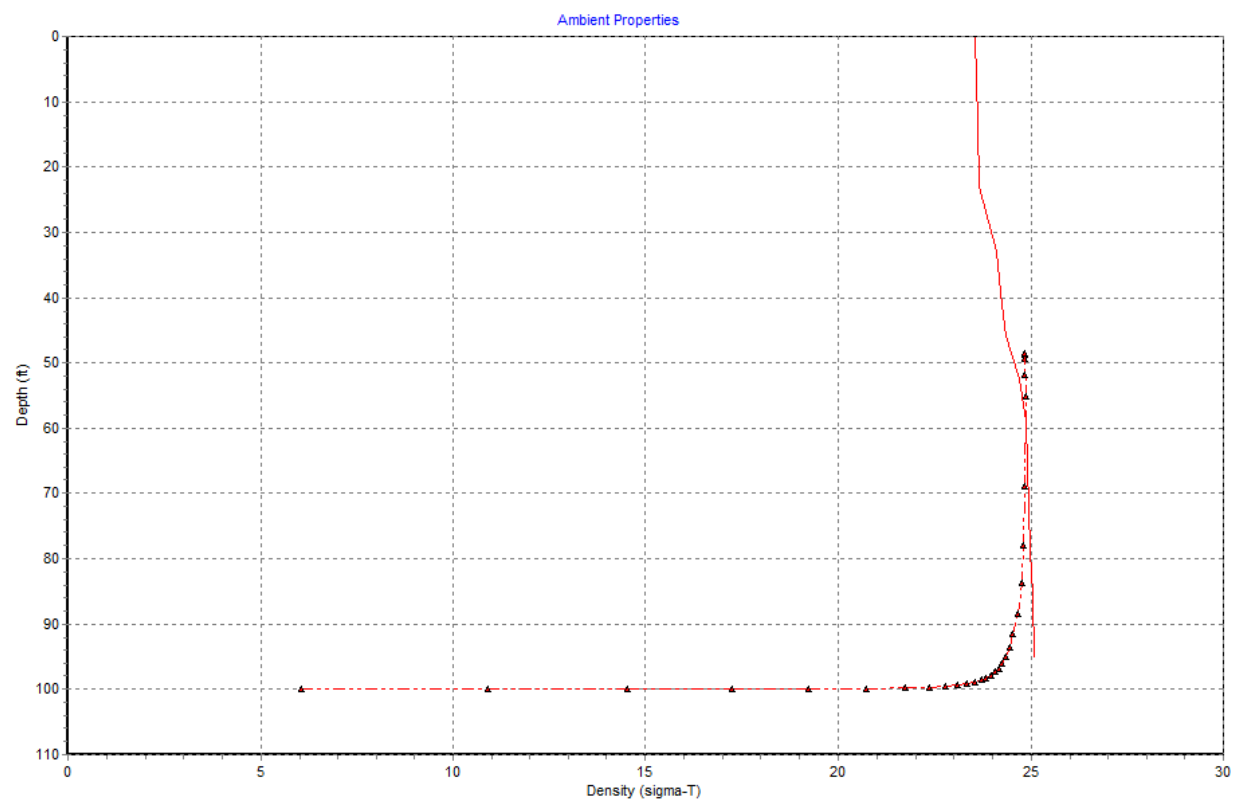

Figure A.1.2: Plumes 18b solution of vertical density profile for discharges of 31 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt. Discharge effluent density shown as black triangles. Ambient water mass density profile shown as solid red line.

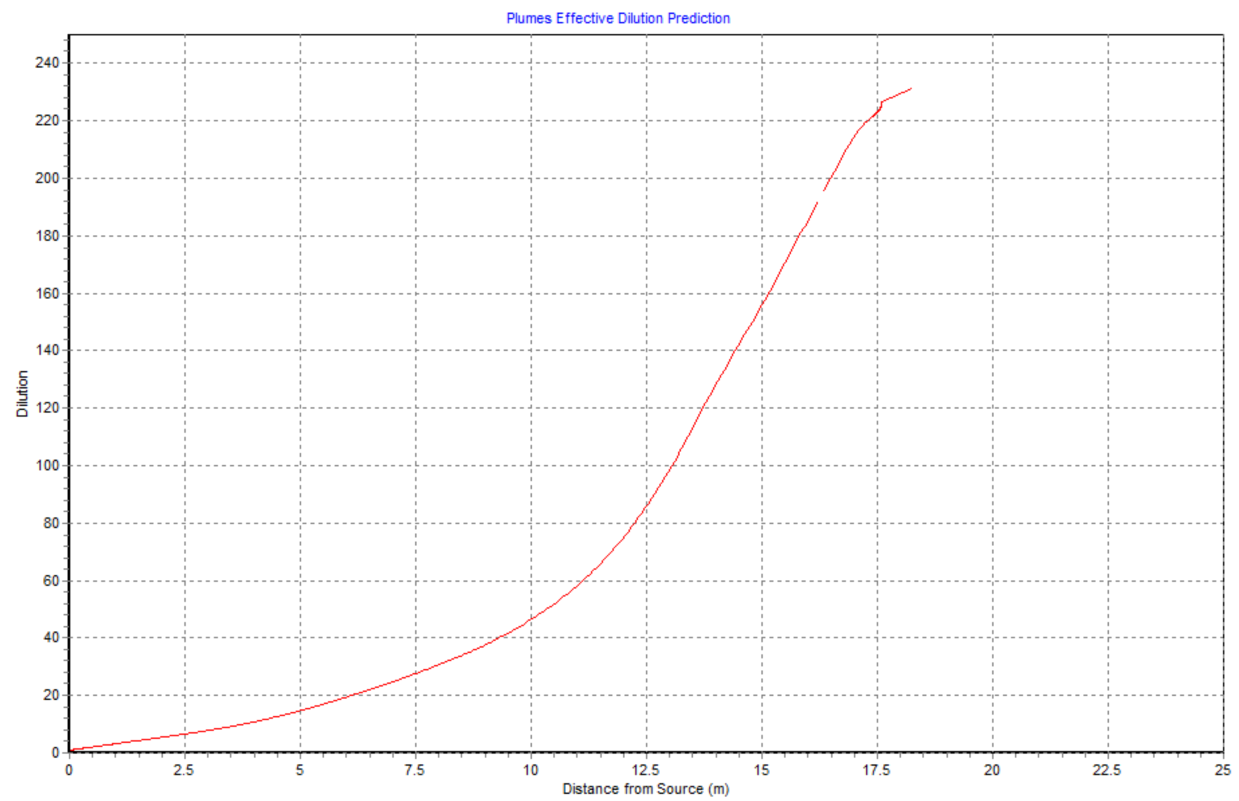

Figure A.1.3: Plumes 18b solution of effective (bulk average) dilution as a function of vertical distance from the point of discharge for discharges of 31 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt.

A.2: Plumes 18b Results for SJCOO discharges of 18.9 mgd Wastewater Only:

SJCOO discharging 18.9 mgd of wastewater at TDS = 1.25
ppt

Model configuration items checked:
Channel width (m) 100
Start case for graphs 1
Max detailed graphs 10 (limits plots that can overflow memory)
Elevation Projection Plane (deg) 0
Shore vector (m,deg) not checked
Bacteria model : Mancini (1978) coliform model
PDS sfc. model heat transfer : Medium
Equation of State : S, T
Similarity Profile : Default profile (k=2.0, ...)
Diffuser port contraction coefficient 1
Light absorption coefficient 0.16
Farfield increment (m) 200
UM3 aspiration coefficient 0.1
Output file: text output tab
Output each ?? steps 10
Maximum dilution reported 1000
Text output format : Standard
Max vertical reversals : to max rise or fall

/ UM3. 1/9/2019 3:16:57 PM
Case 1; ambient file C:\Plumes18\SJCOO_WW18.9mgd_b0mgd_T-1.001.db; Diffuser table record 1: ----------------------------------

Ambient Table:

Depth	Amb-cur	Amb-dir	Amb-sal	Amb-tem	Amb-pol	Decay	Far-spd	Far-dir	Disprsn	Density
m	m/s	deg	psu	C	kg/kg	s-1	m/s	deg	m0.67/s2	sigma-T
0.0	0.0	0.0	33.38	19.97	0.0	0.0	0.0	0.0	0.0003	23.55719
2.000	0.0	0.0	33.39	19.87	0.0	0.0	0.0	0.0	0.0003	23.59288
4.000	0.0	0.0	33.40	19.81	0.0	0.0	0.0	0.0	0.0003	23.61484
7.000	0.0	0.0	33.41	19.64	0.0	0.0	0.0	0.0	0.0003	23.67096
10.00	0.0	0.0	33.40	17.76	0.0	0.0	0.0	0.0	0.0003	24.12888
12.00	0.0	0.0	33.38	17.34	0.0	0.0	0.0	0.0	0.0003	24.21549
14.00	0.0	0.0	33.38	16.73	0.0	0.0	0.0	0.0	0.0003	24.35932
16.00	0.0	0.0	33.38	15.17	0.0	0.0	0.0	0.0	0.0003	24.71340
18.00	0.0	0.0	33.37	14.37	0.0	0.0	0.0	0.0	0.0003	24.87044
20.00	0.0	0.0	33.35	14.18	0.0	0.0	0.0	0.0	0.0003	24.89660
22.00	0.0	0.0	33.33	13.83	0.0	0.0	0.0	0.0	0.0003	24.95172
24.00	0.0	0.0	33.32	13.75	0.0	0.0	0.0	0.0	0.0003	24.96707
26.00	0.0	0.0	33.36	13.42	0.0	0.0	0.0	0.0	0.0003	25.06459
29.00	0.0	0.0	33.36	13.39	0.0	0.0	0.0	0.0	0.0003	25.07096

Diffuser table:

P-dia	Ver angl	H-Angle	SourceX	SourceY	Ports	Spacing	MZ-dis	Isoplth	P-depth	Ttl-flo	Eff-sal	Temp	Polutnt
(in)	(deg)	(deg)	(ft)	(ft)	()	(ft)	(ft)	(concent)	(ft)	(MGD)	(psu)	(C)	(ppm)
3.0500	0.0	0.0	0.0	0.0	125.00	12.000	1000.0	0.0	100.00	18.900	1.2500	20.660	1250.0

Simulation:

Froude No: 9.996; Strat No: 1.48E-4; Spcg No: 47.21; k: 1.41E+5; eff den (sigmaT) -0.921168; eff vel 1.405(m/s);
Current is very small, flow regime may be transient.

Step	Depth (ft)	Amb-cur (m/s)	P-dia (in)	Eff-sal (psu)	Polutnt (ppm)	Dilutn ()	x-posn (m)	y-posn (m)	Iso dia (m)
0	100.0	1.000E-5	3.050	1.250	1250.0	1.000	0.0	0.0	0.07747;
1	100.00	0.0	3.083	1.952	1223.4	1.022	0.0135	0.0	0.07831; bottom hit;
10	100.00	0.0	3.681	7.079	1027.9	1.216	0.0809	0.0	0.0935;
20	100.00	0.0	4.479	11.80	846.4	1.477	0.166	0.0	0.1138;
30	100.00	0.0	5.451	15.67	696.5	1.795	0.271	0.0	0.1385;
40	100.00	0.0	6.635	18.85	572.9	2.182	0.400	0.0	0.1685;
50	99.99	0.0	8.078	21.46	470.9	2.654	0.556	0.0	0.2052;
60	99.98	0.0	9.834	23.60	387.0	3.230	0.747	0.0	0.2498;
70	99.96	0.0	11.96	25.35	317.9	3.932	0.978	0.0	0.3039;
80	99.93	0.0	14.50	26.77	262.0	4.770	1.250	0.0	0.3682;
90	99.89	0.0	17.09	27.79	221.7	5.638	1.497	0.0	0.4342;
100	99.84	0.0	19.71	28.55	191.4	6.530	1.713	0.0	0.5007;
110	99.79	0.0	22.38	29.15	167.7	7.455	1.905	0.0	0.5683;
120	99.73	0.0	25.09	29.63	148.5	8.420	2.079	0.0	0.6372;
130	99.67	0.0	27.84	30.03	132.6	9.426	2.240	0.0	0.7071;
140	99.60	0.0	30.62	30.37	119.3	10.48	2.390	0.0	0.7777;
150	99.53	0.0	33.40	30.65	108.1	11.57	2.532	0.0	0.8484;
160	99.45	0.0	36.16	30.89	98.48	12.69	2.668	0.0	0.9186;
167	99.39	0.0	38.07	31.04	92.57	13.50	2.761	0.0	0.9670; begin overlap;
170	99.37	0.0	38.87	31.10	90.25	13.85	2.800	0.0	0.9873;
180	99.27	0.0	41.37	31.27	83.38	14.99	2.929	0.0	1.0507;
190	99.17	0.0	43.67	31.42	77.50	16.13	3.059	0.0	1.1091;
200	99.06	0.0	45.82	31.55	72.32	17.28	3.189	0.0	1.1638;
210	98.93	0.0	47.87	31.67	67.63	18.48	3.322	0.0	1.2158;
219	98.80	0.0	49.66	31.76	63.70	19.62	3.446	0.0	1.2613; end overlap;
220	98.78	0.0	49.85	31.78	63.28	19.75	3.460	0.0	1.2662;
230	98.61	0.0	51.68	31.87	59.33	21.07	3.604	0.0	1.3127;
240	98.42	0.0	53.32	31.96	55.72	22.43	3.756	0.0	1.3544;
250	98.18	0.0	54.79	32.05	52.37	23.87	3.920	0.0	1.3916;
260	97.89	0.0	56.11	32.13	49.18	25.42	4.098	0.0	1.4253;
270	97.54	0.0	57.37	32.21	46.05	27.14	4.295	0.0	1.4571;
280	97.09	0.0	58.66	32.29	42.88	29.15	4.516	0.0	1.4899;
290	96.51	0.0	60.15	32.37	39.55	31.60	4.768	0.0	1.5278;
300	95.74	0.0	62.10	32.46	35.93	34.79	5.060	0.0	1.5774;
310	94.66	0.0	64.93	32.56	31.89	39.20	5.405	0.0	1.6493;
320	93.13	0.0	69.35	32.68	27.32	45.76	5.819	0.0	1.7616;
330	90.97	0.0	76.09	32.80	22.49	55.58	6.293	0.0	1.9328;
340	88.52	0.0	84.34	32.90	18.45	67.75	6.733	0.0	2.1423;
350	85.78	0.0	94.01	32.98	15.14	82.58	7.142	0.0	2.3878;
360	82.69	0.0	105.4	33.05	12.42	100.7	7.527	0.0	2.6770;
370	79.20	0.0	119.8	33.10	10.19	122.7	7.902	0.0	3.0441;
380	75.20	0.0	138.1	33.14	8.357	149.6	8.283	0.0	3.5083;
383	73.88	0.0	144.2	33.15	7.875	158.7	8.400	0.0	3.6615; merging;
390	69.82	0.0	165.8	33.18	6.856	182.3	8.736	0.0	4.2112;
400	61.78	0.0	231.8	33.21	5.624	222.3	9.349	0.0	5.8881; trap level;
410	52.59	0.0	428.0	33.23	4.707	265.6	10.10	0.0	10.872;
413	51.93	0.0	546.2	33.23	4.655	268.5	10.18	0.0	13.874; begin overlap;
420	51.33	0.0	768.8	33.23	4.625	270.2	10.27	0.0	19.527;
430	51.03	0.0	1051.1	33.23	4.608	271.3	10.32	0.0	26.699;
440	50.89	0.0	1324.1	33.24	4.595	272.0	10.36	0.0	33.633;
450	50.82	0.0	1588.6	33.24	4.585	272.6	10.38	0.0	40.351;

460	50.81	0.0	1690.5	33.24	4.578	273.0	10.38	0.0	42.938;
470	50.81	0.0	1699.5	33.24	4.571	273.4	10.38	0.0	43.167;
480	50.81	0.0	1704.4	33.24	4.565	273.8	10.38	0.0	43.293; local maximum rise or fall;
485	57.56	0.0	1723.7	33.24	4.473	279.5	10.91	0.0	43.782;

Horiz plane projections in effluent direction: radius(m): 0.0; CL(m): 3.3248
Lmz(m): 3.3248
forced entrain 1 0.0 12.94 43.78 0.323
Rate sec-1 0.0 dy-1 0.0 kt: 0.0 Amb Sal 33.3837
;
3:16:57 PM. amb fills: 4

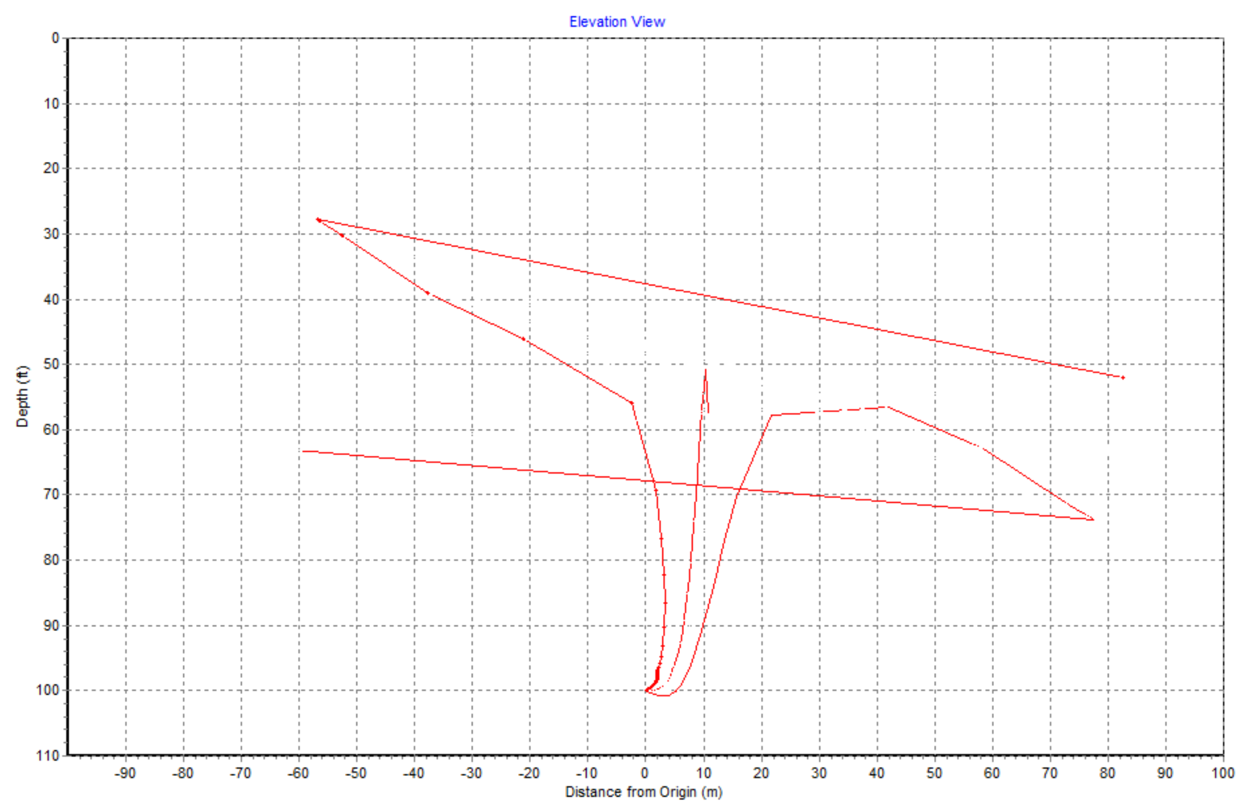

Figure A.2.1: Plumes 18b solution of discharge plume trajectories for discharges of 18.9 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt. ZID is defined by the maximum horizontal excursion of trajectories from the origin. From the maximum horizontal spreading of the plume, the ZID extends from *X* = -60 m to *X* = +83 m so that ZID = 143 m

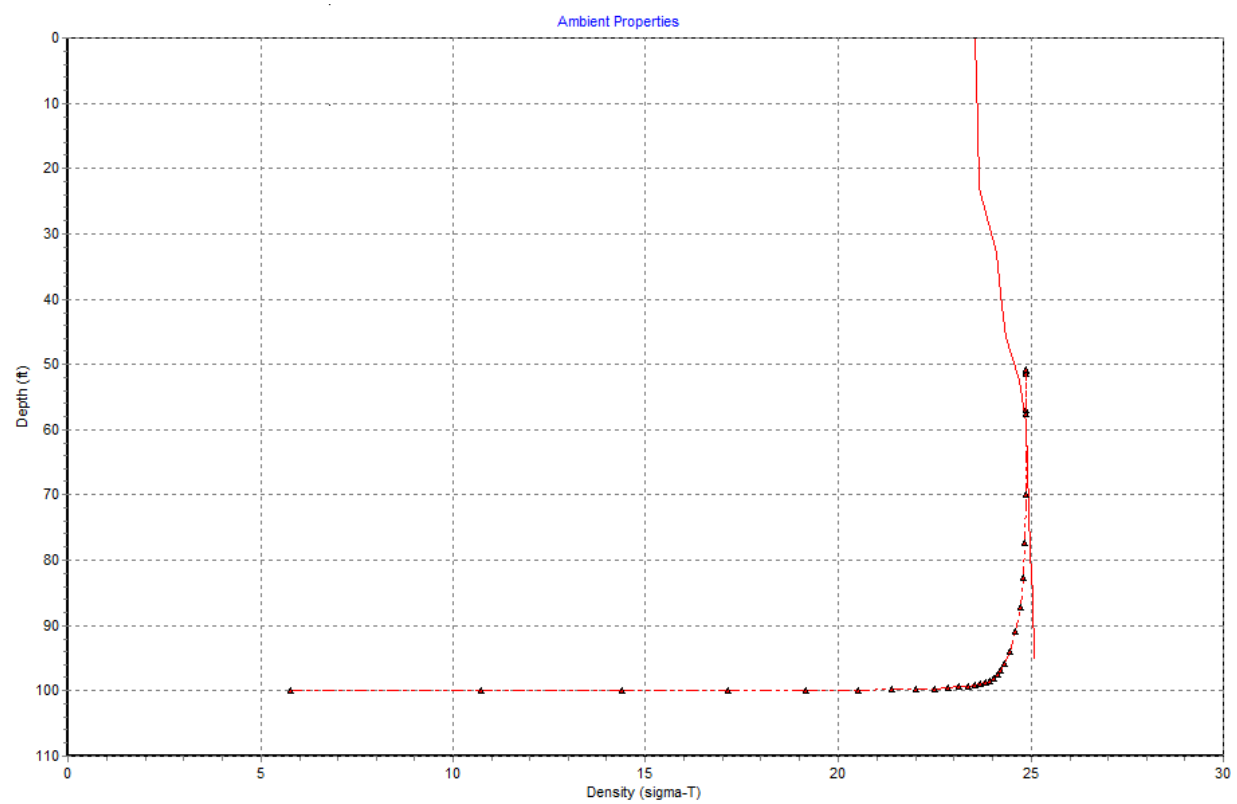

Figure A.2.2: Plumes 18b solution of vertical density profile for discharges of 18.9 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt. Discharge effluent density shown as black triangles. Ambient water mass density profile shown as solid red line.

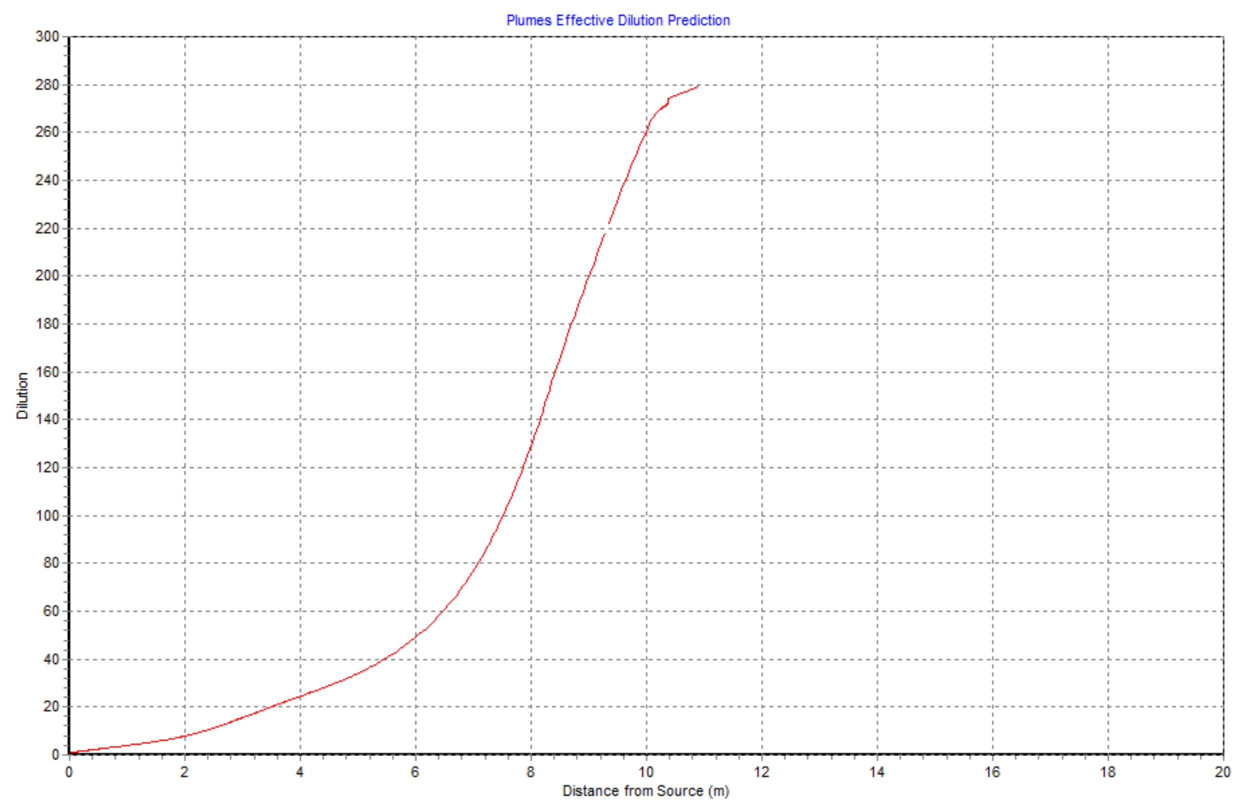

Figure A.2.3: Plumes 18b solution of effective (bulk average) dilution as a function of vertical distance from the point of discharge for discharges of 18.9 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt.

A.3: Plumes 18b Results for SJCOO discharges of 13 mgd Wastewater Only:

SJCOO discharging 13 mgd of wastewater at TDS = 1.25
ppt

Model configuration items checked:
Channel width (m) 100
Start case for graphs 1
Max detailed graphs 10 (limits plots that can overflow memory)
Elevation Projection Plane (deg) 0
Shore vector (m,deg) not checked
Bacteria model : Mancini (1978) coliform model
PDS sfc. model heat transfer : Medium
Equation of State : S, T
Similarity Profile : Default profile (k=2.0, ...)
Diffuser port contraction coefficient 1
Light absorption coefficient 0.16
Farfield increment (m) 200
UM3 aspiration coefficient 0.1
Output file: text output tab
Output each ?? steps 10
Maximum dilution reported 1000
Text output format : Standard
Max vertical reversals : to max rise or fall

/ UM3. 1/10/2019 10:42:10 AM
Case 1; ambient file C:\Plumes18\SJCOO_WW13mgd_b0mgd_T-2.001.db; Diffuser table record 1: ----------------------------------

Ambient Table:

Depth	Amb-cur	Amb-dir	Amb-sal	Amb-tem	Amb-pol	Decay	Far-spd	Far-dir	Disprsn	Density
m	m/s	deg	psu	C	kg/kg	s-1	m/s	deg	m0.67/s2	sigma-T
0.0	0.0	0.0	33.38	19.97	0.0	0.0	0.0	0.0	0.0003	23.55719
2.000	0.0	0.0	33.39	19.87	0.0	0.0	0.0	0.0	0.0003	23.59288
4.000	0.0	0.0	33.40	19.81	0.0	0.0	0.0	0.0	0.0003	23.61484
7.000	0.0	0.0	33.41	19.64	0.0	0.0	0.0	0.0	0.0003	23.67096
10.00	0.0	0.0	33.40	17.76	0.0	0.0	0.0	0.0	0.0003	24.12888
12.00	0.0	0.0	33.38	17.34	0.0	0.0	0.0	0.0	0.0003	24.21549
14.00	0.0	0.0	33.38	16.73	0.0	0.0	0.0	0.0	0.0003	24.35932
16.00	0.0	0.0	33.38	15.17	0.0	0.0	0.0	0.0	0.0003	24.71340
18.00	0.0	0.0	33.37	14.37	0.0	0.0	0.0	0.0	0.0003	24.87044
20.00	0.0	0.0	33.35	14.18	0.0	0.0	0.0	0.0	0.0003	24.89660
22.00	0.0	0.0	33.33	13.83	0.0	0.0	0.0	0.0	0.0003	24.95172
24.00	0.0	0.0	33.32	13.75	0.0	0.0	0.0	0.0	0.0003	24.96707
26.00	0.0	0.0	33.36	13.42	0.0	0.0	0.0	0.0	0.0003	25.06459
29.00	0.0	0.0	33.36	13.39	0.0	0.0	0.0	0.0	0.0003	25.07096

Diffuser table:

P-dia	Ver angl	H-Angle	SourceX	SourceY	Ports	Spacing	MZ-dis	Isoplth	P-depth	Ttl-flo	Eff-sal	Temp	Polutnt
(in)	(deg)	(deg)	(ft)	(ft)	()	(ft)	(ft)	(concent)	(ft)	(MGD)	(psu)	(C)	(ppm)
3.0500	0.0	0.0	0.0	0.0	125.00	12.000	1000.0	0.0	100.00	13.000	1.2500	20.660	1250.0

Simulation:
Froude No: 6.875; Strat No: 1.48E-4; Spcg No: 47.21; k: 96666.1; eff den (sigmaT) -0.921168; eff vel 0.967(m/s);

Current is very small, flow regime may be transient.

Step	Depth (ft)	Amb-cur (m/s)	P-dia (in)	Eff-sal (psu)	Polutnt (ppm)	Dilutn ()	x-posn (m)	y-posn (m)	Iso dia (m)
0	100.0	1.000E-5	3.050	1.250	1250.0	1.000	0.0	0.0	0.07747;
1	100.00	0.0	3.073	1.736	1231.6	1.015	0.00937	0.0	0.07805; bottom hit;
10	100.00	0.0	3.656	6.899	1034.8	1.208	0.053	0.0	0.09287;
20	100.00	0.0	4.448	11.65	852.2	1.467	0.103	0.0	0.1130;
30	100.00	0.0	5.414	15.55	701.3	1.783	0.164	0.0	0.1375;
40	100.00	0.0	6.590	18.75	576.7	2.167	0.239	0.0	0.1674;
50	99.99	0.0	8.022	21.38	474.1	2.636	0.331	0.0	0.2038;
60	99.99	0.0	9.762	23.53	389.6	3.208	0.444	0.0	0.2480;
70	99.97	0.0	11.87	25.30	320.1	3.905	0.582	0.0	0.3015;
80	99.95	0.0	14.22	26.65	266.8	4.686	0.723	0.0	0.3611;
90	99.93	0.0	16.71	27.67	226.1	5.528	0.844	0.0	0.4245;
100	99.90	0.0	19.38	28.49	194.0	6.444	0.949	0.0	0.4922;
110	99.87	0.0	22.22	29.14	168.0	7.439	1.042	0.0	0.5644;
112	99.87	0.0	22.81	29.26	163.5	7.647	1.059	0.0	0.5794; begin overlap;
120	99.84	0.0	25.00	29.64	148.3	8.429	1.126	0.0	0.6350;
130	99.81	0.0	27.40	30.00	134.1	9.324	1.204	0.0	0.6960;
140	99.77	0.0	29.53	30.27	123.0	10.16	1.278	0.0	0.7502;
150	99.74	0.0	31.45	30.50	114.1	10.96	1.350	0.0	0.7989;
160	99.69	0.0	33.20	30.69	106.5	11.73	1.421	0.0	0.8433;
170	99.64	0.0	34.80	30.85	99.99	12.50	1.492	0.0	0.8840;
180	99.59	0.0	36.28	31.00	94.19	13.27	1.563	0.0	0.9215;
190	99.53	0.0	37.65	31.13	88.95	14.05	1.636	0.0	0.9563;
200	99.46	0.0	38.93	31.25	84.13	14.86	1.710	0.0	0.9888;
210	99.38	0.0	40.14	31.37	79.60	15.70	1.788	0.0	1.0195;
220	99.29	0.0	41.29	31.47	75.29	16.60	1.870	0.0	1.0488;
230	99.18	0.0	42.42	31.58	71.10	17.58	1.957	0.0	1.0776;
240	99.06	0.0	43.57	31.68	66.94	18.67	2.049	0.0	1.1068;
248	98.94	0.0	44.54	31.77	63.59	19.66	2.129	0.0	1.1314; end overlap;
250	98.90	0.0	44.77	31.79	62.78	19.91	2.150	0.0	1.1373;
260	98.72	0.0	45.81	31.89	58.89	21.23	2.259	0.0	1.1635;
270	98.48	0.0	46.65	31.98	55.22	22.64	2.382	0.0	1.1848;
280	98.18	0.0	47.38	32.07	51.60	24.22	2.522	0.0	1.2034;
290	97.77	0.0	48.14	32.16	47.84	26.13	2.685	0.0	1.2227;
300	97.22	0.0	49.16	32.27	43.72	28.59	2.878	0.0	1.2487;
310	96.42	0.0	50.83	32.39	38.97	32.08	3.112	0.0	1.2911;
320	95.22	0.0	53.83	32.53	33.34	37.49	3.403	0.0	1.3673;
330	93.53	0.0	58.72	32.68	27.39	45.63	3.730	0.0	1.4915;
340	91.64	0.0	64.84	32.80	22.47	55.62	4.028	0.0	1.6469;
350	89.53	0.0	72.10	32.90	18.44	67.80	4.302	0.0	1.8314;
360	87.17	0.0	80.54	32.98	15.13	82.64	4.558	0.0	2.0457;
370	84.51	0.0	90.23	33.05	12.41	100.7	4.800	0.0	2.2918;
380	81.53	0.0	102.0	33.11	10.18	122.8	5.033	0.0	2.5900;
390	78.12	0.0	117.0	33.15	8.352	149.7	5.265	0.0	2.9726;
400	74.18	0.0	135.7	33.18	6.851	182.4	5.506	0.0	3.4465;
405	71.98	0.0	146.1	33.19	6.205	201.4	5.631	0.0	3.7107; merging;
410	69.06	0.0	162.8	33.20	5.620	222.4	5.789	0.0	4.1340;
416	64.56	0.0	199.7	33.22	4.991	250.5	6.022	0.0	5.0723; trap level;
420	61.00	0.0	241.4	33.23	4.611	271.1	6.209	0.0	6.1323;
430	53.65	0.0	568.5	33.25	4.011	311.7	6.682	0.0	14.439; begin overlap;
440	53.08	0.0	982.9	33.25	3.991	313.2	6.753	0.0	24.965;
450	52.92	0.0	1354.7	33.25	3.982	313.9	6.783	0.0	34.410;
460	52.84	0.0	1716.3	33.25	3.975	314.5	6.801	0.0	43.594;
470	52.82	0.0	1953.0	33.25	3.970	314.8	6.807	0.0	49.607;

480	52.82	0.0	1973.2	33.25	3.967	315.1	6.807	0.0	50.119;
490	52.82	0.0	1977.2	33.25	3.963	315.4	6.807	0.0	50.221; local maximum rise or fall;
497	63.67	0.0	1999.3	33.25	3.883	321.9	7.268	0.0	50.782;

Horiz plane projections in effluent direction: radius(m): 0.0; CL(m): 2.2154
Lmz(m): 2.2154
forced entrain 1 0.0 11.07 50.78 0.283
Rate sec-1 0.0 dy-1 0.0 kt: 0.0 Amb Sal 33.3841
;
10:42:10 AM. amb fills: 4

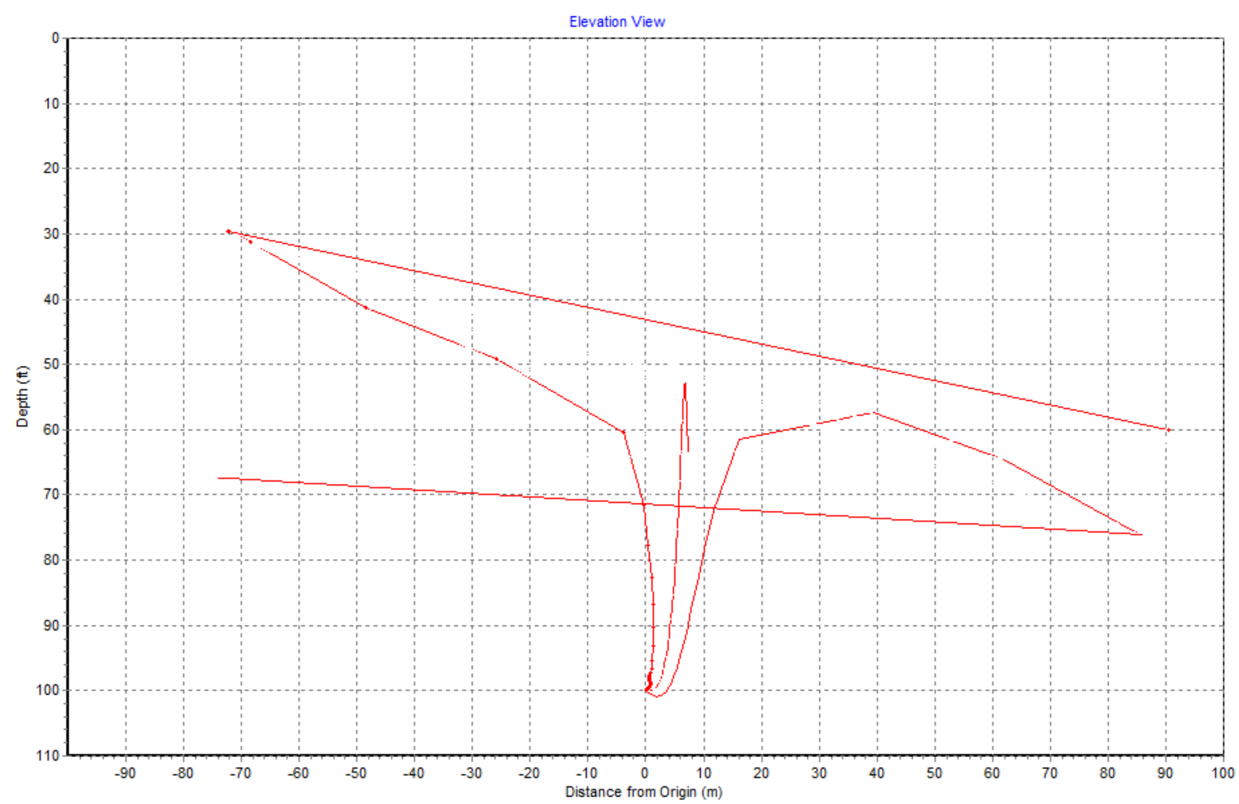

Figure A.3.1: Plumes 18b solution of discharge plume trajectories for discharges of 13 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt. ZID is defined by the maximum horizontal excursion of trajectories from the origin. From the maximum horizontal spreading of the plume, the ZID extends from *X* = -74 m to *X* = +91 m so that ZID = 165 m

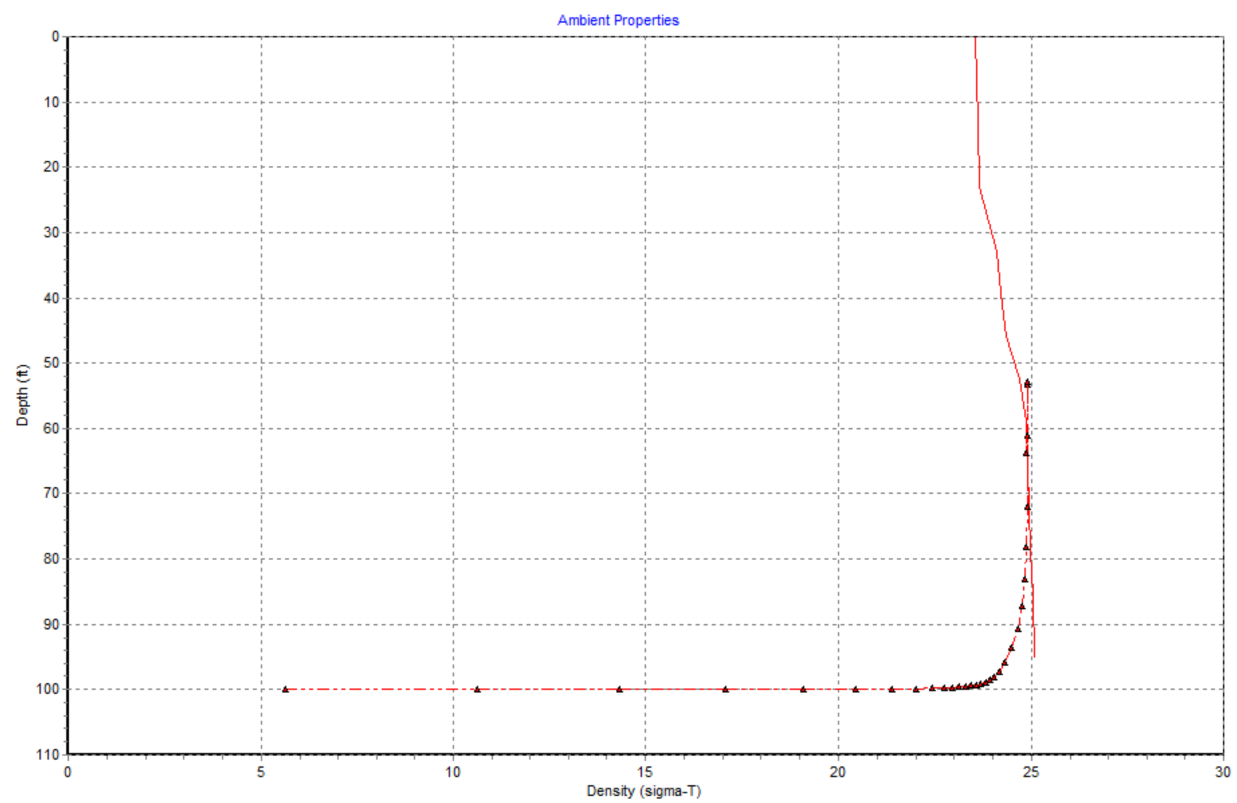

Figure A.3.2: Plumes 18b solution of vertical density profile for discharges of 13 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt. Discharge effluent density shown as black triangles. Ambient water mass density profile shown as solid red line.

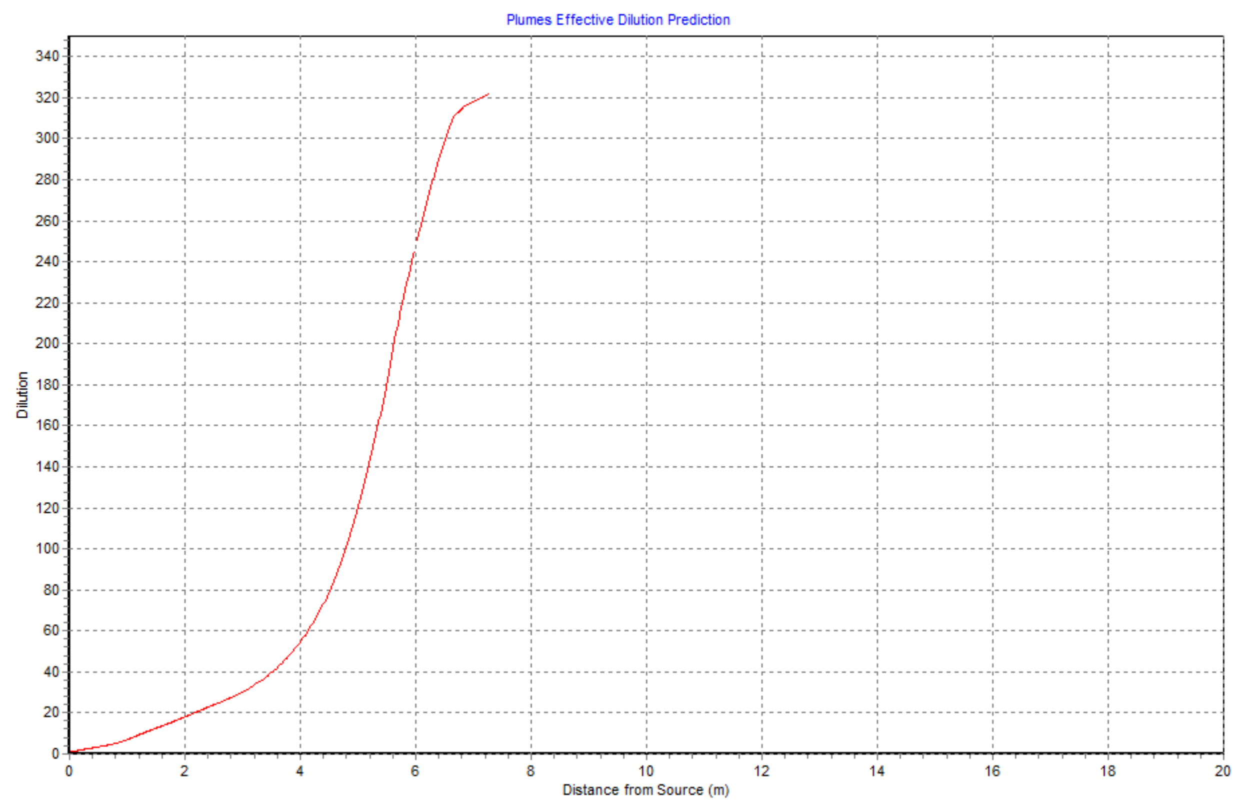

Figure A.3.3: Plumes 18b solution of effective (bulk average) dilution as a function of vertical distance from the point of discharge for discharges of 13 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt.

A.4: Plumes 18b Results for SJCOO discharges of 8 mgd Wastewater Only:

SJCOO discharging 8 mgd of wastewater at TDS = 1.25 ppt

Model configuration items checked:
Channel width (m) 100
Start case for graphs 1
Max detailed graphs 10 (limits plots that can overflow memory)
Elevation Projection Plane (deg) 0
Shore vector (m,deg) not checked
Bacteria model : Mancini (1978) coliform model
PDS sfc. model heat transfer : Medium
Equation of State : S, T
Similarity Profile : Default profile (k=2.0, ...)
Diffuser port contraction coefficient 1
Light absorption coefficient 0.16
Farfield increment (m) 200
UM3 aspiration coefficient 0.1
Output file: text output tab
Output each ?? steps 10
Maximum dilution reported 1000
Text output format : Standard
Max vertical reversals : to max rise or fall

/ UM3. 1/10/2019 11:08:09 AM
Case 1; ambient file C:\Plumes18\SJCOO_WW8mgd_b0mgd_T-1.001.db; Diffuser table record 1: ----------------------------------

Ambient Table:

Depth	Amb-cur	Amb-dir	Amb-sal	Amb-tem	Amb-pol	Decay	Far-spd	Far-dir	Disprsn	Density
m	m/s	deg	psu	C	kg/kg	s-1	m/s	deg	m0.67/s2	sigma-T
0.0	0.0	0.0	33.38	19.97	0.0	0.0	0.0	0.0	0.0003	23.55719
2.000	0.0	0.0	33.39	19.87	0.0	0.0	0.0	0.0	0.0003	23.59288
4.000	0.0	0.0	33.40	19.81	0.0	0.0	0.0	0.0	0.0003	23.61484
7.000	0.0	0.0	33.41	19.64	0.0	0.0	0.0	0.0	0.0003	23.67096
10.00	0.0	0.0	33.40	17.76	0.0	0.0	0.0	0.0	0.0003	24.12888
12.00	0.0	0.0	33.38	17.34	0.0	0.0	0.0	0.0	0.0003	24.21549
14.00	0.0	0.0	33.38	16.73	0.0	0.0	0.0	0.0	0.0003	24.35932
16.00	0.0	0.0	33.38	15.17	0.0	0.0	0.0	0.0	0.0003	24.71340
18.00	0.0	0.0	33.37	14.37	0.0	0.0	0.0	0.0	0.0003	24.87044
20.00	0.0	0.0	33.35	14.18	0.0	0.0	0.0	0.0	0.0003	24.89660
22.00	0.0	0.0	33.33	13.83	0.0	0.0	0.0	0.0	0.0003	24.95172
24.00	0.0	0.0	33.32	13.75	0.0	0.0	0.0	0.0	0.0003	24.96707
26.00	0.0	0.0	33.36	13.42	0.0	0.0	0.0	0.0	0.0003	25.06459
29.00	0.0	0.0	33.36	13.39	0.0	0.0	0.0	0.0	0.0003	25.07096

Diffuser table:

P-dia	Ver angl	H-Angle	SourceX	SourceY	Ports	Spacing	MZ-dis	Isoplth	P-depth	Ttl-flo	Eff-sal	Temp	Polutnt
(in)	(deg)	(deg)	(ft)	(ft)	()	(ft)	(ft)	(concent)	(ft)	(MGD)	(psu)	(C)	(ppm)
3.0500	0.0	0.0	0.0	0.0	125.00	12.000	1000.0	0.0	100.00	8.0000	1.2500	20.660	1250.0

Simulation:
Froude No: 4.231; Strat No: 1.48E-4; Spcg No: 47.21; k: 59486.8; eff den (sigmaT) -0.921168; eff vel 0.595(m/s);

Current is very small, flow regime may be transient.

Step	Depth (ft)	Amb-cur (m/s)	P-dia (in)	Eff-sal (psu)	Polutnt (ppm)	Dilutn ()	x-posn (m)	y-posn (m)	Iso dia (m)
0	100.0	1.000E-5	3.050	1.250	1250.0	1.000	0.0	0.0	0.07747;
1	100.00	0.0	3.064	1.551	1238.6	1.009	0.0058	0.0	0.07783; bottom hit;
10	100.00	0.0	3.635	6.744	1040.8	1.201	0.0201	0.0	0.09233;
20	100.00	0.0	4.423	11.53	857.1	1.458	0.0234	0.0	0.1123;
30	100.00	0.0	5.383	15.45	705.3	1.772	0.0244	0.0	0.1367;
40	100.00	0.0	6.553	18.67	580.1	2.155	0.0248	0.0	0.1665;
50	100.00	0.0	7.980	21.31	476.9	2.621	0.025	0.0	0.2027;
60	100.00	0.0	9.719	23.47	391.9	3.190	0.0251	0.0	0.2469;
70	100.00	0.0	11.84	25.25	321.9	3.883	0.0251	0.0	0.3007;
76	100.00	0.0	13.32	26.15	286.2	4.367	0.0251	0.0	0.3384; begin overlap;
80	100.00	0.0	14.31	26.64	267.0	4.682	0.0251	0.0	0.3634;
90	100.00	0.0	16.50	27.53	232.0	5.388	0.0251	0.0	0.4192;
100	100.00	0.0	18.44	28.13	208.0	6.010	0.0251	0.0	0.4683;
110	100.00	0.0	20.18	28.58	190.2	6.572	0.0251	0.0	0.5126;
120	100.00	0.0	21.79	28.93	176.3	7.090	0.0251	0.0	0.5534;
130	100.00	0.0	23.27	29.21	165.2	7.567	0.0251	0.0	0.5910;
140	100.00	0.0	24.73	29.46	155.5	8.038	0.0251	0.0	0.6282;
150	99.98	0.0	25.95	29.68	146.5	8.530	0.140	0.0	0.6591;
160	99.97	0.0	27.18	29.86	139.3	8.972	0.160	0.0	0.6905;
170	99.97	0.0	28.39	30.03	132.9	9.405	0.176	0.0	0.7210;
180	99.96	0.0	29.54	30.17	127.1	9.832	0.192	0.0	0.7502;
190	99.96	0.0	30.61	30.30	121.9	10.26	0.213	0.0	0.7775;
200	99.95	0.0	31.53	30.42	117.2	10.67	0.238	0.0	0.8009;
210	99.93	0.0	32.37	30.53	112.9	11.07	0.263	0.0	0.8222;
220	99.92	0.0	33.13	30.63	108.9	11.48	0.288	0.0	0.8416;
230	99.91	0.0	33.83	30.72	105.1	11.89	0.314	0.0	0.8593;
240	99.89	0.0	34.46	30.81	101.6	12.31	0.340	0.0	0.8753;
250	99.87	0.0	35.02	30.90	98.17	12.73	0.367	0.0	0.8896;
260	99.85	0.0	35.52	30.98	94.90	13.17	0.396	0.0	0.9022;
270	99.82	0.0	35.95	31.06	91.73	13.63	0.426	0.0	0.9132;
280	99.79	0.0	36.32	31.14	88.62	14.10	0.457	0.0	0.9226;
290	99.75	0.0	36.64	31.22	85.56	14.61	0.491	0.0	0.9306;
300	99.71	0.0	36.89	31.29	82.49	15.15	0.528	0.0	0.9371;
310	99.65	0.0	37.11	31.37	79.40	15.74	0.568	0.0	0.9425;
320	99.59	0.0	37.29	31.45	76.21	16.40	0.612	0.0	0.9471;
330	99.51	0.0	37.46	31.53	72.86	17.16	0.661	0.0	0.9515;
340	99.40	0.0	37.68	31.63	69.24	18.05	0.716	0.0	0.9571;
350	99.27	0.0	38.04	31.73	65.19	19.17	0.779	0.0	0.9661; end overlap;
360	99.09	0.0	38.30	31.83	61.14	20.45	0.853	0.0	0.9729;
370	98.85	0.0	38.38	31.93	57.11	21.89	0.941	0.0	0.9749;
380	98.50	0.0	38.47	32.04	52.73	23.71	1.050	0.0	0.9772;
390	97.95	0.0	38.93	32.17	47.47	26.33	1.189	0.0	0.9889;
400	97.05	0.0	40.47	32.34	40.68	30.73	1.372	0.0	1.0278;
410	95.78	0.0	43.60	32.53	33.38	37.44	1.575	0.0	1.1075;
420	94.38	0.0	47.83	32.68	27.39	45.64	1.753	0.0	1.2150;
430	92.82	0.0	53.00	32.80	22.47	55.63	1.915	0.0	1.3462;
440	91.09	0.0	59.06	32.90	18.44	67.80	2.066	0.0	1.5002;
450	89.14	0.0	66.07	32.98	15.12	82.65	2.207	0.0	1.6781;
460	86.96	0.0	74.08	33.05	12.41	100.7	2.341	0.0	1.8817;
470	84.50	0.0	83.22	33.11	10.18	122.8	2.469	0.0	2.1138;
480	81.73	0.0	94.33	33.15	8.351	149.7	2.592	0.0	2.3960;
490	78.57	0.0	108.7	33.18	6.851	182.5	2.717	0.0	2.7621;
500	74.89	0.0	127.3	33.21	5.620	222.4	2.849	0.0	3.2327;

508	71.50	0.0	144.5	33.23	4.797	260.6	2.963	0.0	3.6706; merging;
510	70.49	0.0	149.8	33.23	4.610	271.1	2.995	0.0	3.8059;
515	67.29	0.0	174.0	33.24	4.176	299.3	3.097	0.0	4.4204; trap level;
520	63.22	0.0	224.9	33.25	3.782	330.5	3.233	0.0	5.7134;
529	56.84	0.0	640.9	33.26	3.348	373.4	3.537	0.0	16.278; begin overlap;
530	56.76	0.0	717.8	33.26	3.346	373.6	3.544	0.0	18.232;
540	56.46	0.0	1308.9	33.26	3.339	374.3	3.578	0.0	33.246;
550	56.38	0.0	1837.7	33.26	3.336	374.7	3.592	0.0	46.677;
560	56.35	0.0	2290.4	33.26	3.333	375.1	3.598	0.0	58.177;
570	56.35	0.0	2354.0	33.26	3.331	375.3	3.599	0.0	59.791;
580	56.35	0.0	2357.4	33.26	3.329	375.5	3.599	0.0	59.877; local maximum rise or fall;
587	76.20	0.0	2382.9	33.27	3.263	383.1	3.988	0.0	60.525;

Horiz plane projections in effluent direction: radius(m): 0.0; CL(m): 1.2157
Lmz(m): 1.2157
forced entrain 1 0.0 7.253 60.52 0.238
Rate sec-1 0.0 dy-1 0.0 kt: 0.0 Amb Sal 33.3739
;
11:08:09 AM. amb fills: 4

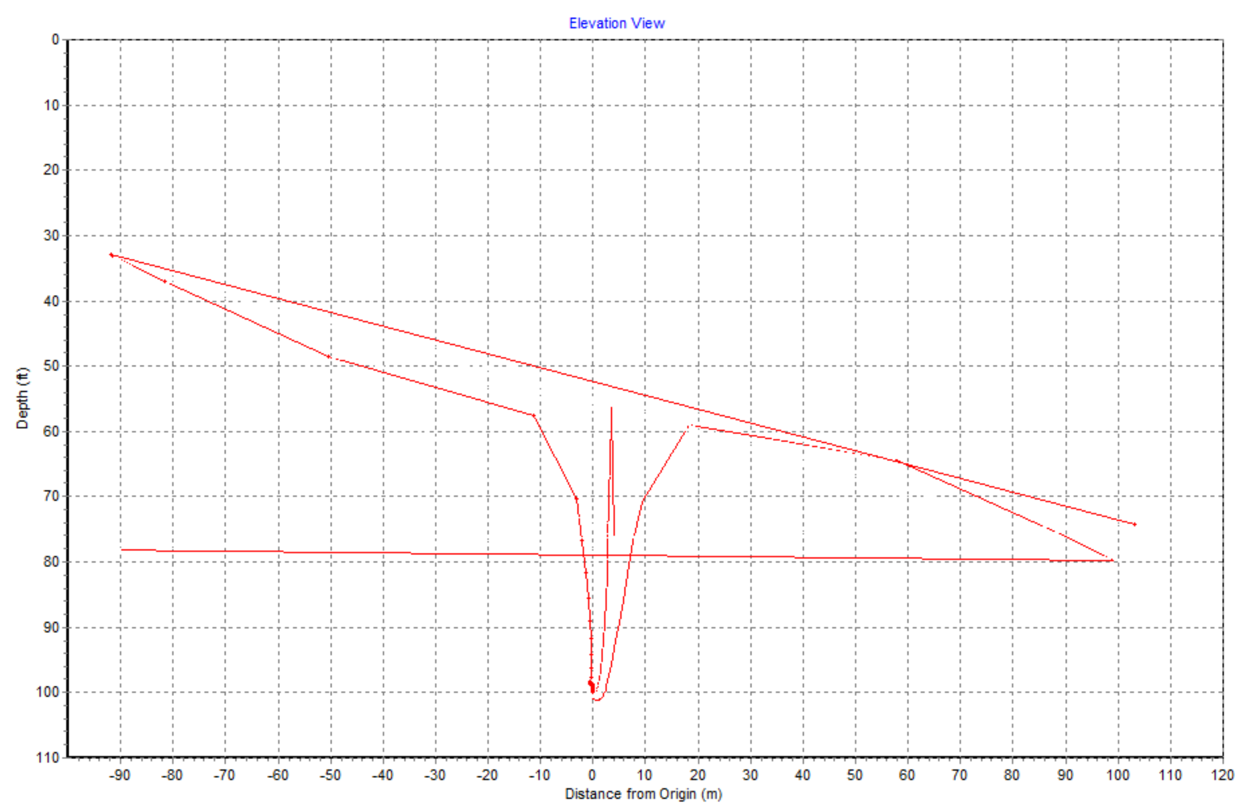

Figure A.4.1: Plumes 18b solution of discharge plume trajectories for discharges of 8 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt. ZID is defined by the maximum horizontal excursion of trajectories from the origin. From the maximum horizontal spreading of the plume, the ZID extends from X = -92 m to X = +104 m so that ZID = 196 m

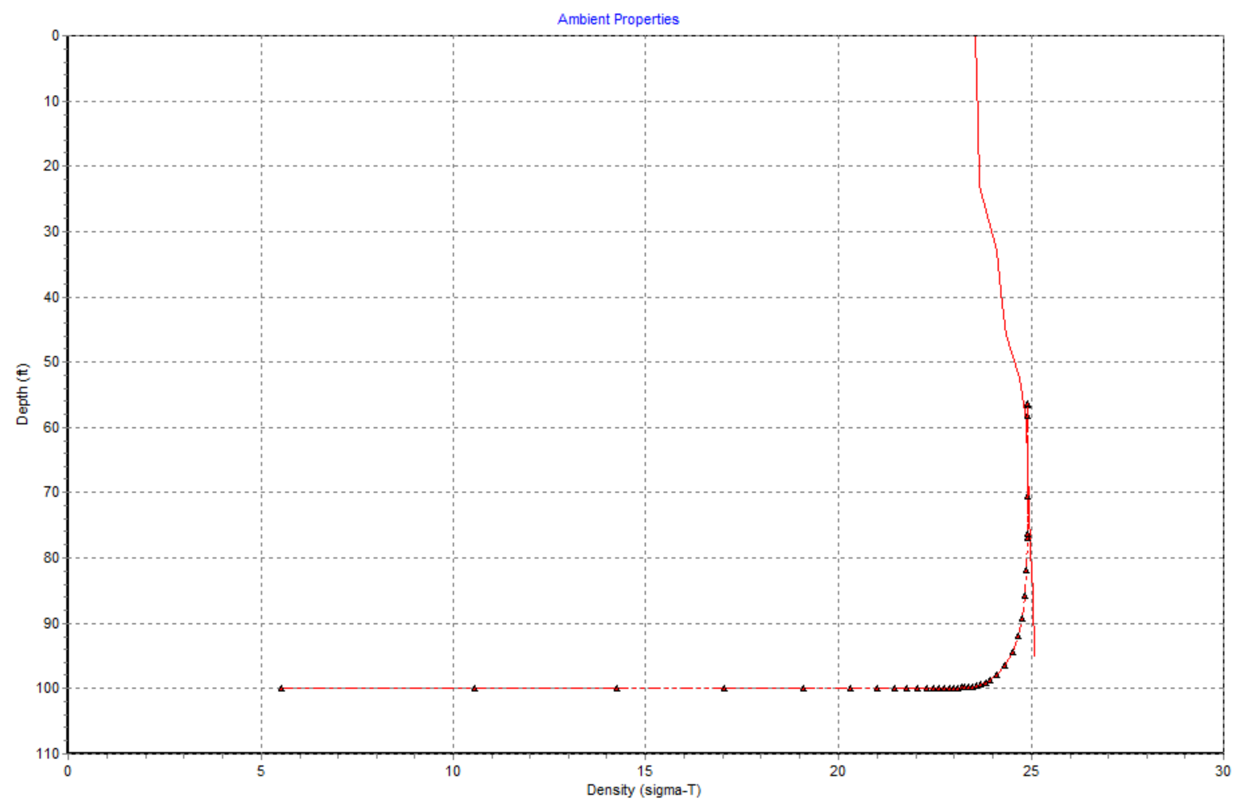

Figure A.4.2: Plumes 18b solution of vertical density profile for discharges of 8 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt. Discharge effluent density shown as black triangles. Ambient water mass density profile shown as solid red line.

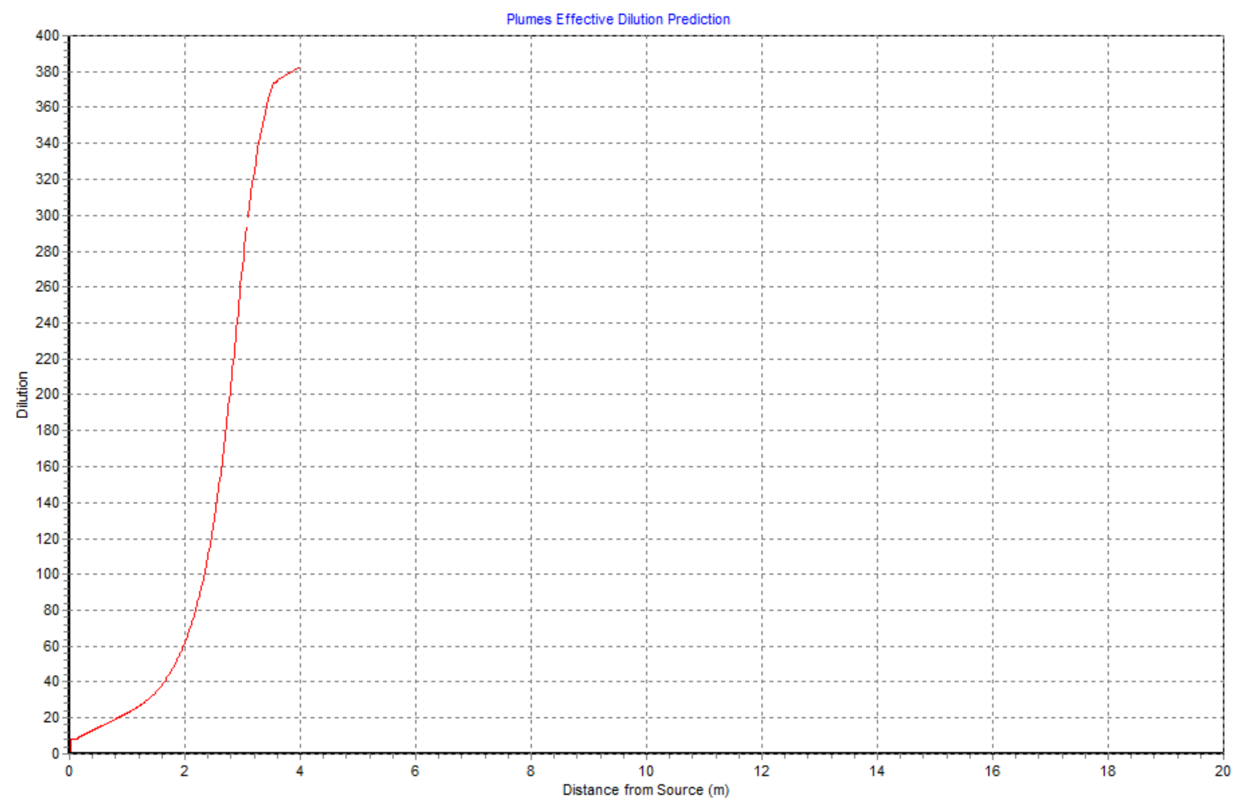

Figure A.4.3: Plumes 18b solution of effective (bulk average) dilution as a function of vertical distance from the point of discharge for discharges of 8 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt.

A.5: Plumes 18b Results for SJCOO discharges of 0.35 mgd Wastewater Only:

SJCOO discharging 0.35 mgd of wastewater at TDS = 1.25
ppt

Model configuration items checked:
Channel width (m) 100
Start case for graphs 1
Max detailed graphs 10 (limits plots that can overflow memory)
Elevation Projection Plane (deg) 0
Shore vector (m,deg) not checked
Bacteria model : Mancini (1978) coliform model
PDS sfc. model heat transfer : Medium
Equation of State : S, T
Similarity Profile : Default profile (k=2.0, ...)
Diffuser port contraction coefficient 1
Light absorption coefficient 0.16
Farfield increment (m) 200
UM3 aspiration coefficient 0.1
Output file: text output tab
Output each ?? steps 10
Maximum dilution reported 1000
Text output format : Standard
Max vertical reversals : to max rise or fall

/ UM3. 1/10/2019 11:27:04 AM
Case 1; ambient file C:\Plumes18\SJCOO_WW0.35mgd_b0mgd_T-2.001.db; Diffuser table record 1: ----------------------------------

Ambient Table:

Depth	Amb-cur	Amb-dir	Amb-sal	Amb-tem	Amb-pol	Decay	Far-spd	Far-dir	Disprsn	Density
m	m/s	deg	psu	C	kg/kg	s-1	m/s	deg	m0.67/s2	sigma-T
0.0	0.0	0.0	33.38	19.97	0.0	0.0	0.0	0.0	0.0003	23.55719
2.000	0.0	0.0	33.39	19.87	0.0	0.0	0.0	0.0	0.0003	23.59288
4.000	0.0	0.0	33.40	19.81	0.0	0.0	0.0	0.0	0.0003	23.61484
7.000	0.0	0.0	33.41	19.64	0.0	0.0	0.0	0.0	0.0003	23.67096
10.00	0.0	0.0	33.40	17.76	0.0	0.0	0.0	0.0	0.0003	24.12888
12.00	0.0	0.0	33.38	17.34	0.0	0.0	0.0	0.0	0.0003	24.21549
14.00	0.0	0.0	33.38	16.73	0.0	0.0	0.0	0.0	0.0003	24.35932
16.00	0.0	0.0	33.38	15.17	0.0	0.0	0.0	0.0	0.0003	24.71340
18.00	0.0	0.0	33.37	14.37	0.0	0.0	0.0	0.0	0.0003	24.87044
20.00	0.0	0.0	33.35	14.18	0.0	0.0	0.0	0.0	0.0003	24.89660
22.00	0.0	0.0	33.33	13.83	0.0	0.0	0.0	0.0	0.0003	24.95172
24.00	0.0	0.0	33.32	13.75	0.0	0.0	0.0	0.0	0.0003	24.96707
26.00	0.0	0.0	33.36	13.42	0.0	0.0	0.0	0.0	0.0003	25.06459
29.00	0.0	0.0	33.36	13.39	0.0	0.0	0.0	0.0	0.0003	25.07096

Diffuser table:

P-dia	Ver angl	H-Angle	SourceX	SourceY	Ports	Spacing	MZ-dis	Isoplth	P-depth	Ttl-flo	Eff-sal	Temp	Polutnt
(in)	(deg)	(deg)	(ft)	(ft)	()	(ft)	(ft)	(concent)	(ft)	(MGD)	(psu)	(C)	(ppm)
3.0500	0.0	0.0	0.0	0.0	125.00	12.000	1000.0	0.0	100.00	0.3500	1.2500	20.660	1250.0

Simulation:

Froude No: 0.185; Strat No: 1.48E-4; Spcg No: 47.21; k: 2602.5; eff den (sigmaT) -0.921168; eff vel 0.026(m/s);
Current is very small, flow regime may be transient.
Absolute value Froude No. < 1, possible intrusion and/or plume diameter reduction

Step	Depth (ft)	Amb-cur (m/s)	P-dia (in)	Eff-sal (psu)	Polutnt (ppm)	Dilutn ()	x-posn (m)	y-posn (m)	Iso dia (m)
0	100.0	1.000E-5	3.050	1.250	1250.0	1.000	0.0	0.0	0.07747;
1	100.00	0.0	3.051	1.263	1249.5	1.000	0.000256	0.0	0.07749; bottom hit;
2	100.00	0.0	3.052	1.285	1248.7	1.001	0.000266	0.0	0.07751; begin overlap;
10	99.98	0.0	2.365	2.302	1210.1	1.033	0.0124	0.0	0.06008;
20	99.97	0.0	2.263	4.671	1119.9	1.116	0.0169	0.0	0.05747;
30	99.96	0.0	2.323	7.904	996.3	1.255	0.0205	0.0	0.0590;
40	99.95	0.0	2.438	11.12	872.5	1.433	0.0238	0.0	0.06193;
50	99.95	0.0	2.872	14.54	740.4	1.688	0.0239	0.0	0.07295;
57	99.94	0.0	2.896	16.44	666.8	1.875	0.0273	0.0	0.07355; end overlap;
60	99.94	0.0	3.058	17.42	628.8	1.988	0.0274	0.0	0.07766;
62	99.94	0.0	3.177	18.03	604.8	2.067	0.0275	0.0	0.0807; begin overlap;
70	99.94	0.0	3.637	19.96	529.8	2.360	0.0275	0.0	0.09237;
80	99.94	0.0	4.143	21.58	466.2	2.681	0.0275	0.0	0.1052;
85	99.92	0.0	3.876	22.41	433.7	2.882	0.0316	0.0	0.09844; end overlap;
90	99.91	0.0	4.114	23.44	393.2	3.179	0.0329	0.0	0.1045;
100	99.89	0.0	4.330	25.09	328.0	3.810	0.0374	0.0	0.1100;
110	99.81	0.0	4.227	26.44	274.8	4.549	0.0452	0.0	0.1074;
120	99.70	0.0	4.281	27.69	225.6	5.540	0.0534	0.0	0.1087;
130	99.57	0.0	4.556	28.71	185.2	6.748	0.0603	0.0	0.1157;
140	99.43	0.0	4.961	29.54	152.1	8.220	0.0663	0.0	0.1260;
150	99.27	0.0	5.469	30.23	124.8	10.01	0.0717	0.0	0.1389;
160	99.09	0.0	6.074	30.79	102.4	12.20	0.0767	0.0	0.1543;
170	98.89	0.0	6.777	31.25	84.06	14.87	0.0814	0.0	0.1721;
180	98.67	0.0	7.584	31.63	68.98	18.12	0.0858	0.0	0.1926;
190	98.42	0.0	8.503	31.94	56.60	22.08	0.090	0.0	0.2160;
200	98.14	0.0	9.546	32.20	46.44	26.91	0.094	0.0	0.2425;
210	97.82	0.0	10.73	32.41	38.11	32.80	0.0978	0.0	0.2725;
220	97.47	0.0	12.06	32.58	31.27	39.98	0.102	0.0	0.3064;
230	97.07	0.0	13.57	32.72	25.65	48.73	0.105	0.0	0.3447;
240	96.62	0.0	15.27	32.83	21.04	59.40	0.108	0.0	0.3879;
250	96.11	0.0	17.19	32.93	17.27	72.40	0.112	0.0	0.4367;
260	95.54	0.0	19.35	33.01	14.16	88.25	0.115	0.0	0.4916;
270	94.89	0.0	21.79	33.07	11.62	107.6	0.118	0.0	0.5536;
280	94.16	0.0	24.54	33.12	9.533	131.1	0.121	0.0	0.6234;
290	93.35	0.0	27.65	33.17	7.821	159.8	0.123	0.0	0.7022;
300	92.43	0.0	31.14	33.20	6.416	194.8	0.126	0.0	0.7911;
310	91.39	0.0	35.09	33.23	5.263	237.5	0.129	0.0	0.8913;
320	90.22	0.0	39.55	33.25	4.318	289.5	0.131	0.0	1.0046;
330	88.90	0.0	44.59	33.27	3.542	352.9	0.133	0.0	1.1326;
340	87.41	0.0	50.29	33.29	2.906	430.2	0.136	0.0	1.2775;
350	85.74	0.0	56.76	33.30	2.384	524.4	0.138	0.0	1.4418;
360	83.84	0.0	64.53	33.31	1.956	639.2	0.140	0.0	1.6390;
367	82.35	0.0	72.65	33.32	1.703	734.2	0.142	0.0	1.8453; trap level;
370	81.65	0.0	77.39	33.32	1.604	779.2	0.142	0.0	1.9658;
380	78.93	0.0	114.5	33.32	1.316	949.8	0.146	0.0	2.9083;
382	78.49	0.0	152.0	33.32	1.265	988.2	0.148	0.0	3.8607; merging;
383	78.61	0.0	293.5	33.32	1.263	990.0	0.149	0.0	7.4546; local maximum rise or fall;

Horiz plane projections in effluent direction: radius(m): 0.0; CL(m): 0.0454
Lmz(m): 0.0454
forced entrain 1 0.0 6.520 7.455 0.00893

Rate sec-1 0.0 dy-1 0.0 kt: 0.0 Amb Sal 33.3233
;
11:27:04 AM. amb fills: 4

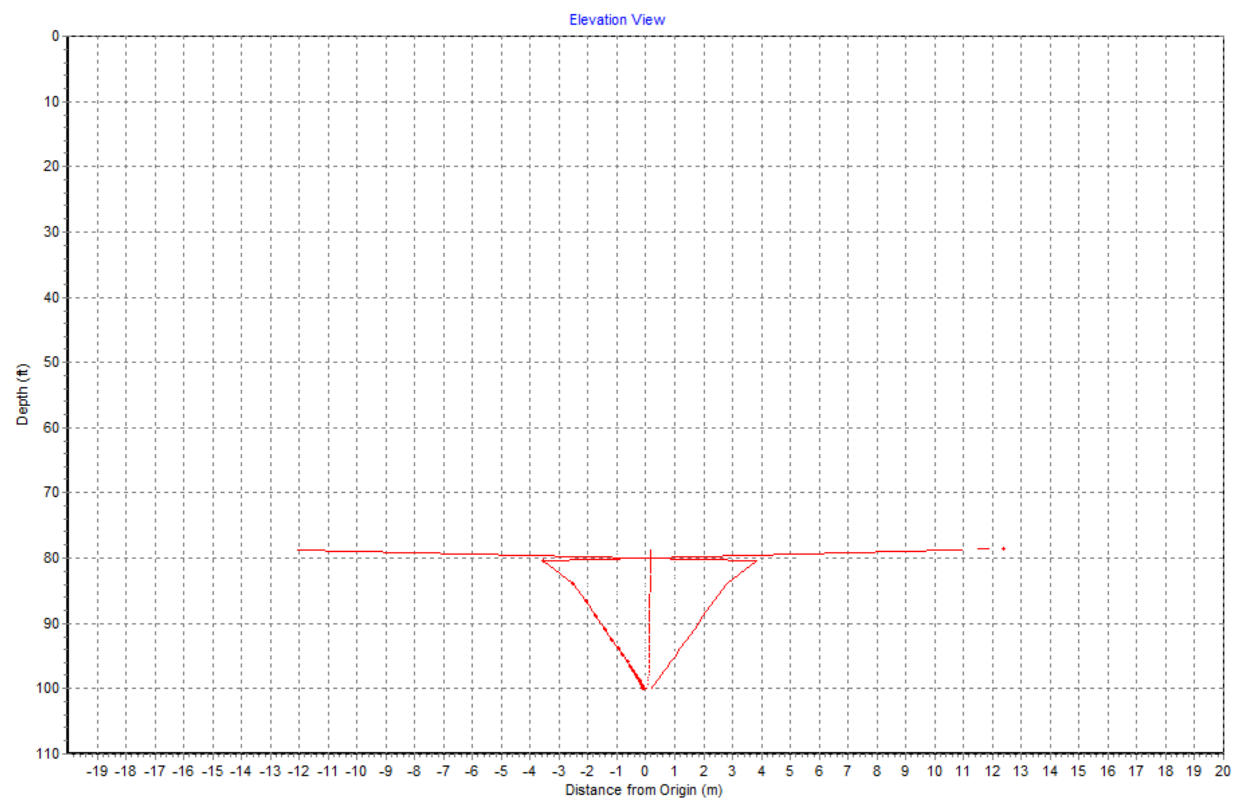

Figure A.5.1: Plumes 18b solution of discharge plume trajectories for discharges of 0.35 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt. ZID is defined by the maximum horizontal excursion of trajectories from the origin. From the maximum horizontal spreading of the plume, the ZID extends from X = -12 m to X = +12.5 m so that ZID = 24.5 m

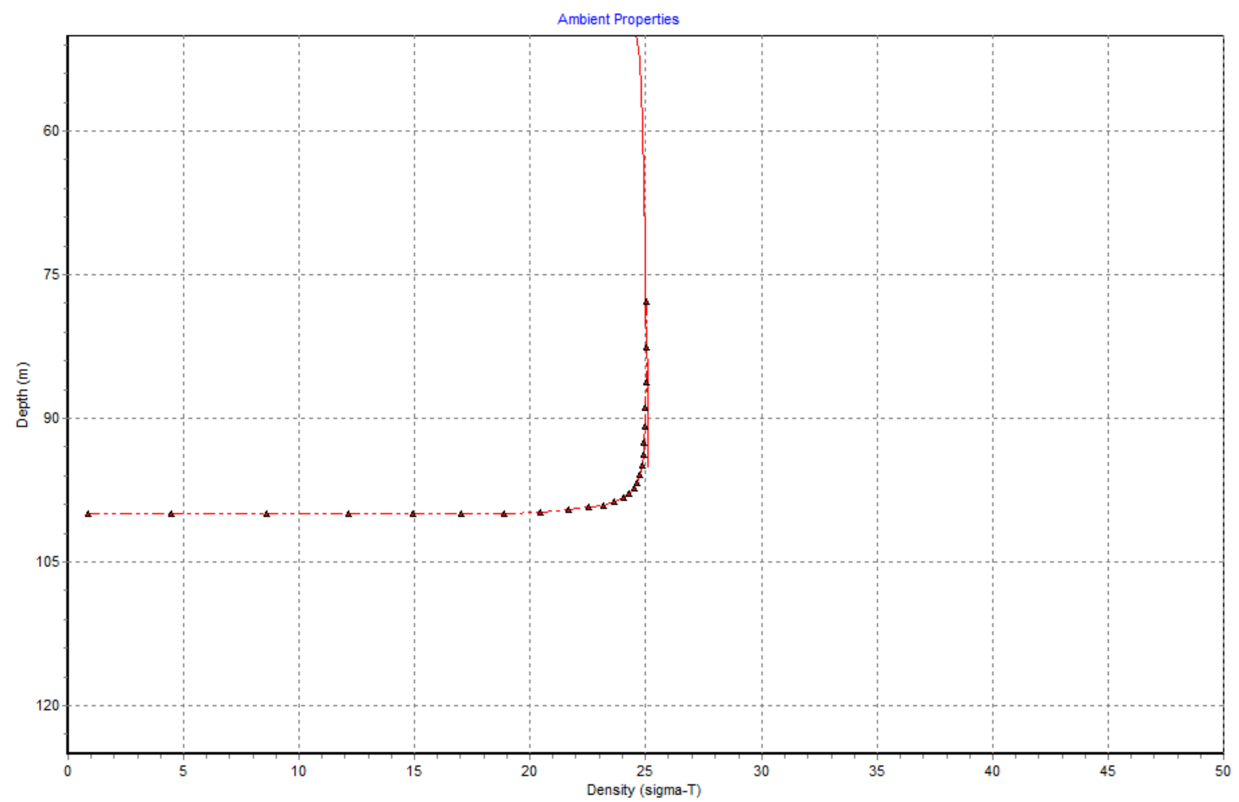

Figure A.5.2: Plumes 18b solution of vertical density profile for discharges of 0.35 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt. Discharge effluent density shown as black triangles. Ambient water mass density profile shown as solid red line.

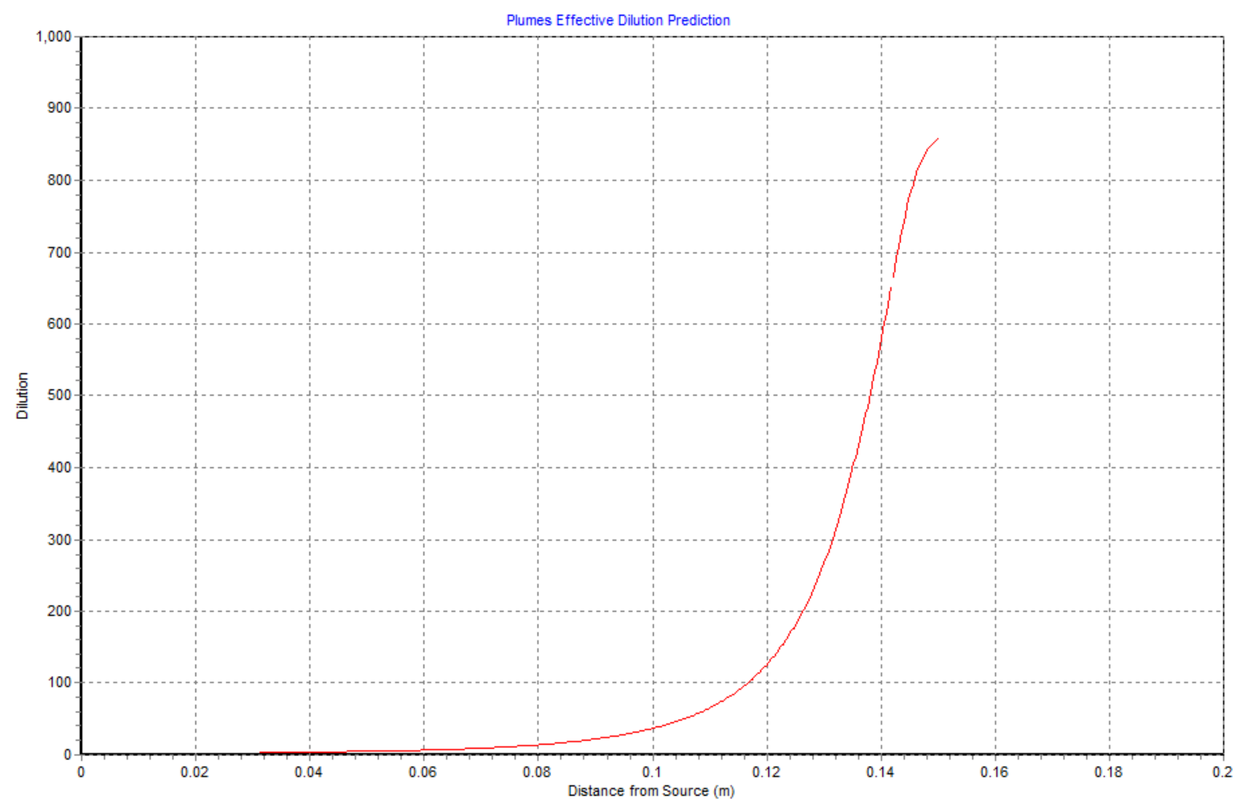

Figure A.5.3: Plumes 18b solution of effective (bulk average) dilution as a function of vertical distance from the point of discharge for discharges of 0.35 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt.

APPENDIX-B: Results for Buoyant Combined Discharges of Brine and Wastewater

B.1: Plumes 18b Results for SJCOO discharges of 31 mgd Wastewater and 5 mgd Brine:

Project "C:\Plumes18\SJCOO_WW31mgd_b5mgd_T-3"
memo
SJCOO discharging 31 mgd wastewater and 5 mgd brine

Model configuration items checked:
Channel width (m) 100
Start case for graphs 1
Max detailed graphs 10 (limits plots that can overflow memory)
Elevation Projection Plane (deg) 0
Shore vector (m,deg) not checked
Bacteria model : Mancini (1978) coliform model
PDS sfc. model heat transfer : Medium
Equation of State : S, T
Similarity Profile : Default profile (k=2.0, ...)
Diffuser port contraction coefficient 1
Light absorption coefficient 0.16
Farfield increment (m) 200
UM3 aspiration coefficient 0.1
Output file: text output tab
Output each ?? steps 10
Maximum dilution reported 1000
Text output format : Standard
Max vertical reversals : to max rise or fall

/ UM3. 1/10/2019 11:52:37 AM
Case 1; ambient file C:\Plumes18\SJCOO_WW31mgd_b5mgd_T-3.001.db; Diffuser table record 1: ----------------------------------

Ambient Table:

Depth	Amb-cur	Amb-dir	Amb-sal	Amb-tem	Amb-pol	Decay	Far-spd	Far-dir	Disprsn	Density
m	m/s	deg	psu	C	kg/kg	s-1	m/s	deg	m0.67/s2	sigma-T
0.0	0.0	0.0	33.38	19.97	0.0	0.0	0.0	0.0	0.0003	23.55719
2.000	0.0	0.0	33.39	19.87	0.0	0.0	0.0	0.0	0.0003	23.59288
4.000	0.0	0.0	33.40	19.81	0.0	0.0	0.0	0.0	0.0003	23.61484
7.000	0.0	0.0	33.41	19.64	0.0	0.0	0.0	0.0	0.0003	23.67096
10.00	0.0	0.0	33.40	17.76	0.0	0.0	0.0	0.0	0.0003	24.12888
12.00	0.0	0.0	33.38	17.34	0.0	0.0	0.0	0.0	0.0003	24.21549
14.00	0.0	0.0	33.38	16.73	0.0	0.0	0.0	0.0	0.0003	24.35932
16.00	0.0	0.0	33.38	15.17	0.0	0.0	0.0	0.0	0.0003	24.71340
18.00	0.0	0.0	33.37	14.37	0.0	0.0	0.0	0.0	0.0003	24.87044
20.00	0.0	0.0	33.35	14.18	0.0	0.0	0.0	0.0	0.0003	24.89660
22.00	0.0	0.0	33.33	13.83	0.0	0.0	0.0	0.0	0.0003	24.95172
24.00	0.0	0.0	33.32	13.75	0.0	0.0	0.0	0.0	0.0003	24.96707
26.00	0.0	0.0	33.36	13.42	0.0	0.0	0.0	0.0	0.0003	25.06459
29.00	0.0	0.0	33.36	13.39	0.0	0.0	0.0	0.0	0.0003	25.07096

Diffuser table:

P-dia	Ver angl	H-Angle	SourceX	SourceY	Ports	Spacing	MZ-dis	Isoplth	P-depth	Ttl-flo	Eff-sal	Temp	Polutnt
(in)	(deg)	(deg)	(ft)	(ft)	()	(ft)	(ft)	(concent)	(ft)	(MGD)	(psu)	(C)	(ppm)
3.0500	0.0	0.0	0.0	0.0	125.00	12.000	1000.0	0.0	100.00	36.000	9.3000	20.660	9300.0

Simulation:
Froude No: 21.83; Strat No: 1.94E-4; Spcg No: 47.21; k: 2.68E+5; eff den (sigmaT) 5.178451; eff vel 2.677(m/s);
Current is very small, flow regime may be transient.

Step	Depth (ft)	Amb-cur (m/s)	P-dia (in)	Eff-sal (psu)	Polutnt (ppm)	Dilutn ()	x-posn (m)	y-posn (m)	Iso dia (m)
0	100.0	1.000E-5	3.050	9.300	9300.0	1.000	0.0	0.0	0.07747;
1	100.00	0.0	3.113	10.28	8929.9	1.041	0.0253	0.0	0.07906; bottom hit;
10	100.00	0.0	3.754	14.04	7495.2	1.241	0.129	0.0	0.09536;
20	100.00	0.0	4.570	17.52	6166.1	1.508	0.265	0.0	0.1161;
30	100.00	0.0	5.565	20.36	5070.1	1.834	0.432	0.0	0.1413;
40	100.00	0.0	6.777	22.70	4167.2	2.232	0.635	0.0	0.1721;
50	99.99	0.0	8.254	24.61	3423.8	2.716	0.883	0.0	0.2097;
60	99.99	0.0	10.05	26.18	2812.3	3.307	1.185	0.0	0.2554;
70	99.98	0.0	12.25	27.47	2309.5	4.027	1.552	0.0	0.3111;
80	99.96	0.0	14.92	28.53	1896.2	4.904	1.998	0.0	0.3790;
90	99.93	0.0	18.17	29.40	1556.7	5.974	2.538	0.0	0.4615;
100	99.87	0.0	22.11	30.11	1277.7	7.278	3.189	0.0	0.5615;
110	99.77	0.0	26.64	30.67	1058.7	8.784	3.923	0.0	0.6766;
120	99.64	0.0	30.91	31.05	910.0	10.22	4.573	0.0	0.7851;
130	99.51	0.0	34.93	31.32	801.4	11.60	5.151	0.0	0.8873;
140	99.35	0.0	38.79	31.54	717.2	12.97	5.679	0.0	0.9852;
150	99.18	0.0	42.51	31.71	649.2	14.33	6.169	0.0	1.0799;
160	98.99	0.0	46.14	31.86	592.5	15.70	6.631	0.0	1.1720;
170	98.79	0.0	49.69	31.98	544.2	17.09	7.073	0.0	1.2620;
180	98.56	0.0	53.15	32.09	502.4	18.51	7.500	0.0	1.3500;
190	98.32	0.0	56.54	32.18	465.6	19.97	7.917	0.0	1.4361;
200	98.05	0.0	59.86	32.26	432.8	21.49	8.329	0.0	1.5204;
210	97.74	0.0	63.11	32.34	403.2	23.06	8.740	0.0	1.6031;
220	97.41	0.0	66.31	32.41	376.3	24.72	9.153	0.0	1.6843;
230	97.03	0.0	69.47	32.47	351.4	26.47	9.572	0.0	1.7645;
240	96.60	0.0	72.61	32.53	328.1	28.34	10.00	0.0	1.8442;
250	96.11	0.0	75.75	32.58	306.2	30.37	10.45	0.0	1.9242;
260	95.55	0.0	78.96	32.64	285.2	32.61	10.91	0.0	2.0056;
270	94.90	0.0	82.29	32.69	264.8	35.12	11.40	0.0	2.0903;
280	94.12	0.0	85.85	32.74	244.8	38.00	11.92	0.0	2.1805;
290	93.19	0.0	89.75	32.79	224.8	41.37	12.48	0.0	2.2797;
300	92.06	0.0	94.20	32.84	204.7	45.44	13.08	0.0	2.3926;
310	90.66	0.0	99.46	32.89	184.2	50.50	13.75	0.0	2.5262;
320	88.88	0.0	105.9	32.95	163.1	57.03	14.49	0.0	2.6908;
330	86.57	0.0	114.3	33.00	141.3	65.80	15.32	0.0	2.9024;
340	83.44	0.0	125.6	33.06	118.8	78.29	16.27	0.0	3.1913;
350	79.39	0.0	142.1	33.11	97.46	95.43	17.32	0.0	3.6105;
351	78.96	0.0	144.1	33.12	95.55	97.34	17.42	0.0	3.6596; merging;
360	73.60	0.0	174.1	33.15	79.95	116.3	18.61	0.0	4.4226;
370	65.40	0.0	232.9	33.18	65.59	141.8	20.21	0.0	5.9165;
375	60.57	0.0	281.7	33.20	59.40	156.6	21.10	0.0	7.1559; trap level;
380	55.25	0.0	353.2	33.21	53.80	172.8	22.10	0.0	8.9724;
387	52.25	0.0	505.5	33.22	51.00	182.4	22.75	0.0	12.839; begin overlap;
390	51.81	0.0	554.7	33.22	50.63	183.7	22.86	0.0	14.090;
400	51.01	0.0	696.1	33.23	50.08	185.7	23.09	0.0	17.680;
410	50.61	0.0	827.2	33.23	49.76	186.9	23.23	0.0	21.012;
420	50.38	0.0	955.2	33.23	49.50	187.9	23.32	0.0	24.262;
430	50.23	0.0	1080.8	33.23	49.28	188.7	23.39	0.0	27.451;
440	50.12	0.0	1203.9	33.23	49.09	189.5	23.45	0.0	30.578;

450	50.06	0.0	1313.3	33.23	48.92	190.1	23.48	0.0	33.358;
460	50.05	0.0	1349.8	33.23	48.78	190.6	23.49	0.0	34.285;
470	50.05	0.0	1359.6	33.23	48.65	191.1	23.49	0.0	34.534;
480	50.05	0.0	1367.0	33.23	48.53	191.7	23.49	0.0	34.722;
490	50.05	0.0	1374.2	33.23	48.40	192.2	23.49	0.0	34.905;
500	50.05	0.0	1382.3	33.23	48.26	192.7	23.49	0.0	35.111; local maximum rise or fall;
502	52.48	0.0	1396.7	33.23	47.32	196.6	24.42	0.0	35.475;

Horiz plane projections in effluent direction: radius(m): 0.0; CL(m): 7.4435
Lmz(m): 7.4435
forced entrain 1 0.0 14.49 35.48 0.533
Rate sec-1 0.0 dy-1 0.0 kt: 0.0 Amb Sal 33.3831
;
11:52:38 AM. amb fills: 4

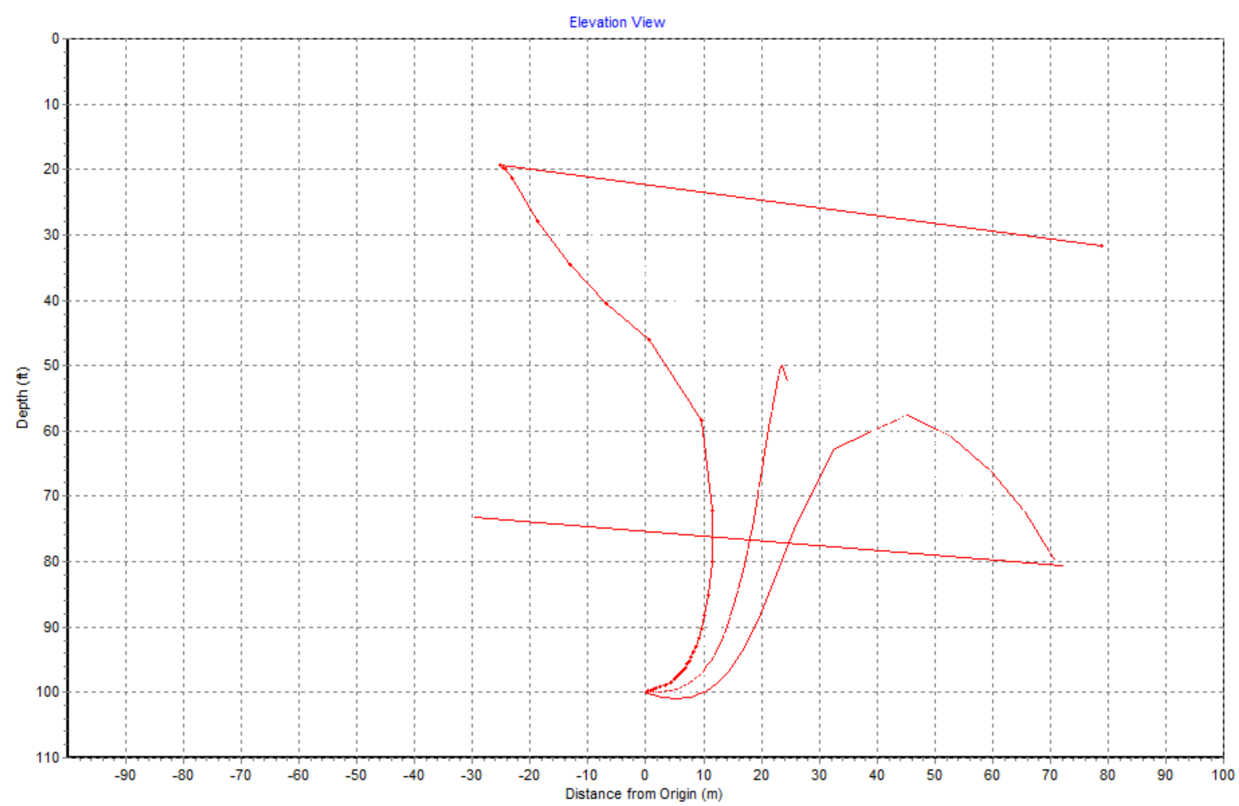

Figure B.1.1: Plumes 18b solution of discharge plume trajectories for discharges of 31 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt., and 5 mgd of brine from the Doheny Desalination Project with a brine salinity of 67 ppt. ZID is defined by the maximum horizontal excursion of trajectories from the origin. From the maximum horizontal spreading of the plume, the ZID extends from *X* = -30 m to *X* = +80 m so that ZID = 110 m

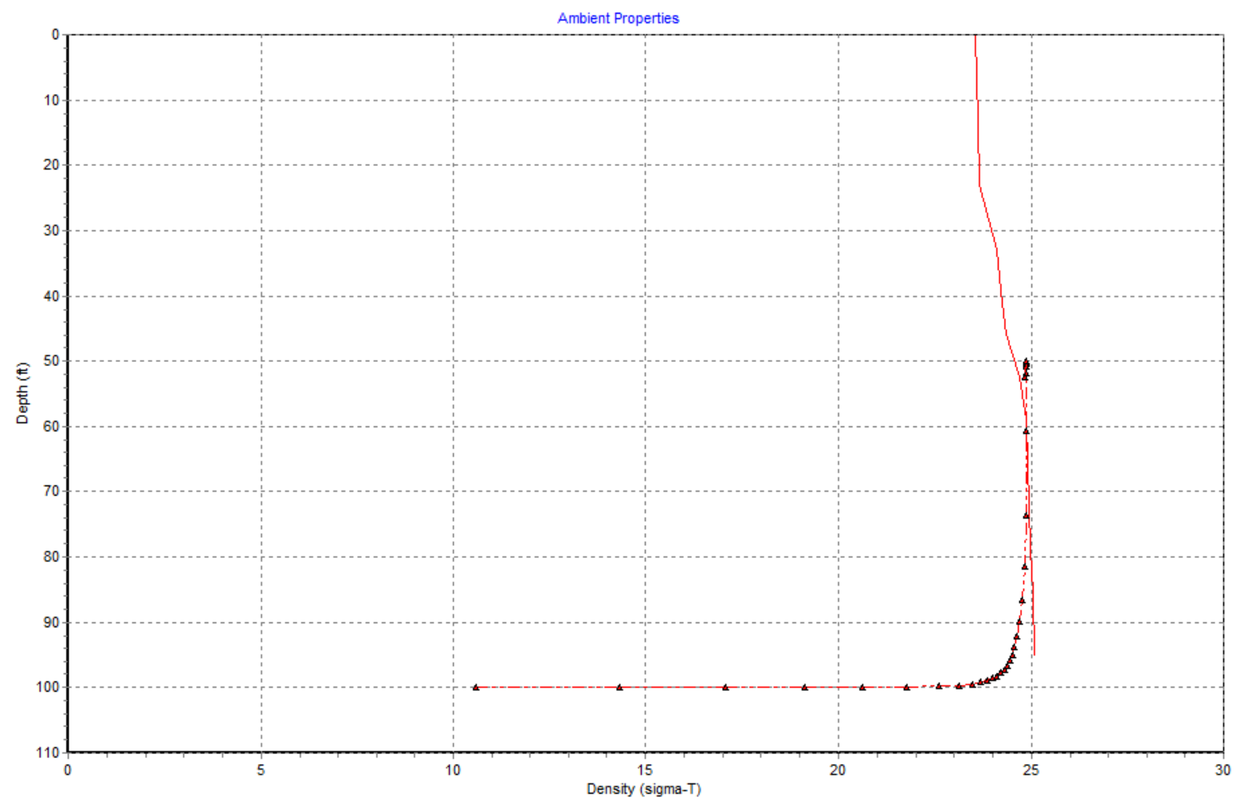

Figure B.1.2: Plumes 18b solution of vertical density profile for discharges of 31 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt. and 5 mgd of brine from the Doheny Desalination Project with a brine salinity of 67 ppt. Discharge effluent density shown as black triangles. Ambient water mass density profile shown as solid red line.

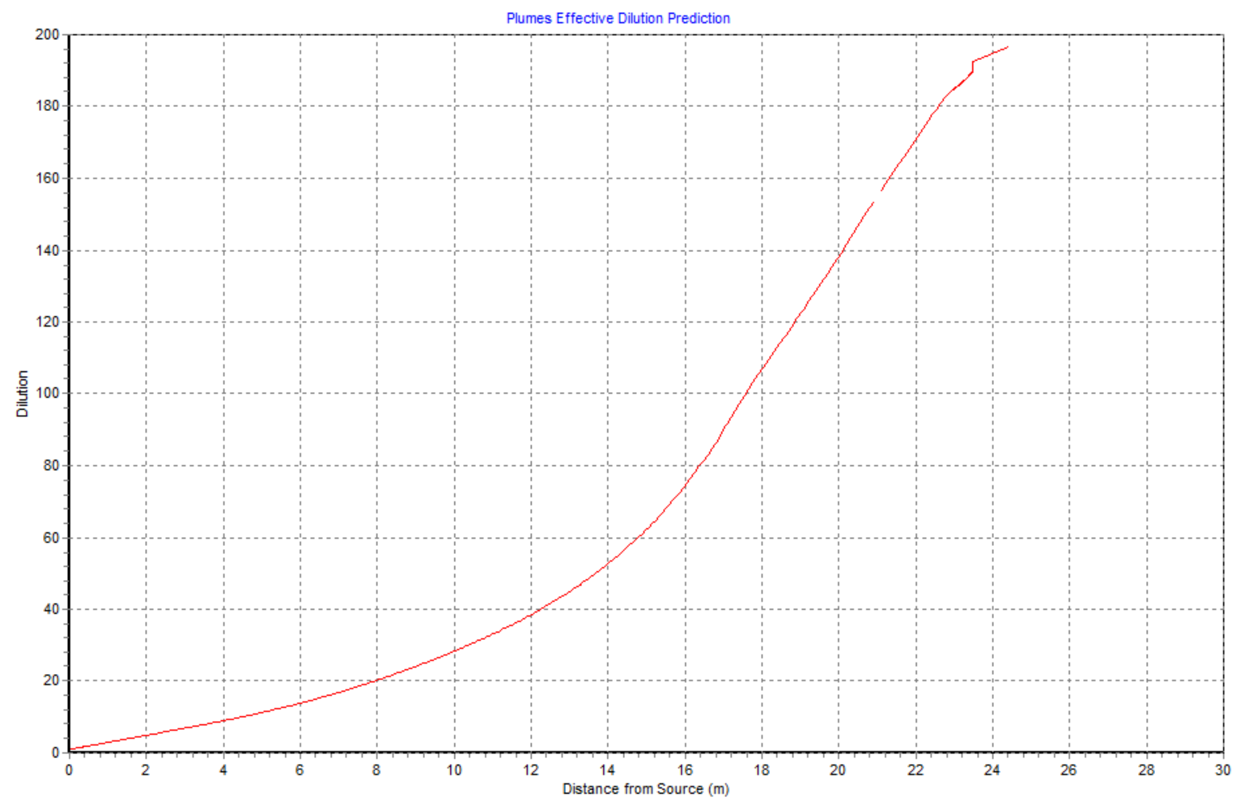

Figure B.1.3: Plumes 18b solution of effective (bulk average) dilution as a function of vertical distance from the point of discharge for discharges of 31 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt. and 5 mgd of brine from the Doheny Desalination Project with a brine salinity of 67 ppt.

B.2: Plumes 18b Results for SJCOO discharges of 31 mgd Wastewater and 15 mgd Brine:

Contents of the memo box (may not be current and must be updated manually)
Project "C:\Plumes18\SJCOO_WW31mgd_b15mgd_T-1"
memo
SJCOO discharging 31 mgd wastewater and 15 mgd brine

Model configuration items checked:
Channel width (m) 100
Start case for graphs 1
Max detailed graphs 10 (limits plots that can overflow memory)
Elevation Projection Plane (deg) 0
Shore vector (m,deg) not checked
Bacteria model : Mancini (1978) coliform model
PDS sfc. model heat transfer : Medium
Equation of State : S, T
Similarity Profile : Default profile (k=2.0, ...)
Diffuser port contraction coefficient 1
Light absorption coefficient 0.16
Farfield increment (m) 200
UM3 aspiration coefficient 0.1
Output file: text output tab
Output each ?? steps 10
Maximum dilution reported 1000
Text output format : Standard
Max vertical reversals : to max rise or fall

/ UM3. 1/10/2019 12:20:30 PM
Case 1; ambient file C:\Plumes18\SJCOO_WW31mgd_b15mgd_T-1.001.db; Diffuser table record 1: ----------------------------------

Ambient Table:

Depth	Amb-cur	Amb-dir	Amb-sal	Amb-tem	Amb-pol	Decay	Far-spd	Far-dir	Disprsn	Density
m	m/s	deg	psu	C	kg/kg	s-1	m/s	deg	m0.67/s2	sigma-T
0.0	0.0	0.0	33.38	19.97	0.0	0.0	0.0	0.0	0.0003	23.55719
2.000	0.0	0.0	33.39	19.87	0.0	0.0	0.0	0.0	0.0003	23.59288
4.000	0.0	0.0	33.40	19.81	0.0	0.0	0.0	0.0	0.0003	23.61484
7.000	0.0	0.0	33.41	19.64	0.0	0.0	0.0	0.0	0.0003	23.67096
10.00	0.0	0.0	33.40	17.76	0.0	0.0	0.0	0.0	0.0003	24.12888
12.00	0.0	0.0	33.38	17.34	0.0	0.0	0.0	0.0	0.0003	24.21549
14.00	0.0	0.0	33.38	16.73	0.0	0.0	0.0	0.0	0.0003	24.35932
16.00	0.0	0.0	33.38	15.17	0.0	0.0	0.0	0.0	0.0003	24.71340
18.00	0.0	0.0	33.37	14.37	0.0	0.0	0.0	0.0	0.0003	24.87044
20.00	0.0	0.0	33.35	14.18	0.0	0.0	0.0	0.0	0.0003	24.89660
22.00	0.0	0.0	33.33	13.83	0.0	0.0	0.0	0.0	0.0003	24.95172
24.00	0.0	0.0	33.32	13.75	0.0	0.0	0.0	0.0	0.0003	24.96707
26.00	0.0	0.0	33.36	13.42	0.0	0.0	0.0	0.0	0.0003	25.06459
29.00	0.0	0.0	33.36	13.39	0.0	0.0	0.0	0.0	0.0003	25.07096

Diffuser table:

P-dia	Ver angl	H-Angle	SourceX	SourceY	Ports	MZ-dis	Isoplth	P-depth	Ttl-flo	Eff-sal	Temp	Polutnt
(in)	(deg)	(deg)	(ft)	(ft)	()	(ft)	(concent)	(ft)	(MGD)	(psu)	(C)	(ppm)
3.0500	0.0	0.0	0.0	0.0	125.00	1000.0	0.0	100.00	46.000	21.850	20.660	21850.0

Simulation:
Froude No: 38.72; Strat No: 3.70E-4; Spcg No: 14.39; k: 3.42E+5; eff den (sigmaT) 14.65328; eff vel 3.420(m/s);
Current is very small, flow regime may be transient.

Step	Depth (ft)	Amb-cur (m/s)	P-dia (in)	Eff-sal (psu)	Polutnt (ppm)	Dilutn ()	x-posn (m)	y-posn (m)	Iso dia (m)
0	100.0	1.000E-5	3.050	21.85	21850.0	1.000	0.0	0.0	0.07747;
1	100.00	0.0	3.130	22.43	20750.8	1.053	0.032	0.0	0.0795; bottom hit;
10	100.00	0.0	3.798	24.22	17391.1	1.256	0.151	0.0	0.09648;
20	100.00	0.0	4.627	25.86	14287.7	1.529	0.310	0.0	0.1175;
30	100.00	0.0	5.637	27.21	11734.9	1.862	0.503	0.0	0.1432;
40	100.00	0.0	6.867	28.31	9636.2	2.267	0.739	0.0	0.1744;
50	100.00	0.0	8.368	29.22	7911.4	2.762	1.027	0.0	0.2125;
60	100.00	0.0	10.20	29.96	6494.3	3.364	1.378	0.0	0.2590;
70	99.99	0.0	12.43	30.57	5330.5	4.099	1.806	0.0	0.3156;
80	99.98	0.0	15.14	31.08	4374.8	4.995	2.326	0.0	0.3847;
90	99.97	0.0	18.46	31.49	3590.1	6.086	2.958	0.0	0.4688;
100	99.94	0.0	22.49	31.82	2946.0	7.417	3.727	0.0	0.5712;
110	99.89	0.0	27.40	32.10	2417.4	9.039	4.658	0.0	0.6959;
120	99.81	0.0	33.36	32.33	1983.5	11.02	5.782	0.0	0.8473;
130	99.65	0.0	40.57	32.51	1627.6	13.42	7.130	0.0	1.0304;
135	99.55	0.0	44.35	32.58	1488.1	14.68	7.820	0.0	1.1266; merging;
140	99.44	0.0	47.81	32.64	1387.2	15.75	8.463	0.0	1.2144;
150	99.18	0.0	54.23	32.71	1241.0	17.61	9.655	0.0	1.3776;
160	98.89	0.0	60.48	32.77	1131.6	19.31	10.76	0.0	1.5363;
170	98.55	0.0	66.79	32.82	1042.9	20.95	11.80	0.0	1.6965;
180	98.17	0.0	73.29	32.86	967.2	22.59	12.80	0.0	1.8617;
190	97.75	0.0	80.09	32.89	900.8	24.26	13.77	0.0	2.0342;
200	97.29	0.0	87.24	32.92	841.2	25.98	14.71	0.0	2.2160;
210	96.77	0.0	94.84	32.95	786.7	27.77	15.64	0.0	2.4089;
220	96.19	0.0	102.9	32.98	736.4	29.67	16.57	0.0	2.6144;
230	95.54	0.0	111.6	33.00	689.5	31.69	17.49	0.0	2.8342;
240	94.82	0.0	120.9	33.03	645.3	33.86	18.42	0.0	3.0698;
250	94.01	0.0	130.8	33.05	603.6	36.20	19.36	0.0	3.3234;
260	93.10	0.0	141.6	33.07	563.8	38.75	20.33	0.0	3.5971;
270	92.06	0.0	153.3	33.09	525.7	41.56	21.32	0.0	3.8945;
280	90.87	0.0	166.1	33.11	488.9	44.69	22.35	0.0	4.2200;
290	89.48	0.0	180.4	33.13	453.2	48.21	23.44	0.0	4.5810;
300	87.85	0.0	196.4	33.14	418.1	52.26	24.59	0.0	4.9883;
310	85.92	0.0	214.9	33.16	383.4	56.99	25.81	0.0	5.4594;
320	83.53	0.0	237.6	33.18	348.1	62.77	27.17	0.0	6.0361;
330	79.75	0.0	276.0	33.20	305.5	71.52	29.07	0.0	7.0103;
340	73.12	0.0	367.9	33.22	251.4	86.90	32.09	0.0	9.3445;
347	67.93	0.0	466.2	33.24	218.9	99.82	34.32	0.0	11.842; trap level;
350	65.60	0.0	525.6	33.24	206.3	105.9	35.32	0.0	13.350;
360	60.46	0.0	788.2	33.26	179.6	121.7	37.70	0.0	20.021;
361	60.18	0.0	812.7	33.26	177.9	122.8	37.84	0.0	20.643; begin overlap;
370	58.29	0.0	1001.1	33.26	169.1	129.2	38.85	0.0	25.428;
380	57.30	0.0	1163.5	33.27	164.6	132.8	39.44	0.0	29.552;
390	56.74	0.0	1314.1	33.27	161.1	135.6	39.82	0.0	33.379;
400	56.46	0.0	1442.8	33.27	158.4	137.9	40.03	0.0	36.646;
410	56.43	0.0	1500.7	33.27	156.3	139.8	40.06	0.0	38.119;
420	56.42	0.0	1540.9	33.27	154.3	141.6	40.06	0.0	39.140;
430	56.42	0.0	1579.5	33.27	152.4	143.3	40.06	0.0	40.118;
440	56.42	0.0	1621.8	33.27	150.4	145.2	40.06	0.0	41.194;

442	55.79	0.0	1834.6	33.28	147.5	148.2	40.77	0.0	46.599; surface;
450	55.78	0.0	2082.0	33.28	147.0	148.7	40.78	0.0	52.884;
460	55.57	0.0	2818.3	33.28	143.8	152.0	41.80	0.0	71.584; local maximum rise or fall;
465	66.22	0.0	2875.3	33.28	140.9	155.1	45.81	0.0	73.033;

Horiz plane projections in effluent direction: radius(m): 0.0; CL(m): 13.963
Lmz(m): 13.963
forced entrain 1 0.0 10.30 73.03 0.981
Rate sec-1 0.0 dy-1 0.0 kt: 0.0 Amb Sal 33.3761
;
12:20:30 PM. amb fills: 4

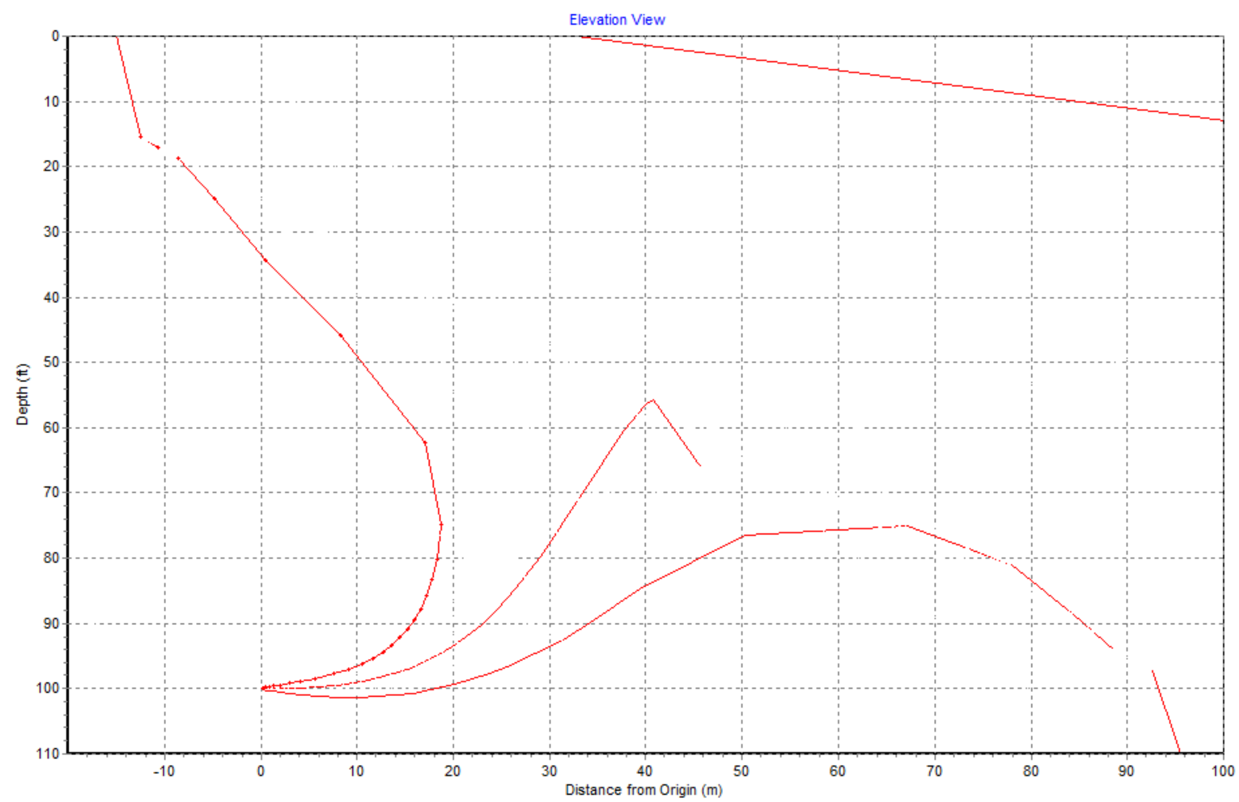

Figure B.2.1: Plumes 18b solution of discharge plume trajectories for discharges of 31 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt., and 15 mgd of brine from the Doheny Desalination Project with a brine salinity of 67 ppt. ZID is defined by the maximum horizontal excursion of trajectories from the origin. From the maximum horizontal spreading of the plume, the ZID extends from X = -15 m to X = +94 m so that ZID = 109 m

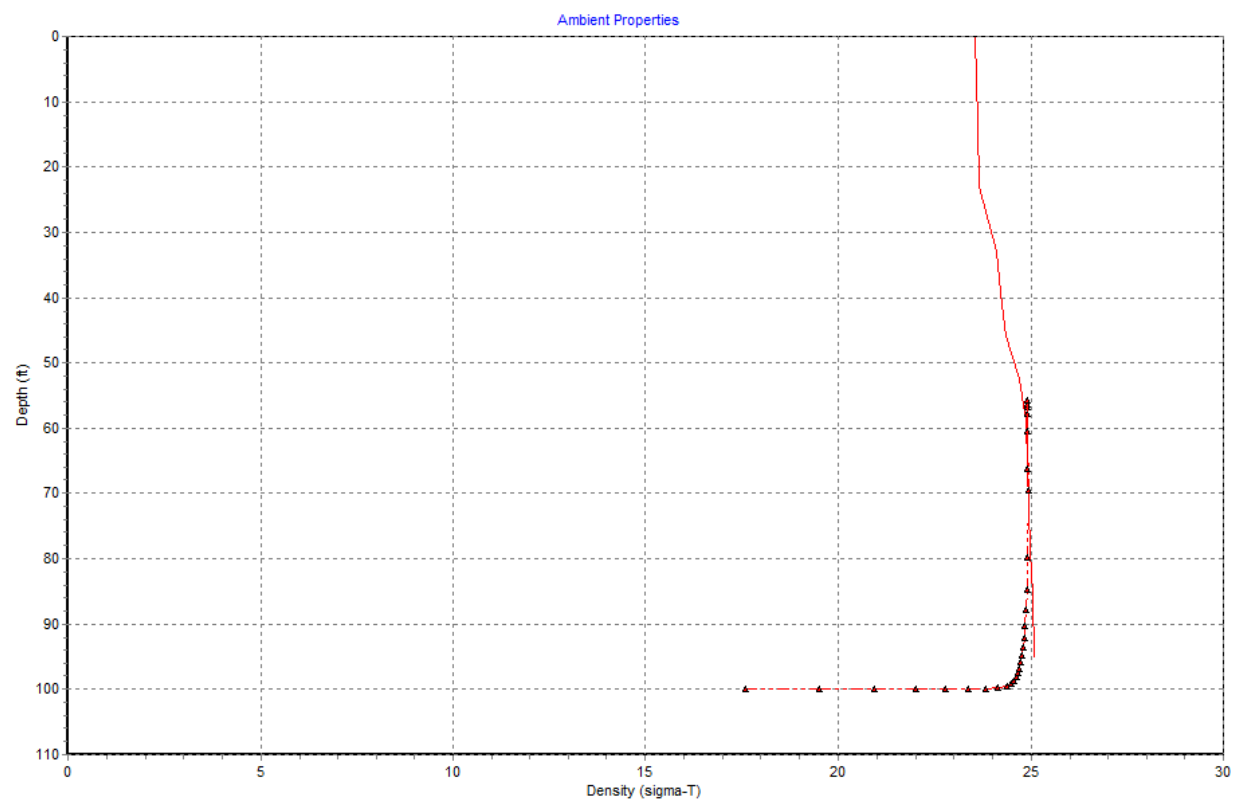

Figure B.2.2: Plumes 18b solution of vertical density profile for discharges of 31 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt. and 15 mgd of brine from the Doheny Desalination Project with a brine salinity of 67 ppt. Discharge effluent density shown as black triangles. Ambient water mass density profile shown as solid red line.

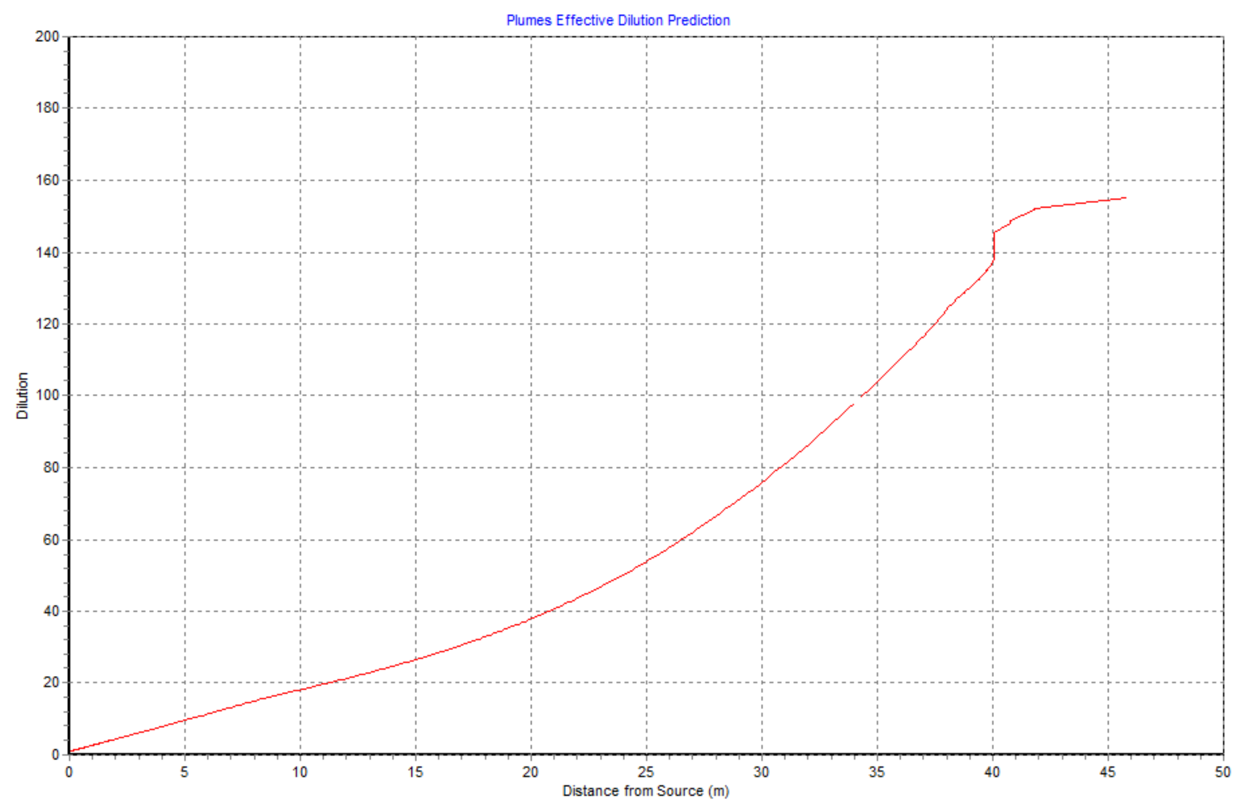

Figure B.2.3: Plumes 18b solution of effective (bulk average) dilution as a function of vertical distance from the point of discharge for discharges of 31 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt. and 15 mgd of brine from the Doheny Desalination Project with a brine salinity of 67 ppt.

B.3: Plumes 18b Results for SJCOO discharges of 18.9 mgd Wastewater and 5 mgd Brine:

Contents of the memo box (may not be current and must be updated manually)
Project "C:\Plumes18\SJCOO_WW18.9mgd_b5mgd_T-1"
memo
SJCOO discharging 18.9 mgd wastewater and 5 mgd brine

Model configuration items checked:
Channel width (m) 100
Start case for graphs 1
Max detailed graphs 10 (limits plots that can overflow memory)
Elevation Projection Plane (deg) 0
Shore vector (m,deg) not checked
Bacteria model : Mancini (1978) coliform model
PDS sfc. model heat transfer : Medium
Equation of State : S, T
Similarity Profile : Default profile (k=2.0, ...)
Diffuser port contraction coefficient 1
Light absorption coefficient 0.16
Farfield increment (m) 200
UM3 aspiration coefficient 0.1
Output file: text output tab
Output each ?? steps 10
Maximum dilution reported 1000
Text output format : Standard
Max vertical reversals : to max rise or fall

/ UM3. 1/10/2019 12:59:47 PM
Case 1; ambient file C:\Plumes18\SJCOO_WW18.9mgd_b5mgd_T-1.001.db; Diffuser table record 1: ----------------------------------

Ambient Table:

Depth	Amb-cur	Amb-dir	Amb-sal	Amb-tem	Amb-pol	Decay	Far-spd	Far-dir	Disprsn	Density
m	m/s	deg	psu	C	kg/kg	s-1	m/s	deg	m0.67/s2	sigma-T
0.0	0.0	0.0	33.38	19.97	0.0	0.0	0.0	0.0	0.0003	23.55719
2.000	0.0	0.0	33.39	19.87	0.0	0.0	0.0	0.0	0.0003	23.59288
4.000	0.0	0.0	33.40	19.81	0.0	0.0	0.0	0.0	0.0003	23.61484
7.000	0.0	0.0	33.41	19.64	0.0	0.0	0.0	0.0	0.0003	23.67096
10.00	0.0	0.0	33.40	17.76	0.0	0.0	0.0	0.0	0.0003	24.12888
12.00	0.0	0.0	33.38	17.34	0.0	0.0	0.0	0.0	0.0003	24.21549
14.00	0.0	0.0	33.38	16.73	0.0	0.0	0.0	0.0	0.0003	24.35932
16.00	0.0	0.0	33.38	15.17	0.0	0.0	0.0	0.0	0.0003	24.71340
18.00	0.0	0.0	33.37	14.37	0.0	0.0	0.0	0.0	0.0003	24.87044
20.00	0.0	0.0	33.35	14.18	0.0	0.0	0.0	0.0	0.0003	24.89660
22.00	0.0	0.0	33.33	13.83	0.0	0.0	0.0	0.0	0.0003	24.95172
24.00	0.0	0.0	33.32	13.75	0.0	0.0	0.0	0.0	0.0003	24.96707
26.00	0.0	0.0	33.36	13.42	0.0	0.0	0.0	0.0	0.0003	25.06459
29.00	0.0	0.0	33.36	13.39	0.0	0.0	0.0	0.0	0.0003	25.07096

Diffuser table:

P-dia	Ver angl	H-Angle	SourceX	SourceY	Ports	MZ-dis	Isoplth	P-depth	Ttl-flo	Eff-sal	Temp	Polutnt
(in)	(deg)	(deg)	(ft)	(ft)	()	(ft)	(concent)	(ft)	(MGD)	(psu)	(C)	(ppm)
3.0500	0.0	0.0	0.0	0.0	125.00	1000.0	0.0	100.00	23.900	14.020	20.660	14020.0

Simulation:

Froude No: 16.02; Strat No: 2.36E-4; Spcg No: 14.39; k: 1.78E+5; eff den (sigmaT) 8.743853; eff vel 1.777(m/s);
Current is very small, flow regime may be transient.

Step	Depth (ft)	Amb-cur (m/s)	P-dia (in)	Eff-sal (psu)	Polutnt (ppm)	Dilutn ()	x-posn (m)	y-posn (m)	Iso dia (m)
0	100.0	1.000E-5	3.050	14.02	14020.0	1.000	0.0	0.0	0.07747;
1	100.00	0.0	3.092	14.55	13644.5	1.028	0.017	0.0	0.07853; bottom hit;
10	100.00	0.0	3.705	17.62	11446.4	1.225	0.113	0.0	0.09409;
20	100.00	0.0	4.510	20.45	9412.2	1.490	0.241	0.0	0.1146;
30	100.00	0.0	5.493	22.77	7736.2	1.812	0.397	0.0	0.1395;
40	100.00	0.0	6.690	24.67	6356.4	2.206	0.587	0.0	0.1699;
50	99.99	0.0	8.149	26.23	5221.2	2.685	0.818	0.0	0.2070;
60	99.98	0.0	9.927	27.51	4287.7	3.270	1.100	0.0	0.2522;
70	99.97	0.0	12.09	28.56	3520.5	3.982	1.441	0.0	0.3071;
80	99.94	0.0	14.72	29.43	2890.1	4.851	1.854	0.0	0.3739;
90	99.89	0.0	17.90	30.13	2372.3	5.910	2.352	0.0	0.4547;
100	99.81	0.0	21.31	30.65	1990.2	7.044	2.861	0.0	0.5413;
110	99.72	0.0	24.50	31.02	1724.7	8.129	3.304	0.0	0.6224;
120	99.62	0.0	27.57	31.29	1525.1	9.193	3.701	0.0	0.7002;
130	99.50	0.0	30.54	31.50	1367.3	10.25	4.065	0.0	0.7756;
140	99.38	0.0	33.44	31.68	1238.2	11.32	4.403	0.0	0.8493;
150	99.24	0.0	36.28	31.83	1130.0	12.41	4.724	0.0	0.9214;
160	99.09	0.0	39.06	31.95	1037.5	13.51	5.030	0.0	0.9921;
170	98.93	0.0	41.79	32.06	957.2	14.65	5.327	0.0	1.0614;
178	98.79	0.0	43.92	32.14	900.0	15.58	5.560	0.0	1.1157; merging;
180	98.75	0.0	44.45	32.16	886.8	15.81	5.618	0.0	1.1291;
190	98.56	0.0	47.09	32.24	826.7	16.96	5.906	0.0	1.1961;
200	98.34	0.0	49.76	32.31	773.1	18.13	6.194	0.0	1.2638;
210	98.09	0.0	52.44	32.38	724.4	19.35	6.486	0.0	1.3321;
220	97.81	0.0	55.15	32.44	679.5	20.63	6.785	0.0	1.4009;
230	97.50	0.0	57.87	32.50	637.7	21.98	7.094	0.0	1.4700;
240	97.13	0.0	60.60	32.55	598.5	23.42	7.415	0.0	1.5393;
250	96.70	0.0	63.35	32.60	561.4	24.97	7.755	0.0	1.6091;
260	96.20	0.0	66.13	32.65	525.7	26.67	8.116	0.0	1.6798;
270	95.60	0.0	69.01	32.69	490.8	28.56	8.506	0.0	1.7529;
280	94.86	0.0	72.09	32.74	456.3	30.73	8.932	0.0	1.8310;
290	93.95	0.0	75.53	32.79	421.3	33.28	9.404	0.0	1.9185;
300	92.78	0.0	79.65	32.84	385.1	36.41	9.932	0.0	2.0230;
310	91.26	0.0	84.92	32.89	347.0	40.40	10.53	0.0	2.1571;
320	89.22	0.0	92.23	32.95	306.3	45.78	11.23	0.0	2.3427;
330	86.36	0.0	103.2	33.01	262.3	53.45	12.05	0.0	2.6210;
340	82.32	0.0	120.9	33.07	216.5	64.76	13.01	0.0	3.0701;
350	77.55	0.0	149.4	33.12	177.6	78.94	13.96	0.0	3.7946;
360	72.06	0.0	192.2	33.15	145.7	96.22	14.93	0.0	4.8807;
370	65.85	0.0	254.3	33.19	119.5	117.3	15.90	0.0	6.4600;
378	60.31	0.0	336.5	33.21	102.0	137.4	16.73	0.0	8.5480; trap level;
380	58.84	0.0	363.7	33.22	98.06	143.0	16.95	0.0	9.2392;
390	52.96	0.0	618.0	33.24	83.72	167.5	17.91	0.0	15.697; begin overlap;
400	51.76	0.0	876.1	33.24	81.02	173.0	18.17	0.0	22.253;
410	51.31	0.0	1107.1	33.25	79.66	176.0	18.30	0.0	28.121;
420	51.19	0.0	1255.4	33.25	78.79	177.9	18.34	0.0	31.887;
430	51.19	0.0	1286.0	33.25	78.09	179.5	18.34	0.0	32.665;
440	51.19	0.0	1308.8	33.25	77.41	181.1	18.34	0.0	33.244;
446	50.90	0.0	2033.2	33.25	75.64	185.3	18.63	0.0	51.643; surface;
450	50.90	0.0	3055.0	33.25	75.63	185.4	18.63	0.0	77.598; local maximum rise or fall;
453	56.95	0.0	3160.3	33.26	74.15	189.1	19.60	0.0	80.271;

Horiz plane projections in effluent direction: radius(m): 0.0; CL(m): 5.9730
Lmz(m): 5.9730
forced entrain 1 0.0 13.12 80.27 0.724
Rate sec-1 0.0 dy-1 0.0 kt: 0.0 Amb Sal 33.3837
;
12:59:47 PM. amb fills: 4

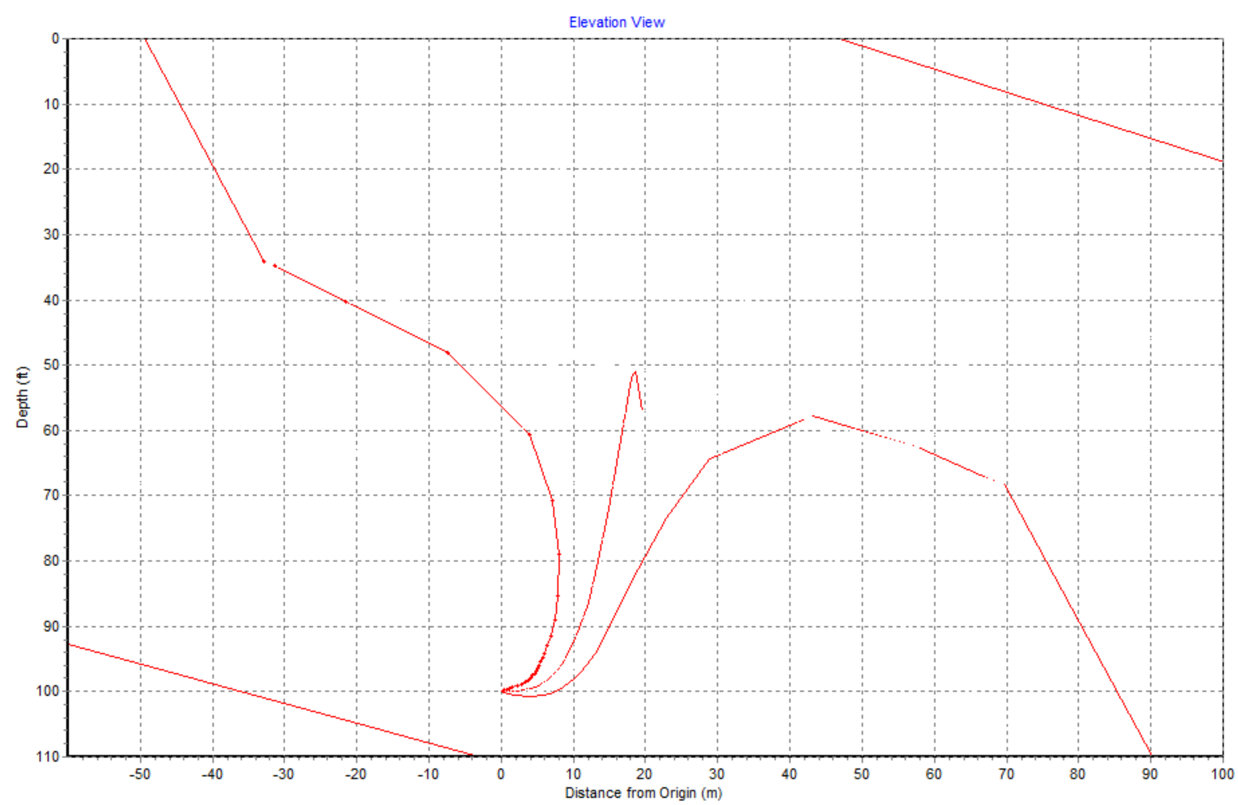

Figure B.3.1: Plumes 18b solution of discharge plume trajectories for discharges of 18.9 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt., and 5 mgd of brine from the Doheny Desalination Project with a brine salinity of 67 ppt. ZID is defined by the maximum horizontal excursion of trajectories from the origin. From the maximum horizontal spreading of the plume, the ZID extends from *X* = -50 m to *X* = +85 m so that ZID = 135 m

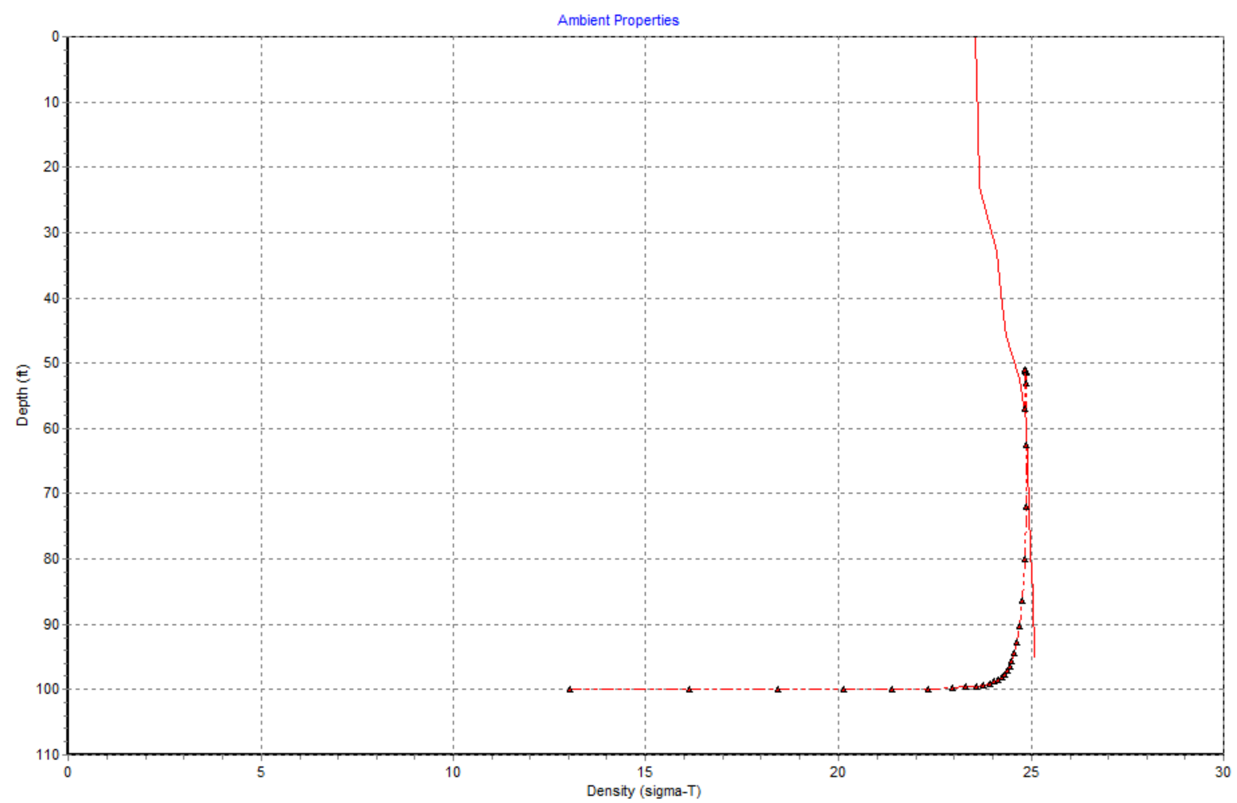

Figure B.3.2: Plumes 18b solution of vertical density profile for discharges of 18.9 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt. and 5 mgd of brine from the Doheny Desalination Project with a brine salinity of 67 ppt. Discharge effluent density shown as black triangles. Ambient water mass density profile shown as solid red line.

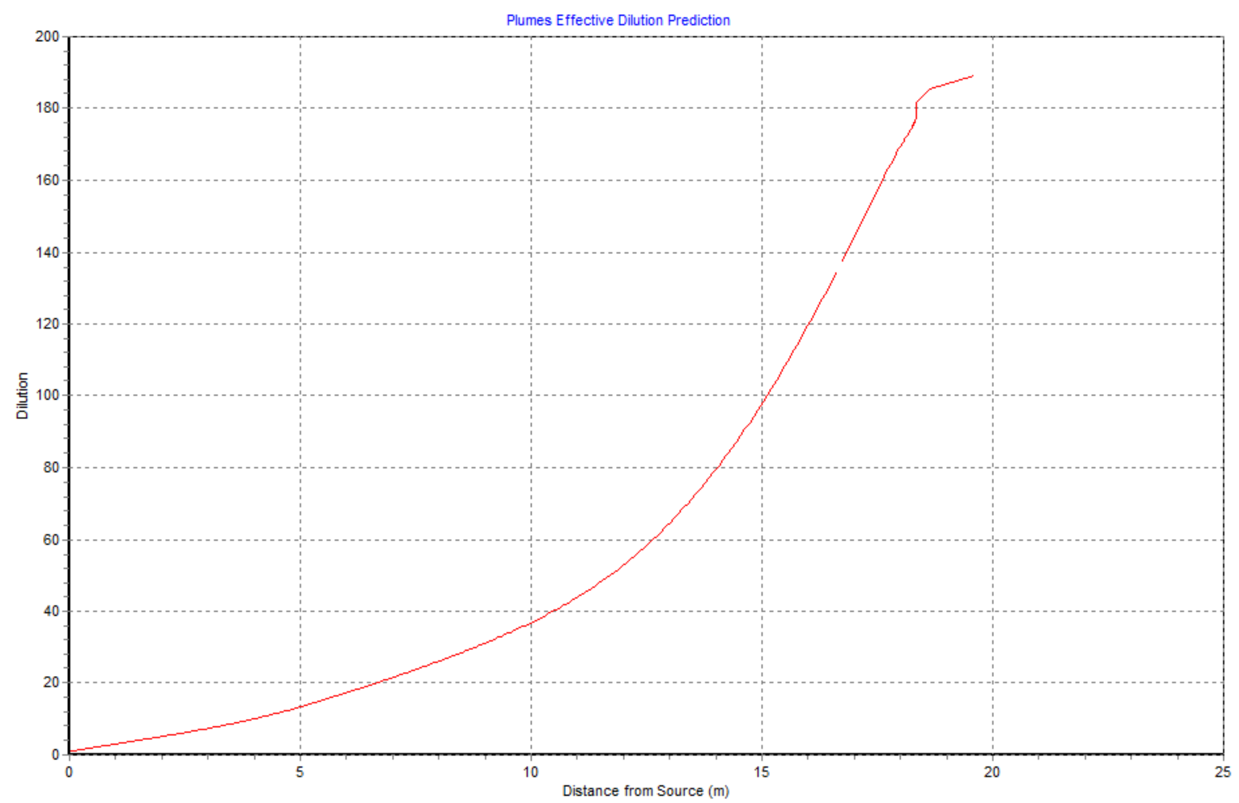

Figure B.3.3: Plumes 18b solution of effective (bulk average) dilution as a function of vertical distance from the point of discharge for discharges of 18.9 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt. and 5 mgd of brine from the Doheny Desalination Project with a brine salinity of 67 ppt.

B.4: Plumes 18b Results for SJCOO discharges of 18.9 mgd Wastewater and 15 mgd Brine:

Contents of the memo box (may not be current and must be updated manually)
Project "C:\Plumes18\SJCOO_WW18.9mgd_b15mgd_T-1"
memo
SJCOO discharging 18.9 mgd wastewater and 15 mgd brine

Model configuration items checked:
Channel width (m) 100
Start case for graphs 1
Max detailed graphs 10 (limits plots that can overflow memory)
Elevation Projection Plane (deg) 0
Shore vector (m,deg) not checked
Bacteria model : Mancini (1978) coliform model
PDS sfc. model heat transfer : Medium
Equation of State : S, T
Similarity Profile : Default profile (k=2.0, ...)
Diffuser port contraction coefficient 1
Light absorption coefficient 0.16
Farfield increment (m) 200
UM3 aspiration coefficient 0.1
Output file: text output tab
Output each ?? steps 10
Maximum dilution reported 1000
Text output format : Standard
Max vertical reversals : to max rise or fall

/ UM3. 1/10/2019 1:29:39 PM
Case 1; ambient file C:\Plumes18\SJCOO_WW18.9mgd_b15mgd_T-1.001.db; Diffuser table record 1: ----------------------------------

Ambient Table:

Depth	Amb-cur	Amb-dir	Amb-sal	Amb-tem	Amb-pol	Decay	Far-spd	Far-dir	Disprsn	Density
m	m/s	deg	psu	C	kg/kg	s-1	m/s	deg	m0.67/s2	sigma-T
0.0	0.0	0.0	33.38	19.97	0.0	0.0	0.0	0.0	0.0003	23.55719
2.000	0.0	0.0	33.39	19.87	0.0	0.0	0.0	0.0	0.0003	23.59288
4.000	0.0	0.0	33.40	19.81	0.0	0.0	0.0	0.0	0.0003	23.61484
7.000	0.0	0.0	33.41	19.64	0.0	0.0	0.0	0.0	0.0003	23.67096
10.00	0.0	0.0	33.40	17.76	0.0	0.0	0.0	0.0	0.0003	24.12888
12.00	0.0	0.0	33.38	17.34	0.0	0.0	0.0	0.0	0.0003	24.21549
14.00	0.0	0.0	33.38	16.73	0.0	0.0	0.0	0.0	0.0003	24.35932
16.00	0.0	0.0	33.38	15.17	0.0	0.0	0.0	0.0	0.0003	24.71340
18.00	0.0	0.0	33.37	14.37	0.0	0.0	0.0	0.0	0.0003	24.87044
20.00	0.0	0.0	33.35	14.18	0.0	0.0	0.0	0.0	0.0003	24.89660
22.00	0.0	0.0	33.33	13.83	0.0	0.0	0.0	0.0	0.0003	24.95172
24.00	0.0	0.0	33.32	13.75	0.0	0.0	0.0	0.0	0.0003	24.96707
26.00	0.0	0.0	33.36	13.42	0.0	0.0	0.0	0.0	0.0003	25.06459
29.00	0.0	0.0	33.36	13.39	0.0	0.0	0.0	0.0	0.0003	25.07096

Diffuser table:

P-dia	Ver angl	H-Angle	SourceX	SourceY	Ports	MZ-dis	Isoplth	P-depth	Ttl-flo	Eff-sal	Temp	Polutnt
(in)	(deg)	(deg)	(ft)	(ft)	()	(ft)	(concent)	(ft)	(MGD)	(psu)	(C)	(ppm)
3.0500	0.0	0.0	0.0	0.0	125.00	1000.0	0.0	100.00	33.900	29.640	20.660	29640.0

Simulation:

Froude No: 43.40; Strat No: 8.51E-4; Spcg No: 14.39; k: 2.52E+5; eff den (sigmaT) 20.54180; eff vel 2.521(m/s);
Current is very small, flow regime may be transient.

Step	Depth (ft)	Amb-cur (m/s)	P-dia (in)	Eff-sal (psu)	Polutnt (ppm)	Dilutn ()	x-posn (m)	y-posn (m)	Iso dia (m)
0	100.0	1.000E-5	3.050	29.64	29640.0	1.000	0.0	0.0	0.07747;
1	100.00	0.0	3.109	29.78	28526.4	1.039	0.0239	0.0	0.07897; bottom hit;
10	100.00	0.0	3.749	30.37	23886.8	1.241	0.146	0.0	0.09522;
20	100.00	0.0	4.569	30.90	19608.3	1.512	0.308	0.0	0.1160;
30	100.00	0.0	5.568	31.34	16094.2	1.842	0.507	0.0	0.1414;
40	100.00	0.0	6.785	31.71	13208.5	2.244	0.749	0.0	0.1723;
50	100.00	0.0	8.270	32.00	10839.4	2.734	1.044	0.0	0.2101;
60	100.00	0.0	10.08	32.25	8894.6	3.332	1.404	0.0	0.2560;
70	99.99	0.0	12.29	32.45	7298.4	4.061	1.842	0.0	0.3120;
80	99.99	0.0	14.97	32.61	5988.4	4.950	2.376	0.0	0.3803;
90	99.97	0.0	18.25	32.75	4913.3	6.033	3.025	0.0	0.4636;
100	99.95	0.0	22.24	32.86	4031.1	7.353	3.814	0.0	0.5650;
110	99.91	0.0	27.10	32.95	3307.3	8.962	4.771	0.0	0.6884;
120	99.84	0.0	33.02	33.02	2713.4	10.92	5.930	0.0	0.8386;
130	99.72	0.0	40.19	33.08	2226.1	13.32	7.326	0.0	1.0208;
135	99.62	0.0	44.32	33.11	2016.3	14.70	8.124	0.0	1.1257; merging;
140	99.50	0.0	48.45	33.13	1856.9	15.96	8.945	0.0	1.2307;
150	99.23	0.0	55.90	33.16	1642.4	18.05	10.45	0.0	1.4199;
160	98.91	0.0	62.94	33.18	1490.7	19.88	11.82	0.0	1.5988;
170	98.54	0.0	69.90	33.19	1371.7	21.61	13.12	0.0	1.7755;
180	98.11	0.0	76.94	33.20	1272.6	23.29	14.35	0.0	1.9542;
190	97.64	0.0	84.16	33.21	1186.9	24.97	15.55	0.0	2.1378;
200	97.10	0.0	91.68	33.22	1110.7	26.69	16.72	0.0	2.3286;
210	96.50	0.0	99.55	33.23	1041.5	28.46	17.86	0.0	2.5286;
220	95.84	0.0	107.9	33.24	977.7	30.32	19.00	0.0	2.7397;
230	95.09	0.0	116.7	33.25	918.1	32.28	20.15	0.0	2.9642;
240	94.26	0.0	126.2	33.25	861.8	34.39	21.30	0.0	3.2045;
250	93.31	0.0	136.4	33.26	808.1	36.68	22.46	0.0	3.4637;
260	92.25	0.0	147.5	33.27	756.6	39.18	23.66	0.0	3.7458;
270	91.03	0.0	159.7	33.27	706.6	41.95	24.89	0.0	4.0564;
280	89.64	0.0	173.3	33.28	657.8	45.06	26.18	0.0	4.4030;
290	88.03	0.0	188.9	33.29	609.7	48.62	27.52	0.0	4.7970;
300	86.13	0.0	206.9	33.29	562.0	52.74	28.95	0.0	5.2551;
310	83.81	0.0	229.1	33.30	513.0	57.78	30.52	0.0	5.8200;
319	79.50	0.0	277.7	33.30	444.6	66.66	33.18	0.0	7.0529; trap level;
320	78.85	0.0	287.9	33.30	435.9	68.00	33.57	0.0	7.3118;
330	75.07	0.0	400.7	33.31	387.5	76.48	36.01	0.0	10.177;
340	73.17	0.0	496.5	33.31	361.5	81.98	37.40	0.0	12.612; begin overlap;
350	71.91	0.0	583.6	33.31	345.5	85.79	38.42	0.0	14.822;
360	71.06	0.0	657.2	33.31	335.8	88.27	39.18	0.0	16.692;
370	70.48	0.0	724.0	33.31	328.6	90.21	39.75	0.0	18.389;
380	70.06	0.0	787.9	33.31	322.5	91.89	40.22	0.0	20.013;
390	69.74	0.0	850.1	33.31	317.2	93.44	40.62	0.0	21.593;
400	69.48	0.0	910.9	33.31	312.4	94.88	40.96	0.0	23.137;
410	69.28	0.0	970.3	33.31	307.9	96.26	41.26	0.0	24.646;
420	69.12	0.0	1028.2	33.31	303.7	97.58	41.53	0.0	26.117;
430	69.00	0.0	1083.3	33.31	299.9	98.83	41.76	0.0	27.516;
440	68.95	0.0	1124.0	33.31	296.6	99.94	41.87	0.0	28.549;
450	68.94	0.0	1152.3	33.31	293.5	101.0	41.90	0.0	29.267;
460	68.93	0.0	1176.6	33.31	290.5	102.0	41.90	0.0	29.886;
470	68.93	0.0	1200.0	33.31	287.7	103.0	41.90	0.0	30.479;

480	68.93	0.0	1223.0	33.31	285.0	104.0	41.90	0.0	31.065;
490	68.93	0.0	1245.8	33.31	282.4	105.0	41.90	0.0	31.643; local maximum rise or fall;
498	68.99	0.0	1274.4	33.31	275.3	107.6	43.60	0.0	32.371;

Horiz plane projections in effluent direction: radius(m): 0.0; CL(m): 13.290
Lmz(m): 13.290
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Rate sec-1 0.0 dy-1 0.0 kt: 0.0 Amb Sal 33.3361
;
1:29:39 PM. amb fills: 4

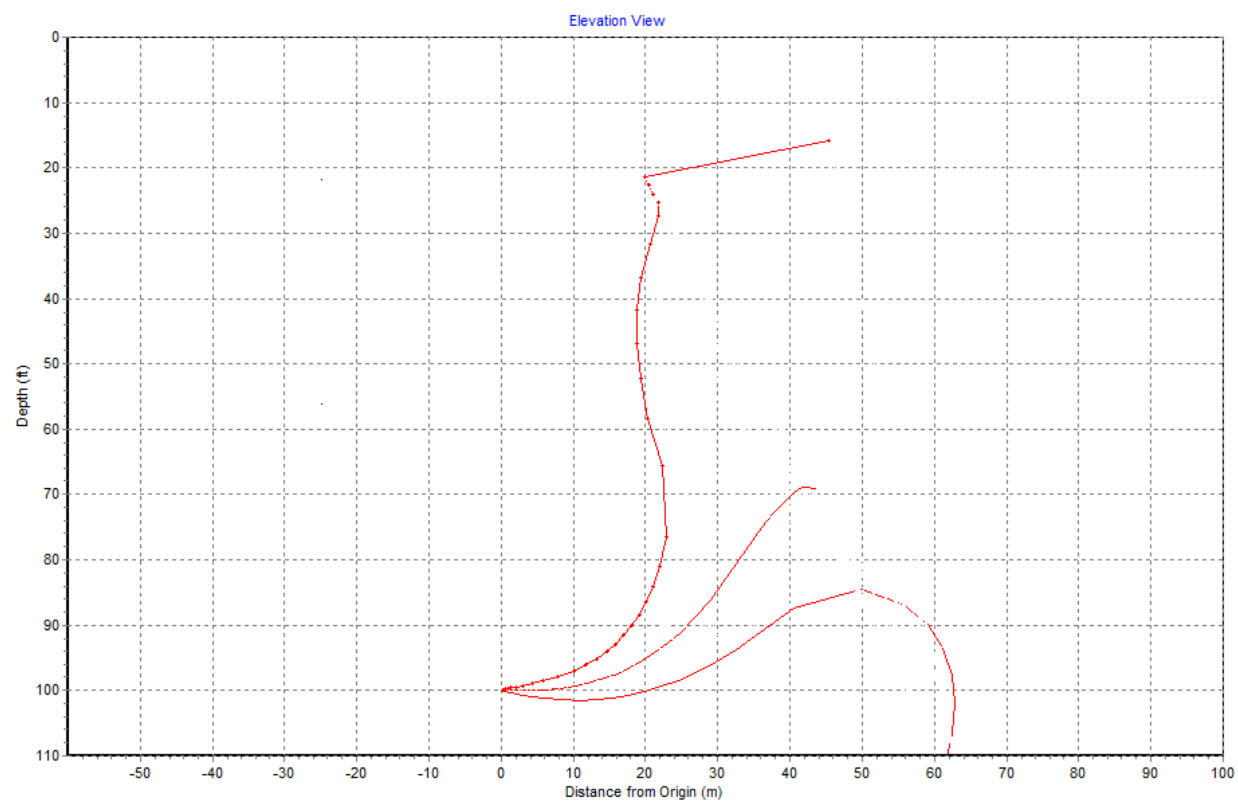

Figure B.4.1: Plumes 18b solution of discharge plume trajectories for discharges of 18.9 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt., and 15 mgd of brine from the Doheny Desalination Project with a brine salinity of 67 ppt. ZID is defined by the maximum horizontal excursion of trajectories from the origin. From the maximum horizontal spreading of the plume, the ZID extends from *X* = -0 m to *X* = +63 m so that ZID = 63 m

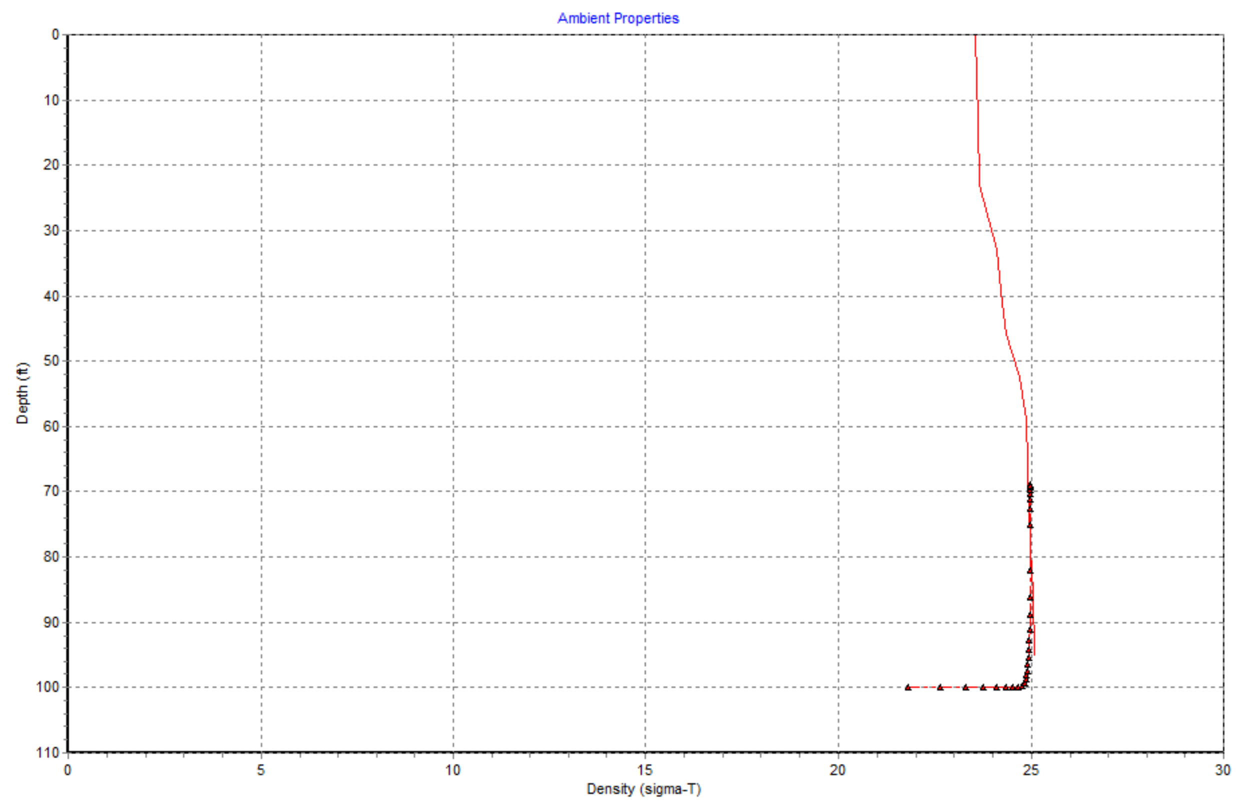

Figure B.4.2: Plumes 18b solution of vertical density profile for discharges of 18.9 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt. and 15 mgd of brine from the Doheny Desalination Project with a brine salinity of 67 ppt. Discharge effluent density shown as black triangles. Ambient water mass density profile shown as solid red line.

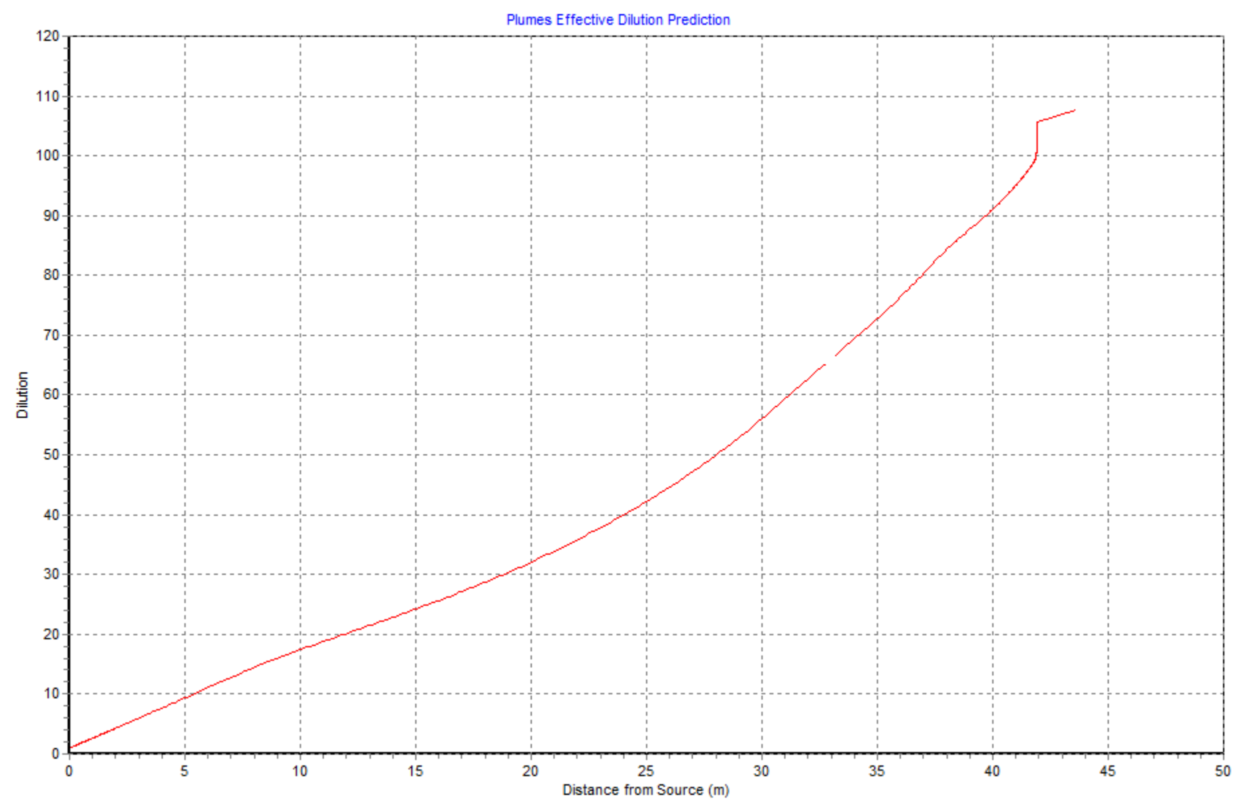

Figure B.4.3: Plumes 18b solution of effective (bulk average) dilution as a function of vertical distance from the point of discharge for discharges of 18.9 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt. and 15 mgd of brine from the Doheny Desalination Project with a brine salinity of 67 ppt.

B.5: Plumes 18b Results for SJCOO discharges of 13 mgd Wastewater and 5 mgd Brine:

Contents of the memo box (may not be current and must be updated manually)
Project "C:\Plumes18\SJCOO_WW13mgd_b5mgd_T-1"
memo
SJCOO discharging 13 mgd wastewater and 5 mgd brine

Model configuration items checked:
Channel width (m) 100
Start case for graphs 1
Max detailed graphs 10 (limits plots that can overflow memory)
Elevation Projection Plane (deg) 0
Shore vector (m,deg) not checked
Bacteria model : Mancini (1978) coliform model
PDS sfc. model heat transfer : Medium
Equation of State : S, T
Similarity Profile : Default profile (k=2.0, ...)
Diffuser port contraction coefficient 1
Light absorption coefficient 0.16
Farfield increment (m) 200
UM3 aspiration coefficient 0.1
Output file: text output tab
Output each ?? steps 10
Maximum dilution reported 1000
Text output format : Standard
Max vertical reversals : to max rise or fall

/ UM3. 1/10/2019 1:51:56 PM
Case 1; ambient file C:\Plumes18\SJCOO_WW13mgd_b5mgd_T-1.001.db; Diffuser table record 1: ----------------------------------

Ambient Table:

Depth	Amb-cur	Amb-dir	Amb-sal	Amb-tem	Amb-pol	Decay	Far-spd	Far-dir	Disprsn	Density
m	m/s	deg	psu	C	kg/kg	s-1	m/s	deg	m0.67/s2	sigma-T
0.0	0.0	0.0	33.38	19.97	0.0	0.0	0.0	0.0	0.0003	23.55719
2.000	0.0	0.0	33.39	19.87	0.0	0.0	0.0	0.0	0.0003	23.59288
4.000	0.0	0.0	33.40	19.81	0.0	0.0	0.0	0.0	0.0003	23.61484
7.000	0.0	0.0	33.41	19.64	0.0	0.0	0.0	0.0	0.0003	23.67096
10.00	0.0	0.0	33.40	17.76	0.0	0.0	0.0	0.0	0.0003	24.12888
12.00	0.0	0.0	33.38	17.34	0.0	0.0	0.0	0.0	0.0003	24.21549
14.00	0.0	0.0	33.38	16.73	0.0	0.0	0.0	0.0	0.0003	24.35932
16.00	0.0	0.0	33.38	15.17	0.0	0.0	0.0	0.0	0.0003	24.71340
18.00	0.0	0.0	33.37	14.37	0.0	0.0	0.0	0.0	0.0003	24.87044
20.00	0.0	0.0	33.35	14.18	0.0	0.0	0.0	0.0	0.0003	24.89660
22.00	0.0	0.0	33.33	13.83	0.0	0.0	0.0	0.0	0.0003	24.95172
24.00	0.0	0.0	33.32	13.75	0.0	0.0	0.0	0.0	0.0003	24.96707
26.00	0.0	0.0	33.36	13.42	0.0	0.0	0.0	0.0	0.0003	25.06459
29.00	0.0	0.0	33.36	13.39	0.0	0.0	0.0	0.0	0.0003	25.07096

Diffuser table:

P-dia	Ver angl	H-Angle	SourceX	SourceY	Ports	MZ-dis	Isoplth	P-depth	Ttl-flo	Eff-sal	Temp	Polutnt
(in)	(deg)	(deg)	(ft)	(ft)	()	(ft)	(concent)	(ft)	(MGD)	(psu)	(C)	(ppm)
3.0500	0.0	0.0	0.0	0.0	125.00	1000.0	0.0	100.00	18.000	18.610	20.660	18610.0

Simulation:
Froude No: 13.62; Strat No: 3.00E-4; Spcg No: 14.39; k: 1.34E+5; eff den (sigmaT) 12.20791; eff vel 1.338(m/s);
Current is very small, flow regime may be transient.

Step	Depth (ft)	Amb-cur (m/s)	P-dia (in)	Eff-sal (psu)	Polutnt (ppm)	Dilutn ()	x-posn (m)	y-posn (m)	Iso dia (m)
0	100.0	1.000E-5	3.050	18.61	18610.0	1.000	0.0	0.0	0.07747;
1	100.00	0.0	3.081	18.91	18232.1	1.021	0.0129	0.0	0.07827; bottom hit;
10	100.00	0.0	3.681	21.27	15286.9	1.217	0.107	0.0	0.09349;
20	100.00	0.0	4.482	23.44	12564.0	1.481	0.233	0.0	0.1139;
30	100.00	0.0	5.460	25.23	10322.6	1.803	0.386	0.0	0.1387;
40	99.99	0.0	6.651	26.69	8478.7	2.195	0.573	0.0	0.1689;
50	99.99	0.0	8.102	27.89	6962.6	2.673	0.800	0.0	0.2058;
60	99.98	0.0	9.869	28.87	5716.6	3.255	1.076	0.0	0.2507;
70	99.96	0.0	12.02	29.68	4692.8	3.966	1.409	0.0	0.3052;
80	99.92	0.0	14.62	30.34	3851.9	4.831	1.812	0.0	0.3714;
90	99.86	0.0	17.56	30.85	3202.8	5.811	2.253	0.0	0.4460;
100	99.78	0.0	20.34	31.20	2757.0	6.750	2.639	0.0	0.5166;
110	99.70	0.0	23.00	31.46	2426.4	7.670	2.983	0.0	0.5841;
120	99.61	0.0	25.58	31.66	2167.7	8.585	3.296	0.0	0.6496;
130	99.51	0.0	28.09	31.83	1957.6	9.507	3.587	0.0	0.7136;
140	99.39	0.0	30.56	31.96	1782.4	10.44	3.860	0.0	0.7763;
150	99.27	0.0	32.98	32.08	1633.4	11.39	4.121	0.0	0.8378;
160	99.14	0.0	35.36	32.18	1504.5	12.37	4.373	0.0	0.8981;
170	98.99	0.0	37.68	32.27	1391.7	13.37	4.618	0.0	0.9570;
180	98.83	0.0	39.95	32.35	1291.8	14.41	4.861	0.0	1.0146;
190	98.65	0.0	42.16	32.42	1202.3	15.48	5.102	0.0	1.0708;
199	98.47	0.0	44.10	32.48	1129.1	16.48	5.320	0.0	1.1200; merging;
200	98.45	0.0	44.31	32.48	1121.5	16.59	5.344	0.0	1.1254;
210	98.23	0.0	46.40	32.54	1050.7	17.71	5.591	0.0	1.1786;
220	97.98	0.0	48.47	32.59	986.2	18.87	5.845	0.0	1.2311;
230	97.68	0.0	50.51	32.64	926.2	20.09	6.109	0.0	1.2831;
240	97.34	0.0	52.56	32.68	869.6	21.40	6.387	0.0	1.3349;
250	96.94	0.0	54.61	32.72	815.2	22.83	6.683	0.0	1.3872;
260	96.47	0.0	56.73	32.76	762.3	24.41	7.001	0.0	1.4409;
270	95.89	0.0	58.97	32.81	709.8	26.22	7.347	0.0	1.4979;
280	95.17	0.0	61.46	32.85	656.8	28.34	7.729	0.0	1.5612;
290	94.26	0.0	64.40	32.89	602.1	30.91	8.156	0.0	1.6358;
300	93.08	0.0	68.10	32.94	544.8	34.16	8.640	0.0	1.7298;
310	91.52	0.0	73.09	32.98	483.8	38.47	9.198	0.0	1.8565;
320	89.35	0.0	80.34	33.03	418.3	44.49	9.852	0.0	2.0406;
330	86.22	0.0	91.68	33.09	348.3	53.43	10.63	0.0	2.3286;
340	82.35	0.0	108.2	33.14	285.7	65.13	11.42	0.0	2.7478;
350	77.85	0.0	134.5	33.17	234.4	79.39	12.21	0.0	3.4160;
360	72.61	0.0	175.6	33.20	192.3	96.77	13.03	0.0	4.4605;
370	66.57	0.0	237.5	33.22	157.8	118.0	13.89	0.0	6.0325;
371	65.92	0.0	246.1	33.23	154.7	120.3	13.98	0.0	6.2519; trap level;
380	59.69	0.0	365.8	33.25	129.4	143.8	14.90	0.0	9.2922;
388	55.64	0.0	587.5	33.26	116.0	160.4	15.59	0.0	14.923; begin overlap;
390	55.38	0.0	637.5	33.26	115.2	161.5	15.64	0.0	16.192;
400	54.63	0.0	846.1	33.26	113.0	164.7	15.82	0.0	21.491;
410	54.28	0.0	1044.1	33.26	111.6	166.8	15.93	0.0	26.521;
420	54.13	0.0	1206.9	33.26	110.6	168.3	15.98	0.0	30.656;
430	54.12	0.0	1244.3	33.27	109.8	169.5	15.98	0.0	31.606;
440	54.12	0.0	1261.9	33.27	109.1	170.6	15.98	0.0	32.052;
450	54.12	0.0	1278.7	33.27	108.4	171.7	15.98	0.0	32.479;

453 53.98 0.0 2141.7 33.27 106.2 175.2 16.30 0.0 54.400; surface; local maximum rise or fall;
459 87.41 0.0 3571.6 33.27 104.1 178.7 18.45 0.0 90.719;
Horiz plane projections in effluent direction: radius(m): 0.0; CL(m): 5.6237
Lmz(m): 5.6237
forced entrain 1 0.0 3.837 90.72 0.923
Rate sec-1 0.0 dy-1 0.0 kt: 0.0 Amb Sal 33.3807
;
1:51:56 PM. amb fills: 4

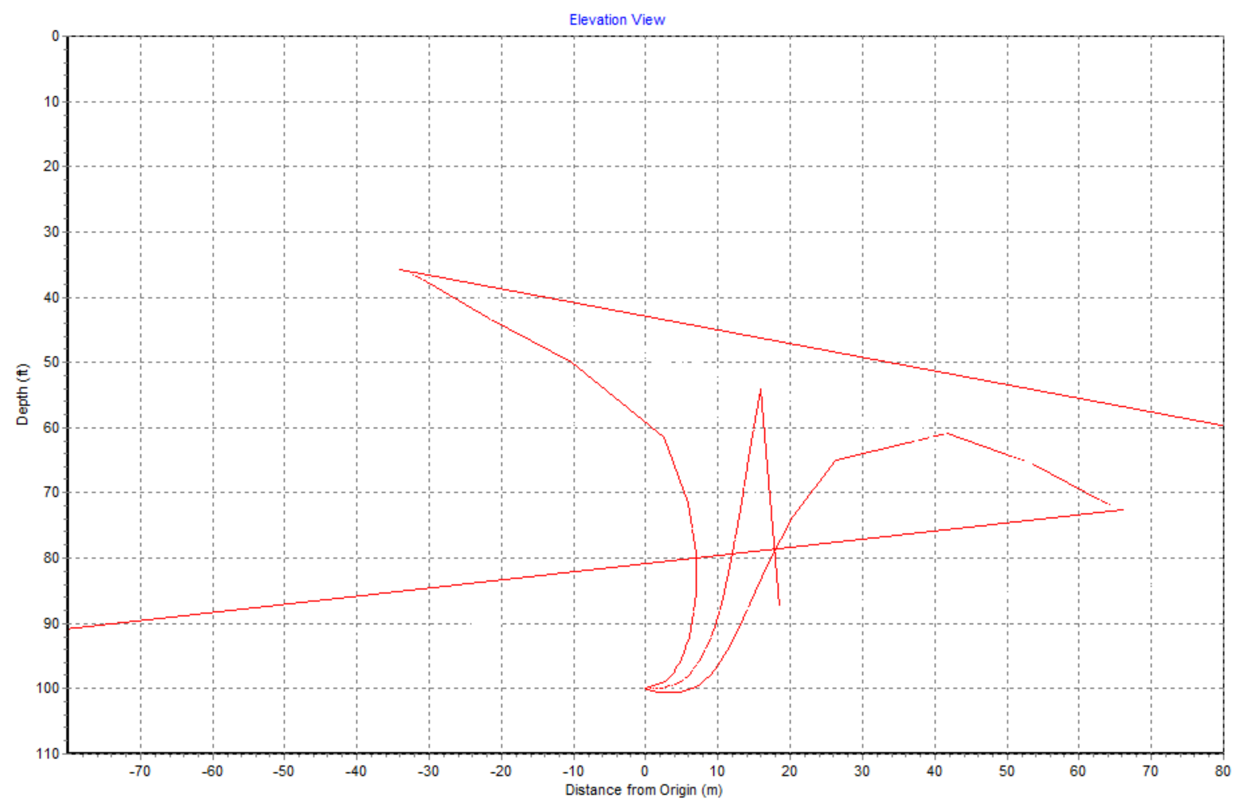

Figure B.5.1: Plumes 18b solution of discharge plume trajectories for discharges of 13 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt., and 5 mgd of brine from the Doheny Desalination Project with a brine salinity of 67 ppt. ZID is defined by the maximum horizontal excursion of trajectories from the origin. From the maximum horizontal spreading of the plume, the ZID extends from X = -80 m to X = +80 m so that ZID = 160 m

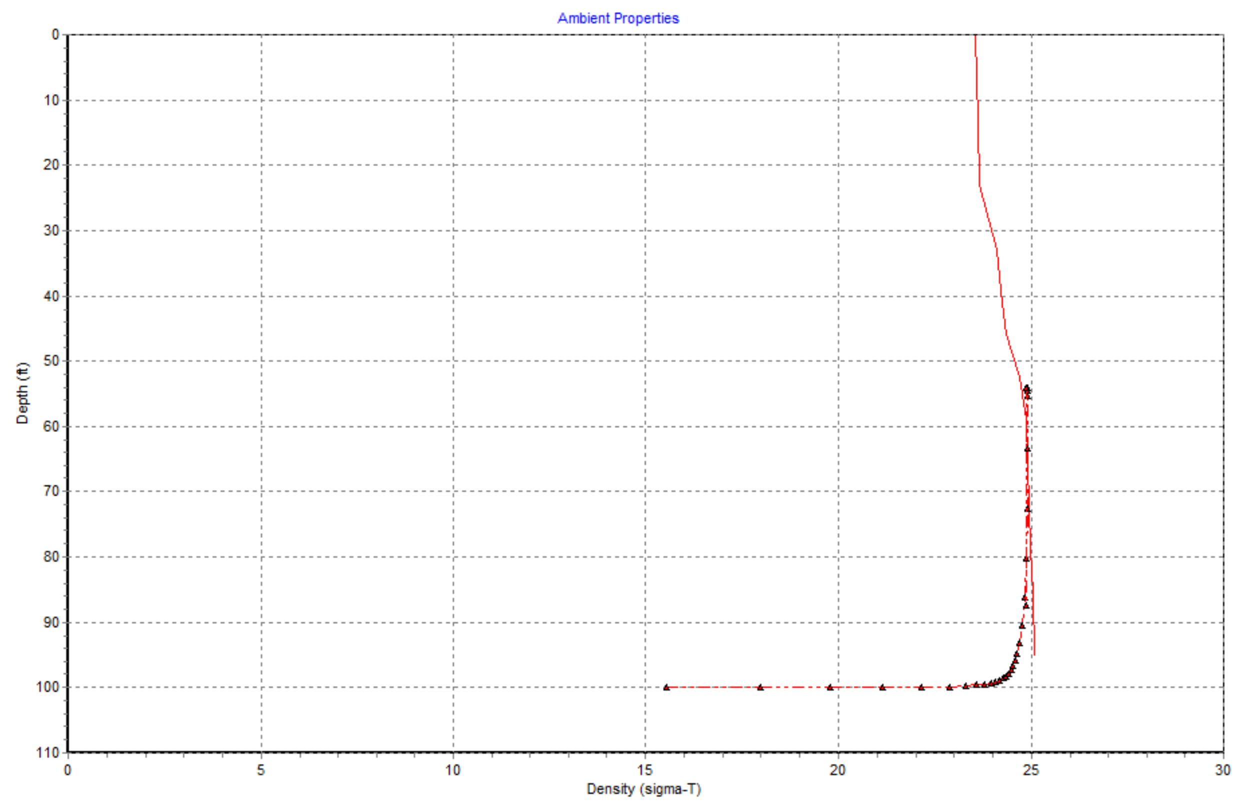

Figure B.5.2: Plumes 18b solution of vertical density profile for discharges of 13 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt. and 5 mgd of brine from the Doheny Desalination Project with a brine salinity of 67 ppt. Discharge effluent density shown as black triangles. Ambient water mass density profile shown as solid red line.

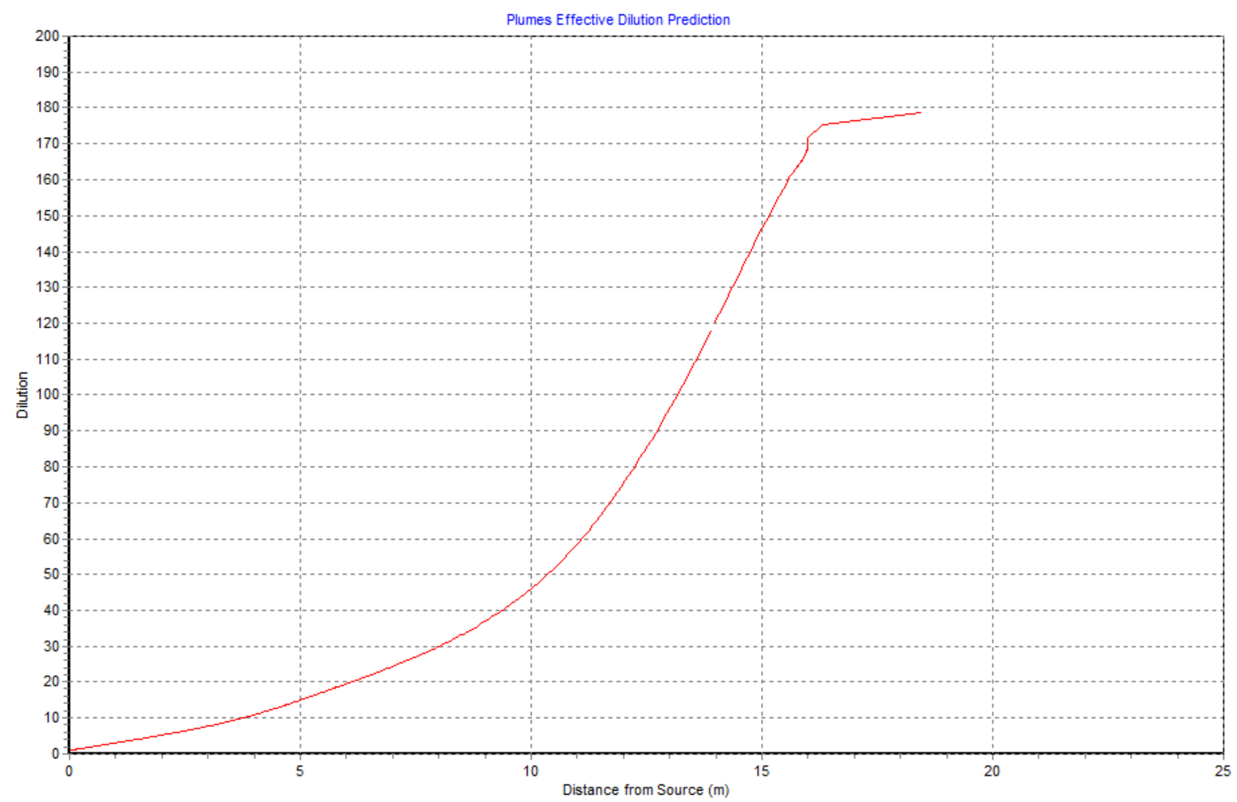

Figure B.5.3: Plumes 18b solution of effective (bulk average) dilution as a function of vertical distance from the point of discharge for discharges of 13 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt. and 5 mgd of brine from the Doheny Desalination Project with a brine salinity of 67 ppt.

B.6: Plumes 18b Results for SJCOO discharges of 13 mgd Wastewater and 5 mgd Brine:

Contents of the memo box (may not be current and must be updated manually)
Project "C:\Plumes18\SJCOO_WW8mgd_b5mgd_T-1"
memo
SJCOO discharging 8 mgd wastewater and 5 mgd brine

Model configuration items checked:
Channel width (m) 100
Start case for graphs 1
Max detailed graphs 10 (limits plots that can overflow memory)
Elevation Projection Plane (deg) 0
Shore vector (m,deg) not checked
Bacteria model : Mancini (1978) coliform model
PDS sfc. model heat transfer : Medium
Equation of State : S, T
Similarity Profile : Default profile (k=2.0, ...)
Diffuser port contraction coefficient 1
Light absorption coefficient 0.16
Farfield increment (m) 200
UM3 aspiration coefficient 0.1
Output file: text output tab
Output each ?? steps 10
Maximum dilution reported 1000
Text output format : Standard
Max vertical reversals : to max rise or fall

/ UM3. 1/10/2019 2:20:30 PM
Case 1; ambient file C:\Plumes18\SJCOO_WW8mgd_b5mgd_T-1.001.db; Diffuser table record 1: ----------------------------------

Ambient Table:

Depth	Amb-cur	Amb-dir	Amb-sal	Amb-tem	Amb-pol	Decay	Far-spd	Far-dir	Disprsn	Density
m	m/s	deg	psu	C	kg/kg	s-1	m/s	deg	m0.67/s2	sigma-T
0.0	0.0	0.0	33.38	19.97	0.0	0.0	0.0	0.0	0.0003	23.55719
2.000	0.0	0.0	33.39	19.87	0.0	0.0	0.0	0.0	0.0003	23.59288
4.000	0.0	0.0	33.40	19.81	0.0	0.0	0.0	0.0	0.0003	23.61484
7.000	0.0	0.0	33.41	19.64	0.0	0.0	0.0	0.0	0.0003	23.67096
10.00	0.0	0.0	33.40	17.76	0.0	0.0	0.0	0.0	0.0003	24.12888
12.00	0.0	0.0	33.38	17.34	0.0	0.0	0.0	0.0	0.0003	24.21549
14.00	0.0	0.0	33.38	16.73	0.0	0.0	0.0	0.0	0.0003	24.35932
16.00	0.0	0.0	33.38	15.17	0.0	0.0	0.0	0.0	0.0003	24.71340
18.00	0.0	0.0	33.37	14.37	0.0	0.0	0.0	0.0	0.0003	24.87044
20.00	0.0	0.0	33.35	14.18	0.0	0.0	0.0	0.0	0.0003	24.89660
22.00	0.0	0.0	33.33	13.83	0.0	0.0	0.0	0.0	0.0003	24.95172
24.00	0.0	0.0	33.32	13.75	0.0	0.0	0.0	0.0	0.0003	24.96707
26.00	0.0	0.0	33.36	13.42	0.0	0.0	0.0	0.0	0.0003	25.06459
29.00	0.0	0.0	33.36	13.39	0.0	0.0	0.0	0.0	0.0003	25.07096

Diffuser table:

P-dia	Ver angl	H-Angle	SourceX	SourceY	Ports	MZ-dis	Isoplth	P-depth	Ttl-flo	Eff-sal	Temp	Polutnt
(in)	(deg)	(deg)	(ft)	(ft)	()	(ft)	(concent)	(ft)	(MGD)	(psu)	(C)	(ppm)
3.0500	0.0	0.0	0.0	0.0	125.00	1000.0	0.0	100.00	13.000	25.770	20.660	25770.0

Simulation:
Froude No: 12.95; Strat No: 5.17E-4; Spcg No: 14.39; k: 96666.1; eff den (sigmaT) 17.61415; eff vel 0.967(m/s);
Current is very small, flow regime may be transient.

Step	Depth (ft)	Amb-cur (m/s)	P-dia (in)	Eff-sal (psu)	Polutnt (ppm)	Dilutn ()	x-posn (m)	y-posn (m)	Iso dia (m)
0	100.0	1.000E-5	3.050	25.77	25770.0	1.000	0.0	0.0	0.07747;
1	100.00	0.0	3.073	25.88	25389.9	1.015	0.00937	0.0	0.07805; bottom hit;
10	100.00	0.0	3.661	27.10	21270.5	1.212	0.109	0.0	0.09299;
20	100.00	0.0	4.460	28.23	17468.3	1.475	0.243	0.0	0.1133;
30	100.00	0.0	5.435	29.15	14342.9	1.797	0.406	0.0	0.1380;
40	99.99	0.0	6.622	29.91	11774.7	2.189	0.604	0.0	0.1682;
50	99.99	0.0	8.068	30.53	9665.1	2.666	0.845	0.0	0.2049;
60	99.97	0.0	9.828	31.04	7932.6	3.249	1.138	0.0	0.2496;
70	99.95	0.0	11.97	31.45	6510.1	3.958	1.492	0.0	0.3040;
80	99.90	0.0	14.55	31.80	5343.5	4.823	1.917	0.0	0.3696;
90	99.83	0.0	17.21	32.04	4510.5	5.713	2.334	0.0	0.4372;
100	99.76	0.0	19.69	32.21	3928.3	6.560	2.699	0.0	0.5002;
110	99.67	0.0	22.05	32.34	3488.9	7.386	3.028	0.0	0.5601;
120	99.58	0.0	24.32	32.44	3140.6	8.206	3.330	0.0	0.6178;
130	99.47	0.0	26.53	32.53	2854.6	9.028	3.614	0.0	0.6739;
140	99.36	0.0	28.69	32.60	2613.7	9.860	3.883	0.0	0.7286;
150	99.23	0.0	30.79	32.66	2406.7	10.71	4.142	0.0	0.7821;
160	99.09	0.0	32.85	32.71	2226.1	11.58	4.393	0.0	0.8344;
170	98.94	0.0	34.87	32.76	2066.2	12.47	4.640	0.0	0.8856;
180	98.77	0.0	36.84	32.80	1922.9	13.40	4.885	0.0	0.9357;
190	98.58	0.0	38.78	32.84	1793.1	14.37	5.129	0.0	0.9849;
200	98.37	0.0	40.68	32.87	1674.1	15.39	5.377	0.0	1.0333;
210	98.13	0.0	42.57	32.90	1563.8	16.48	5.629	0.0	1.0812;
218	97.92	0.0	44.07	32.93	1480.6	17.41	5.836	0.0	1.1193; merging;
220	97.86	0.0	44.44	32.93	1461.0	17.64	5.888	0.0	1.1287;
230	97.55	0.0	46.26	32.96	1368.5	18.83	6.158	0.0	1.1749;
240	97.19	0.0	48.08	32.99	1281.5	20.11	6.443	0.0	1.2213;
250	96.77	0.0	49.97	33.01	1197.5	21.52	6.746	0.0	1.2692;
260	96.26	0.0	51.98	33.04	1114.8	23.12	7.073	0.0	1.3202;
270	95.64	0.0	54.19	33.06	1032.1	24.97	7.430	0.0	1.3765;
280	94.86	0.0	56.76	33.08	947.8	27.19	7.823	0.0	1.4417;
290	93.88	0.0	59.89	33.11	860.7	29.94	8.263	0.0	1.5213;
300	92.61	0.0	63.94	33.14	769.6	33.48	8.763	0.0	1.6241;
310	90.89	0.0	69.51	33.16	673.5	38.26	9.339	0.0	1.7654;
320	88.49	0.0	77.70	33.19	571.8	45.07	10.02	0.0	1.9736;
330	85.18	0.0	90.09	33.22	470.2	54.81	10.79	0.0	2.2883;
340	81.30	0.0	108.3	33.25	385.7	66.81	11.54	0.0	2.7506;
350	76.70	0.0	142.8	33.26	316.4	81.44	12.34	0.0	3.6277;
353	75.14	0.0	158.3	33.27	298.2	86.42	12.61	0.0	4.0211; trap level;
360	71.13	0.0	205.9	33.27	259.6	99.27	13.30	0.0	5.2296;
370	65.96	0.0	341.8	33.28	221.3	116.5	14.29	0.0	8.6817;
375	65.09	0.0	413.3	33.28	215.0	119.9	14.50	0.0	10.497; begin overlap;
380	64.53	0.0	476.2	33.29	211.6	121.8	14.64	0.0	12.096;
390	63.85	0.0	590.2	33.29	208.0	123.9	14.85	0.0	14.990;
400	63.46	0.0	699.5	33.29	205.6	125.3	15.00	0.0	17.766;
410	63.21	0.0	807.2	33.29	203.8	126.5	15.10	0.0	20.503;
420	63.04	0.0	913.8	33.29	202.2	127.5	15.18	0.0	23.210;
430	62.93	0.0	1016.2	33.29	200.8	128.3	15.25	0.0	25.812;
440	62.90	0.0	1064.4	33.29	199.8	129.0	15.26	0.0	27.037;
450	62.90	0.0	1078.8	33.29	198.8	129.6	15.26	0.0	27.401;

460	62.90	0.0	1089.5	33.29	197.9	130.2	15.26	0.0	27.672;
470	62.90	0.0	1099.9	33.29	196.9	130.9	15.26	0.0	27.936; local maximum rise or fall;
477	62.96	0.0	1118.2	33.29	192.5	133.9	15.78	0.0	28.401;

Horiz plane projections in effluent direction: radius(m): 0.0; CL(m): 4.8084
Lmz(m): 4.8084
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Rate sec-1 0.0 dy-1 0.0 kt: 0.0 Amb Sal 33.3550
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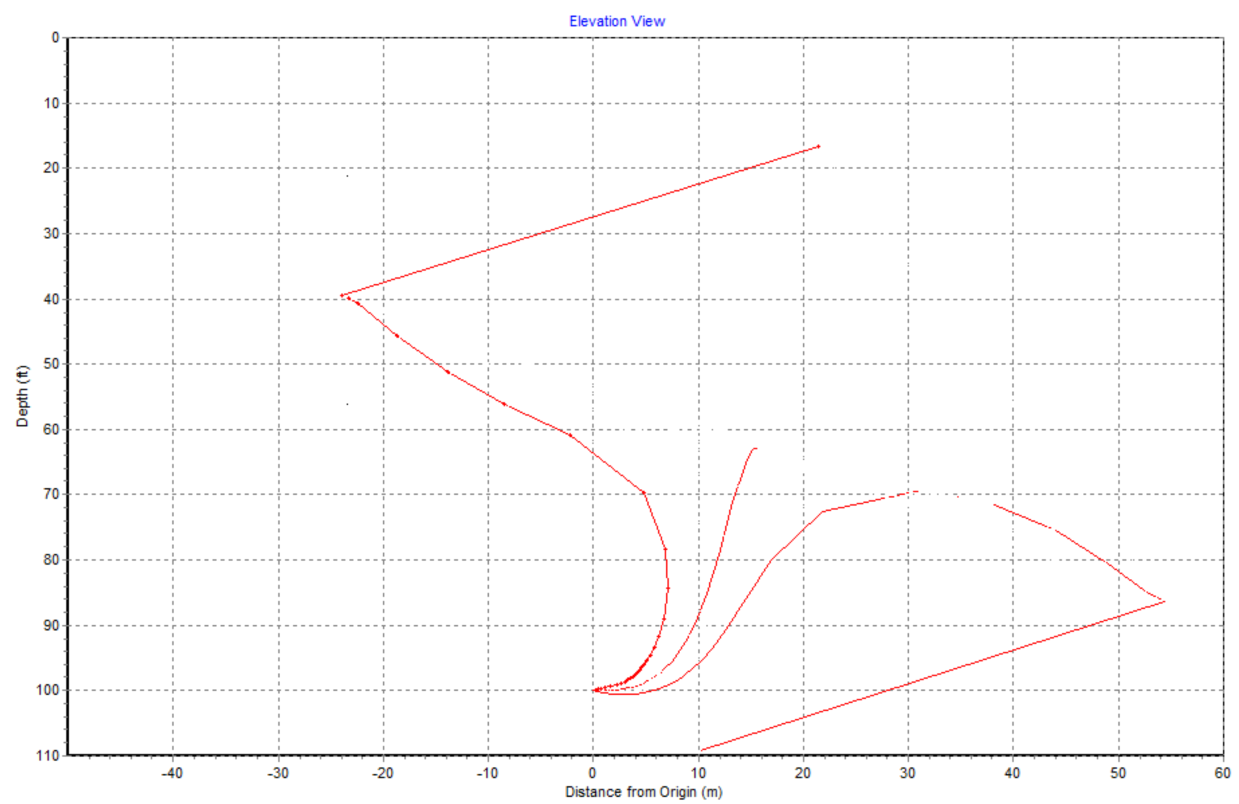

Figure B.6.1: Plumes 18b solution of discharge plume trajectories for discharges of 8 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt., and 5 mgd of brine from the Doheny Desalination Project with a brine salinity of 67 ppt. ZID is defined by the maximum horizontal excursion of trajectories from the origin. From the maximum horizontal spreading of the plume, the ZID extends from *X* = -24 m to *X* = +54 m so that ZID = 78 m

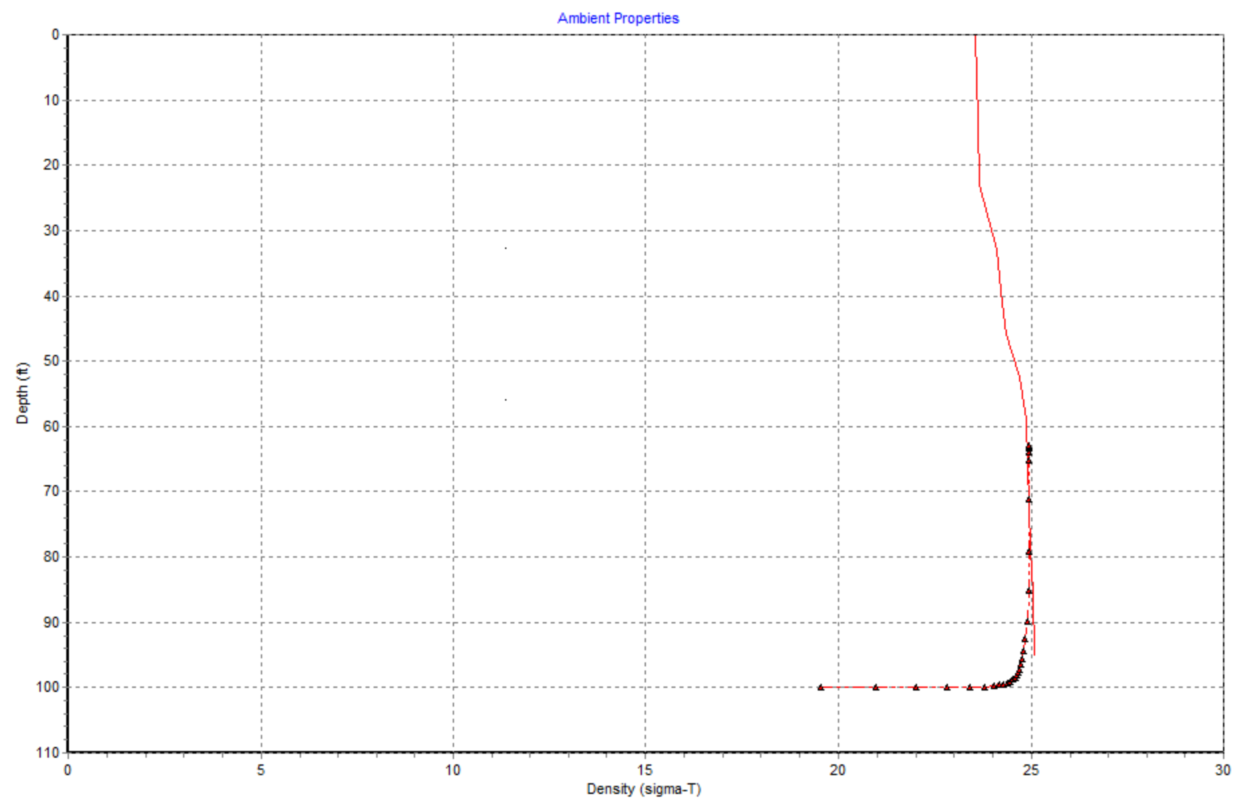

Figure B.6.2: Plumes 18b solution of vertical density profile for discharges of 8 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt. and 5 mgd of brine from the Doheny Desalination Project with a brine salinity of 67 ppt. Discharge effluent density shown as black triangles. Ambient water mass density profile shown as solid red line.

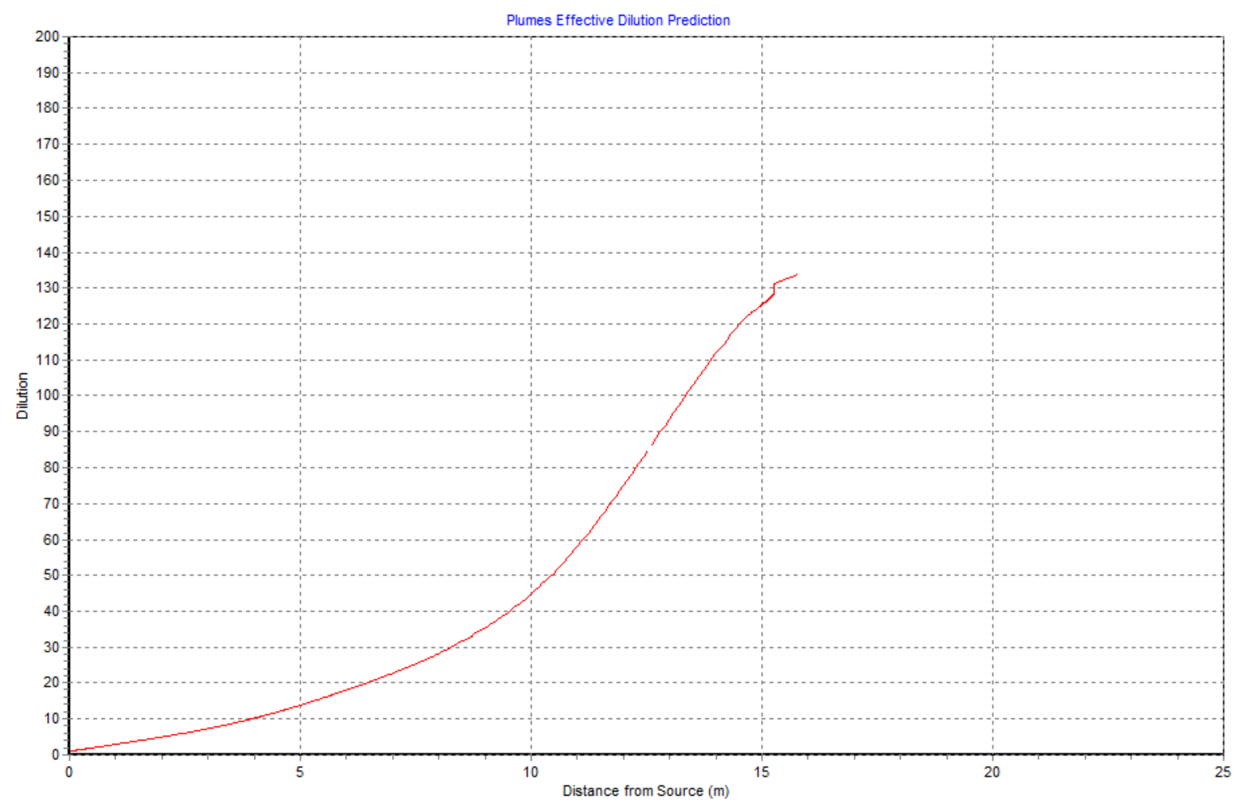

Figure B.6.3: Plumes 18b solution of effective (bulk average) dilution as a function of vertical distance from the point of discharge for discharges of 8 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt. and 5 mgd of brine from the Doheny Desalination Project with a brine salinity of 67 ppt.

APPENDIX-C: Results for Dense (Negatively Buoyant) Combined Discharges of Brine and Wastewater

C.1: Plumes 18b Results for SJCOO discharges of 13 mgd Wastewater and 15 mgd Brine:

Contents of the memo box (may not be current and must be updated manually)
Project "C:\Plumes18\SJCOO_WW13mgd_b15mgd_T-4"
memomemomemo
SJCOO discharging 13 mgd wastewater and 15 mgd brine

Model configuration items checked:
Channel width (m) 100
Start case for graphs 1
Max detailed graphs 10 (limits plots that can overflow memory)
Elevation Projection Plane (deg) 0
Shore vector (m,deg) not checked
Bacteria model : Mancini (1978) coliform model
PDS sfc. model heat transfer : Medium
Equation of State : S, T
Similarity Profile : Default profile (k=2.0, ...)
Diffuser port contraction coefficient 1
Light absorption coefficient 0.16
Farfield increment (m) 200
UM3 aspiration coefficient 0.1
Output file: text output tab
Output each ?? steps 1
Maximum dilution reported 1000
Text output format : Standard
Max vertical reversals : to max rise or fall

/ UM3. 1/14/2019 11:25:09 AM
Case 1; ambient file C:\Plumes18\SJCOO_WW13mgd_b15mgd_T-4.001.db; Diffuser table record 1: ----------------------------------

Ambient Table:

Depth	Amb-cur	Amb-dir	Amb-sal	Amb-tem	Amb-pol	Decay	Far-spd	Far-dir	Disprsn	Density
m	m/s	deg	psu	C	kg/kg	s-1	m/s	deg	m0.67/s2	sigma-T
0.0	0.0	0.0	33.38	19.97	0.0	0.0	0.0	0.0	0.0003	23.55719
2.000	0.0	0.0	33.39	19.87	0.0	0.0	0.0	0.0	0.0003	23.59288
4.000	0.0	0.0	33.40	19.81	0.0	0.0	0.0	0.0	0.0003	23.61484
7.000	0.0	0.0	33.41	19.64	0.0	0.0	0.0	0.0	0.0003	23.67096
10.00	0.0	0.0	33.40	17.76	0.0	0.0	0.0	0.0	0.0003	24.12888
12.00	0.0	0.0	33.38	17.34	0.0	0.0	0.0	0.0	0.0003	24.21549
14.00	0.0	0.0	33.38	16.73	0.0	0.0	0.0	0.0	0.0003	24.35932
16.00	0.0	0.0	33.38	15.17	0.0	0.0	0.0	0.0	0.0003	24.71340
18.00	0.0	0.0	33.37	14.37	0.0	0.0	0.0	0.0	0.0003	24.87044
20.00	0.0	0.0	33.35	14.18	0.0	0.0	0.0	0.0	0.0003	24.89660
22.00	0.0	0.0	33.33	13.83	0.0	0.0	0.0	0.0	0.0003	24.95172
24.00	0.0	0.0	33.32	13.75	0.0	0.0	0.0	0.0	0.0003	24.96707
26.00	0.0	0.0	33.36	13.42	0.0	0.0	0.0	0.0	0.0003	25.06459
31.85	0.0	0.0	33.36	13.39	0.0	0.0	0.0	0.0	0.0003	25.07201

Diffuser table:
P-diaVer angl H-Angle SourceX SourceY Ports MZ-dis Isoplth P-depth Ttl-flo Eff-sal Temp Polutnt

(in)	(deg)	(deg)	(ft)	(ft)	()	(ft)(concent)	(ft)	(MGD)	(psu)	(C)	(ppm)
3.0500	0.0	0.0	0.0	0.0	125.00	1000.0	0.0	100.00	28.000	35.890	20.660 35890.0

Simulation:
Froude No: -165.0; Strat No:-0.01789; Spcg No: 14.39; k: 2.08E+5; eff den (sigmaT) 25.28520; eff vel 2.082(m/s);
Current is very small, flow regime may be transient.

Step	Depth (ft)	Amb-cur (m/s)	P-dia (in)	Eff-sal (psu)	Polutnt (ppm)	Dilutn ()	x-posn (ft)	y-posn (ft)	Iso dia (m)
0	100.0	1.000E-5	3.050	35.89	35890.0	1.000	0.0	0.0	0.07747;
1	100.0	0.0	3.099	35.81	34769.0	1.032	0.0199	0.0	0.07871;
2	100.0	0.0	3.180	35.76	34087.2	1.053	0.0329	0.0	0.08076;
3	100.0	0.0	3.243	35.72	33418.9	1.074	0.0461	0.0	0.08238;
4	100.0	0.0	3.308	35.67	32763.6	1.095	0.0596	0.0	0.08403;
5	100.0	0.0	3.374	35.62	32121.2	1.117	0.0733	0.0	0.08571;
6	100.0	0.0	3.442	35.58	31491.3	1.140	0.0874	0.0	0.08742;
7	100.0	0.0	3.511	35.54	30873.8	1.162	0.102	0.0	0.08917;
8	100.0	0.0	3.581	35.49	30268.4	1.186	0.116	0.0	0.09095;
9	100.0	0.0	3.652	35.45	29674.9	1.209	0.131	0.0	0.09277;
10	100.0	0.0	3.725	35.41	29093.0	1.234	0.146	0.0	0.09463;
11	100.0	0.0	3.800	35.37	28522.6	1.258	0.162	0.0	0.09652;
12	100.0	0.0	3.876	35.33	27963.3	1.283	0.178	0.0	0.09845;
13	100.0	0.0	3.954	35.29	27414.9	1.309	0.194	0.0	0.1004;
14	100.0	0.0	4.033	35.26	26877.4	1.335	0.210	0.0	0.1024;
15	100.0	0.0	4.113	35.22	26350.3	1.362	0.227	0.0	0.1045;
16	100.0	0.0	4.196	35.18	25833.6	1.389	0.244	0.0	0.1066;
17	100.0	0.0	4.279	35.15	25327.0	1.417	0.261	0.0	0.1087;
18	100.0	0.0	4.365	35.11	24830.4	1.445	0.279	0.0	0.1109;
19	100.0	0.0	4.452	35.08	24343.5	1.474	0.297	0.0	0.1131;
20	100.0	0.0	4.541	35.04	23866.1	1.504	0.316	0.0	0.1154;
21	100.0	0.0	4.632	35.01	23398.1	1.534	0.335	0.0	0.1177;
22	100.0	0.0	4.725	34.98	22939.3	1.565	0.354	0.0	0.1200;
23	100.0	0.0	4.819	34.95	22489.5	1.596	0.373	0.0	0.1224;
24	100.0	0.0	4.916	34.92	22048.5	1.628	0.393	0.0	0.1249;
25	100.0	0.0	5.014	34.89	21616.1	1.660	0.414	0.0	0.1274;
26	100.0	0.0	5.114	34.86	21192.2	1.694	0.435	0.0	0.1299;
27	100.0	0.0	5.217	34.83	20776.6	1.727	0.456	0.0	0.1325;
28	100.0	0.0	5.321	34.80	20369.2	1.762	0.478	0.0	0.1352;
29	100.0	0.0	5.427	34.77	19969.8	1.797	0.500	0.0	0.1379;
30	100.0	0.0	5.536	34.74	19578.2	1.833	0.522	0.0	0.1406;
31	100.0	0.0	5.647	34.71	19194.3	1.870	0.545	0.0	0.1434;
32	100.0	0.0	5.760	34.69	18817.9	1.907	0.569	0.0	0.1463;
33	100.0	0.0	5.875	34.66	18448.8	1.945	0.592	0.0	0.1492;
34	100.0	0.0	5.992	34.64	18087.1	1.984	0.617	0.0	0.1522;
35	100.0	0.0	6.112	34.61	17732.4	2.024	0.642	0.0	0.1552;
36	100.0	0.0	6.234	34.59	17384.7	2.064	0.667	0.0	0.1584;
37	100.0	0.0	6.359	34.56	17043.7	2.106	0.693	0.0	0.1615;
38	100.0	0.0	6.486	34.54	16709.5	2.148	0.719	0.0	0.1648;
39	100.0	0.0	6.616	34.52	16381.8	2.191	0.746	0.0	0.1680;
40	100.0	0.0	6.748	34.49	16060.6	2.235	0.774	0.0	0.1714;
41	100.0	0.0	6.883	34.47	15745.7	2.279	0.802	0.0	0.1748;
42	100.0	0.0	7.021	34.45	15436.9	2.325	0.830	0.0	0.1783;
43	100.0	0.0	7.161	34.43	15134.2	2.371	0.859	0.0	0.1819;
44	100.0	0.0	7.305	34.41	14837.4	2.419	0.889	0.0	0.1855;
45	100.0	0.0	7.451	34.39	14546.4	2.467	0.919	0.0	0.1893;
46	100.0	0.0	7.600	34.37	14261.2	2.517	0.950	0.0	0.1930;

47	100.0	0.0	7.752	34.35	13981.5	2.567	0.982	0.0	0.1969;
48	100.0	0.0	7.907	34.33	13707.3	2.618	1.014	0.0	0.2008;
49	100.0	0.0	8.065	34.31	13438.5	2.671	1.047	0.0	0.2049;
50	100.0	0.0	8.226	34.29	13175.0	2.724	1.080	0.0	0.2089;
51	100.0	0.0	8.391	34.27	12916.7	2.779	1.114	0.0	0.2131;
52	100.0	0.0	8.559	34.25	12663.4	2.834	1.149	0.0	0.2174;
53	100.0	0.0	8.730	34.24	12415.0	2.891	1.185	0.0	0.2217;
54	100.0	0.0	8.904	34.22	12171.6	2.949	1.221	0.0	0.2262;
55	100.0	0.0	9.083	34.20	11932.9	3.008	1.258	0.0	0.2307;
56	100.0	0.0	9.264	34.19	11698.9	3.068	1.295	0.0	0.2353;
57	100.0	0.0	9.450	34.17	11469.5	3.129	1.334	0.0	0.2400;
58	100.0	0.0	9.639	34.15	11244.6	3.192	1.373	0.0	0.2448;
59	100.0	0.0	9.831	34.14	11024.1	3.256	1.413	0.0	0.2497;
60	100.0	0.0	10.03	34.12	10807.9	3.321	1.454	0.0	0.2547;
61	100.0	0.0	10.23	34.11	10596.0	3.387	1.495	0.0	0.2598;
62	100.0	0.0	10.43	34.09	10388.2	3.455	1.538	0.0	0.2650;
63	100.0	0.0	10.64	34.08	10184.5	3.524	1.581	0.0	0.2703;
64	100.0	0.0	10.85	34.07	9984.8	3.594	1.625	0.0	0.2757;
65	100.0	0.0	11.07	34.05	9789.0	3.666	1.670	0.0	0.2812;
66	100.0	0.0	11.29	34.04	9597.0	3.740	1.716	0.0	0.2868;
67	100.0	0.0	11.52	34.03	9408.8	3.815	1.763	0.0	0.2926;
68	100.0	0.0	11.75	34.01	9224.3	3.891	1.811	0.0	0.2984;
69	100.0	0.0	11.98	34.00	9043.4	3.969	1.859	0.0	0.3044;
70	100.0	0.0	12.22	33.99	8866.1	4.048	1.909	0.0	0.3105;
71	100.0	0.0	12.47	33.98	8692.2	4.129	1.960	0.0	0.3167;
72	100.0	0.0	12.72	33.96	8521.8	4.212	2.011	0.0	0.3230;
73	100.0	0.0	12.97	33.95	8354.7	4.296	2.064	0.0	0.3295;
74	100.0	0.0	13.23	33.94	8190.8	4.382	2.118	0.0	0.3361;
75	100.0	0.0	13.50	33.93	8030.2	4.469	2.173	0.0	0.3428;
76	100.0	0.0	13.77	33.92	7872.8	4.559	2.229	0.0	0.3497;
77	100.0	0.0	14.04	33.91	7718.4	4.650	2.286	0.0	0.3567;
78	100.0	0.0	14.32	33.90	7567.0	4.743	2.344	0.0	0.3638;
79	100.0	0.0	14.61	33.89	7418.6	4.838	2.403	0.0	0.3711;
80	100.0	0.0	14.90	33.88	7273.2	4.935	2.464	0.0	0.3785;
81	100.0	0.0	15.20	33.87	7130.5	5.033	2.525	0.0	0.3861;
82	100.0	0.0	15.50	33.86	6990.7	5.134	2.588	0.0	0.3938;
83	100.0	0.0	15.81	33.85	6853.6	5.237	2.653	0.0	0.4017;
84	100.0	0.0	16.13	33.84	6719.2	5.341	2.718	0.0	0.4097;
85	100.0	0.0	16.45	33.83	6587.5	5.448	2.785	0.0	0.4179;
86	100.0	0.0	16.78	33.82	6458.3	5.557	2.853	0.0	0.4262;
87	100.0	0.0	17.12	33.81	6331.7	5.668	2.923	0.0	0.4348;
88	100.0	0.0	17.46	33.80	6207.5	5.782	2.993	0.0	0.4435;
89	100.0	0.0	17.81	33.79	6085.8	5.897	3.066	0.0	0.4523;
90	100.0	0.0	18.16	33.78	5966.4	6.015	3.140	0.0	0.4614;
91	100.0	0.0	18.53	33.78	5849.4	6.136	3.215	0.0	0.4706;
92	100.0	0.0	18.90	33.77	5734.7	6.258	3.291	0.0	0.4800;
93	100.0	0.0	19.28	33.76	5622.3	6.384	3.370	0.0	0.4896;
94	100.0	0.0	19.66	33.75	5512.0	6.511	3.450	0.0	0.4994;
95	100.0	0.0	20.06	33.74	5404.0	6.641	3.531	0.0	0.5094;
96	100.0	0.0	20.46	33.74	5298.0	6.774	3.614	0.0	0.5196;
97	100.0	0.0	20.87	33.73	5194.1	6.910	3.699	0.0	0.5300;
98	100.0	0.0	21.28	33.72	5092.3	7.048	3.785	0.0	0.5406;
99	100.0	0.0	21.71	33.71	4992.4	7.189	3.873	0.0	0.5514;
100	100.0	0.0	22.14	33.71	4894.5	7.333	3.963	0.0	0.5624;
101	100.0	0.0	22.59	33.70	4798.5	7.479	4.055	0.0	0.5737;
102	100.0	0.0	23.04	33.69	4704.4	7.629	4.148	0.0	0.5851;

103	100.0	0.0	23.50	33.69	4612.2	7.782	4.243	0.0	0.5968;	
104	100.0	0.0	23.97	33.68	4521.7	7.937	4.341	0.0	0.6088;	
105	100.0	0.0	24.45	33.68	4433.1	8.096	4.440	0.0	0.6210;	
106	100.0	0.0	24.94	33.67	4346.2	8.258	4.541	0.0	0.6334;	
107	100.0	0.0	25.43	33.66	4260.9	8.423	4.644	0.0	0.6460;	
108	100.0	0.0	25.94	33.66	4177.4	8.592	4.749	0.0	0.6590;	
109	100.0	0.0	26.46	33.65	4095.5	8.763	4.856	0.0	0.6721;	
110	100.0	0.0	26.99	33.65	4015.2	8.939	4.966	0.0	0.6856;	
111	100.0	0.0	27.53	33.64	3936.4	9.117	5.077	0.0	0.6993;	
112	100.0	0.0	28.08	33.63	3859.2	9.300	5.191	0.0	0.7133;	
113	100.0	0.0	28.64	33.63	3783.6	9.486	5.307	0.0	0.7276;	
114	100.0	0.0	29.22	33.62	3709.4	9.675	5.426	0.0	0.7421;	
115	100.0	0.0	29.80	33.62	3636.6	9.869	5.547	0.0	0.7569;	
116	100.0	0.0	30.40	33.61	3565.3	10.07	5.670	0.0	0.7721;	
117	100.0	0.0	31.00	33.61	3495.4	10.27	5.795	0.0	0.7875;	
118	100.0	0.0	31.62	33.60	3426.9	10.47	5.923	0.0	0.8033;	
119	100.0	0.0	32.26	33.60	3359.7	10.68	6.054	0.0	0.8193;	
120	100.0	0.0	32.90	33.60	3293.8	10.90	6.187	0.0	0.8357;	
121	100.0	0.0	33.56	33.59	3229.2	11.11	6.323	0.0	0.8524;	
122	100.0	0.0	34.23	33.59	3165.9	11.34	6.462	0.0	0.8695;	
123	100.0	0.0	34.92	33.58	3103.8	11.56	6.603	0.0	0.8869;	
124	100.0	0.0	35.61	33.58	3043.0	11.79	6.747	0.0	0.9046;	
125	100.0	0.0	36.33	33.57	2983.3	12.03	6.894	0.0	0.9227;	
126	100.0	0.0	37.05	33.57	2924.8	12.27	7.044	0.0	0.9411;	
127	100.0	0.0	37.79	33.57	2867.4	12.52	7.197	0.0	0.9600;	
128	100.0	0.0	38.55	33.56	2811.2	12.77	7.352	0.0	0.9792;	
129	100.0	0.0	39.32	33.56	2756.1	13.02	7.511	0.0	0.9987;	
130	100.0	0.0	40.11	33.55	2702.0	13.28	7.673	0.0	1.0187;	
131	100.0	0.0	40.91	33.55	2649.1	13.55	7.839	0.0	1.0391;	
132	100.1	0.0	41.73	33.55	2597.1	13.82	8.007	0.0	1.0599;	
133	100.1	0.0	42.56	33.54	2546.2	14.10	8.179	0.0	1.0810;	
134	100.1	0.0	43.41	33.54	2496.3	14.38	8.354	0.0	1.1027;	
135	100.1	0.0	44.28	33.54	2447.3	14.67	8.533	0.0	1.1247;	merging;
136	100.1	0.0	45.17	33.53	2399.3	14.96	8.727	0.0	1.1474;	
137	100.1	0.0	46.12	33.53	2352.3	15.26	8.937	0.0	1.1716;	
138	100.1	0.0	47.12	33.53	2306.2	15.56	9.159	0.0	1.1968;	
139	100.1	0.0	48.16	33.52	2260.9	15.87	9.394	0.0	1.2232;	
140	100.1	0.0	49.24	33.52	2216.6	16.19	9.640	0.0	1.2507;	
141	100.1	0.0	50.36	33.52	2173.1	16.52	9.899	0.0	1.2792;	
142	100.1	0.0	51.53	33.51	2130.5	16.85	10.17	0.0	1.3088;	
143	100.1	0.0	52.74	33.51	2088.8	17.18	10.45	0.0	1.3397;	
144	100.1	0.0	54.00	33.51	2047.8	17.53	10.75	0.0	1.3717;	
145	100.1	0.0	55.31	33.50	2007.6	17.88	11.05	0.0	1.4049;	
146	100.1	0.0	56.67	33.50	1968.3	18.23	11.37	0.0	1.4394;	
147	100.2	0.0	58.08	33.50	1929.7	18.60	11.71	0.0	1.4753;	
148	100.2	0.0	59.55	33.50	1891.8	18.97	12.05	0.0	1.5126;	
149	100.2	0.0	61.08	33.49	1854.7	19.35	12.42	0.0	1.5513;	
150	100.2	0.0	62.66	33.49	1818.4	19.74	12.79	0.0	1.5916;	
151	100.2	0.0	64.31	33.49	1782.7	20.13	13.18	0.0	1.6335;	
152	100.2	0.0	66.02	33.49	1747.8	20.53	13.58	0.0	1.6770;	
153	100.3	0.0	67.80	33.48	1713.5	20.95	14.00	0.0	1.7222;	
154	100.3	0.0	69.66	33.48	1679.9	21.36	14.44	0.0	1.7693;	
155	100.3	0.0	71.59	33.48	1647.0	21.79	14.89	0.0	1.8183;	
156	100.3	0.0	73.59	33.48	1614.7	22.23	15.36	0.0	1.8693;	
157	100.4	0.0	75.68	33.47	1583.0	22.67	15.85	0.0	1.9223;	
158	100.4	0.0	77.86	33.47	1552.0	23.13	16.35	0.0	1.9776;	

159	100.4	0.0	80.12	33.47	1521.5	23.59	16.87	0.0	2.0350;
160	100.5	0.0	82.48	33.47	1491.7	24.06	17.41	0.0	2.0949;
161	100.5	0.0	84.93	33.47	1462.4	24.54	17.97	0.0	2.1572;
162	100.6	0.0	87.48	33.46	1433.8	25.03	18.55	0.0	2.2221;
163	100.6	0.0	90.14	33.46	1405.7	25.53	19.14	0.0	2.2897;
164	100.7	0.0	92.91	33.46	1378.2	26.04	19.76	0.0	2.3599; bottom hit;

Horiz plane projections in effluent direction: radius(m): 0.0; CL(m): 6.0215
Lmz(m): 6.0215
forced entrain 1 0.0 -0.209 2.360 0.996
Rate sec-1 0.0 dy-1 0.0 kt: 0.0 Amb Sal 33.3632
;
11:25:10 AM. amb fills: 4

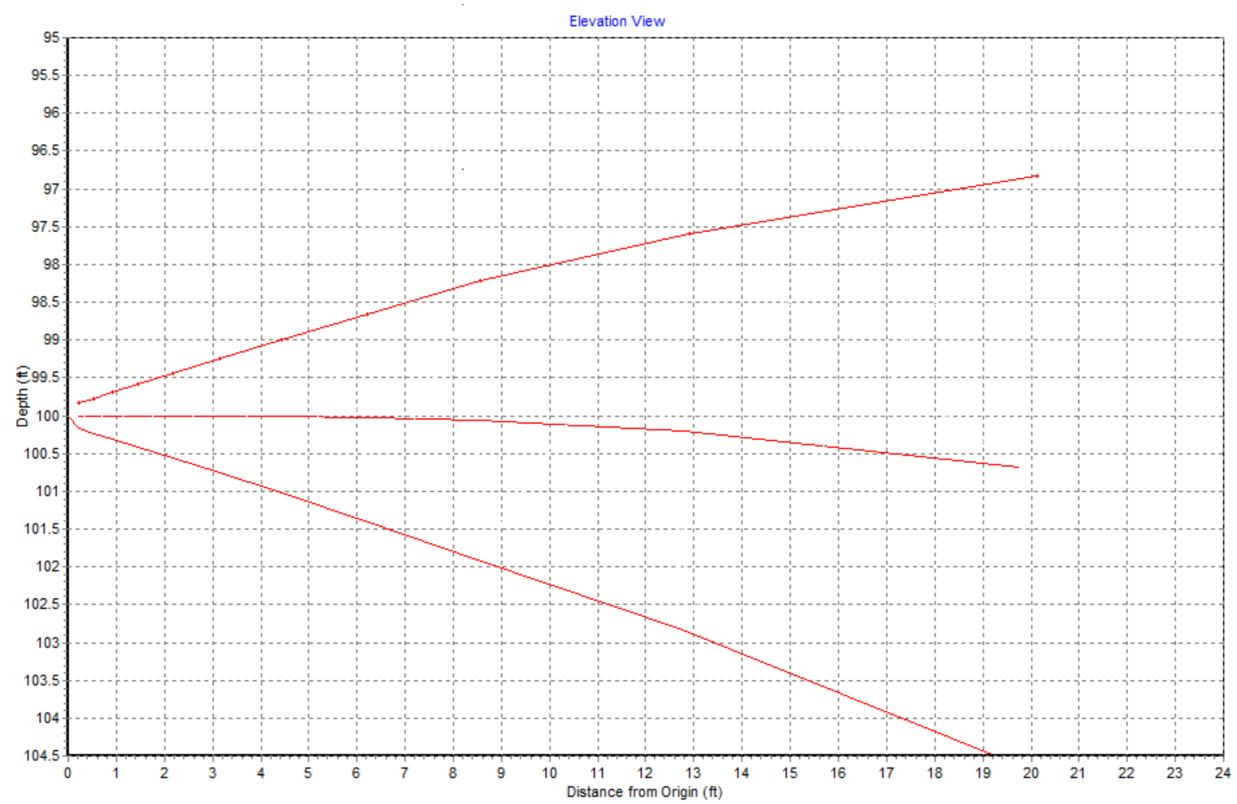

Figure C.1.1: Plumes 18b solution of discharge plume trajectories for discharges of 13 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt., and 15 mgd of brine from the Doheny Desalination Project with a brine salinity of 67 ppt. Depth of maximum rise of the plume is Z = 96.8 ft. at X_a = 20.1 ft from the point of discharge. The centerline of the plume is at an average distance of X_b = 19.76 ft from the point of discharge as the plume begins to impact the bottom at a depth of Z = 104.5 ft.

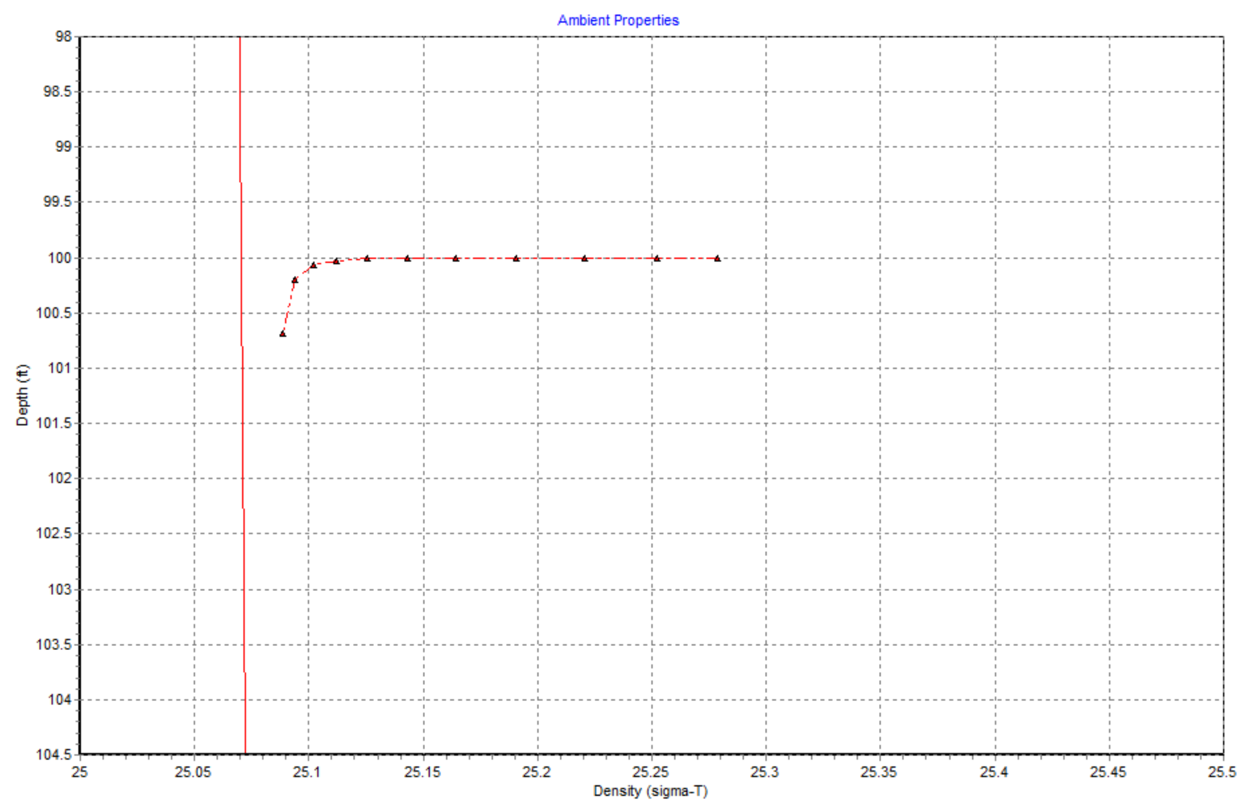

Figure C.1.2: Plumes 18b solution of vertical density profile for discharges of 13 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt. and 15 mgd of brine from the Doheny Desalination Project with a brine salinity of 67 ppt. Discharge effluent density shown as black triangles. Ambient water mass density profile shown as solid red line.

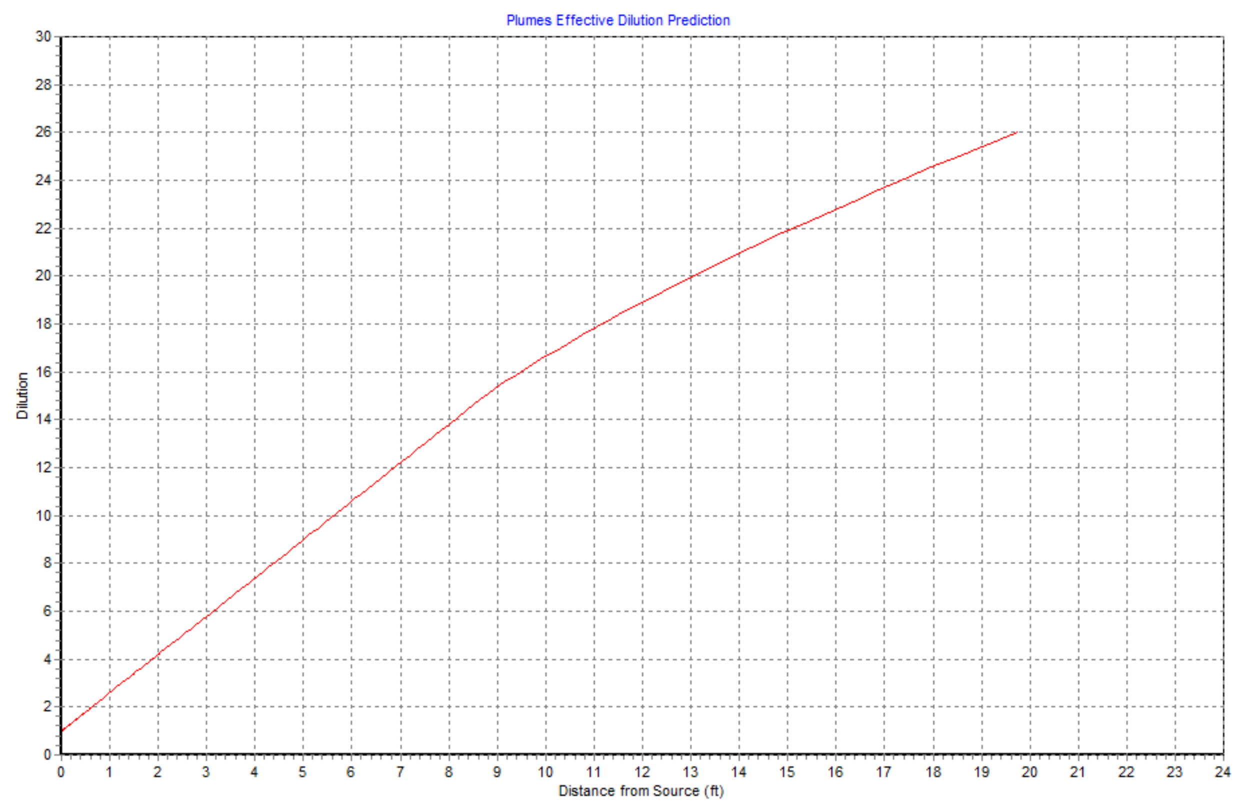

Figure C.1.3: Plumes 18b solution of effective (bulk average) dilution as a function of vertical distance from the point of discharge for discharges of 13 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt. and 15 mgd of brine from the Doheny Desalination Project with a brine salinity of 67 ppt. As the plume begins to impact the bottom, the plume centerline is at a distance of X_b = 19.76 ft from the point of discharge, where the effective dilution reaches S_a = 26.04.

C.2: Plumes 18b Results for SJCOO discharges of 8 mgd Wastewater and 15 mgd Brine:

Contents of the memo box (may not be current and must be updated manually)
Project "C:\Plumes18\SJCOO_WW8mgd_b15mgd_D-1"
memo
SJCOO discharging 8 mgd wastewater and 15 mgd brine

Model configuration items checked:
Channel width (m) 100
Start case for graphs 1
Max detailed graphs 10 (limits plots that can overflow memory)
Elevation Projection Plane (deg) 0
Shore vector (m,deg) not checked
Bacteria model : Mancini (1978) coliform model
PDS sfc. model heat transfer : Medium
Equation of State : S, T
Similarity Profile : Default profile (k=2.0, ...)
Diffuser port contraction coefficient 1
Light absorption coefficient 0.16
Farfield increment (m) 200
UM3 aspiration coefficient 0.1
Output file: text output tab
Output each ?? steps 1
Maximum dilution reported 1000
Text output format : Standard
Max vertical reversals : to max rise or fall

/ UM3. 1/14/2019 1:05:33 PM
Case 1; ambient file C:\Plumes18\SJCOO_WW8mgd_b15mgd_D-1.001.db; Diffuser table record 1: ----------------------------------

Ambient Table:

Depth	Amb-cur	Amb-dir	Amb-sal	Amb-tem	Amb-pol	Decay	Far-spd	Far-dir	Disprsn	Density
m	m/s	deg	psu	C	kg/kg	s-1	m/s	deg	m0.67/s2	sigma-T
0.0	0.0	0.0	33.38	19.97	0.0	0.0	0.0	0.0	0.0003	23.55719
2.000	0.0	0.0	33.39	19.87	0.0	0.0	0.0	0.0	0.0003	23.59288
4.000	0.0	0.0	33.40	19.81	0.0	0.0	0.0	0.0	0.0003	23.61484
7.000	0.0	0.0	33.41	19.64	0.0	0.0	0.0	0.0	0.0003	23.67096
10.00	0.0	0.0	33.40	17.76	0.0	0.0	0.0	0.0	0.0003	24.12888
12.00	0.0	0.0	33.38	17.34	0.0	0.0	0.0	0.0	0.0003	24.21549
14.00	0.0	0.0	33.38	16.73	0.0	0.0	0.0	0.0	0.0003	24.35932
16.00	0.0	0.0	33.38	15.17	0.0	0.0	0.0	0.0	0.0003	24.71340
18.00	0.0	0.0	33.37	14.37	0.0	0.0	0.0	0.0	0.0003	24.87044
20.00	0.0	0.0	33.35	14.18	0.0	0.0	0.0	0.0	0.0003	24.89660
22.00	0.0	0.0	33.33	13.83	0.0	0.0	0.0	0.0	0.0003	24.95172
24.00	0.0	0.0	33.32	13.75	0.0	0.0	0.0	0.0	0.0003	24.96707
26.00	0.0	0.0	33.36	13.42	0.0	0.0	0.0	0.0	0.0003	25.06459
31.85	0.0	0.0	33.36	13.39	0.0	0.0	0.0	0.0	0.0003	25.07201

Diffuser table:

P-dia	Ver angl	H-Angle	SourceX	SourceY	Ports	MZ-dis	Isoplth	P-depth	Ttl-flo	Eff-sal	Temp	Polutnt
(in)	(deg)	(deg)	(ft)	(ft)	()	(ft)	(concent)	(ft)	(MGD)	(psu)	(C)	(ppm)
3.0500	0.0	0.0	0.0	0.0	125.00	1000.0	0.0	100.00	23.000	43.690	20.660	43690.0

Simulation:

Froude No: -25.36; Strat No:-6.23E-4; Spcg No: 14.39; k: 1.71E+5; eff den (sigmaT) 31.24414; eff vel 1.710(m/s);
Current is very small, flow regime may be transient.

Step	Depth (ft)	Amb-cur (m/s)	P-dia (in)	Eff-sal (psu)	Polutnt (ppm)	Dilutn ()	x-posn (ft)	y-posn (ft)	Iso dia (m)
0	100.0	1.000E-5	3.050	43.69	43690.0	1.000	0.0	0.0	0.07747;
1	100.0	0.0	3.090	43.43	42562.8	1.026	0.0164	0.0	0.07849;
2	100.0	0.0	3.162	43.23	41723.7	1.047	0.0286	0.0	0.08031;
3	100.0	0.0	3.225	43.03	40901.2	1.068	0.0407	0.0	0.08192;
4	100.0	0.0	3.290	42.84	40095.0	1.090	0.053	0.0	0.08357;
5	100.0	0.0	3.356	42.66	39304.7	1.112	0.0656	0.0	0.08524;
6	100.0	0.0	3.423	42.48	38530.1	1.134	0.0784	0.0	0.08695;
7	100.0	0.0	3.492	42.30	37770.8	1.157	0.0914	0.0	0.08869;
8	100.0	0.0	3.562	42.12	37026.6	1.180	0.105	0.0	0.09047;
9	100.0	0.0	3.633	41.95	36297.1	1.204	0.118	0.0	0.09229;
10	100.0	0.0	3.706	41.78	35582.0	1.228	0.132	0.0	0.09414;
11	100.0	0.0	3.780	41.62	34881.1	1.253	0.146	0.0	0.09602;
12	100.0	0.0	3.856	41.46	34194.0	1.278	0.161	0.0	0.09795;
13	100.0	0.0	3.934	41.30	33520.6	1.303	0.175	0.0	0.09991;
14	100.0	0.0	4.012	41.14	32860.4	1.330	0.190	0.0	0.1019;
15	100.0	0.0	4.093	40.99	32213.3	1.356	0.206	0.0	0.1040;
16	100.0	0.0	4.175	40.84	31579.0	1.384	0.221	0.0	0.1060;
17	100.0	0.0	4.258	40.69	30957.2	1.411	0.237	0.0	0.1082;
18	100.0	0.0	4.344	40.55	30347.7	1.440	0.253	0.0	0.1103;
19	100.0	0.0	4.431	40.41	29750.3	1.469	0.270	0.0	0.1125;
20	100.0	0.0	4.520	40.27	29164.7	1.498	0.287	0.0	0.1148;
21	100.0	0.0	4.610	40.13	28590.6	1.528	0.304	0.0	0.1171;
22	100.0	0.0	4.703	40.00	28027.9	1.559	0.321	0.0	0.1194;
23	100.0	0.0	4.797	39.87	27476.3	1.590	0.339	0.0	0.1218;
24	100.0	0.0	4.893	39.74	26935.5	1.622	0.357	0.0	0.1243;
25	100.0	0.0	4.991	39.62	26405.5	1.655	0.376	0.0	0.1268;
26	100.0	0.0	5.091	39.50	25885.9	1.688	0.395	0.0	0.1293;
27	100.0	0.0	5.193	39.38	25376.6	1.722	0.414	0.0	0.1319;
28	100.0	0.0	5.297	39.26	24877.3	1.756	0.434	0.0	0.1345;
29	100.0	0.0	5.403	39.14	24387.9	1.791	0.454	0.0	0.1372;
30	100.0	0.0	5.511	39.03	23908.2	1.827	0.475	0.0	0.1400;
31	100.0	0.0	5.622	38.92	23437.9	1.864	0.496	0.0	0.1428;
32	100.0	0.0	5.734	38.81	22976.8	1.901	0.517	0.0	0.1457;
33	100.0	0.0	5.849	38.70	22524.9	1.940	0.539	0.0	0.1486;
34	100.0	0.0	5.966	38.60	22081.9	1.979	0.561	0.0	0.1515;
35	100.0	0.0	6.086	38.50	21647.7	2.018	0.583	0.0	0.1546;
36	100.0	0.0	6.208	38.39	21221.9	2.059	0.607	0.0	0.1577;
37	100.0	0.0	6.332	38.30	20804.6	2.100	0.630	0.0	0.1608;
38	100.0	0.0	6.459	38.20	20395.6	2.142	0.654	0.0	0.1641;
39	100.0	0.0	6.588	38.10	19994.5	2.185	0.678	0.0	0.1673;
40	100.0	0.0	6.720	38.01	19601.4	2.229	0.703	0.0	0.1707;
41	100.0	0.0	6.855	37.92	19216.1	2.274	0.729	0.0	0.1741;
42	100.0	0.0	6.992	37.83	18838.3	2.319	0.755	0.0	0.1776;
43	100.0	0.0	7.132	37.74	18468.0	2.366	0.781	0.0	0.1812;
44	100.0	0.0	7.275	37.66	18104.9	2.413	0.808	0.0	0.1848;
45	100.0	0.0	7.421	37.57	17749.1	2.462	0.836	0.0	0.1885;
46	100.0	0.0	7.569	37.49	17400.2	2.511	0.864	0.0	0.1923;
47	100.0	0.0	7.721	37.41	17058.2	2.561	0.892	0.0	0.1961;
48	100.0	0.0	7.875	37.33	16723.0	2.613	0.922	0.0	0.2000;
49	100.0	0.0	8.033	37.25	16394.3	2.665	0.951	0.0	0.2040;
50	100.0	0.0	8.194	37.18	16072.1	2.718	0.982	0.0	0.2081;

51	100.0	0.0	8.358	37.10	15756.3	2.773	1.013	0.0	0.2123;
52	100.0	0.0	8.525	37.03	15446.7	2.828	1.044	0.0	0.2165;
53	100.0	0.0	8.696	36.96	15143.2	2.885	1.076	0.0	0.2209;
54	100.0	0.0	8.870	36.89	14845.6	2.943	1.109	0.0	0.2253;
55	100.0	0.0	9.047	36.82	14553.9	3.002	1.143	0.0	0.2298;
56	100.0	0.0	9.229	36.75	14268.0	3.062	1.177	0.0	0.2344;
57	100.0	0.0	9.413	36.68	13987.7	3.123	1.212	0.0	0.2391;
58	100.0	0.0	9.602	36.62	13712.9	3.186	1.247	0.0	0.2439;
59	100.0	0.0	9.794	36.55	13443.5	3.250	1.283	0.0	0.2488;
60	100.0	0.0	9.990	36.49	13179.4	3.315	1.320	0.0	0.2537;
61	100.0	0.0	10.19	36.43	12920.5	3.381	1.358	0.0	0.2588;
62	100.0	0.0	10.39	36.37	12666.7	3.449	1.396	0.0	0.2640;
63	100.0	0.0	10.60	36.31	12417.9	3.518	1.435	0.0	0.2693;
64	100.0	0.0	10.81	36.25	12174.0	3.589	1.475	0.0	0.2747;
65	100.0	0.0	11.03	36.20	11934.9	3.661	1.516	0.0	0.2802;
66	100.0	0.0	11.25	36.14	11700.5	3.734	1.557	0.0	0.2858;
67	100.0	0.0	11.48	36.09	11470.7	3.809	1.599	0.0	0.2915;
68	100.0	0.0	11.71	36.03	11245.4	3.885	1.643	0.0	0.2973;
69	100.0	0.0	11.94	35.98	11024.6	3.963	1.686	0.0	0.3033;
70	100.0	0.0	12.18	35.93	10808.1	4.042	1.731	0.0	0.3093;
71	100.0	0.0	12.42	35.88	10595.8	4.123	1.777	0.0	0.3155;
72	100.0	0.0	12.67	35.83	10387.8	4.206	1.823	0.0	0.3218;
73	100.0	0.0	12.92	35.78	10183.8	4.290	1.871	0.0	0.3283;
74	100.0	0.0	13.18	35.73	9983.8	4.376	1.919	0.0	0.3348;
75	100.0	0.0	13.45	35.69	9787.8	4.464	1.969	0.0	0.3415;
76	100.0	0.0	13.71	35.64	9595.6	4.553	2.019	0.0	0.3483;
77	100.0	0.0	13.99	35.60	9407.2	4.644	2.070	0.0	0.3553;
78	100.0	0.0	14.27	35.55	9222.5	4.737	2.123	0.0	0.3624;
79	100.0	0.0	14.55	35.51	9041.5	4.832	2.176	0.0	0.3697;
80	100.0	0.0	14.84	35.47	8864.0	4.929	2.230	0.0	0.3770;
81	100.0	0.0	15.14	35.43	8689.9	5.028	2.285	0.0	0.3846;
82	100.0	0.0	15.44	35.39	8519.3	5.128	2.342	0.0	0.3923;
83	100.0	0.0	15.75	35.35	8352.1	5.231	2.399	0.0	0.4001;
84	100.0	0.0	16.07	35.31	8188.1	5.336	2.458	0.0	0.4081;
85	100.1	0.0	16.39	35.27	8027.4	5.443	2.518	0.0	0.4162;
86	100.1	0.0	16.71	35.23	7869.8	5.552	2.579	0.0	0.4245;
87	100.1	0.0	17.05	35.20	7715.3	5.663	2.641	0.0	0.4330;
88	100.1	0.0	17.39	35.16	7563.9	5.776	2.704	0.0	0.4416;
89	100.1	0.0	17.73	35.12	7415.4	5.892	2.768	0.0	0.4504;
90	100.1	0.0	18.09	35.09	7269.9	6.010	2.834	0.0	0.4594;
91	100.1	0.0	18.45	35.06	7127.2	6.130	2.901	0.0	0.4686;
92	100.1	0.0	18.82	35.02	6987.3	6.253	2.969	0.0	0.4779;
93	100.1	0.0	19.19	34.99	6850.2	6.378	3.038	0.0	0.4874;
94	100.1	0.0	19.57	34.96	6715.7	6.506	3.109	0.0	0.4971;
95	100.1	0.0	19.96	34.93	6583.9	6.636	3.181	0.0	0.5070;
96	100.1	0.0	20.36	34.90	6454.7	6.769	3.255	0.0	0.5171;
97	100.1	0.0	20.76	34.87	6328.0	6.904	3.330	0.0	0.5274;
98	100.1	0.0	21.18	34.84	6203.8	7.042	3.406	0.0	0.5379;
99	100.1	0.0	21.60	34.81	6082.1	7.183	3.483	0.0	0.5486;
100	100.1	0.0	22.03	34.78	5962.7	7.327	3.562	0.0	0.5594;
101	100.1	0.0	22.46	34.75	5845.7	7.474	3.643	0.0	0.5705;
102	100.1	0.0	22.91	34.72	5731.0	7.623	3.725	0.0	0.5819;
103	100.2	0.0	23.36	34.70	5618.5	7.776	3.808	0.0	0.5934;
104	100.2	0.0	23.82	34.67	5508.3	7.932	3.893	0.0	0.6051;
105	100.2	0.0	24.29	34.65	5400.2	8.090	3.980	0.0	0.6171;
106	100.2	0.0	24.77	34.62	5294.2	8.252	4.068	0.0	0.6293;

107	100.2	0.0	25.26	34.60	5190.3	8.418	4.157	0.0	0.6417;
108	100.2	0.0	25.76	34.57	5088.5	8.586	4.248	0.0	0.6543;
109	100.2	0.0	26.26	34.55	4990.9	8.754	4.339	0.0	0.6671;
110	100.2	0.0	26.76	34.53	4898.2	8.920	4.428	0.0	0.6797;
111	100.2	0.0	27.25	34.51	4809.9	9.083	4.516	0.0	0.6921;
112	100.3	0.0	27.73	34.49	4725.7	9.245	4.602	0.0	0.7044;
113	100.3	0.0	28.21	34.47	4645.2	9.405	4.687	0.0	0.7165;
114	100.3	0.0	28.68	34.45	4568.1	9.564	4.770	0.0	0.7284;
115	100.3	0.0	29.14	34.43	4494.2	9.721	4.852	0.0	0.7403;
116	100.3	0.0	29.61	34.41	4423.3	9.877	4.933	0.0	0.7520;
117	100.3	0.0	30.06	34.40	4355.2	10.03	5.013	0.0	0.7636;
118	100.3	0.0	30.51	34.38	4289.7	10.18	5.092	0.0	0.7750;
119	100.4	0.0	30.96	34.37	4226.6	10.34	5.170	0.0	0.7864;
120	100.4	0.0	31.40	34.35	4165.8	10.49	5.247	0.0	0.7976;
121	100.4	0.0	31.84	34.34	4107.1	10.64	5.323	0.0	0.8088;
122	100.4	0.0	32.28	34.33	4050.4	10.79	5.398	0.0	0.8198;
123	100.4	0.0	32.71	34.31	3995.5	10.93	5.472	0.0	0.8307;
124	100.4	0.0	33.13	34.30	3942.5	11.08	5.546	0.0	0.8416;
125	100.4	0.0	33.56	34.29	3891.1	11.23	5.618	0.0	0.8524;
126	100.5	0.0	33.98	34.28	3841.3	11.37	5.690	0.0	0.8631;
127	100.5	0.0	34.40	34.26	3793.0	11.52	5.761	0.0	0.8737;
128	100.5	0.0	34.81	34.25	3746.2	11.66	5.831	0.0	0.8842;
129	100.5	0.0	35.22	34.24	3700.7	11.81	5.901	0.0	0.8946;
130	100.5	0.0	35.63	34.23	3656.4	11.95	5.970	0.0	0.9050;
131	100.5	0.0	36.04	34.22	3613.4	12.09	6.038	0.0	0.9153;
132	100.6	0.0	36.44	34.21	3571.5	12.23	6.105	0.0	0.9256;
133	100.6	0.0	36.84	34.20	3530.8	12.37	6.172	0.0	0.9357;
134	100.6	0.0	37.24	34.19	3491.1	12.51	6.239	0.0	0.9459;
135	100.6	0.0	37.63	34.18	3452.4	12.66	6.305	0.0	0.9559;
136	100.6	0.0	38.03	34.17	3414.6	12.79	6.370	0.0	0.9659;
137	100.7	0.0	38.42	34.17	3377.8	12.93	6.435	0.0	0.9758;
138	100.7	0.0	38.81	34.16	3341.8	13.07	6.499	0.0	0.9857;
139	100.7	0.0	39.19	34.15	3306.7	13.21	6.563	0.0	0.9955;
140	100.7	0.0	39.58	34.14	3272.4	13.35	6.626	0.0	1.0053;
141	100.7	0.0	39.96	34.13	3238.9	13.49	6.689	0.0	1.0150;
142	100.7	0.0	40.34	34.13	3206.1	13.63	6.751	0.0	1.0247;
143	100.8	0.0	40.72	34.12	3174.0	13.76	6.813	0.0	1.0344;
144	100.8	0.0	41.10	34.11	3142.6	13.90	6.874	0.0	1.0439;
145	100.8	0.0	41.48	34.10	3111.9	14.04	6.935	0.0	1.0535;
146	100.8	0.0	41.85	34.10	3081.8	14.18	6.996	0.0	1.0630;
147	100.8	0.0	42.22	34.09	3052.3	14.31	7.056	0.0	1.0724;
148	100.9	0.0	42.59	34.08	3023.4	14.45	7.116	0.0	1.0818;
149	100.9	0.0	42.96	34.08	2995.0	14.59	7.176	0.0	1.0912;
150	100.9	0.0	43.33	34.07	2967.2	14.72	7.235	0.0	1.1005;
151	100.9	0.0	43.69	34.06	2939.9	14.86	7.294	0.0	1.1098;
152	100.9	0.0	44.06	34.06	2913.1	15.00	7.352	0.0	1.1191; merging;
153	101.0	0.0	44.42	34.05	2887.6	15.13	7.411	0.0	1.1282;
154	101.0	0.0	44.77	34.04	2863.2	15.26	7.468	0.0	1.1373;
155	101.0	0.0	45.13	34.04	2839.4	15.39	7.526	0.0	1.1463;
156	101.0	0.0	45.48	34.03	2816.3	15.51	7.583	0.0	1.1552;
157	101.1	0.0	45.83	34.03	2793.8	15.64	7.641	0.0	1.1641;
158	101.1	0.0	46.18	34.02	2771.8	15.76	7.698	0.0	1.1729;
159	101.1	0.0	46.52	34.02	2750.3	15.89	7.754	0.0	1.1817;
160	101.1	0.0	46.87	34.01	2729.3	16.01	7.811	0.0	1.1905;
161	101.1	0.0	47.22	34.01	2708.6	16.13	7.867	0.0	1.1993;
162	101.2	0.0	47.56	34.00	2688.4	16.25	7.923	0.0	1.2080;

163	101.2	0.0	47.90	34.00	2668.5	16.37	7.979	0.0	1.2167;
164	101.2	0.0	48.25	33.99	2649.0	16.49	8.035	0.0	1.2254;
165	101.2	0.0	48.59	33.99	2629.8	16.61	8.090	0.0	1.2341;
166	101.3	0.0	48.93	33.98	2610.9	16.73	8.146	0.0	1.2428;
167	101.3	0.0	49.27	33.98	2592.3	16.85	8.201	0.0	1.2515;
168	101.3	0.0	49.61	33.98	2574.1	16.97	8.256	0.0	1.2602;
169	101.3	0.0	49.95	33.97	2556.0	17.09	8.311	0.0	1.2688;
170	101.4	0.0	50.29	33.97	2538.3	17.21	8.366	0.0	1.2775;
171	101.4	0.0	50.63	33.96	2520.8	17.33	8.421	0.0	1.2861;
172	101.4	0.0	50.98	33.96	2503.5	17.45	8.475	0.0	1.2948;
173	101.5	0.0	51.32	33.95	2486.5	17.57	8.530	0.0	1.3034;
174	101.5	0.0	51.66	33.95	2469.7	17.69	8.584	0.0	1.3121;
175	101.5	0.0	52.00	33.95	2453.2	17.81	8.638	0.0	1.3208;
176	101.5	0.0	52.34	33.94	2436.8	17.93	8.692	0.0	1.3294;
177	101.6	0.0	52.68	33.94	2420.6	18.05	8.746	0.0	1.3381;
178	101.6	0.0	53.02	33.93	2404.7	18.17	8.800	0.0	1.3468;
179	101.6	0.0	53.36	33.93	2388.9	18.29	8.854	0.0	1.3555;
180	101.6	0.0	53.71	33.93	2373.3	18.41	8.908	0.0	1.3642;
181	101.7	0.0	54.05	33.92	2357.9	18.53	8.962	0.0	1.3729;
182	101.7	0.0	54.39	33.92	2342.6	18.65	9.016	0.0	1.3816;
183	101.7	0.0	54.74	33.92	2327.6	18.77	9.069	0.0	1.3903;
184	101.8	0.0	55.08	33.91	2312.6	18.89	9.123	0.0	1.3990;
185	101.8	0.0	55.42	33.91	2297.9	19.01	9.176	0.0	1.4078;
186	101.8	0.0	55.77	33.91	2283.3	19.13	9.230	0.0	1.4165;
187	101.9	0.0	56.11	33.90	2268.8	19.26	9.283	0.0	1.4253;
188	101.9	0.0	56.46	33.90	2254.5	19.38	9.337	0.0	1.4341;
189	101.9	0.0	56.81	33.90	2240.3	19.50	9.390	0.0	1.4429;
190	101.9	0.0	57.15	33.89	2226.3	19.62	9.444	0.0	1.4517;
191	102.0	0.0	57.50	33.89	2212.4	19.75	9.497	0.0	1.4605;
192	102.0	0.0	57.85	33.89	2198.6	19.87	9.550	0.0	1.4694;
193	102.0	0.0	58.20	33.88	2185.0	20.00	9.604	0.0	1.4782;
194	102.1	0.0	58.55	33.88	2171.4	20.12	9.657	0.0	1.4871;
195	102.1	0.0	58.90	33.88	2158.0	20.25	9.710	0.0	1.4960;
196	102.1	0.0	59.25	33.87	2144.7	20.37	9.763	0.0	1.5049;
197	102.2	0.0	59.60	33.87	2131.6	20.50	9.817	0.0	1.5139;
198	102.2	0.0	59.95	33.87	2118.5	20.62	9.870	0.0	1.5228;
199	102.3	0.0	60.31	33.86	2105.5	20.75	9.923	0.0	1.5318;
200	102.3	0.0	60.66	33.86	2092.7	20.88	9.977	0.0	1.5408;
201	102.3	0.0	61.02	33.86	2079.9	21.01	10.03	0.0	1.5498;
202	102.4	0.0	61.37	33.85	2067.3	21.13	10.08	0.0	1.5588;
203	102.4	0.0	61.73	33.85	2054.7	21.26	10.14	0.0	1.5679; bottom hit;

Horiz plane projections in effluent direction: radius(m): 0.0; CL(m): 3.0898

Lmz(m): 3.0898

forced entrain 1 0.0 -0.730 1.568 0.827

Rate sec-1 0.0 dy-1 0.0 kt: 0.0 Amb Sal 33.3632

;

1:05:33 PM. amb fills: 4

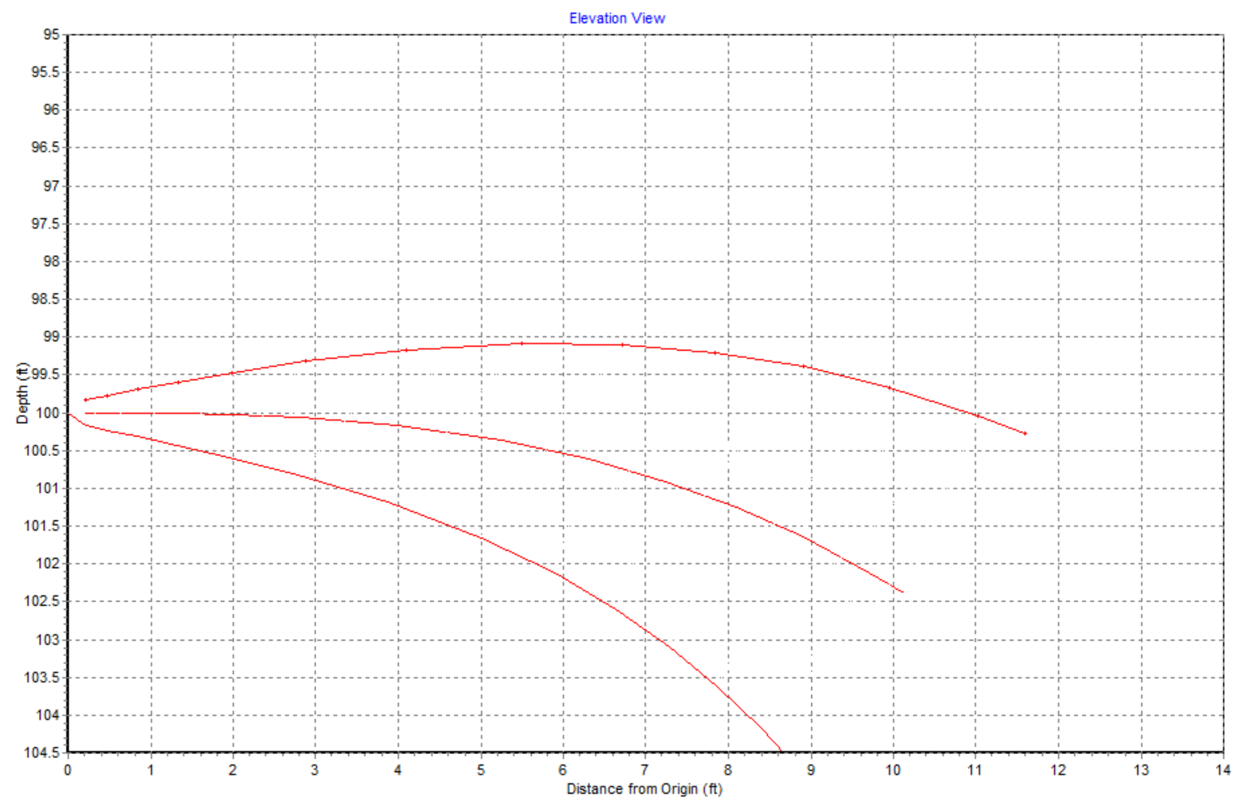

Figure C.2.1: Plumes 18b solution of discharge plume trajectories for discharges of 8 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt., and 15 mgd of brine from the Doheny Desalination Project with a brine salinity of 67 ppt. Depth of maximum rise of the plume is Z = 99.1 ft. at X_a = 6.038 ft from the point of discharge. The centerline of the plume is at an average distance of X_b = 10.14 ft from the point of discharge as the plume begins to impact the bottom at a depth of Z = 104.5 ft.

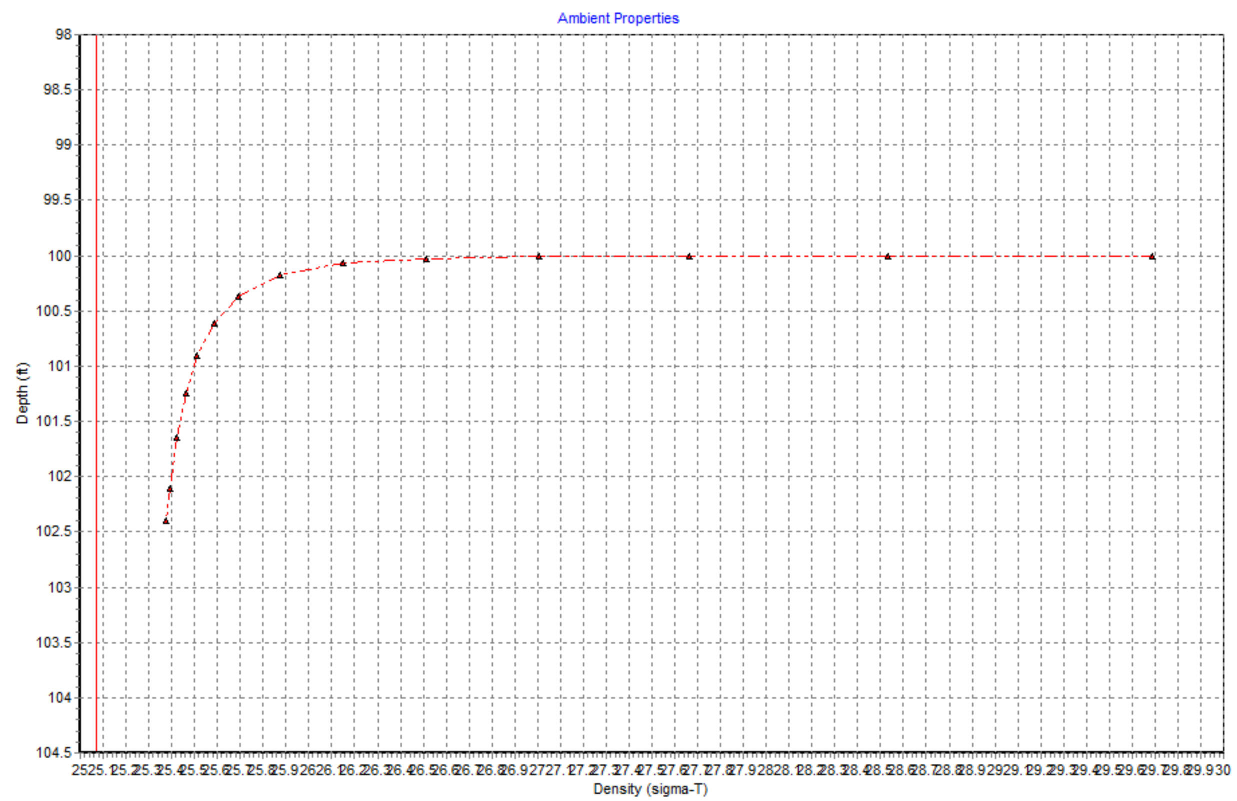

Figure C.2.2: Plumes 18b solution of vertical density profile for discharges of 8 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt. and 15 mgd of brine from the Doheny Desalination Project with a brine salinity of 67 ppt. Discharge effluent density shown as black triangles. Ambient water mass density profile shown as solid red line.

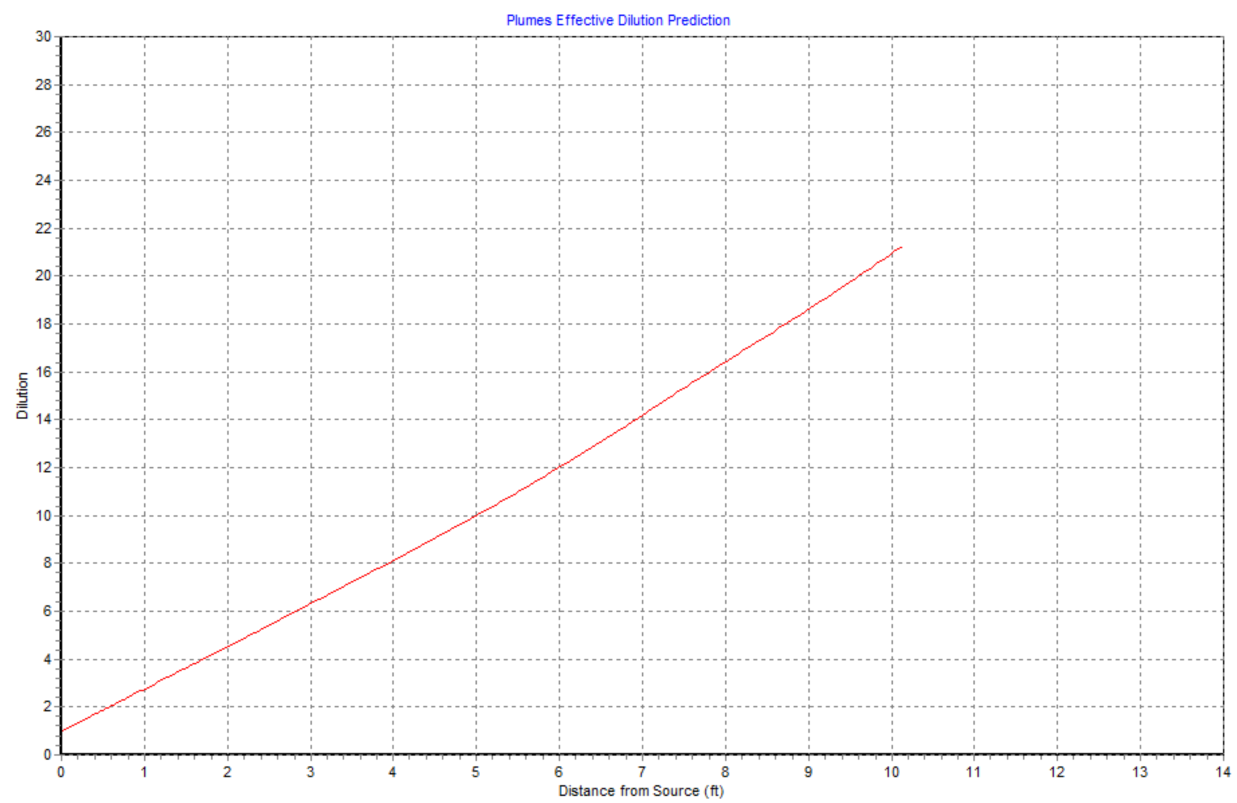

Figure C.2.3: Plumes 18b solution of effective (bulk average) dilution as a function of vertical distance from the point of discharge for discharges of 8 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt. and 15 mgd of brine from the Doheny Desalination Project with a brine salinity of 67 ppt. Effective dilution is S_a = 12.09 at the maximum rise of the plume at X_a = 6.038 ft. from the point of discharge. As the plume begins to impact the bottom, the plume centerline is at a distance of X_b = 10.14 ft from the point of discharge, where the effective dilution reaches S_a = 21.26.

C.3: Plumes 18b Results for SJCOO discharges of 0 mgd Wastewater and 15 mgd Brine:

Contents of the memo box (may not be current and must be updated manually)
Project "C:\Plumes18\SJCOO_WW0mgd_b15mgd_D-1"
memo
SJCOO discharging 0 mgd wastewater and 15 mgd brine

Model configuration items checked:
Channel width (m) 100
Start case for graphs 1
Max detailed graphs 10 (limits plots that can overflow memory)
Elevation Projection Plane (deg) 0
Shore vector (m,deg) not checked
Bacteria model : Mancini (1978) coliform model
PDS sfc. model heat transfer : Medium
Equation of State : S, T
Similarity Profile : Default profile (k=2.0, ...)
Diffuser port contraction coefficient 1
Light absorption coefficient 0.16
Farfield increment (m) 200
UM3 aspiration coefficient 0.1
Output file: text output tab
Output each ?? steps 1
Maximum dilution reported 1000
Text output format : Standard
Max vertical reversals : to max rise or fall

/ UM3. 1/14/2019 1:30:39 PM
Case 1; ambient file C:\Plumes18\SJCOO_WW0mgd_b15mgd_D-1.001.db; Diffuser table record 1: ----------------------------------

Ambient Table:

Depth	Amb-cur	Amb-dir	Amb-sal	Amb-tem	Amb-pol	Decay	Far-spd	Far-dir	Disprsn	Density
m	m/s	deg	psu	C	kg/kg	s-1	m/s	deg	m0.67/s2	sigma-T
0.0	0.0	0.0	33.38	19.97	0.0	0.0	0.0	0.0	0.0003	23.55719
2.000	0.0	0.0	33.39	19.87	0.0	0.0	0.0	0.0	0.0003	23.59288
4.000	0.0	0.0	33.40	19.81	0.0	0.0	0.0	0.0	0.0003	23.61484
7.000	0.0	0.0	33.41	19.64	0.0	0.0	0.0	0.0	0.0003	23.67096
10.00	0.0	0.0	33.40	17.76	0.0	0.0	0.0	0.0	0.0003	24.12888
12.00	0.0	0.0	33.38	17.34	0.0	0.0	0.0	0.0	0.0003	24.21549
14.00	0.0	0.0	33.38	16.73	0.0	0.0	0.0	0.0	0.0003	24.35932
16.00	0.0	0.0	33.38	15.17	0.0	0.0	0.0	0.0	0.0003	24.71340
18.00	0.0	0.0	33.37	14.37	0.0	0.0	0.0	0.0	0.0003	24.87044
20.00	0.0	0.0	33.35	14.18	0.0	0.0	0.0	0.0	0.0003	24.89660
22.00	0.0	0.0	33.33	13.83	0.0	0.0	0.0	0.0	0.0003	24.95172
24.00	0.0	0.0	33.32	13.75	0.0	0.0	0.0	0.0	0.0003	24.96707
26.00	0.0	0.0	33.36	13.42	0.0	0.0	0.0	0.0	0.0003	25.06459
31.85	0.0	0.0	33.36	13.39	0.0	0.0	0.0	0.0	0.0003	25.07201

Diffuser table:

P-dia	Ver angl	H-Angle	SourceX	SourceY	Ports	MZ-dis	Isoplth	P-depth	Ttl-flo	Eff-sal	Temp	Polutnt
(in)	(deg)	(deg)	(ft)	(ft)	()	(ft)	(concent)	(ft)	(MGD)	(psu)	(C)	(ppm)
3.0500	0.0	0.0	0.0	0.0	125.00	1000.0	0.0	100.00	15.000	67.000	20.660	67000.0

Simulation:
Froude No: -8.389; Strat No:-1.57E-4; Spcg No: 14.39; k: 1.11E+5; eff den (sigmaT) 49.48870; eff vel 1.115(m/s);
Current is very small, flow regime may be transient.

Step	Depth (ft)	Amb-cur (m/s)	P-dia (in)	Eff-sal (psu)	Polutnt (ppm)	Dilutn ()	x-posn (ft)	y-posn (ft)	Iso dia (m)
0	100.0	1.000E-5	3.050	67.00	67000.0	1.000	0.0	0.0	0.07747;
1	100.0	0.0	3.076	66.44	65862.4	1.017	0.0108	0.0	0.07814;
2	100.0	0.0	3.134	65.79	64541.7	1.038	0.0189	0.0	0.07959;
3	100.0	0.0	3.197	65.16	63248.1	1.059	0.0261	0.0	0.0812;
4	100.0	0.0	3.262	64.53	61981.0	1.081	0.0331	0.0	0.08285;
5	100.0	0.0	3.328	63.92	60739.7	1.103	0.0399	0.0	0.08452;
6	100.0	0.0	3.395	63.32	59523.8	1.126	0.0468	0.0	0.08623;
7	100.0	0.0	3.463	62.74	58332.8	1.149	0.0538	0.0	0.08797;
8	100.0	0.0	3.533	62.16	57166.1	1.172	0.0609	0.0	0.08975;
9	100.0	0.0	3.605	61.60	56023.1	1.196	0.0681	0.0	0.09156;
10	100.0	0.0	3.678	61.04	54903.4	1.220	0.0754	0.0	0.09341;
11	100.0	0.0	3.752	60.50	53806.5	1.245	0.0828	0.0	0.0953;
12	100.0	0.0	3.828	59.97	52731.9	1.271	0.0904	0.0	0.09722;
13	100.0	0.0	3.905	59.45	51679.2	1.296	0.0982	0.0	0.09918;
14	100.0	0.0	3.984	58.93	50647.8	1.323	0.106	0.0	0.1012;
15	100.0	0.0	4.064	58.43	49637.4	1.350	0.114	0.0	0.1032;
16	100.0	0.0	4.146	57.94	48647.4	1.377	0.122	0.0	0.1053;
17	100.0	0.0	4.230	57.46	47677.6	1.405	0.131	0.0	0.1074;
18	100.0	0.0	4.315	56.99	46727.3	1.434	0.139	0.0	0.1096;
19	100.0	0.0	4.402	56.52	45796.3	1.463	0.148	0.0	0.1118;
20	100.0	0.0	4.491	56.07	44884.2	1.493	0.157	0.0	0.1141;
21	100.0	0.0	4.581	55.62	43990.5	1.523	0.165	0.0	0.1164;
22	100.0	0.0	4.674	55.19	43114.8	1.554	0.175	0.0	0.1187;
23	100.0	0.0	4.768	54.76	42256.8	1.586	0.184	0.0	0.1211;
24	100.0	0.0	4.864	54.34	41416.1	1.618	0.193	0.0	0.1235;
25	100.0	0.0	4.962	53.93	40592.4	1.651	0.203	0.0	0.1260;
26	100.0	0.0	5.062	53.53	39785.3	1.684	0.213	0.0	0.1286;
27	100.0	0.0	5.163	53.13	38994.5	1.718	0.223	0.0	0.1312;
28	100.0	0.0	5.267	52.74	38219.5	1.753	0.233	0.0	0.1338;
29	100.0	0.0	5.373	52.36	37460.2	1.789	0.244	0.0	0.1365;
30	100.0	0.0	5.482	51.99	36716.2	1.825	0.254	0.0	0.1392;
31	100.0	0.0	5.592	51.62	35987.1	1.862	0.265	0.0	0.1420;
32	100.0	0.0	5.704	51.27	35272.7	1.899	0.276	0.0	0.1449;
33	100.0	0.0	5.819	50.92	34572.6	1.938	0.287	0.0	0.1478;
34	100.0	0.0	5.936	50.57	33886.5	1.977	0.299	0.0	0.1508;
35	100.0	0.0	6.055	50.23	33214.3	2.017	0.311	0.0	0.1538;
36	100.0	0.0	6.177	49.90	32555.5	2.058	0.322	0.0	0.1569;
37	100.0	0.0	6.301	49.58	31910.0	2.100	0.335	0.0	0.1601;
38	100.0	0.0	6.428	49.26	31277.3	2.142	0.347	0.0	0.1633;
39	100.0	0.0	6.557	48.95	30657.4	2.185	0.360	0.0	0.1665;
40	100.0	0.0	6.689	48.64	30049.8	2.230	0.372	0.0	0.1699;
41	100.0	0.0	6.823	48.34	29454.5	2.275	0.385	0.0	0.1733;
42	100.0	0.0	6.960	48.05	28871.0	2.321	0.399	0.0	0.1768;
43	100.0	0.0	7.100	47.76	28299.2	2.368	0.412	0.0	0.1803;
44	100.0	0.0	7.242	47.48	27738.9	2.415	0.426	0.0	0.1840;
45	100.0	0.0	7.387	47.20	27189.7	2.464	0.440	0.0	0.1876;
46	100.0	0.0	7.536	46.93	26651.5	2.514	0.455	0.0	0.1914;
47	100.0	0.0	7.687	46.67	26124.1	2.565	0.469	0.0	0.1952;
48	100.0	0.0	7.841	46.41	25607.2	2.616	0.484	0.0	0.1992;
49	100.0	0.0	7.998	46.15	25100.6	2.669	0.499	0.0	0.2031;

50	100.0	0.0	8.158	45.90	24604.1	2.723	0.515	0.0	0.2072;
51	100.0	0.0	8.321	45.65	24117.5	2.778	0.530	0.0	0.2114;
52	100.0	0.0	8.488	45.41	23640.7	2.834	0.546	0.0	0.2156;
53	100.0	0.0	8.658	45.18	23173.3	2.891	0.563	0.0	0.2199;
54	100.0	0.0	8.831	44.94	22715.2	2.950	0.579	0.0	0.2243;
55	100.0	0.0	9.008	44.72	22266.3	3.009	0.596	0.0	0.2288;
56	100.0	0.0	9.188	44.49	21826.3	3.070	0.614	0.0	0.2334;
57	100.0	0.0	9.371	44.28	21395.1	3.132	0.631	0.0	0.2380;
58	100.0	0.0	9.558	44.06	20972.4	3.195	0.649	0.0	0.2428;
59	100.0	0.0	9.749	43.85	20558.2	3.259	0.667	0.0	0.2476;
60	100.0	0.0	9.943	43.65	20152.2	3.325	0.686	0.0	0.2526;
61	100.0	0.0	10.14	43.44	19754.2	3.392	0.705	0.0	0.2576;
62	100.0	0.0	10.34	43.25	19364.2	3.460	0.724	0.0	0.2627;
63	100.0	0.0	10.55	43.05	18981.9	3.530	0.744	0.0	0.2680;
64	100.0	0.0	10.76	42.86	18607.3	3.601	0.764	0.0	0.2733;
65	100.0	0.0	10.97	42.68	18240.0	3.673	0.784	0.0	0.2787;
66	100.0	0.0	11.19	42.49	17880.1	3.747	0.805	0.0	0.2843;
67	100.0	0.0	11.41	42.32	17527.3	3.823	0.826	0.0	0.2899;
68	100.0	0.0	11.64	42.14	17181.5	3.900	0.848	0.0	0.2956;
69	100.0	0.0	11.87	41.97	16842.6	3.978	0.870	0.0	0.3015;
70	100.0	0.0	12.10	41.80	16510.4	4.058	0.892	0.0	0.3074;
71	100.0	0.0	12.34	41.64	16196.3	4.137	0.914	0.0	0.3134;
72	100.1	0.0	12.57	41.49	15893.1	4.216	0.936	0.0	0.3193;
73	100.1	0.0	12.80	41.34	15600.4	4.295	0.957	0.0	0.3252;
74	100.1	0.0	13.04	41.19	15317.3	4.374	0.978	0.0	0.3312;
75	100.1	0.0	13.27	41.05	15043.5	4.454	0.998	0.0	0.3371;
76	100.1	0.0	13.51	40.92	14778.4	4.534	1.018	0.0	0.3430;
77	100.1	0.0	13.74	40.79	14521.5	4.614	1.038	0.0	0.3490;
78	100.1	0.0	13.98	40.66	14272.5	4.694	1.058	0.0	0.3550;
79	100.1	0.0	14.21	40.54	14030.9	4.775	1.077	0.0	0.3610;
80	100.1	0.0	14.45	40.42	13796.4	4.856	1.096	0.0	0.3670;
81	100.1	0.0	14.68	40.30	13568.7	4.938	1.114	0.0	0.3730;
82	100.1	0.0	14.92	40.19	13347.4	5.020	1.133	0.0	0.3790;
83	100.1	0.0	15.16	40.08	13132.2	5.102	1.151	0.0	0.3851;
84	100.1	0.0	15.40	39.97	12922.9	5.185	1.169	0.0	0.3912;
85	100.1	0.0	15.64	39.87	12719.2	5.268	1.186	0.0	0.3972;
86	100.1	0.0	15.88	39.77	12520.9	5.351	1.204	0.0	0.4034;
87	100.1	0.0	16.12	39.67	12327.7	5.435	1.221	0.0	0.4095;
88	100.1	0.0	16.36	39.58	12139.4	5.519	1.238	0.0	0.4156;
89	100.1	0.0	16.61	39.48	11955.9	5.604	1.255	0.0	0.4218;
90	100.1	0.0	16.85	39.39	11776.9	5.689	1.271	0.0	0.4280;
91	100.1	0.0	17.10	39.30	11602.3	5.775	1.287	0.0	0.4342;
92	100.1	0.0	17.34	39.21	11431.9	5.861	1.303	0.0	0.4405;
93	100.1	0.0	17.59	39.13	11265.6	5.947	1.319	0.0	0.4467;
94	100.1	0.0	17.83	39.05	11103.2	6.034	1.335	0.0	0.4530;
95	100.1	0.0	18.08	38.97	10944.5	6.122	1.350	0.0	0.4593;
96	100.1	0.0	18.33	38.89	10789.5	6.210	1.366	0.0	0.4656;
97	100.1	0.0	18.58	38.81	10637.9	6.298	1.381	0.0	0.4720;
98	100.1	0.0	18.83	38.73	10489.8	6.387	1.396	0.0	0.4783;
99	100.1	0.0	19.08	38.66	10344.9	6.477	1.411	0.0	0.4847;
100	100.2	0.0	19.34	38.59	10203.2	6.567	1.425	0.0	0.4911;
101	100.2	0.0	19.59	38.52	10064.5	6.657	1.440	0.0	0.4976;
102	100.2	0.0	19.84	38.45	9928.9	6.748	1.454	0.0	0.5040;
103	100.2	0.0	20.10	38.38	9796.0	6.840	1.468	0.0	0.5105;
104	100.2	0.0	20.35	38.31	9666.0	6.932	1.482	0.0	0.5170;
105	100.2	0.0	20.61	38.25	9538.7	7.024	1.496	0.0	0.5235;

106	100.2	0.0	20.87	38.19	9413.9	7.117	1.510	0.0	0.5301;
107	100.2	0.0	21.13	38.12	9291.7	7.211	1.523	0.0	0.5366;
108	100.2	0.0	21.39	38.06	9172.0	7.305	1.537	0.0	0.5432;
109	100.2	0.0	21.65	38.00	9054.7	7.399	1.550	0.0	0.5498;
110	100.2	0.0	21.91	37.94	8939.7	7.495	1.563	0.0	0.5565;
111	100.2	0.0	22.17	37.89	8827.0	7.590	1.577	0.0	0.5631;
112	100.2	0.0	22.43	37.83	8716.4	7.687	1.590	0.0	0.5698;
113	100.2	0.0	22.70	37.77	8608.0	7.783	1.602	0.0	0.5765;
114	100.2	0.0	22.96	37.72	8501.7	7.881	1.615	0.0	0.5832;
115	100.2	0.0	23.23	37.67	8397.4	7.979	1.628	0.0	0.5899;
116	100.2	0.0	23.49	37.61	8295.1	8.077	1.640	0.0	0.5967;
117	100.2	0.0	23.76	37.56	8194.7	8.176	1.653	0.0	0.6035;
118	100.2	0.0	24.03	37.51	8096.1	8.276	1.665	0.0	0.6103;
119	100.2	0.0	24.29	37.46	7999.4	8.376	1.677	0.0	0.6171;
120	100.2	0.0	24.56	37.41	7904.5	8.476	1.689	0.0	0.6239;
121	100.2	0.0	24.83	37.37	7811.3	8.577	1.701	0.0	0.6307;
122	100.3	0.0	25.10	37.32	7719.8	8.679	1.713	0.0	0.6376;
123	100.3	0.0	25.37	37.27	7630.0	8.781	1.725	0.0	0.6445;
124	100.3	0.0	25.64	37.23	7541.8	8.884	1.737	0.0	0.6514;
125	100.3	0.0	25.92	37.18	7455.1	8.987	1.748	0.0	0.6583;
126	100.3	0.0	26.19	37.14	7370.0	9.091	1.760	0.0	0.6652;
127	100.3	0.0	26.46	37.10	7286.4	9.195	1.771	0.0	0.6721;
128	100.3	0.0	26.74	37.06	7204.2	9.300	1.783	0.0	0.6791;
129	100.3	0.0	27.01	37.02	7123.5	9.406	1.794	0.0	0.6860;
130	100.3	0.0	27.28	36.97	7044.2	9.511	1.805	0.0	0.6930;
131	100.3	0.0	27.56	36.93	6966.2	9.618	1.816	0.0	0.7000;
132	100.3	0.0	27.83	36.90	6889.6	9.725	1.827	0.0	0.7070;
133	100.3	0.0	28.11	36.86	6814.3	9.832	1.838	0.0	0.7140;
134	100.3	0.0	28.39	36.82	6740.3	9.940	1.849	0.0	0.7210;
135	100.3	0.0	28.66	36.78	6667.5	10.05	1.860	0.0	0.7281;
136	100.3	0.0	28.94	36.75	6596.0	10.16	1.871	0.0	0.7351;
137	100.3	0.0	29.22	36.71	6525.6	10.27	1.882	0.0	0.7421;
138	100.3	0.0	29.49	36.67	6457.2	10.38	1.892	0.0	0.7491; begin overlap;
139	100.3	0.0	29.77	36.64	6390.8	10.48	1.903	0.0	0.7561;
140	100.3	0.0	30.04	36.61	6326.2	10.59	1.914	0.0	0.7629;
141	100.3	0.0	30.30	36.58	6263.3	10.70	1.924	0.0	0.7696;
142	100.4	0.0	30.56	36.54	6202.2	10.80	1.935	0.0	0.7763;
143	100.4	0.0	30.82	36.51	6142.6	10.91	1.945	0.0	0.7828;
144	100.4	0.0	31.08	36.48	6084.5	11.01	1.955	0.0	0.7893;
145	100.4	0.0	31.33	36.45	6027.9	11.12	1.966	0.0	0.7957;
146	100.4	0.0	31.58	36.43	5972.6	11.22	1.976	0.0	0.8021;
147	100.4	0.0	31.82	36.40	5918.6	11.32	1.986	0.0	0.8083;
148	100.4	0.0	32.07	36.37	5865.9	11.42	1.997	0.0	0.8145;
149	100.4	0.0	32.31	36.35	5814.3	11.52	2.007	0.0	0.8206;
150	100.4	0.0	32.55	36.32	5763.9	11.62	2.017	0.0	0.8267;
151	100.4	0.0	32.78	36.29	5714.6	11.72	2.027	0.0	0.8327;
152	100.4	0.0	33.02	36.27	5666.4	11.82	2.037	0.0	0.8386;
153	100.4	0.0	33.25	36.25	5619.1	11.92	2.047	0.0	0.8445;
154	100.4	0.0	33.47	36.22	5572.8	12.02	2.057	0.0	0.8502;
155	100.4	0.0	33.70	36.20	5527.4	12.12	2.067	0.0	0.8560;
156	100.4	0.0	33.92	36.18	5482.9	12.22	2.077	0.0	0.8617;
157	100.4	0.0	34.15	36.15	5439.2	12.32	2.087	0.0	0.8673;
158	100.4	0.0	34.36	36.13	5396.4	12.42	2.097	0.0	0.8729;
159	100.5	0.0	34.58	36.11	5354.3	12.51	2.107	0.0	0.8784;
160	100.5	0.0	34.80	36.09	5313.0	12.61	2.117	0.0	0.8838;
161	100.5	0.0	35.01	36.07	5272.4	12.71	2.127	0.0	0.8893;

162	100.5	0.0	35.22	36.05	5232.5	12.80	2.137	0.0	0.8946;
163	100.5	0.0	35.43	36.03	5193.3	12.90	2.147	0.0	0.8999;
164	100.5	0.0	35.64	36.01	5154.7	13.00	2.157	0.0	0.9052;
165	100.5	0.0	35.84	35.99	5116.8	13.09	2.167	0.0	0.9104;
166	100.5	0.0	36.05	35.97	5079.4	13.19	2.176	0.0	0.9156;
167	100.5	0.0	36.25	35.95	5042.7	13.29	2.186	0.0	0.9207;
168	100.5	0.0	36.45	35.93	5006.5	13.38	2.196	0.0	0.9258;
169	100.5	0.0	36.65	35.91	4970.8	13.48	2.206	0.0	0.9308;
170	100.5	0.0	36.84	35.90	4935.7	13.57	2.216	0.0	0.9358;
171	100.5	0.0	37.04	35.88	4901.1	13.67	2.226	0.0	0.9408;
172	100.5	0.0	37.23	35.86	4867.0	13.77	2.236	0.0	0.9457;
173	100.6	0.0	37.43	35.84	4833.3	13.86	2.246	0.0	0.9506;
174	100.6	0.0	37.62	35.83	4800.1	13.96	2.255	0.0	0.9554;
175	100.6	0.0	37.80	35.81	4767.3	14.05	2.265	0.0	0.9602;
176	100.6	0.0	37.99	35.79	4735.0	14.15	2.275	0.0	0.9650;
177	100.6	0.0	38.18	35.78	4703.1	14.25	2.285	0.0	0.9697;
178	100.6	0.0	38.36	35.76	4671.6	14.34	2.295	0.0	0.9744;
179	100.6	0.0	38.55	35.74	4640.5	14.44	2.305	0.0	0.9791;
180	100.6	0.0	38.73	35.73	4609.7	14.53	2.315	0.0	0.9837;
181	100.6	0.0	38.91	35.71	4579.3	14.63	2.325	0.0	0.9883;
182	100.6	0.0	39.09	35.70	4549.3	14.73	2.335	0.0	0.9929;
183	100.6	0.0	39.27	35.68	4519.6	14.82	2.345	0.0	0.9974;
184	100.6	0.0	39.45	35.67	4490.2	14.92	2.355	0.0	1.0019;
185	100.6	0.0	39.62	35.65	4461.2	15.02	2.365	0.0	1.0064;
186	100.7	0.0	39.80	35.64	4432.4	15.12	2.375	0.0	1.0108;
187	100.7	0.0	39.97	35.62	4404.0	15.21	2.385	0.0	1.0153;
188	100.7	0.0	40.14	35.61	4375.8	15.31	2.395	0.0	1.0196;
189	100.7	0.0	40.31	35.59	4348.0	15.41	2.405	0.0	1.0240;
190	100.7	0.0	40.49	35.58	4320.4	15.51	2.415	0.0	1.0283;
191	100.7	0.0	40.66	35.57	4293.0	15.61	2.425	0.0	1.0326;
192	100.7	0.0	40.82	35.55	4266.0	15.71	2.435	0.0	1.0369;
193	100.7	0.0	40.99	35.54	4239.1	15.81	2.445	0.0	1.0412;
194	100.7	0.0	41.16	35.53	4212.5	15.90	2.455	0.0	1.0454;
195	100.7	0.0	41.32	35.51	4186.2	16.01	2.466	0.0	1.0496;
196	100.7	0.0	41.49	35.50	4160.0	16.11	2.476	0.0	1.0538;
197	100.8	0.0	41.65	35.48	4134.1	16.21	2.486	0.0	1.0580;
198	100.8	0.0	41.82	35.47	4108.4	16.31	2.497	0.0	1.0621;
199	100.8	0.0	41.98	35.46	4082.9	16.41	2.507	0.0	1.0663;
200	100.8	0.0	42.14	35.45	4057.5	16.51	2.517	0.0	1.0704;
201	100.8	0.0	42.30	35.43	4032.4	16.62	2.528	0.0	1.0745;
202	100.8	0.0	42.46	35.42	4007.4	16.72	2.538	0.0	1.0785;
203	100.8	0.0	42.62	35.41	3982.7	16.82	2.549	0.0	1.0826;
204	100.8	0.0	42.78	35.39	3958.0	16.93	2.559	0.0	1.0866;
205	100.8	0.0	42.94	35.38	3933.6	17.03	2.570	0.0	1.0907;
206	100.8	0.0	43.10	35.37	3909.3	17.14	2.580	0.0	1.0947;
207	100.9	0.0	43.26	35.36	3885.2	17.25	2.591	0.0	1.0987;
208	100.9	0.0	43.41	35.35	3861.2	17.35	2.602	0.0	1.1027;
209	100.9	0.0	43.57	35.33	3837.3	17.46	2.612	0.0	1.1067;
210	100.9	0.0	43.73	35.32	3813.6	17.57	2.623	0.0	1.1106;
211	100.9	0.0	43.88	35.31	3790.0	17.68	2.634	0.0	1.1146;
212	100.9	0.0	44.04	35.30	3766.5	17.79	2.645	0.0	1.1185; merging;
213	100.9	0.0	44.19	35.28	3743.3	17.90	2.656	0.0	1.1225;
214	100.9	0.0	44.35	35.27	3720.2	18.01	2.667	0.0	1.1265;
215	101.0	0.0	44.51	35.26	3697.3	18.12	2.678	0.0	1.1305;
216	101.0	0.0	44.67	35.25	3674.5	18.23	2.689	0.0	1.1345;
217	101.0	0.0	44.82	35.24	3651.8	18.35	2.700	0.0	1.1385;

218	101.0	0.0	44.98	35.23	3629.2	18.46	2.711	0.0	1.1426;
219	101.0	0.0	45.14	35.21	3606.7	18.58	2.722	0.0	1.1467;
220	101.0	0.0	45.31	35.20	3584.2	18.69	2.734	0.0	1.1508;
221	101.0	0.0	45.47	35.19	3561.9	18.81	2.745	0.0	1.1549;
222	101.0	0.0	45.63	35.18	3539.5	18.93	2.756	0.0	1.1590;
223	101.1	0.0	45.79	35.17	3517.3	19.05	2.768	0.0	1.1631;
224	101.1	0.0	45.96	35.16	3495.1	19.17	2.779	0.0	1.1673;
225	101.1	0.0	46.12	35.15	3473.0	19.29	2.791	0.0	1.1715;
226	101.1	0.0	46.29	35.13	3450.9	19.42	2.803	0.0	1.1757;
227	101.1	0.0	46.46	35.12	3428.8	19.54	2.814	0.0	1.1800;
228	101.1	0.0	46.62	35.11	3406.8	19.67	2.826	0.0	1.1843;
229	101.1	0.0	46.79	35.10	3384.9	19.79	2.838	0.0	1.1885;
230	101.2	0.0	46.96	35.09	3363.2	19.92	2.850	0.0	1.1928; end overlap;
231	101.2	0.0	47.13	35.08	3341.8	20.05	2.862	0.0	1.1970;
232	101.2	0.0	47.29	35.07	3320.6	20.18	2.874	0.0	1.2012;
233	101.2	0.0	47.45	35.06	3299.6	20.31	2.887	0.0	1.2053;
234	101.2	0.0	47.61	35.05	3278.8	20.43	2.899	0.0	1.2094;
235	101.2	0.0	47.77	35.04	3258.2	20.56	2.911	0.0	1.2134;
236	101.3	0.0	47.93	35.03	3237.7	20.69	2.924	0.0	1.2174;
237	101.3	0.0	48.08	35.02	3217.5	20.82	2.936	0.0	1.2213;
238	101.3	0.0	48.23	35.00	3197.5	20.95	2.949	0.0	1.2252;
239	101.3	0.0	48.39	34.99	3177.6	21.08	2.962	0.0	1.2290;
240	101.3	0.0	48.53	34.98	3158.0	21.22	2.975	0.0	1.2327;
241	101.4	0.0	48.68	34.97	3138.4	21.35	2.988	0.0	1.2364;
242	101.4	0.0	48.82	34.96	3119.1	21.48	3.001	0.0	1.2401;
243	101.4	0.0	48.96	34.95	3099.9	21.61	3.014	0.0	1.2437;
244	101.4	0.0	49.10	34.95	3080.9	21.75	3.027	0.0	1.2472;
245	101.4	0.0	49.24	34.94	3062.1	21.88	3.040	0.0	1.2507;
246	101.5	0.0	49.37	34.93	3043.4	22.02	3.054	0.0	1.2541;
247	101.5	0.0	49.51	34.92	3024.8	22.15	3.068	0.0	1.2575;
248	101.5	0.0	49.64	34.91	3006.4	22.29	3.081	0.0	1.2608;
249	101.5	0.0	49.76	34.90	2988.1	22.42	3.095	0.0	1.2640;
250	101.5	0.0	49.89	34.89	2970.0	22.56	3.109	0.0	1.2672;
251	101.6	0.0	50.01	34.88	2951.9	22.70	3.124	0.0	1.2703;
252	101.6	0.0	50.13	34.87	2934.0	22.84	3.138	0.0	1.2734;
253	101.6	0.0	50.25	34.86	2916.2	22.97	3.152	0.0	1.2764;
254	101.6	0.0	50.37	34.85	2898.5	23.12	3.167	0.0	1.2794;
255	101.7	0.0	50.48	34.84	2881.0	23.26	3.182	0.0	1.2823;
256	101.7	0.0	50.60	34.83	2863.5	23.40	3.197	0.0	1.2851;
257	101.7	0.0	50.71	34.82	2846.1	23.54	3.212	0.0	1.2879;
258	101.7	0.0	50.81	34.82	2828.8	23.68	3.227	0.0	1.2907;
259	101.8	0.0	50.92	34.81	2811.6	23.83	3.242	0.0	1.2934;
260	101.8	0.0	51.02	34.80	2794.4	23.98	3.258	0.0	1.2960;
261	101.8	0.0	51.12	34.79	2777.4	24.12	3.274	0.0	1.2986;
262	101.9	0.0	51.22	34.78	2760.4	24.27	3.290	0.0	1.3011;
263	101.9	0.0	51.32	34.77	2743.4	24.42	3.306	0.0	1.3036;
264	101.9	0.0	51.42	34.76	2726.5	24.57	3.322	0.0	1.3060;
265	102.0	0.0	51.51	34.75	2709.6	24.73	3.339	0.0	1.3084;
266	102.0	0.0	51.61	34.75	2692.8	24.88	3.356	0.0	1.3108;
267	102.0	0.0	51.70	34.74	2676.0	25.04	3.373	0.0	1.3131;
268	102.1	0.0	51.79	34.73	2659.3	25.19	3.390	0.0	1.3154;
269	102.1	0.0	51.87	34.72	2642.5	25.35	3.407	0.0	1.3176;
270	102.1	0.0	51.96	34.71	2625.8	25.52	3.425	0.0	1.3198;
271	102.2	0.0	52.05	34.70	2609.0	25.68	3.443	0.0	1.3220;
272	102.2	0.0	52.13	34.69	2592.3	25.85	3.461	0.0	1.3242;
273	102.2	0.0	52.22	34.69	2575.5	26.01	3.479	0.0	1.3263;

274	102.3	0.0	52.30	34.68	2558.7	26.18	3.498	0.0	1.3284;
275	102.3	0.0	52.38	34.67	2541.9	26.36	3.517	0.0	1.3305;
276	102.4	0.0	52.47	34.66	2525.1	26.53	3.536	0.0	1.3326;
277	102.4	0.0	52.55	34.65	2508.2	26.71	3.556	0.0	1.3347;
278	102.5	0.0	52.63	34.64	2491.2	26.89	3.575	0.0	1.3368;
279	102.5	0.0	52.71	34.63	2474.2	27.08	3.595	0.0	1.3389;
280	102.5	0.0	52.80	34.63	2457.1	27.27	3.616	0.0	1.3410;
281	102.6	0.0	52.88	34.62	2440.0	27.46	3.636	0.0	1.3431;
282	102.6	0.0	52.96	34.61	2422.7	27.66	3.657	0.0	1.3452;
283	102.7	0.0	53.05	34.60	2405.3	27.85	3.679	0.0	1.3474;
284	102.8	0.0	53.13	34.59	2387.8	28.06	3.701	0.0	1.3496;
285	102.8	0.0	53.22	34.58	2370.2	28.27	3.723	0.0	1.3518;
286	102.9	0.0	53.31	34.57	2352.5	28.48	3.745	0.0	1.3541;
287	102.9	0.0	53.40	34.56	2334.6	28.70	3.768	0.0	1.3564;
288	103.0	0.0	53.50	34.55	2316.6	28.92	3.791	0.0	1.3588;
289	103.0	0.0	53.59	34.54	2298.4	29.15	3.815	0.0	1.3613;
290	103.1	0.0	53.69	34.53	2280.1	29.38	3.839	0.0	1.3638;
291	103.2	0.0	53.80	34.52	2261.5	29.63	3.863	0.0	1.3664;
292	103.2	0.0	53.90	34.52	2242.8	29.87	3.888	0.0	1.3692;
293	103.3	0.0	54.02	34.51	2223.9	30.13	3.913	0.0	1.3720;
294	103.4	0.0	54.13	34.50	2204.7	30.39	3.939	0.0	1.3750;
295	103.5	0.0	54.25	34.49	2185.3	30.66	3.966	0.0	1.3780;
296	103.5	0.0	54.38	34.48	2165.7	30.94	3.992	0.0	1.3813;
297	103.6	0.0	54.51	34.47	2145.8	31.22	4.020	0.0	1.3846;
298	103.7	0.0	54.65	34.45	2125.7	31.52	4.048	0.0	1.3882;
299	103.8	0.0	54.80	34.44	2105.3	31.83	4.076	0.0	1.3919; bottom hit;

Horiz plane projections in effluent direction: radius(m): 0.0; CL(m): 1.2424
Lmz(m): 1.2424
forced entrain 1 0.0 -1.159 1.392 0.312
Rate sec-1 0.0 dy-1 0.0 kt: 0.0 Amb Sal 33.3632
;
1:30:39 PM. amb fills: 4

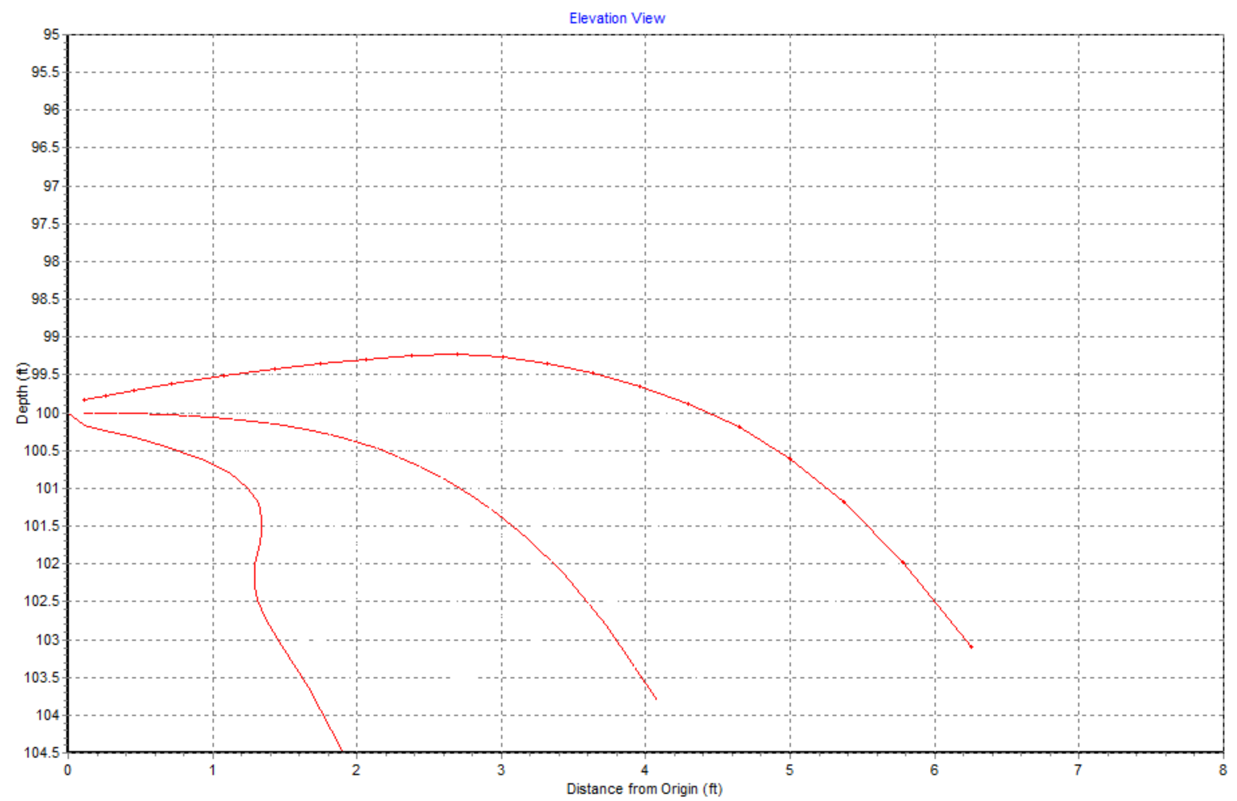

Figure C.3.1: Plumes 18b solution of discharge plume trajectories for discharges of 0 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt., and 15 mgd of brine from the Doheny Desalination Project with a brine salinity of 67 ppt. Depth of maximum rise of the plume is Z = 99.2 ft. at Xa = 2.803 ft from the point of discharge. The centerline of the plume is at an average distance of Xb = 4.076 ft from the point of discharge as the plume begins to impact the bottom at a depth of Z = 104.5 ft.

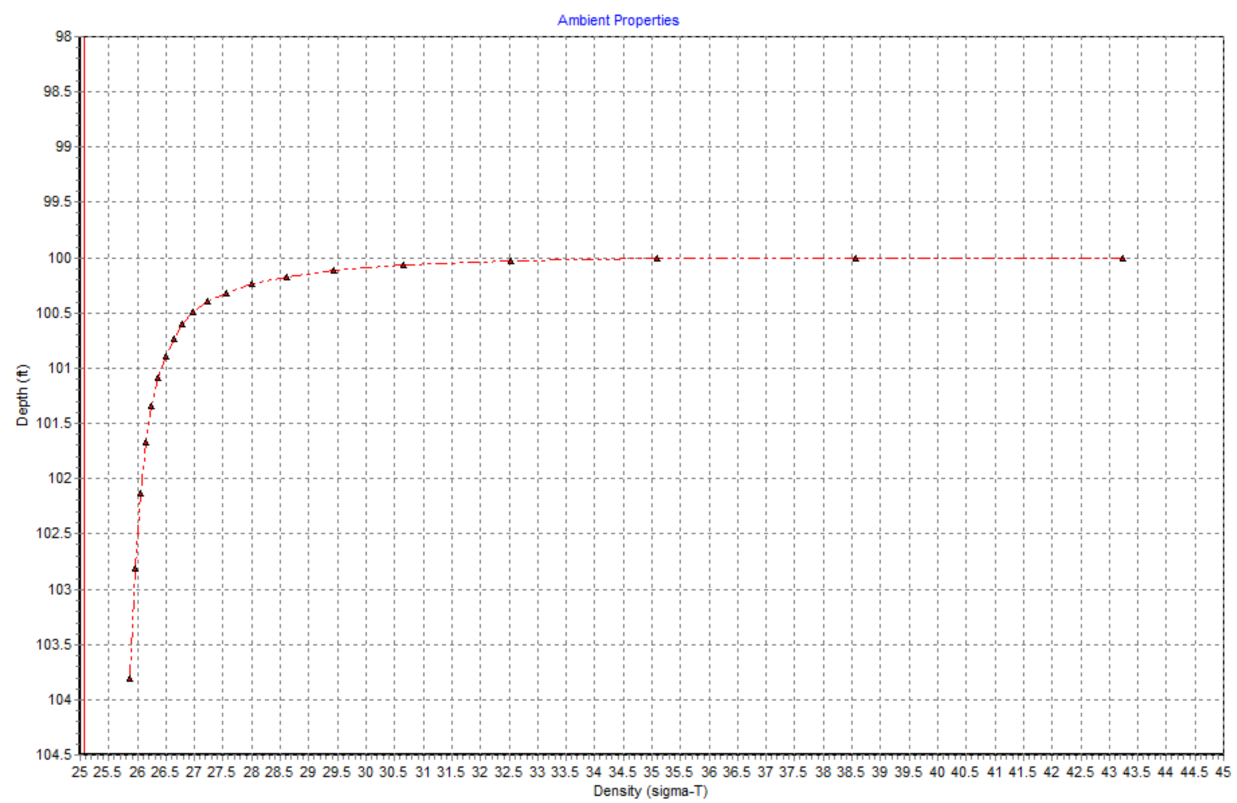

Figure C.3.2: Plumes 18b solution of vertical density profile for discharges of 0 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt. and 15 mgd of brine from the Doheny Desalination Project with a brine salinity of 67 ppt. Discharge effluent density shown as black triangles. Ambient water mass density profile shown as solid red line.

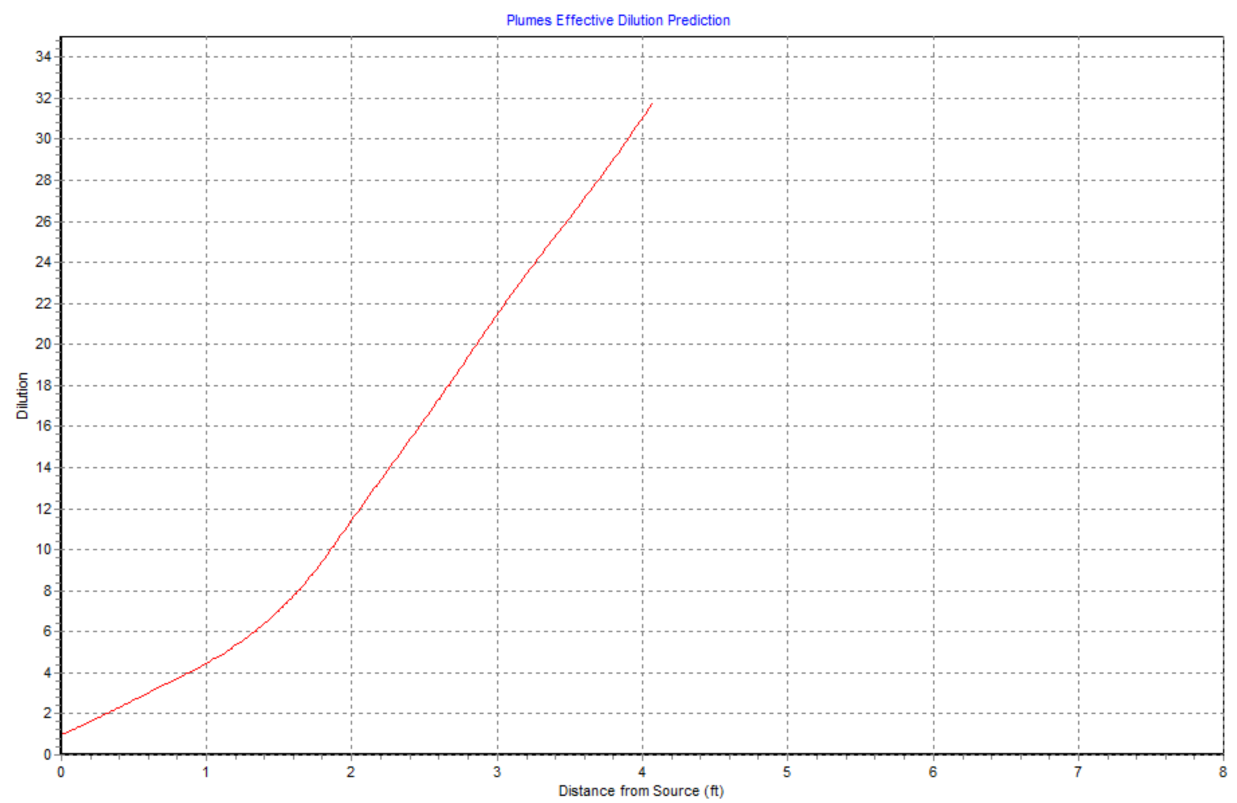

Figure C.3.3: Plumes 18b solution of effective (bulk average) dilution as a function of vertical distance from the point of discharge for discharges of 0 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt. and 15 mgd of brine from the Doheny Desalination Project with a brine salinity of 67 ppt. Effective dilution is S_a = 19.42 at the maximum rise of the plume at Xa = 2.803 ft. from the point of discharge. As the plume begins to impact the bottom, the plume centerline is at a distance of Xb = 4.076 ft from the point of discharge, where the effective dilution reaches S_a = 31.83.

C.4: Plumes 18b Results for SJCOO discharges of 0 mgd Wastewater and 10 mgd Brine:

Contents of the memo box (may not be current and must be updated manually)
Project "C:\Plumes18\SJCOO_WW0mgd_b10mgd_D-1"
memomemo
SJCOO discharging 0 mgd wastewater and 10 mgd brine

Model configuration items checked:
Channel width (m) 100
Start case for graphs 1
Max detailed graphs 10 (limits plots that can overflow memory)
Elevation Projection Plane (deg) 0
Shore vector (m,deg) not checked
Bacteria model : Mancini (1978) coliform model
PDS sfc. model heat transfer : Medium
Equation of State : S, T
Similarity Profile : Default profile (k=2.0, ...)
Diffuser port contraction coefficient 1
Light absorption coefficient 0.16
Farfield increment (m) 200
UM3 aspiration coefficient 0.1
Output file: text output tab
Output each ?? steps 1
Maximum dilution reported 1000
Text output format : Standard
Max vertical reversals : to max rise or fall

/ UM3. 1/14/2019 1:47:00 PM
Case 1; ambient file C:\Plumes18\SJCOO_WW0mgd_b10mgd_D-1.001.db; Diffuser table record 1: ----------------------------------

Ambient Table:

Depth	Amb-cur	Amb-dir	Amb-sal	Amb-tem	Amb-pol	Decay	Far-spd	Far-dir	Disprsn	Density
m	m/s	deg	psu	C	kg/kg	s-1	m/s	deg	m0.67/s2	sigma-T
0.0	0.0	0.0	33.38	19.97	0.0	0.0	0.0	0.0	0.0003	23.55719
2.000	0.0	0.0	33.39	19.87	0.0	0.0	0.0	0.0	0.0003	23.59288
4.000	0.0	0.0	33.40	19.81	0.0	0.0	0.0	0.0	0.0003	23.61484
7.000	0.0	0.0	33.41	19.64	0.0	0.0	0.0	0.0	0.0003	23.67096
10.00	0.0	0.0	33.40	17.76	0.0	0.0	0.0	0.0	0.0003	24.12888
12.00	0.0	0.0	33.38	17.34	0.0	0.0	0.0	0.0	0.0003	24.21549
14.00	0.0	0.0	33.38	16.73	0.0	0.0	0.0	0.0	0.0003	24.35932
16.00	0.0	0.0	33.38	15.17	0.0	0.0	0.0	0.0	0.0003	24.71340
18.00	0.0	0.0	33.37	14.37	0.0	0.0	0.0	0.0	0.0003	24.87044
20.00	0.0	0.0	33.35	14.18	0.0	0.0	0.0	0.0	0.0003	24.89660
22.00	0.0	0.0	33.33	13.83	0.0	0.0	0.0	0.0	0.0003	24.95172
24.00	0.0	0.0	33.32	13.75	0.0	0.0	0.0	0.0	0.0003	24.96707
26.00	0.0	0.0	33.36	13.42	0.0	0.0	0.0	0.0	0.0003	25.06459
31.85	0.0	0.0	33.36	13.39	0.0	0.0	0.0	0.0	0.0003	25.07201

Diffuser table:

P-dia	Ver angl	H-Angle	SourceX	SourceY	Ports	MZ-dis	Isoplth	P-depth	Ttl-flo	Eff-sal	Temp	Polutnt
(in)	(deg)	(deg)	(ft)	(ft)	()	(ft)	(concent)	(ft)	(MGD)	(psu)	(C)	(ppm)
3.0500	0.0	0.0	0.0	0.0	125.00	1000.0	0.0	100.00	10.000	67.000	20.660	67000.0

Simulation:

Froude No: -5.593; Strat No:-1.57E-4; Spcg No: 14.39; k: 74358.5; eff den (sigmaT) 49.48870; eff vel 0.744(m/s);
Current is very small, flow regime may be transient.

Step	Depth (ft)	Amb-cur (m/s)	P-dia (in)	Eff-sal (psu)	Polutnt (ppm)	Dilutn ()	x-posn (ft)	y-posn (ft)	Iso dia (m)
0	100.0	1.000E-5	3.050	67.00	67000.0	1.000	0.0	0.0	0.07747;
1	100.0	0.0	3.068	66.63	66237.3	1.012	0.00724	0.0	0.07791;
2	100.0	0.0	3.116	65.97	64908.9	1.032	0.0127	0.0	0.07915;
3	100.0	0.0	3.179	65.33	63607.8	1.053	0.0174	0.0	0.08075;
4	100.0	0.0	3.243	64.71	62333.3	1.075	0.0215	0.0	0.08238;
5	100.0	0.0	3.309	64.09	61084.8	1.097	0.0255	0.0	0.08405;
6	100.0	0.0	3.376	63.49	59861.9	1.119	0.0292	0.0	0.08575;
7	100.0	0.0	3.444	62.90	58664.0	1.142	0.0329	0.0	0.08748;
8	100.0	0.0	3.514	62.32	57490.5	1.165	0.0364	0.0	0.08925;
9	100.0	0.0	3.585	61.75	56340.9	1.189	0.040	0.0	0.09105;
10	100.0	0.0	3.657	61.20	55214.7	1.213	0.0435	0.0	0.09289;
11	100.0	0.0	3.731	60.65	54111.5	1.238	0.0471	0.0	0.09477;
12	100.0	0.0	3.806	60.12	53030.7	1.263	0.0507	0.0	0.09668;
13	100.0	0.0	3.883	59.59	51971.9	1.289	0.0543	0.0	0.09863;
14	100.0	0.0	3.961	59.08	50934.6	1.315	0.0579	0.0	0.1006;
15	100.0	0.0	4.041	58.57	49918.3	1.342	0.0616	0.0	0.1027;
16	100.0	0.0	4.123	58.08	48922.7	1.370	0.0653	0.0	0.1047;
17	100.0	0.0	4.206	57.59	47947.2	1.397	0.0691	0.0	0.1068;
18	100.0	0.0	4.291	57.12	46991.6	1.426	0.0729	0.0	0.1090;
19	100.0	0.0	4.377	56.65	46055.2	1.455	0.0768	0.0	0.1112;
20	100.0	0.0	4.466	56.20	45137.8	1.484	0.0808	0.0	0.1134;
21	100.0	0.0	4.556	55.75	44239.0	1.515	0.0848	0.0	0.1157;
22	100.0	0.0	4.647	55.31	43358.3	1.545	0.0889	0.0	0.1180;
23	100.0	0.0	4.741	54.88	42495.4	1.577	0.093	0.0	0.1204;
24	100.0	0.0	4.837	54.46	41649.9	1.609	0.0973	0.0	0.1228;
25	100.0	0.0	4.934	54.04	40821.5	1.641	0.102	0.0	0.1253;
26	100.0	0.0	5.033	53.64	40009.7	1.675	0.106	0.0	0.1278;
27	100.0	0.0	5.135	53.24	39214.4	1.709	0.110	0.0	0.1304;
28	100.0	0.0	5.238	52.85	38435.0	1.743	0.115	0.0	0.1330;
29	100.0	0.0	5.343	52.47	37671.4	1.779	0.119	0.0	0.1357;
30	100.0	0.0	5.451	52.09	36923.1	1.815	0.124	0.0	0.1385;
31	100.0	0.0	5.561	51.73	36189.8	1.851	0.129	0.0	0.1412;
32	100.0	0.0	5.673	51.37	35471.3	1.889	0.134	0.0	0.1441;
33	100.0	0.0	5.787	51.01	34767.2	1.927	0.139	0.0	0.1470;
34	100.0	0.0	5.903	50.67	34077.3	1.966	0.144	0.0	0.1499;
35	100.0	0.0	6.022	50.33	33401.2	2.006	0.149	0.0	0.1529;
36	100.0	0.0	6.143	50.00	32738.7	2.047	0.154	0.0	0.1560;
37	100.0	0.0	6.266	49.67	32089.5	2.088	0.159	0.0	0.1592;
38	100.0	0.0	6.392	49.35	31453.3	2.130	0.165	0.0	0.1624;
39	100.0	0.0	6.520	49.04	30829.8	2.173	0.170	0.0	0.1656;
40	100.0	0.0	6.651	48.73	30218.8	2.217	0.176	0.0	0.1689;
41	100.0	0.0	6.785	48.43	29620.0	2.262	0.181	0.0	0.1723;
42	100.0	0.0	6.921	48.13	29033.3	2.308	0.187	0.0	0.1758;
43	100.0	0.0	7.060	47.84	28458.2	2.354	0.193	0.0	0.1793;
44	100.0	0.0	7.202	47.56	27894.7	2.402	0.199	0.0	0.1829;
45	100.0	0.0	7.346	47.28	27342.4	2.450	0.205	0.0	0.1866;
46	100.0	0.0	7.494	47.01	26801.2	2.500	0.211	0.0	0.1903;
47	100.0	0.0	7.644	46.74	26270.8	2.550	0.217	0.0	0.1942;
48	100.0	0.0	7.797	46.48	25750.9	2.602	0.224	0.0	0.1980;
49	100.0	0.0	7.953	46.22	25241.5	2.654	0.230	0.0	0.2020;
50	100.0	0.0	8.113	45.97	24742.2	2.708	0.237	0.0	0.2061;

51	100.0	0.0	8.275	45.72	24252.8	2.763	0.244	0.0	0.2102;
52	100.0	0.0	8.441	45.48	23773.3	2.818	0.251	0.0	0.2144;
53	100.0	0.0	8.610	45.24	23303.3	2.875	0.258	0.0	0.2187;
54	100.0	0.0	8.782	45.01	22842.6	2.933	0.265	0.0	0.2231;
55	100.0	0.0	8.958	44.78	22391.1	2.992	0.272	0.0	0.2275;
56	100.0	0.0	9.137	44.56	21948.7	3.053	0.279	0.0	0.2321;
57	100.0	0.0	9.319	44.34	21515.0	3.114	0.287	0.0	0.2367;
58	100.0	0.0	9.505	44.12	21089.9	3.177	0.295	0.0	0.2414;
59	100.0	0.0	9.695	43.91	20673.4	3.241	0.303	0.0	0.2463;
60	100.0	0.0	9.889	43.70	20265.1	3.306	0.310	0.0	0.2512;
61	100.0	0.0	10.09	43.50	19864.9	3.373	0.319	0.0	0.2562;
62	100.0	0.0	10.29	43.30	19472.7	3.441	0.327	0.0	0.2613;
63	100.0	0.0	10.49	43.11	19088.2	3.510	0.335	0.0	0.2665;
64	100.0	0.0	10.70	42.92	18711.5	3.581	0.344	0.0	0.2718;
65	100.0	0.0	10.91	42.73	18342.2	3.653	0.353	0.0	0.2772;
66	100.0	0.0	11.13	42.55	17980.2	3.726	0.362	0.0	0.2827;
67	100.0	0.0	11.35	42.37	17625.4	3.801	0.371	0.0	0.2883;
68	100.0	0.0	11.58	42.19	17277.7	3.878	0.380	0.0	0.2940;
69	100.0	0.0	11.81	42.02	16936.8	3.956	0.389	0.0	0.2998;
70	100.0	0.0	12.04	41.85	16602.8	4.035	0.399	0.0	0.3058;
71	100.0	0.0	12.28	41.68	16276.6	4.116	0.409	0.0	0.3118;
72	100.0	0.0	12.51	41.52	15967.8	4.196	0.418	0.0	0.3178;
73	100.0	0.0	12.75	41.37	15667.2	4.276	0.428	0.0	0.3238;
74	100.0	0.0	12.99	41.22	15374.5	4.358	0.437	0.0	0.3299;
75	100.0	0.0	13.23	41.08	15089.3	4.440	0.446	0.0	0.3360;
76	100.0	0.0	13.47	40.94	14811.3	4.524	0.455	0.0	0.3422;
77	100.0	0.0	13.72	40.80	14540.4	4.608	0.464	0.0	0.3485;
78	100.0	0.0	13.97	40.66	14276.2	4.693	0.473	0.0	0.3548;
79	100.0	0.0	14.22	40.53	14018.5	4.779	0.481	0.0	0.3611;
80	100.0	0.0	14.47	40.40	13767.1	4.867	0.490	0.0	0.3676;
81	100.0	0.0	14.73	40.28	13521.7	4.955	0.498	0.0	0.3741;
82	100.0	0.0	14.99	40.16	13282.2	5.044	0.506	0.0	0.3807;
83	100.0	0.0	15.25	40.04	13048.4	5.135	0.514	0.0	0.3873;
84	100.0	0.0	15.51	39.92	12820.1	5.226	0.522	0.0	0.3941;
85	100.0	0.0	15.78	39.81	12597.1	5.319	0.530	0.0	0.4009;
86	100.0	0.0	16.05	39.70	12379.2	5.412	0.537	0.0	0.4077;
87	100.0	0.0	16.32	39.59	12166.3	5.507	0.545	0.0	0.4146;
88	100.0	0.0	16.60	39.48	11958.2	5.603	0.552	0.0	0.4216;
89	100.0	0.0	16.88	39.38	11754.8	5.700	0.560	0.0	0.4287;
90	100.0	0.0	17.16	39.28	11556.0	5.798	0.567	0.0	0.4359;
91	100.1	0.0	17.44	39.18	11361.6	5.897	0.574	0.0	0.4431;
92	100.1	0.0	17.73	39.08	11171.4	5.997	0.581	0.0	0.4504;
93	100.1	0.0	18.02	38.99	10990.6	6.096	0.588	0.0	0.4576; begin overlap;
94	100.1	0.0	18.30	38.90	10818.5	6.193	0.594	0.0	0.4647;
95	100.1	0.0	18.57	38.82	10654.5	6.288	0.601	0.0	0.4717;
96	100.1	0.0	18.84	38.74	10497.9	6.382	0.608	0.0	0.4785;
97	100.1	0.0	19.10	38.66	10348.1	6.475	0.614	0.0	0.4852;
98	100.1	0.0	19.36	38.59	10204.7	6.566	0.621	0.0	0.4918;
99	100.1	0.0	19.62	38.52	10067.1	6.655	0.627	0.0	0.4983;
100	100.1	0.0	19.87	38.45	9934.9	6.744	0.633	0.0	0.5047;
101	100.1	0.0	20.12	38.39	9807.8	6.831	0.640	0.0	0.5109;
102	100.1	0.0	20.36	38.32	9685.5	6.918	0.646	0.0	0.5171;
103	100.1	0.0	20.60	38.26	9567.5	7.003	0.652	0.0	0.5232;
104	100.1	0.0	20.83	38.21	9453.8	7.087	0.658	0.0	0.5291;
105	100.1	0.0	21.06	38.15	9343.9	7.170	0.664	0.0	0.5350;
106	100.1	0.0	21.29	38.10	9237.7	7.253	0.670	0.0	0.5408;

107	100.1	0.0	21.52	38.04	9134.9	7.334	0.676	0.0	0.5465;
108	100.1	0.0	21.74	37.99	9035.4	7.415	0.682	0.0	0.5522;
109	100.1	0.0	21.96	37.94	8939.0	7.495	0.688	0.0	0.5578;
110	100.1	0.0	22.17	37.90	8845.5	7.574	0.694	0.0	0.5632;
111	100.1	0.0	22.39	37.85	8754.7	7.653	0.699	0.0	0.5687;
112	100.1	0.0	22.60	37.80	8666.6	7.731	0.705	0.0	0.5740;
113	100.1	0.0	22.81	37.76	8580.9	7.808	0.711	0.0	0.5793;
114	100.1	0.0	23.01	37.72	8497.6	7.885	0.716	0.0	0.5845;
115	100.1	0.0	23.22	37.68	8416.5	7.961	0.722	0.0	0.5897;
116	100.1	0.0	23.42	37.64	8337.5	8.036	0.728	0.0	0.5948;
117	100.1	0.0	23.61	37.60	8260.6	8.111	0.733	0.0	0.5998;
118	100.1	0.0	23.81	37.56	8185.6	8.185	0.739	0.0	0.6048;
119	100.1	0.0	24.00	37.52	8112.5	8.259	0.744	0.0	0.6097;
120	100.1	0.0	24.19	37.48	8041.1	8.332	0.750	0.0	0.6145;
121	100.1	0.0	24.38	37.45	7971.5	8.405	0.755	0.0	0.6193;
122	100.1	0.0	24.57	37.41	7903.4	8.477	0.760	0.0	0.6241;
123	100.1	0.0	24.76	37.38	7836.9	8.549	0.766	0.0	0.6288;
124	100.1	0.0	24.94	37.35	7771.9	8.621	0.771	0.0	0.6335;
125	100.1	0.0	25.12	37.31	7708.3	8.692	0.777	0.0	0.6381;
126	100.1	0.0	25.30	37.28	7646.0	8.763	0.782	0.0	0.6426;
127	100.1	0.0	25.48	37.25	7585.1	8.833	0.787	0.0	0.6471;
128	100.1	0.0	25.65	37.22	7525.5	8.903	0.792	0.0	0.6516;
129	100.1	0.0	25.83	37.19	7467.0	8.973	0.798	0.0	0.6560;
130	100.1	0.0	26.00	37.16	7409.7	9.042	0.803	0.0	0.6604;
131	100.1	0.0	26.17	37.13	7353.6	9.111	0.808	0.0	0.6647;
132	100.1	0.0	26.34	37.10	7298.5	9.180	0.813	0.0	0.6690;
133	100.1	0.0	26.51	37.08	7244.5	9.248	0.819	0.0	0.6732;
134	100.1	0.0	26.67	37.05	7191.5	9.317	0.824	0.0	0.6774;
135	100.1	0.0	26.83	37.02	7139.4	9.384	0.829	0.0	0.6816;
136	100.1	0.0	27.00	37.00	7088.3	9.452	0.834	0.0	0.6857;
137	100.1	0.0	27.16	36.97	7038.1	9.520	0.839	0.0	0.6898;
138	100.1	0.0	27.32	36.95	6988.8	9.587	0.844	0.0	0.6939;
139	100.1	0.0	27.47	36.92	6940.3	9.654	0.849	0.0	0.6979;
140	100.2	0.0	27.63	36.90	6892.6	9.721	0.855	0.0	0.7018;
141	100.2	0.0	27.79	36.87	6845.7	9.787	0.860	0.0	0.7058;
142	100.2	0.0	27.94	36.85	6799.6	9.854	0.865	0.0	0.7097;
143	100.2	0.0	28.09	36.83	6754.2	9.920	0.870	0.0	0.7135;
144	100.2	0.0	28.24	36.80	6709.6	9.986	0.875	0.0	0.7174;
145	100.2	0.0	28.39	36.78	6665.6	10.05	0.880	0.0	0.7212;
146	100.2	0.0	28.54	36.76	6622.3	10.12	0.885	0.0	0.7249;
147	100.2	0.0	28.69	36.74	6579.6	10.18	0.890	0.0	0.7286;
148	100.2	0.0	28.83	36.72	6537.6	10.25	0.895	0.0	0.7323;
149	100.2	0.0	28.98	36.69	6496.2	10.31	0.900	0.0	0.7360;
150	100.2	0.0	29.12	36.67	6455.4	10.38	0.905	0.0	0.7396;
151	100.2	0.0	29.26	36.65	6415.1	10.44	0.910	0.0	0.7432;
152	100.2	0.0	29.40	36.63	6375.5	10.51	0.915	0.0	0.7468;
153	100.2	0.0	29.54	36.61	6336.3	10.57	0.920	0.0	0.7503;
154	100.2	0.0	29.68	36.59	6297.7	10.64	0.925	0.0	0.7538;
155	100.2	0.0	29.81	36.57	6259.6	10.70	0.930	0.0	0.7573;
156	100.2	0.0	29.95	36.55	6222.0	10.77	0.935	0.0	0.7607;
157	100.2	0.0	30.08	36.54	6184.9	10.83	0.940	0.0	0.7642;
158	100.2	0.0	30.22	36.52	6148.2	10.90	0.945	0.0	0.7675;
159	100.2	0.0	30.35	36.50	6112.1	10.96	0.950	0.0	0.7709;
160	100.2	0.0	30.48	36.48	6076.3	11.03	0.956	0.0	0.7742;
161	100.2	0.0	30.61	36.46	6041.0	11.09	0.961	0.0	0.7775;
162	100.2	0.0	30.74	36.44	6006.1	11.16	0.966	0.0	0.7808;

163	100.2	0.0	30.87	36.43	5971.6	11.22	0.971	0.0	0.7841;
164	100.2	0.0	30.99	36.41	5937.5	11.28	0.976	0.0	0.7873;
165	100.2	0.0	31.12	36.39	5903.8	11.35	0.981	0.0	0.7905;
166	100.2	0.0	31.25	36.37	5870.5	11.41	0.986	0.0	0.7936;
167	100.2	0.0	31.37	36.36	5837.5	11.48	0.991	0.0	0.7968;
168	100.2	0.0	31.49	36.34	5804.9	11.54	0.996	0.0	0.7999;
169	100.2	0.0	31.61	36.32	5772.6	11.61	1.001	0.0	0.8030;
170	100.2	0.0	31.73	36.31	5740.7	11.67	1.006	0.0	0.8060;
171	100.2	0.0	31.85	36.29	5709.1	11.74	1.011	0.0	0.8091;
172	100.3	0.0	31.97	36.28	5677.9	11.80	1.016	0.0	0.8121;
173	100.3	0.0	32.09	36.26	5646.9	11.86	1.021	0.0	0.8151;
174	100.3	0.0	32.21	36.24	5616.2	11.93	1.026	0.0	0.8180;
175	100.3	0.0	32.32	36.23	5585.9	11.99	1.032	0.0	0.8210;
176	100.3	0.0	32.44	36.21	5555.8	12.06	1.037	0.0	0.8239;
177	100.3	0.0	32.55	36.20	5526.0	12.12	1.042	0.0	0.8268;
178	100.3	0.0	32.66	36.18	5496.5	12.19	1.047	0.0	0.8296;
179	100.3	0.0	32.78	36.17	5467.3	12.25	1.052	0.0	0.8325;
180	100.3	0.0	32.89	36.15	5438.3	12.32	1.057	0.0	0.8353;
181	100.3	0.0	33.00	36.14	5409.5	12.39	1.063	0.0	0.8381;
182	100.3	0.0	33.11	36.12	5381.0	12.45	1.068	0.0	0.8409;
183	100.3	0.0	33.21	36.11	5352.8	12.52	1.073	0.0	0.8437;
184	100.3	0.0	33.32	36.09	5324.8	12.58	1.078	0.0	0.8464;
185	100.3	0.0	33.43	36.08	5297.0	12.65	1.084	0.0	0.8491;
186	100.3	0.0	33.53	36.07	5269.4	12.71	1.089	0.0	0.8518;
187	100.3	0.0	33.64	36.05	5242.0	12.78	1.094	0.0	0.8545;
188	100.3	0.0	33.74	36.04	5214.9	12.85	1.099	0.0	0.8571;
189	100.3	0.0	33.85	36.02	5187.9	12.91	1.105	0.0	0.8597;
190	100.3	0.0	33.95	36.01	5161.2	12.98	1.110	0.0	0.8623;
191	100.3	0.0	34.05	36.00	5134.6	13.05	1.115	0.0	0.8649;
192	100.3	0.0	34.15	35.98	5108.3	13.12	1.121	0.0	0.8675;
193	100.3	0.0	34.25	35.97	5082.1	13.18	1.126	0.0	0.8700;
194	100.3	0.0	34.35	35.96	5056.1	13.25	1.132	0.0	0.8726;
195	100.4	0.0	34.45	35.94	5030.3	13.32	1.137	0.0	0.8751;
196	100.4	0.0	34.55	35.93	5004.6	13.39	1.142	0.0	0.8776;
197	100.4	0.0	34.65	35.92	4979.1	13.46	1.148	0.0	0.8801;
198	100.4	0.0	34.74	35.90	4953.8	13.53	1.153	0.0	0.8825;
199	100.4	0.0	34.84	35.89	4928.6	13.59	1.159	0.0	0.8849;
200	100.4	0.0	34.94	35.88	4903.5	13.66	1.165	0.0	0.8874;
201	100.4	0.0	35.03	35.87	4878.6	13.73	1.170	0.0	0.8898;
202	100.4	0.0	35.12	35.85	4853.9	13.80	1.176	0.0	0.8921;
203	100.4	0.0	35.22	35.84	4829.2	13.87	1.181	0.0	0.8945;
204	100.4	0.0	35.31	35.83	4804.7	13.94	1.187	0.0	0.8969;
205	100.4	0.0	35.40	35.82	4780.4	14.02	1.193	0.0	0.8992;
206	100.4	0.0	35.49	35.80	4756.1	14.09	1.198	0.0	0.9015;
207	100.4	0.0	35.58	35.79	4732.0	14.16	1.204	0.0	0.9038;
208	100.4	0.0	35.67	35.78	4708.0	14.23	1.210	0.0	0.9061;
209	100.4	0.0	35.76	35.77	4684.1	14.30	1.216	0.0	0.9084;
210	100.4	0.0	35.85	35.75	4660.2	14.38	1.221	0.0	0.9106;
211	100.4	0.0	35.94	35.74	4636.5	14.45	1.227	0.0	0.9129;
212	100.4	0.0	36.03	35.73	4612.9	14.52	1.233	0.0	0.9151;
213	100.5	0.0	36.12	35.72	4589.4	14.60	1.239	0.0	0.9173;
214	100.5	0.0	36.20	35.71	4566.0	14.67	1.245	0.0	0.9195;
215	100.5	0.0	36.29	35.69	4542.6	14.75	1.251	0.0	0.9217;
216	100.5	0.0	36.37	35.68	4519.4	14.83	1.257	0.0	0.9239;
217	100.5	0.0	36.46	35.67	4496.2	14.90	1.263	0.0	0.9261;
218	100.5	0.0	36.54	35.66	4473.1	14.98	1.269	0.0	0.9282;

219	100.5	0.0	36.63	35.65	4450.0	15.06	1.275	0.0	0.9304;
220	100.5	0.0	36.71	35.64	4427.0	15.13	1.282	0.0	0.9325;
221	100.5	0.0	36.80	35.62	4404.1	15.21	1.288	0.0	0.9346;
222	100.5	0.0	36.88	35.61	4381.2	15.29	1.294	0.0	0.9368;
223	100.5	0.0	36.96	35.60	4358.4	15.37	1.300	0.0	0.9389;
224	100.5	0.0	37.05	35.59	4335.6	15.45	1.307	0.0	0.9410;
225	100.5	0.0	37.13	35.58	4312.8	15.53	1.313	0.0	0.9431;
226	100.6	0.0	37.21	35.56	4290.1	15.62	1.319	0.0	0.9452;
227	100.6	0.0	37.29	35.55	4267.5	15.70	1.326	0.0	0.9472;
228	100.6	0.0	37.37	35.54	4244.8	15.78	1.332	0.0	0.9493;
229	100.6	0.0	37.46	35.53	4222.2	15.87	1.339	0.0	0.9514;
230	100.6	0.0	37.54	35.52	4199.6	15.95	1.346	0.0	0.9535;
231	100.6	0.0	37.62	35.51	4177.0	16.04	1.352	0.0	0.9555;
232	100.6	0.0	37.70	35.50	4154.5	16.13	1.359	0.0	0.9576;
233	100.6	0.0	37.78	35.48	4131.9	16.22	1.366	0.0	0.9597;
234	100.6	0.0	37.86	35.47	4109.3	16.30	1.372	0.0	0.9617;
235	100.6	0.0	37.94	35.46	4086.8	16.39	1.379	0.0	0.9638;
236	100.6	0.0	38.03	35.45	4064.2	16.49	1.386	0.0	0.9659;
237	100.6	0.0	38.11	35.44	4041.6	16.58	1.393	0.0	0.9679;
238	100.7	0.0	38.19	35.43	4019.0	16.67	1.400	0.0	0.9700;
239	100.7	0.0	38.27	35.41	3996.3	16.77	1.407	0.0	0.9721;
240	100.7	0.0	38.35	35.40	3973.7	16.86	1.414	0.0	0.9742;
241	100.7	0.0	38.44	35.39	3951.0	16.96	1.422	0.0	0.9763;
242	100.7	0.0	38.52	35.38	3928.2	17.06	1.429	0.0	0.9784;
243	100.7	0.0	38.60	35.37	3905.5	17.16	1.436	0.0	0.9805;
244	100.7	0.0	38.69	35.36	3882.6	17.26	1.444	0.0	0.9826;
245	100.7	0.0	38.77	35.34	3859.7	17.36	1.451	0.0	0.9848;
246	100.7	0.0	38.86	35.33	3836.8	17.46	1.459	0.0	0.9869;
247	100.8	0.0	38.94	35.32	3813.8	17.57	1.466	0.0	0.9891;
248	100.8	0.0	39.03	35.31	3790.7	17.67	1.474	0.0	0.9913;
249	100.8	0.0	39.11	35.30	3767.5	17.78	1.482	0.0	0.9935;
250	100.8	0.0	39.20	35.29	3744.2	17.89	1.489	0.0	0.9957;
251	100.8	0.0	39.29	35.27	3720.9	18.01	1.497	0.0	0.9980;
252	100.8	0.0	39.38	35.26	3697.4	18.12	1.505	0.0	1.0003;
253	100.8	0.0	39.47	35.25	3673.8	18.24	1.513	0.0	1.0026;
254	100.8	0.0	39.57	35.24	3650.1	18.36	1.521	0.0	1.0050;
255	100.9	0.0	39.66	35.22	3626.3	18.48	1.530	0.0	1.0074;
256	100.9	0.0	39.76	35.21	3602.4	18.60	1.538	0.0	1.0098;
257	100.9	0.0	39.85	35.20	3578.6	18.72	1.546	0.0	1.0122;
258	100.9	0.0	39.94	35.19	3555.1	18.85	1.555	0.0	1.0146; end overlap;
259	100.9	0.0	40.04	35.18	3531.8	18.97	1.563	0.0	1.0169;
260	100.9	0.0	40.12	35.16	3508.7	19.10	1.572	0.0	1.0191;
261	101.0	0.0	40.21	35.15	3485.8	19.22	1.581	0.0	1.0213;
262	101.0	0.0	40.29	35.14	3463.1	19.35	1.589	0.0	1.0234;
263	101.0	0.0	40.37	35.13	3440.6	19.47	1.598	0.0	1.0255;
264	101.0	0.0	40.45	35.12	3418.3	19.60	1.607	0.0	1.0275;
265	101.0	0.0	40.53	35.11	3396.1	19.73	1.617	0.0	1.0294;
266	101.0	0.0	40.60	35.10	3374.1	19.86	1.626	0.0	1.0313;
267	101.1	0.0	40.67	35.08	3352.3	19.99	1.635	0.0	1.0331;
268	101.1	0.0	40.74	35.07	3330.5	20.12	1.645	0.0	1.0349;
269	101.1	0.0	40.81	35.06	3308.9	20.25	1.654	0.0	1.0366;
270	101.1	0.0	40.88	35.05	3287.4	20.38	1.664	0.0	1.0383;
271	101.1	0.0	40.94	35.04	3266.0	20.51	1.674	0.0	1.0399;
272	101.2	0.0	41.00	35.03	3244.7	20.65	1.684	0.0	1.0415;
273	101.2	0.0	41.06	35.02	3223.5	20.79	1.694	0.0	1.0430;
274	101.2	0.0	41.12	35.01	3202.3	20.92	1.704	0.0	1.0445;

275	101.2	0.0	41.18	35.00	3181.2	21.06	1.715	0.0	1.0459;
276	101.2	0.0	41.23	34.99	3160.1	21.20	1.725	0.0	1.0473;
277	101.3	0.0	41.29	34.97	3139.0	21.34	1.736	0.0	1.0487;
278	101.3	0.0	41.34	34.96	3118.0	21.49	1.747	0.0	1.0501;
279	101.3	0.0	41.39	34.95	3096.9	21.63	1.758	0.0	1.0514;
280	101.3	0.0	41.45	34.94	3075.9	21.78	1.769	0.0	1.0527;
281	101.4	0.0	41.50	34.93	3054.8	21.93	1.780	0.0	1.0540;
282	101.4	0.0	41.55	34.92	3033.7	22.09	1.792	0.0	1.0553;
283	101.4	0.0	41.60	34.91	3012.5	22.24	1.804	0.0	1.0565;
284	101.5	0.0	41.64	34.90	2991.3	22.40	1.816	0.0	1.0578;
285	101.5	0.0	41.69	34.89	2969.9	22.56	1.828	0.0	1.0590;
286	101.5	0.0	41.74	34.88	2948.5	22.72	1.840	0.0	1.0603;
287	101.6	0.0	41.79	34.87	2927.0	22.89	1.853	0.0	1.0615;
288	101.6	0.0	41.84	34.85	2905.4	23.06	1.866	0.0	1.0628;
289	101.6	0.0	41.89	34.84	2883.6	23.23	1.879	0.0	1.0640;
290	101.7	0.0	41.94	34.83	2861.7	23.41	1.892	0.0	1.0653;
291	101.7	0.0	41.99	34.82	2839.6	23.60	1.905	0.0	1.0666;
292	101.7	0.0	42.05	34.81	2817.3	23.78	1.919	0.0	1.0680;
293	101.8	0.0	42.10	34.80	2794.8	23.97	1.933	0.0	1.0694;
294	101.8	0.0	42.16	34.79	2772.1	24.17	1.947	0.0	1.0708;
295	101.8	0.0	42.22	34.77	2749.1	24.37	1.962	0.0	1.0723;
296	101.9	0.0	42.28	34.76	2725.9	24.58	1.977	0.0	1.0739;
297	101.9	0.0	42.34	34.75	2702.4	24.79	1.992	0.0	1.0755;
298	102.0	0.0	42.41	34.74	2678.6	25.01	2.007	0.0	1.0772;
299	102.0	0.0	42.48	34.73	2654.5	25.24	2.023	0.0	1.0790;
300	102.1	0.0	42.55	34.71	2630.1	25.47	2.039	0.0	1.0809;
301	102.1	0.0	42.63	34.70	2605.3	25.72	2.055	0.0	1.0828;
302	102.2	0.0	42.71	34.69	2580.1	25.97	2.072	0.0	1.0849;
303	102.2	0.0	42.80	34.68	2554.6	26.23	2.089	0.0	1.0872;
304	102.3	0.0	42.90	34.66	2528.6	26.50	2.106	0.0	1.0896;
305	102.4	0.0	43.00	34.65	2502.3	26.78	2.124	0.0	1.0921;
306	102.4	0.0	43.10	34.63	2475.4	27.07	2.142	0.0	1.0948;
307	102.5	0.0	43.21	34.62	2448.1	27.37	2.161	0.0	1.0977;
308	102.6	0.0	43.34	34.61	2420.3	27.68	2.180	0.0	1.1007;
309	102.6	0.0	43.47	34.59	2392.0	28.01	2.199	0.0	1.1040;
310	102.7	0.0	43.60	34.58	2363.1	28.35	2.219	0.0	1.1075;
311	102.8	0.0	43.75	34.56	2333.7	28.71	2.239	0.0	1.1113;
312	102.9	0.0	43.91	34.55	2303.7	29.08	2.260	0.0	1.1153; merging;
313	102.9	0.0	44.08	34.53	2273.5	29.47	2.281	0.0	1.1196;
314	103.0	0.0	44.25	34.52	2243.2	29.87	2.303	0.0	1.1240;
315	103.1	0.0	44.43	34.50	2212.7	30.28	2.325	0.0	1.1285;
316	103.2	0.0	44.62	34.48	2181.8	30.71	2.348	0.0	1.1333;
317	103.3	0.0	44.82	34.47	2150.4	31.16	2.372	0.0	1.1383;
318	103.4	0.0	45.02	34.45	2118.6	31.62	2.396	0.0	1.1436;
319	103.5	0.0	45.25	34.43	2086.3	32.11	2.420	0.0	1.1493;
320	103.7	0.0	45.48	34.42	2053.5	32.63	2.446	0.0	1.1553;
321	103.8	0.0	45.74	34.40	2020.1	33.17	2.472	0.0	1.1618;
322	103.9	0.0	46.01	34.38	1986.2	33.73	2.499	0.0	1.1686;
323	104.0	0.0	46.30	34.37	1951.7	34.33	2.526	0.0	1.1760;
324	104.2	0.0	46.61	34.35	1916.5	34.96	2.555	0.0	1.1839; bottom hit;

Horiz plane projections in effluent direction: radius(m): 0.0; CL(m): 0.7787

Lmz(m): 0.7787

forced entrain 1 0.0 -1.278 1.184 0.198

Rate sec-1 0.0 dy-1 0.0 kt: 0.0 Amb Sal 33.3632

;

1:47:00 PM. amb fills: 4

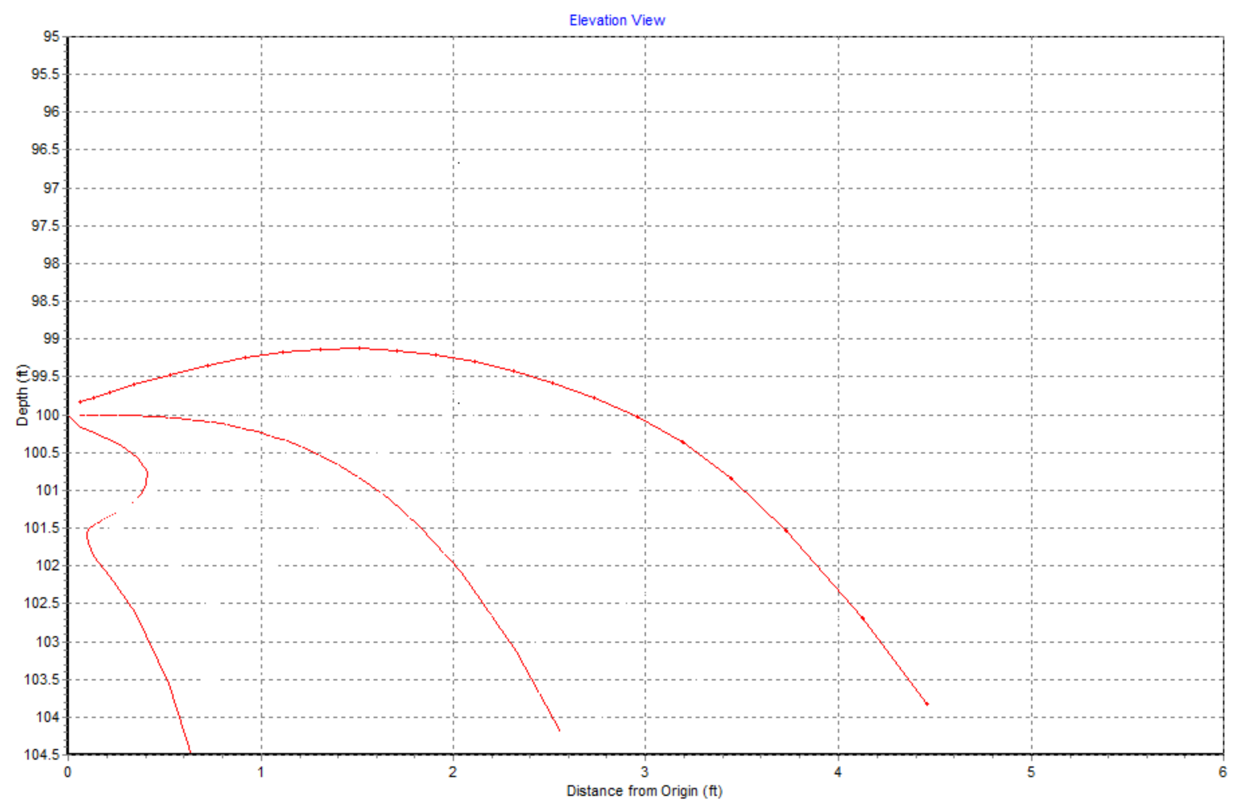

Figure C.4.1: Plumes 18b solution of discharge plume trajectories for discharges of 0 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt., and 10 mgd of brine from the Doheny Desalination Project with a brine salinity of 67 ppt. Depth of maximum rise of the plume is Z = 99.2 ft. at Xa = 1.704 ft from the point of discharge. The centerline of the plume is at an average distance of Xb = 2.555 ft from the point of discharge as the plume begins to impact the bottom at a depth of Z = 104.5 ft.

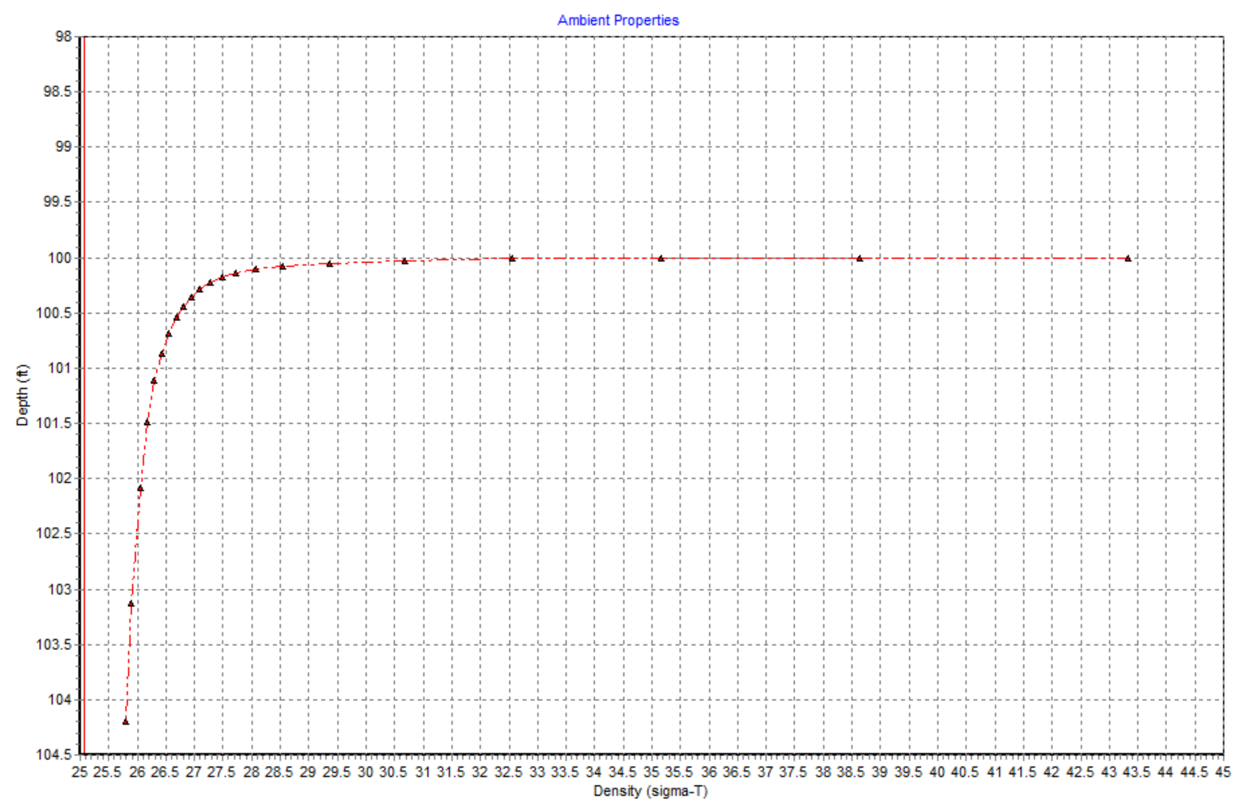

Figure C.4.2: Plumes 18b solution of vertical density profile for discharges of 0 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt. and 10 mgd of brine from the Doheny Desalination Project with a brine salinity of 67 ppt. Discharge effluent density shown as black triangles. Ambient water mass density profile shown as solid red line.

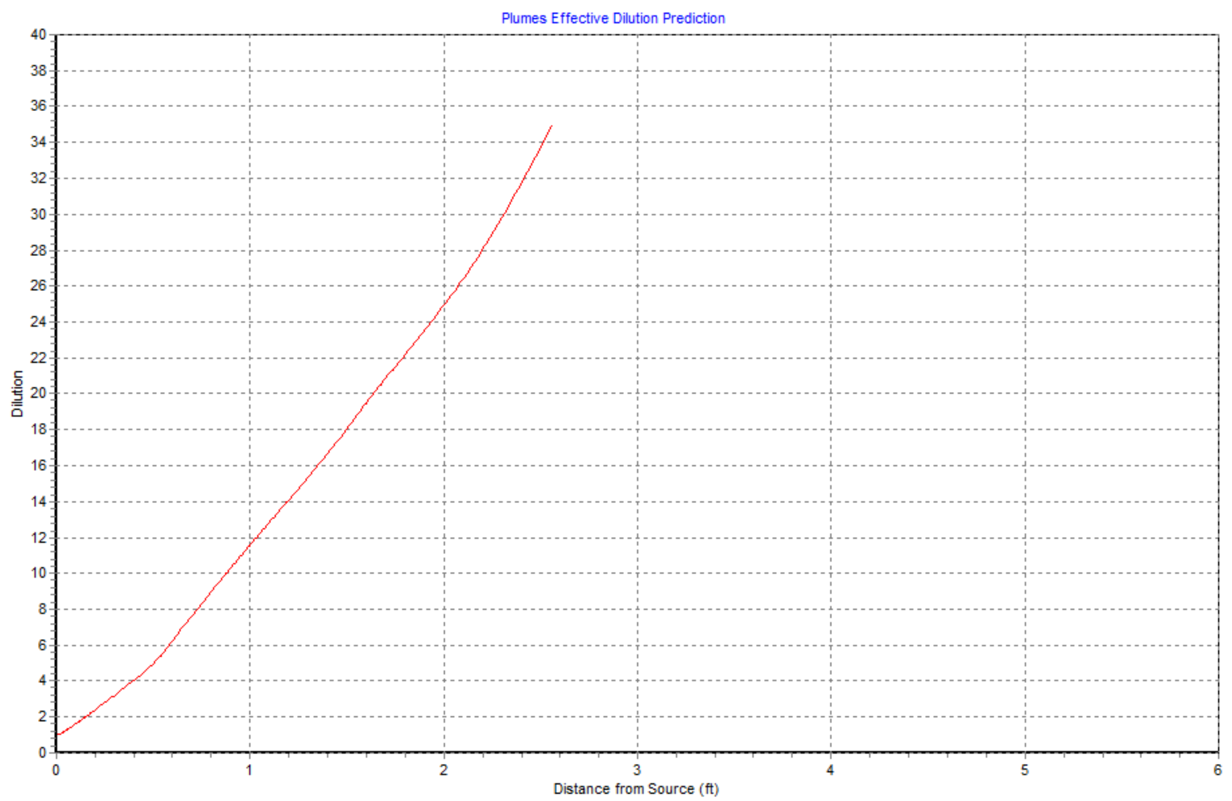

Figure C.4.3: Plumes 18b solution of effective (bulk average) dilution as a function of vertical distance from the point of discharge for discharges of 0 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt. and 10 mgd of brine from the Doheny Desalination Project with a brine salinity of 67 ppt. Effective dilution is S_a = 20.92 at the maximum rise of the plume at Xa = 1.704 ft. from the point of discharge. As the plume begins to impact the bottom, the plume centerline is at a distance of Xb = 2.55 ft from the point of discharge, where the effective dilution reaches S_a = 34.96.

C.5: Plumes 18b Results for SJCOO discharges of 0 mgd Wastewater and 5 mgd Brine:

Contents of the memo box (may not be current and must be updated manually)
Project "C:\Plumes18\SJCOO_WW0mgd_b5mgd_D-1"
memo
SJCOO discharging 0 mgd wastewater and 5 mgd brine

Model configuration items checked:
Channel width (m) 100
Start case for graphs 1
Max detailed graphs 10 (limits plots that can overflow memory)
Elevation Projection Plane (deg) 0
Shore vector (m,deg) not checked
Bacteria model : Mancini (1978) coliform model
PDS sfc. model heat transfer : Medium
Equation of State : S, T
Similarity Profile : Default profile (k=2.0, ...)
Diffuser port contraction coefficient 1
Light absorption coefficient 0.16
Farfield increment (m) 200
UM3 aspiration coefficient 0.1
Output file: text output tab
Output each ?? steps 1
Maximum dilution reported 1000
Text output format : Standard
Max vertical reversals : to max rise or fall

/ UM3. 1/14/2019 2:07:00 PM
Case 1; ambient file C:\Plumes18\SJCOO_WW0mgd_b5mgd_D-1.001.db; Diffuser table record 1: ---------
----

Ambient Table:

Depth	Amb-cur	Amb-dir	Amb-sal	Amb-tem	Amb-pol	Decay	Far-spd	Far-dir	Disprsn	Density
m	m/s	deg	psu	C	kg/kg	s-1	m/s	deg	m0.67/s2	sigma-T
0.0	0.0	0.0	33.38	19.97	0.0	0.0	0.0	0.0	0.0003	23.55719
2.000	0.0	0.0	33.39	19.87	0.0	0.0	0.0	0.0	0.0003	23.59288
4.000	0.0	0.0	33.40	19.81	0.0	0.0	0.0	0.0	0.0003	23.61484
7.000	0.0	0.0	33.41	19.64	0.0	0.0	0.0	0.0	0.0003	23.67096
10.00	0.0	0.0	33.40	17.76	0.0	0.0	0.0	0.0	0.0003	24.12888
12.00	0.0	0.0	33.38	17.34	0.0	0.0	0.0	0.0	0.0003	24.21549
14.00	0.0	0.0	33.38	16.73	0.0	0.0	0.0	0.0	0.0003	24.35932
16.00	0.0	0.0	33.38	15.17	0.0	0.0	0.0	0.0	0.0003	24.71340
18.00	0.0	0.0	33.37	14.37	0.0	0.0	0.0	0.0	0.0003	24.87044
20.00	0.0	0.0	33.35	14.18	0.0	0.0	0.0	0.0	0.0003	24.89660
22.00	0.0	0.0	33.33	13.83	0.0	0.0	0.0	0.0	0.0003	24.95172
24.00	0.0	0.0	33.32	13.75	0.0	0.0	0.0	0.0	0.0003	24.96707
26.00	0.0	0.0	33.36	13.42	0.0	0.0	0.0	0.0	0.0003	25.06459
31.85	0.0	0.0	33.36	13.39	0.0	0.0	0.0	0.0	0.0003	25.07201

Diffuser table:

P-dia	Ver angl	H-Angle	SourceX	SourceY	Ports	MZ-dis	Isoplth	P-depth	Ttl-flo	Eff-sal	Temp	Polutnt
(in)	(deg)	(deg)	(ft)	(ft)	()	(ft)	(concent)	(ft)	(MGD)	(psu)	(C)	(ppm)
3.0500	0.0	0.0	0.0	0.0	125.00	1000.0	0.0	100.00	5.0000	67.000	20.660	67000.0

Simulation:
Froude No: -2.796; Strat No:-1.57E-4; Spcg No: 14.39; k: 37179.3; eff den (sigmaT) 49.48870; eff vel 0.372(m/s);
Current is very small, flow regime may be transient.

Step	Depth (ft)	Amb-cur (m/s)	P-dia (in)	Eff-sal (psu)	Polutnt (ppm)	Dilutn ()	x-posn (ft)	y-posn (ft)	Iso dia (m)
0	100.0	1.000E-5	3.050	67.00	67000.0	1.000	0.0	0.0	0.07747;
1	100.0	0.0	3.059	66.81	66616.5	1.006	0.00364	0.0	0.07769;
2	100.0	0.0	3.099	66.16	65280.3	1.026	0.00565	0.0	0.0787;
3	100.0	0.0	3.161	65.51	63971.6	1.047	0.00684	0.0	0.0803;
4	100.0	0.0	3.225	64.88	62689.6	1.069	0.00758	0.0	0.08192;
5	100.0	0.0	3.290	64.26	61433.9	1.091	0.00804	0.0	0.08358;
6	100.0	0.0	3.357	63.66	60203.8	1.113	0.00834	0.0	0.08527;
7	100.0	0.0	3.425	63.06	58998.9	1.136	0.00853	0.0	0.08699;
8	100.0	0.0	3.494	62.48	57818.6	1.159	0.00865	0.0	0.08875;
9	100.0	0.0	3.565	61.91	56662.3	1.182	0.00873	0.0	0.09054;
10	100.0	0.0	3.637	61.35	55529.6	1.207	0.00879	0.0	0.09237;
11	100.0	0.0	3.710	60.80	54420.0	1.231	0.00882	0.0	0.09423;
12	100.0	0.0	3.785	60.26	53332.9	1.256	0.00884	0.0	0.09614;
13	100.0	0.0	3.861	59.74	52267.9	1.282	0.00885	0.0	0.09808;
14	100.0	0.0	3.939	59.22	51224.6	1.308	0.00886	0.0	0.1001;
15	100.0	0.0	4.019	58.71	50202.5	1.335	0.00887	0.0	0.1021;
16	100.0	0.0	4.100	58.22	49201.1	1.362	0.00887	0.0	0.1041;
17	100.0	0.0	4.183	57.73	48220.0	1.389	0.00888	0.0	0.1062;
18	100.0	0.0	4.267	57.25	47258.8	1.418	0.00888	0.0	0.1084;
19	100.0	0.0	4.353	56.78	46317.0	1.447	0.00888	0.0	0.1106;
20	100.0	0.0	4.441	56.32	45394.3	1.476	0.00888	0.0	0.1128;
21	100.0	0.0	4.530	55.87	44490.3	1.506	0.00888	0.0	0.1151;
22	100.0	0.0	4.622	55.43	43604.5	1.537	0.00888	0.0	0.1174;
23	100.0	0.0	4.715	55.00	42736.7	1.568	0.00888	0.0	0.1198;
24	100.0	0.0	4.810	54.57	41886.3	1.600	0.00888	0.0	0.1222;
25	100.0	0.0	4.907	54.16	41053.1	1.632	0.00888	0.0	0.1246;
26	100.0	0.0	5.005	53.75	40236.7	1.665	0.00888	0.0	0.1271;
27	100.0	0.0	5.106	53.35	39436.8	1.699	0.00888	0.0	0.1297;
28	100.0	0.0	5.209	52.96	38653.0	1.733	0.00888	0.0	0.1323;
29	100.0	0.0	5.314	52.58	37884.9	1.769	0.00888	0.0	0.1350;
30	100.0	0.0	5.421	52.20	37132.3	1.804	0.00888	0.0	0.1377;
31	100.0	0.0	5.530	51.83	36394.9	1.841	0.0317	0.0	0.1405;
32	100.0	0.0	5.618	51.61	35964.4	1.863	0.0371	0.0	0.1427;
33	100.0	0.0	5.707	51.26	35250.4	1.901	0.0401	0.0	0.1450;
34	100.0	0.0	5.822	50.90	34550.8	1.939	0.0419	0.0	0.1479;
35	100.0	0.0	5.939	50.56	33865.2	1.978	0.0429	0.0	0.1508;
36	100.0	0.0	6.058	50.22	33193.4	2.018	0.0436	0.0	0.1539;
37	100.0	0.0	6.180	49.89	32535.0	2.059	0.0441	0.0	0.1570;
38	100.0	0.0	6.304	49.57	31889.9	2.101	0.0443	0.0	0.1601;
39	100.0	0.0	6.431	49.25	31257.6	2.143	0.0445	0.0	0.1633;
40	100.0	0.0	6.560	48.94	30638.1	2.187	0.0446	0.0	0.1666;
41	100.0	0.0	6.692	48.63	30030.9	2.231	0.0447	0.0	0.1700;
42	100.0	0.0	6.827	48.33	29435.9	2.276	0.0447	0.0	0.1734;
43	100.0	0.0	6.964	48.04	28852.8	2.322	0.0447	0.0	0.1769;
44	100.0	0.0	7.104	47.75	28281.4	2.369	0.0448	0.0	0.1804;
45	100.0	0.0	7.247	47.47	27721.4	2.417	0.0448	0.0	0.1841;
46	100.0	0.0	7.392	47.19	27172.6	2.466	0.0448	0.0	0.1878;
47	100.0	0.0	7.541	46.92	26634.8	2.516	0.0448	0.0	0.1915;
48	100.0	0.0	7.692	46.66	26107.7	2.566	0.0448	0.0	0.1954;
49	100.0	0.0	7.847	46.40	25591.1	2.618	0.0448	0.0	0.1993;

50	100.0	0.0	8.005	46.14	25084.8	2.671	0.0448	0.0	0.2033;
51	100.0	0.0	8.166	45.89	24588.7	2.725	0.0448	0.0	0.2074;
52	100.0	0.0	8.330	45.65	24102.4	2.780	0.0448	0.0	0.2116;
53	100.0	0.0	8.497	45.40	23625.8	2.836	0.0448	0.0	0.2158;
54	100.0	0.0	8.667	45.17	23162.2	2.893	0.0448	0.0	0.2201; begin overlap;
55	100.0	0.0	8.836	44.95	22725.1	2.948	0.0448	0.0	0.2244;
56	100.0	0.0	9.002	44.74	22314.3	3.003	0.0448	0.0	0.2286;
57	100.0	0.0	9.163	44.55	21931.1	3.055	0.0448	0.0	0.2327;
58	100.0	0.0	9.321	44.36	21560.8	3.107	0.0448	0.0	0.2367;
59	100.0	0.0	9.484	44.17	21180.5	3.163	0.0448	0.0	0.2409;
60	100.0	0.0	9.643	44.00	20854.3	3.213	0.0448	0.0	0.2449;
61	100.0	0.0	9.782	43.86	20576.3	3.256	0.0448	0.0	0.2485;
62	100.0	0.0	9.911	43.73	20316.3	3.298	0.0448	0.0	0.2517;
63	100.0	0.0	10.01	43.66	20178.5	3.320	0.0448	0.0	0.2542;
64	100.0	0.0	10.05	43.64	20137.9	3.327	0.0448	0.0	0.2553;
65	100.0	0.0	10.16	43.44	19740.2	3.394	0.0857	0.0	0.2581;
66	100.0	0.0	10.25	43.41	19689.8	3.403	0.0886	0.0	0.2604;
67	100.0	0.0	10.33	43.27	19417.1	3.451	0.0905	0.0	0.2624;
68	100.0	0.0	10.47	43.14	19153.7	3.498	0.0916	0.0	0.2660;
69	100.0	0.0	10.61	43.01	18900.6	3.545	0.0923	0.0	0.2696;
70	100.0	0.0	10.75	42.89	18657.2	3.591	0.0928	0.0	0.2731;
71	100.0	0.0	10.89	42.77	18423.0	3.637	0.0931	0.0	0.2766;
72	100.0	0.0	11.03	42.66	18197.4	3.682	0.0933	0.0	0.2801;
73	100.0	0.0	11.16	42.55	17979.9	3.726	0.0934	0.0	0.2835;
74	100.0	0.0	11.29	42.44	17770.1	3.770	0.0935	0.0	0.2869;
75	100.0	0.0	11.42	42.34	17567.5	3.814	0.0935	0.0	0.2902;
76	100.0	0.0	11.55	42.24	17371.7	3.857	0.0936	0.0	0.2935;
77	100.0	0.0	11.68	42.14	17182.4	3.899	0.0936	0.0	0.2967;
78	100.0	0.0	11.81	42.05	16999.3	3.941	0.0936	0.0	0.3000;
79	100.0	0.0	11.94	41.96	16821.8	3.983	0.0936	0.0	0.3032;
80	100.0	0.0	12.06	41.87	16649.9	4.024	0.0936	0.0	0.3063;
81	100.0	0.0	12.18	41.79	16483.2	4.065	0.0936	0.0	0.3094;
82	100.0	0.0	12.30	41.70	16321.5	4.105	0.0936	0.0	0.3125;
83	100.0	0.0	12.42	41.62	16164.4	4.145	0.0936	0.0	0.3156;
84	100.0	0.0	12.54	41.55	16011.8	4.184	0.0936	0.0	0.3186;
85	100.0	0.0	12.66	41.47	15863.5	4.224	0.0936	0.0	0.3216;
86	100.0	0.0	12.78	41.40	15719.3	4.262	0.0936	0.0	0.3246;
87	100.0	0.0	12.89	41.33	15579.0	4.301	0.0936	0.0	0.3275;
88	100.0	0.0	13.01	41.26	15442.4	4.339	0.0936	0.0	0.3304;
89	100.0	0.0	13.12	41.19	15309.4	4.376	0.0936	0.0	0.3333;
90	100.0	0.0	13.24	41.12	15179.9	4.414	0.0936	0.0	0.3362;
91	100.0	0.0	13.35	41.06	15054.2	4.451	0.0936	0.0	0.3390;
92	100.0	0.0	13.46	41.00	14932.5	4.487	0.0936	0.0	0.3418;
93	100.0	0.0	13.56	40.94	14815.1	4.522	0.0936	0.0	0.3445;
94	100.0	0.0	13.67	40.88	14698.3	4.558	0.0936	0.0	0.3473;
95	100.0	0.0	13.79	40.81	14570.1	4.598	0.0936	0.0	0.3502;
96	100.0	0.0	13.92	40.73	14412.4	4.649	0.0936	0.0	0.3536;
97	100.0	0.0	14.05	40.68	14312.3	4.681	0.0936	0.0	0.3568;
98	100.0	0.0	14.14	40.64	14228.7	4.709	0.0936	0.0	0.3591;
99	100.0	0.0	14.32	40.50	13948.3	4.803	0.149	0.0	0.3637;
100	100.0	0.0	14.29	40.49	13938.1	4.807	0.152	0.0	0.3630;
101	100.0	0.0	14.34	40.44	13831.6	4.844	0.153	0.0	0.3643;
102	100.0	0.0	14.45	40.38	13726.8	4.881	0.154	0.0	0.3669;
103	100.0	0.0	14.55	40.33	13624.1	4.918	0.155	0.0	0.3696;
104	100.0	0.0	14.66	40.28	13523.5	4.954	0.155	0.0	0.3723;
105	100.0	0.0	14.76	40.23	13425.0	4.991	0.155	0.0	0.3750;

106	100.0	0.0	14.87	40.18	13328.6	5.027	0.156	0.0	0.3777;
107	100.0	0.0	14.97	40.13	13234.2	5.063	0.156	0.0	0.3803;
108	100.0	0.0	15.08	40.09	13141.7	5.098	0.156	0.0	0.3830;
109	100.0	0.0	15.18	40.04	13051.1	5.134	0.156	0.0	0.3857;
110	100.0	0.0	15.29	39.99	12962.4	5.169	0.156	0.0	0.3883;
111	100.0	0.0	15.39	39.95	12875.5	5.204	0.156	0.0	0.3909;
112	100.0	0.0	15.49	39.91	12790.3	5.238	0.156	0.0	0.3935;
113	100.0	0.0	15.60	39.86	12706.7	5.273	0.156	0.0	0.3961;
114	100.0	0.0	15.70	39.82	12624.8	5.307	0.156	0.0	0.3987;
115	100.0	0.0	15.80	39.78	12544.5	5.341	0.156	0.0	0.4013;
116	100.0	0.0	15.90	39.74	12465.7	5.375	0.156	0.0	0.4038;
117	100.0	0.0	16.00	39.70	12388.3	5.408	0.156	0.0	0.4064;
118	100.0	0.0	16.10	39.66	12312.4	5.442	0.156	0.0	0.4089;
119	100.0	0.0	16.20	39.63	12237.9	5.475	0.156	0.0	0.4114;
120	100.0	0.0	16.29	39.59	12164.7	5.508	0.156	0.0	0.4139;
121	100.0	0.0	16.39	39.55	12092.8	5.540	0.156	0.0	0.4163;
122	100.0	0.0	16.49	39.52	12022.2	5.573	0.156	0.0	0.4188;
123	100.0	0.0	16.58	39.48	11952.8	5.605	0.156	0.0	0.4212;
124	100.0	0.0	16.68	39.45	11884.6	5.638	0.156	0.0	0.4236;
125	100.0	0.0	16.77	39.41	11817.6	5.670	0.156	0.0	0.4261;
126	100.0	0.0	16.87	39.38	11751.7	5.701	0.156	0.0	0.4285;
127	100.0	0.0	16.96	39.34	11687.0	5.733	0.156	0.0	0.4308;
128	100.0	0.0	17.06	39.31	11623.3	5.764	0.156	0.0	0.4332;
129	100.0	0.0	17.15	39.28	11560.0	5.796	0.156	0.0	0.4356;
130	100.0	0.0	17.24	39.25	11497.8	5.827	0.156	0.0	0.4379;
131	100.0	0.0	17.33	39.22	11440.0	5.857	0.156	0.0	0.4402;
132	100.0	0.0	17.42	39.19	11382.9	5.886	0.156	0.0	0.4424;
133	100.0	0.0	17.52	39.15	11304.4	5.927	0.156	0.0	0.4451;
134	100.0	0.0	17.64	39.11	11233.6	5.964	0.156	0.0	0.4480;
135	100.0	0.0	17.74	39.08	11174.7	5.996	0.156	0.0	0.4506;
136	100.0	0.0	17.82	39.06	11133.0	6.018	0.156	0.0	0.4526;
137	100.1	0.0	18.03	38.95	10913.9	6.139	0.221	0.0	0.4580;
138	100.1	0.0	17.63	38.95	10909.7	6.141	0.223	0.0	0.4477;
139	100.1	0.0	17.66	38.92	10844.3	6.178	0.224	0.0	0.4486;
140	100.1	0.0	17.75	38.88	10779.6	6.215	0.226	0.0	0.4509;
141	100.1	0.0	17.85	38.85	10715.6	6.253	0.226	0.0	0.4533;
142	100.1	0.0	17.94	38.82	10652.5	6.290	0.227	0.0	0.4558;
143	100.1	0.0	18.04	38.79	10590.4	6.327	0.228	0.0	0.4583;
144	100.1	0.0	18.14	38.75	10529.1	6.363	0.228	0.0	0.4608;
145	100.1	0.0	18.24	38.72	10468.8	6.400	0.228	0.0	0.4633;
146	100.1	0.0	18.34	38.69	10409.4	6.436	0.229	0.0	0.4659;
147	100.1	0.0	18.44	38.66	10351.0	6.473	0.229	0.0	0.4685;
148	100.1	0.0	18.54	38.63	10293.4	6.509	0.229	0.0	0.4710;
149	100.1	0.0	18.65	38.61	10236.8	6.545	0.229	0.0	0.4736;
150	100.1	0.0	18.75	38.58	10181.1	6.581	0.229	0.0	0.4762;
151	100.1	0.0	18.85	38.55	10126.2	6.616	0.229	0.0	0.4787;
152	100.1	0.0	18.95	38.52	10072.2	6.652	0.229	0.0	0.4813;
153	100.1	0.0	19.05	38.49	10019.1	6.687	0.229	0.0	0.4838;
154	100.1	0.0	19.15	38.47	9966.7	6.722	0.229	0.0	0.4863;
155	100.1	0.0	19.25	38.44	9915.2	6.757	0.229	0.0	0.4889;
156	100.1	0.0	19.35	38.42	9864.5	6.792	0.229	0.0	0.4914;
157	100.1	0.0	19.44	38.39	9814.5	6.827	0.229	0.0	0.4939;
158	100.1	0.0	19.54	38.36	9765.3	6.861	0.230	0.0	0.4964;
159	100.1	0.0	19.64	38.34	9716.8	6.895	0.230	0.0	0.4988;
160	100.1	0.0	19.74	38.32	9669.0	6.929	0.230	0.0	0.5013;
161	100.1	0.0	19.83	38.29	9621.9	6.963	0.230	0.0	0.5038;

162	100.1	0.0	19.93	38.27	9575.5	6.997	0.230	0.0	0.5062;
163	100.1	0.0	20.03	38.24	9529.8	7.031	0.230	0.0	0.5086;
164	100.1	0.0	20.12	38.22	9484.7	7.064	0.230	0.0	0.5111;
165	100.1	0.0	20.22	38.20	9440.3	7.097	0.230	0.0	0.5135;
166	100.1	0.0	20.31	38.18	9396.5	7.130	0.230	0.0	0.5159;
167	100.1	0.0	20.40	38.15	9353.2	7.163	0.230	0.0	0.5183;
168	100.1	0.0	20.50	38.13	9310.6	7.196	0.230	0.0	0.5207;
169	100.1	0.0	20.59	38.11	9268.6	7.229	0.230	0.0	0.5230;
170	100.1	0.0	20.68	38.09	9227.1	7.261	0.230	0.0	0.5254;
171	100.1	0.0	20.78	38.07	9186.2	7.294	0.230	0.0	0.5277;
172	100.1	0.0	20.87	38.05	9145.8	7.326	0.230	0.0	0.5301;
173	100.1	0.0	20.96	38.03	9105.9	7.358	0.230	0.0	0.5324;
174	100.1	0.0	21.05	38.01	9066.6	7.390	0.230	0.0	0.5347;
175	100.1	0.0	21.14	37.99	9027.8	7.422	0.230	0.0	0.5370;
176	100.1	0.0	21.23	37.97	8989.4	7.453	0.230	0.0	0.5393;
177	100.1	0.0	21.32	37.95	8951.6	7.485	0.230	0.0	0.5416;
178	100.1	0.0	21.41	37.93	8914.2	7.516	0.230	0.0	0.5439;
179	100.1	0.0	21.50	37.91	8877.3	7.547	0.230	0.0	0.5461;
180	100.1	0.0	21.59	37.89	8840.9	7.578	0.230	0.0	0.5484;
181	100.1	0.0	21.68	37.87	8804.9	7.609	0.230	0.0	0.5506;
182	100.1	0.0	21.77	37.86	8769.4	7.640	0.230	0.0	0.5529;
183	100.1	0.0	21.85	37.84	8734.3	7.671	0.230	0.0	0.5551;
184	100.1	0.0	21.94	37.82	8699.7	7.701	0.230	0.0	0.5573;
185	100.1	0.0	22.03	37.80	8665.4	7.732	0.230	0.0	0.5595;
186	100.1	0.0	22.12	37.79	8631.5	7.762	0.230	0.0	0.5617;
187	100.1	0.0	22.20	37.77	8597.4	7.793	0.230	0.0	0.5639;
188	100.1	0.0	22.29	37.75	8564.3	7.823	0.230	0.0	0.5661;
189	100.1	0.0	22.38	37.73	8528.5	7.856	0.230	0.0	0.5684;
190	100.1	0.0	22.47	37.72	8496.4	7.886	0.230	0.0	0.5707;
191	100.1	0.0	22.55	37.70	8465.4	7.915	0.230	0.0	0.5728;
192	100.1	0.0	22.64	37.68	8431.8	7.946	0.230	0.0	0.5750;
193	100.1	0.0	22.73	37.67	8399.2	7.977	0.230	0.0	0.5772;
194	100.1	0.0	22.81	37.65	8372.3	8.003	0.230	0.0	0.5793;
195	100.1	0.0	22.89	37.64	8340.0	8.034	0.230	0.0	0.5813;
196	100.1	0.0	22.97	37.62	8313.1	8.060	0.230	0.0	0.5834;
197	100.1	0.0	23.24	37.54	8149.6	8.221	0.303	0.0	0.5902;
198	100.1	0.0	21.90	37.54	8147.5	8.223	0.305	0.0	0.5562;
199	100.1	0.0	21.92	37.52	8101.6	8.270	0.307	0.0	0.5569;
200	100.1	0.0	22.01	37.49	8055.7	8.317	0.309	0.0	0.5590;
201	100.1	0.0	22.09	37.47	8010.1	8.364	0.311	0.0	0.5611;
202	100.1	0.0	22.18	37.45	7964.6	8.412	0.313	0.0	0.5632;
203	100.1	0.0	22.26	37.42	7920.2	8.459	0.315	0.0	0.5653;
204	100.1	0.0	22.33	37.40	7877.2	8.506	0.317	0.0	0.5672;
205	100.1	0.0	22.40	37.38	7834.6	8.552	0.319	0.0	0.5691;
206	100.1	0.0	22.48	37.36	7792.3	8.598	0.321	0.0	0.5709;
207	100.1	0.0	22.55	37.34	7750.5	8.645	0.323	0.0	0.5728;
208	100.1	0.0	22.62	37.31	7709.1	8.691	0.325	0.0	0.5746;
209	100.1	0.0	22.69	37.29	7668.0	8.738	0.327	0.0	0.5764;
210	100.1	0.0	22.76	37.27	7627.2	8.784	0.329	0.0	0.5782;
211	100.1	0.0	22.83	37.25	7586.8	8.831	0.331	0.0	0.5800;
212	100.1	0.0	22.90	37.23	7546.8	8.878	0.333	0.0	0.5817;
213	100.1	0.0	22.97	37.21	7507.1	8.925	0.335	0.0	0.5834;
214	100.1	0.0	23.04	37.19	7467.7	8.972	0.337	0.0	0.5852;
215	100.1	0.0	23.10	37.17	7428.6	9.019	0.339	0.0	0.5869;
216	100.2	0.0	23.17	37.15	7389.8	9.067	0.341	0.0	0.5885;
217	100.2	0.0	23.24	37.13	7351.3	9.114	0.343	0.0	0.5902;

218	100.2	0.0	23.30	37.11	7313.1	9.162	0.345	0.0	0.5919;
219	100.2	0.0	23.37	37.09	7275.2	9.209	0.348	0.0	0.5935;
220	100.2	0.0	23.43	37.07	7237.5	9.257	0.350	0.0	0.5951;
221	100.2	0.0	23.49	37.05	7200.2	9.305	0.352	0.0	0.5967;
222	100.2	0.0	23.55	37.04	7163.1	9.354	0.354	0.0	0.5983;
223	100.2	0.0	23.62	37.02	7126.2	9.402	0.356	0.0	0.5999;
224	100.2	0.0	23.68	37.00	7089.6	9.450	0.358	0.0	0.6014;
225	100.2	0.0	23.74	36.98	7053.3	9.499	0.361	0.0	0.6029;
226	100.2	0.0	23.80	36.96	7017.1	9.548	0.363	0.0	0.6045;
227	100.2	0.0	23.86	36.94	6981.2	9.597	0.365	0.0	0.6060;
228	100.2	0.0	23.92	36.92	6945.6	9.646	0.367	0.0	0.6075;
229	100.2	0.0	23.97	36.91	6910.1	9.696	0.369	0.0	0.6089;
230	100.2	0.0	24.03	36.89	6874.9	9.746	0.372	0.0	0.6104;
231	100.2	0.0	24.09	36.87	6839.8	9.796	0.374	0.0	0.6118;
232	100.2	0.0	24.14	36.85	6805.0	9.846	0.376	0.0	0.6133;
233	100.2	0.0	24.20	36.83	6770.4	9.896	0.378	0.0	0.6147;
234	100.2	0.0	24.25	36.82	6735.9	9.947	0.381	0.0	0.6161;
235	100.2	0.0	24.31	36.80	6701.6	9.998	0.383	0.0	0.6174;
236	100.2	0.0	24.36	36.78	6667.5	10.05	0.385	0.0	0.6188;
237	100.2	0.0	24.42	36.76	6633.6	10.10	0.388	0.0	0.6202;
238	100.2	0.0	24.47	36.75	6599.9	10.15	0.390	0.0	0.6215;
239	100.2	0.0	24.52	36.73	6566.3	10.20	0.393	0.0	0.6228;
240	100.2	0.0	24.57	36.71	6532.8	10.26	0.395	0.0	0.6241;
241	100.2	0.0	24.62	36.70	6499.5	10.31	0.397	0.0	0.6254;
242	100.2	0.0	24.67	36.68	6466.4	10.36	0.400	0.0	0.6267;
243	100.2	0.0	24.72	36.66	6433.4	10.41	0.402	0.0	0.6280;
244	100.2	0.0	24.77	36.65	6400.5	10.47	0.405	0.0	0.6292;
245	100.2	0.0	24.82	36.63	6367.8	10.52	0.407	0.0	0.6304;
246	100.2	0.0	24.87	36.61	6335.1	10.58	0.410	0.0	0.6317;
247	100.2	0.0	24.92	36.60	6302.6	10.63	0.412	0.0	0.6329;
248	100.2	0.0	24.96	36.58	6270.3	10.69	0.415	0.0	0.6341;
249	100.2	0.0	25.01	36.56	6238.0	10.74	0.417	0.0	0.6353;
250	100.2	0.0	25.06	36.55	6205.8	10.80	0.420	0.0	0.6364;
251	100.2	0.0	25.10	36.53	6173.7	10.85	0.422	0.0	0.6376;
252	100.3	0.0	25.15	36.51	6141.7	10.91	0.425	0.0	0.6387;
253	100.3	0.0	25.19	36.50	6109.8	10.97	0.428	0.0	0.6399;
254	100.3	0.0	25.24	36.48	6078.0	11.02	0.430	0.0	0.6410;
255	100.3	0.0	25.28	36.46	6046.2	11.08	0.433	0.0	0.6421;
256	100.3	0.0	25.32	36.45	6014.6	11.14	0.436	0.0	0.6432;
257	100.3	0.0	25.37	36.43	5982.9	11.20	0.439	0.0	0.6443;
258	100.3	0.0	25.41	36.42	5951.4	11.26	0.441	0.0	0.6454;
259	100.3	0.0	25.45	36.40	5919.9	11.32	0.444	0.0	0.6464;
260	100.3	0.0	25.49	36.38	5888.4	11.38	0.447	0.0	0.6475;
261	100.3	0.0	25.53	36.37	5857.0	11.44	0.450	0.0	0.6485;
262	100.3	0.0	25.57	36.35	5825.6	11.50	0.453	0.0	0.6496;
263	100.3	0.0	25.61	36.34	5794.3	11.56	0.456	0.0	0.6506;
264	100.3	0.0	25.65	36.32	5762.9	11.63	0.458	0.0	0.6516;
265	100.3	0.0	25.69	36.30	5731.6	11.69	0.461	0.0	0.6526;
266	100.3	0.0	25.73	36.29	5700.3	11.75	0.464	0.0	0.6536;
267	100.3	0.0	25.77	36.27	5669.0	11.82	0.467	0.0	0.6546;
268	100.3	0.0	25.81	36.26	5637.7	11.88	0.471	0.0	0.6556;
269	100.3	0.0	25.85	36.24	5606.3	11.95	0.474	0.0	0.6566;
270	100.3	0.0	25.89	36.22	5575.0	12.02	0.477	0.0	0.6576;
271	100.3	0.0	25.93	36.21	5543.6	12.09	0.480	0.0	0.6585;
272	100.3	0.0	25.96	36.19	5512.2	12.15	0.483	0.0	0.6595;
273	100.3	0.0	26.00	36.17	5480.7	12.22	0.486	0.0	0.6605;

274	100.4	0.0	26.04	36.16	5449.2	12.30	0.490	0.0	0.6614;
275	100.4	0.0	26.08	36.14	5417.6	12.37	0.493	0.0	0.6624;
276	100.4	0.0	26.12	36.13	5386.0	12.44	0.496	0.0	0.6633;
277	100.4	0.0	26.15	36.11	5354.3	12.51	0.500	0.0	0.6643;
278	100.4	0.0	26.19	36.09	5322.5	12.59	0.503	0.0	0.6653;
279	100.4	0.0	26.23	36.08	5290.6	12.66	0.506	0.0	0.6662;
280	100.4	0.0	26.27	36.06	5258.6	12.74	0.510	0.0	0.6672;
281	100.4	0.0	26.31	36.04	5226.4	12.82	0.513	0.0	0.6682;
282	100.4	0.0	26.34	36.03	5194.2	12.90	0.517	0.0	0.6691;
283	100.4	0.0	26.38	36.01	5161.8	12.98	0.521	0.0	0.6701;
284	100.4	0.0	26.42	35.99	5129.2	13.06	0.524	0.0	0.6711;
285	100.4	0.0	26.46	35.98	5096.5	13.15	0.528	0.0	0.6721;
286	100.4	0.0	26.50	35.96	5063.6	13.23	0.532	0.0	0.6731;
287	100.4	0.0	26.54	35.94	5030.5	13.32	0.536	0.0	0.6742;
288	100.4	0.0	26.58	35.93	4997.2	13.41	0.540	0.0	0.6752;
289	100.5	0.0	26.62	35.91	4963.7	13.50	0.543	0.0	0.6763;
290	100.5	0.0	26.67	35.89	4929.9	13.59	0.547	0.0	0.6774;
291	100.5	0.0	26.71	35.88	4895.9	13.69	0.551	0.0	0.6785;
292	100.5	0.0	26.76	35.86	4861.6	13.78	0.556	0.0	0.6796;
293	100.5	0.0	26.80	35.84	4827.0	13.88	0.560	0.0	0.6808;
294	100.5	0.0	26.85	35.82	4792.1	13.98	0.564	0.0	0.6820;
295	100.5	0.0	26.90	35.80	4756.9	14.08	0.568	0.0	0.6832;
296	100.5	0.0	26.95	35.79	4721.3	14.19	0.573	0.0	0.6844;
297	100.5	0.0	27.00	35.77	4685.4	14.30	0.577	0.0	0.6858;
298	100.5	0.0	27.05	35.75	4650.0	14.41	0.581	0.0	0.6870; end overlap;
299	100.6	0.0	27.10	35.73	4615.1	14.52	0.586	0.0	0.6882;
300	100.6	0.0	27.14	35.71	4580.6	14.63	0.591	0.0	0.6894;
301	100.6	0.0	27.18	35.70	4546.5	14.74	0.595	0.0	0.6904;
302	100.6	0.0	27.22	35.68	4512.8	14.85	0.600	0.0	0.6914;
303	100.6	0.0	27.26	35.66	4479.4	14.96	0.605	0.0	0.6923;
304	100.6	0.0	27.29	35.64	4446.3	15.07	0.610	0.0	0.6931;
305	100.6	0.0	27.32	35.63	4413.5	15.18	0.615	0.0	0.6939;
306	100.6	0.0	27.35	35.61	4381.0	15.29	0.620	0.0	0.6946;
307	100.7	0.0	27.37	35.59	4348.7	15.41	0.625	0.0	0.6952;
308	100.7	0.0	27.39	35.58	4316.7	15.52	0.630	0.0	0.6958;
309	100.7	0.0	27.41	35.56	4284.8	15.64	0.636	0.0	0.6963;
310	100.7	0.0	27.43	35.55	4253.0	15.75	0.641	0.0	0.6967;
311	100.7	0.0	27.45	35.53	4221.3	15.87	0.647	0.0	0.6971;
312	100.7	0.0	27.46	35.51	4189.7	15.99	0.653	0.0	0.6975;
313	100.8	0.0	27.47	35.50	4158.1	16.11	0.659	0.0	0.6978;
314	100.8	0.0	27.48	35.48	4126.6	16.24	0.665	0.0	0.6980;
315	100.8	0.0	27.49	35.46	4095.0	16.36	0.671	0.0	0.6983;
316	100.8	0.0	27.50	35.45	4063.3	16.49	0.677	0.0	0.6985;
317	100.8	0.0	27.51	35.43	4031.6	16.62	0.683	0.0	0.6986;
318	100.9	0.0	27.51	35.42	3999.6	16.75	0.690	0.0	0.6988;
319	100.9	0.0	27.52	35.40	3967.5	16.89	0.697	0.0	0.6989;
320	100.9	0.0	27.52	35.38	3935.2	17.03	0.703	0.0	0.6990;
321	100.9	0.0	27.53	35.37	3902.6	17.17	0.710	0.0	0.6992;
322	100.9	0.0	27.53	35.35	3869.7	17.31	0.718	0.0	0.6993;
323	101.0	0.0	27.54	35.33	3836.4	17.46	0.725	0.0	0.6994;
324	101.0	0.0	27.54	35.32	3802.7	17.62	0.732	0.0	0.6996;
325	101.0	0.0	27.55	35.30	3768.5	17.78	0.740	0.0	0.6997;
326	101.1	0.0	27.56	35.28	3733.8	17.94	0.748	0.0	0.7000;
327	101.1	0.0	27.57	35.26	3698.6	18.11	0.756	0.0	0.7002;
328	101.1	0.0	27.58	35.24	3662.8	18.29	0.765	0.0	0.7005;
329	101.2	0.0	27.59	35.22	3626.2	18.48	0.773	0.0	0.7009;

330	101.2	0.0	27.61	35.21	3589.0	18.67	0.782	0.0	0.7014;
331	101.2	0.0	27.63	35.19	3550.9	18.87	0.791	0.0	0.7019;
332	101.3	0.0	27.66	35.17	3512.0	19.08	0.800	0.0	0.7026;
333	101.3	0.0	27.69	35.15	3472.2	19.30	0.810	0.0	0.7033;
334	101.4	0.0	27.73	35.12	3431.4	19.53	0.820	0.0	0.7042;
335	101.4	0.0	27.77	35.10	3389.5	19.77	0.830	0.0	0.7053;
336	101.4	0.0	27.82	35.08	3346.6	20.02	0.841	0.0	0.7065;
337	101.5	0.0	27.87	35.06	3302.4	20.29	0.852	0.0	0.7080;
338	101.6	0.0	27.94	35.04	3257.0	20.57	0.863	0.0	0.7096;
339	101.6	0.0	28.01	35.01	3210.3	20.87	0.874	0.0	0.7115;
340	101.7	0.0	28.09	34.99	3162.2	21.19	0.886	0.0	0.7136;
341	101.7	0.0	28.19	34.96	3112.7	21.53	0.898	0.0	0.7160;
342	101.8	0.0	28.30	34.94	3061.6	21.88	0.911	0.0	0.7187;
343	101.9	0.0	28.42	34.91	3008.9	22.27	0.924	0.0	0.7218;
344	102.0	0.0	28.55	34.88	2954.5	22.68	0.938	0.0	0.7253;
345	102.0	0.0	28.71	34.85	2898.4	23.12	0.952	0.0	0.7292;
346	102.1	0.0	28.87	34.82	2841.6	23.58	0.966	0.0	0.7334;
347	102.2	0.0	29.05	34.79	2785.8	24.05	0.980	0.0	0.7378;
348	102.3	0.0	29.23	34.77	2731.1	24.53	0.994	0.0	0.7425;
349	102.4	0.0	29.43	34.74	2677.5	25.02	1.007	0.0	0.7474;
350	102.5	0.0	29.63	34.71	2624.9	25.52	1.020	0.0	0.7526;
351	102.6	0.0	29.85	34.68	2573.4	26.04	1.033	0.0	0.7581;
352	102.7	0.0	30.07	34.66	2522.9	26.56	1.046	0.0	0.7638;
353	102.7	0.0	30.30	34.63	2473.4	27.09	1.059	0.0	0.7697;
354	102.8	0.0	30.54	34.61	2424.9	27.63	1.071	0.0	0.7758;
355	102.9	0.0	30.79	34.58	2377.3	28.18	1.084	0.0	0.7822;
356	103.0	0.0	31.05	34.56	2330.6	28.75	1.096	0.0	0.7887;
357	103.1	0.0	31.32	34.54	2284.9	29.32	1.108	0.0	0.7955;
358	103.2	0.0	31.59	34.51	2240.1	29.91	1.120	0.0	0.8024;
359	103.3	0.0	31.87	34.49	2196.1	30.51	1.131	0.0	0.8096;
360	103.4	0.0	32.16	34.47	2153.0	31.12	1.143	0.0	0.8169;
361	103.5	0.0	32.46	34.45	2110.8	31.74	1.154	0.0	0.8244;
362	103.6	0.0	32.76	34.43	2069.3	32.38	1.166	0.0	0.8320;
363	103.7	0.0	33.07	34.41	2028.7	33.03	1.177	0.0	0.8399;
364	103.8	0.0	33.38	34.38	1988.9	33.69	1.188	0.0	0.8479;
365	103.9	0.0	33.70	34.36	1949.9	34.36	1.199	0.0	0.8561;
366	104.0	0.0	34.03	34.35	1911.6	35.05	1.210	0.0	0.8645;
367	104.1	0.0	34.37	34.33	1874.1	35.75	1.221	0.0	0.8730;
368	104.2	0.0	34.71	34.31	1837.4	36.47	1.231	0.0	0.8817;
369	104.3	0.0	35.06	34.29	1801.3	37.20	1.242	0.0	0.8905;
370	104.5	0.0	35.42	34.27	1766.0	37.94	1.252	0.0	0.8996; bottom hit;

Horiz plane projections in effluent direction: radius(m): 0.0; CL(m): 0.3817
Lmz(m): 0.3817
forced entrain 1 0.0 -1.357 0.900 0.0978
Rate sec-1 0.0 dy-1 0.0 kt: 0.0 Amb Sal 33.3632
;
2:07:00 PM. amb fills: 4

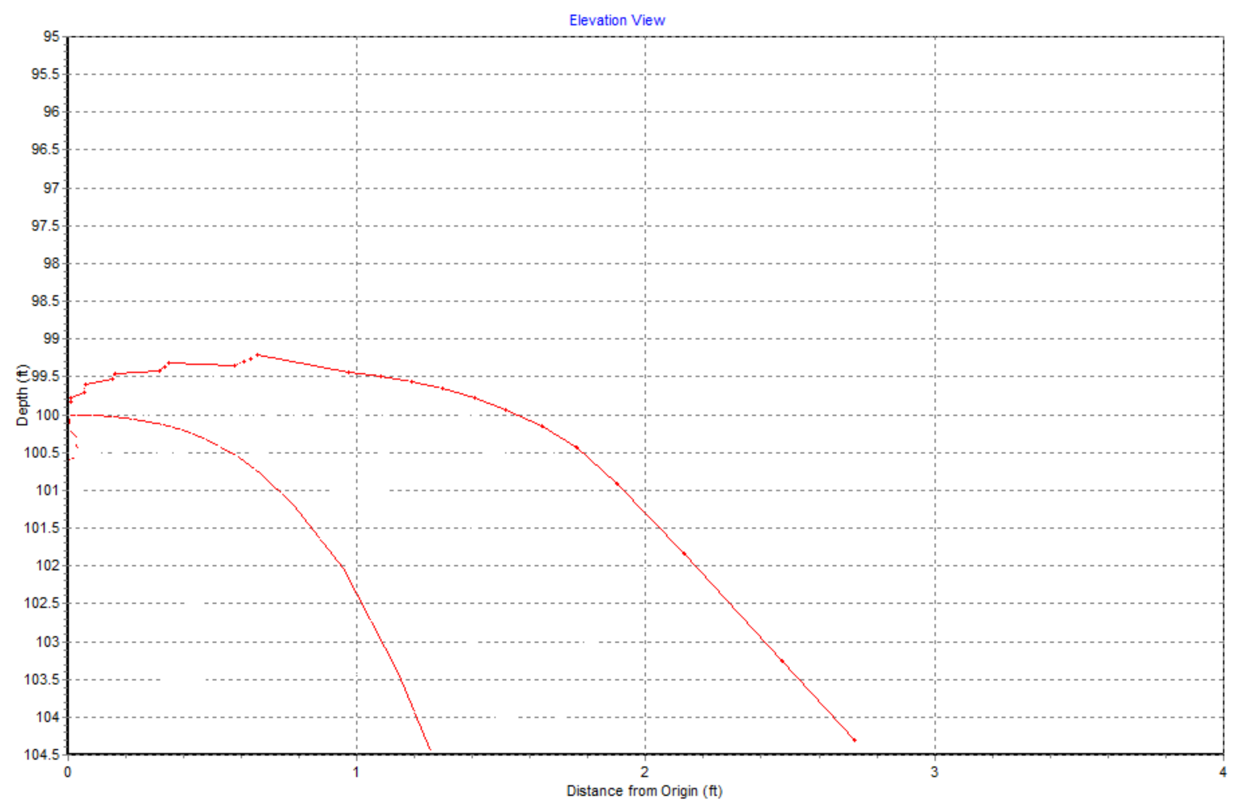

Figure C.5.1: Plumes 18b solution of discharge plume trajectories for discharges of 0 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt., and 5 mgd of brine from the Doheny Desalination Project with a brine salinity of 67 ppt. Depth of maximum rise of the plume is Z = 99.3 ft. at Xa = 0.800 ft from the point of discharge. The centerline of the plume is at an average distance of Xb = 1.252 ft from the point of discharge as the plume begins to impact the bottom at a depth of Z = 104.5 ft.

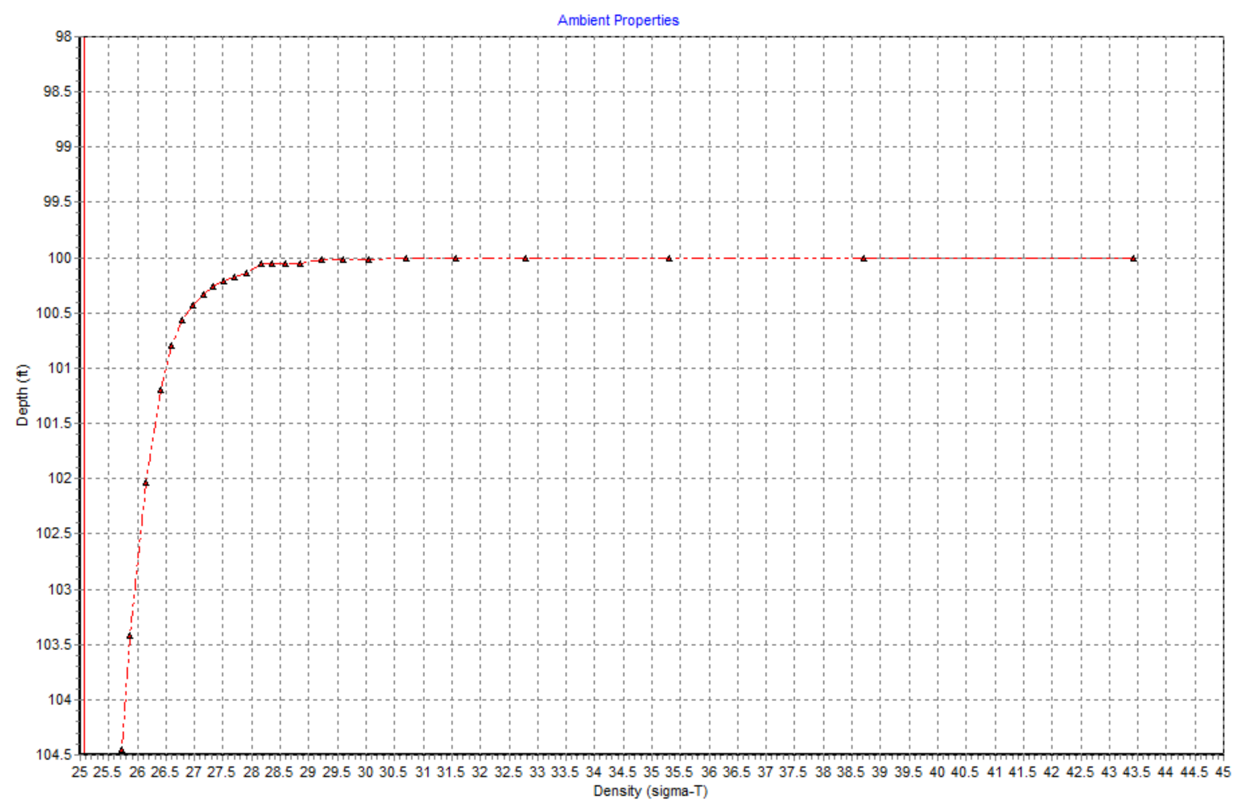

Figure C.5.2: Plumes 18b solution of vertical density profile for discharges of 0 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt. and 5 mgd of brine from the Doheny Desalination Project with a brine salinity of 67 ppt. Discharge effluent density shown as black triangles. Ambient water mass density profile shown as solid red line.

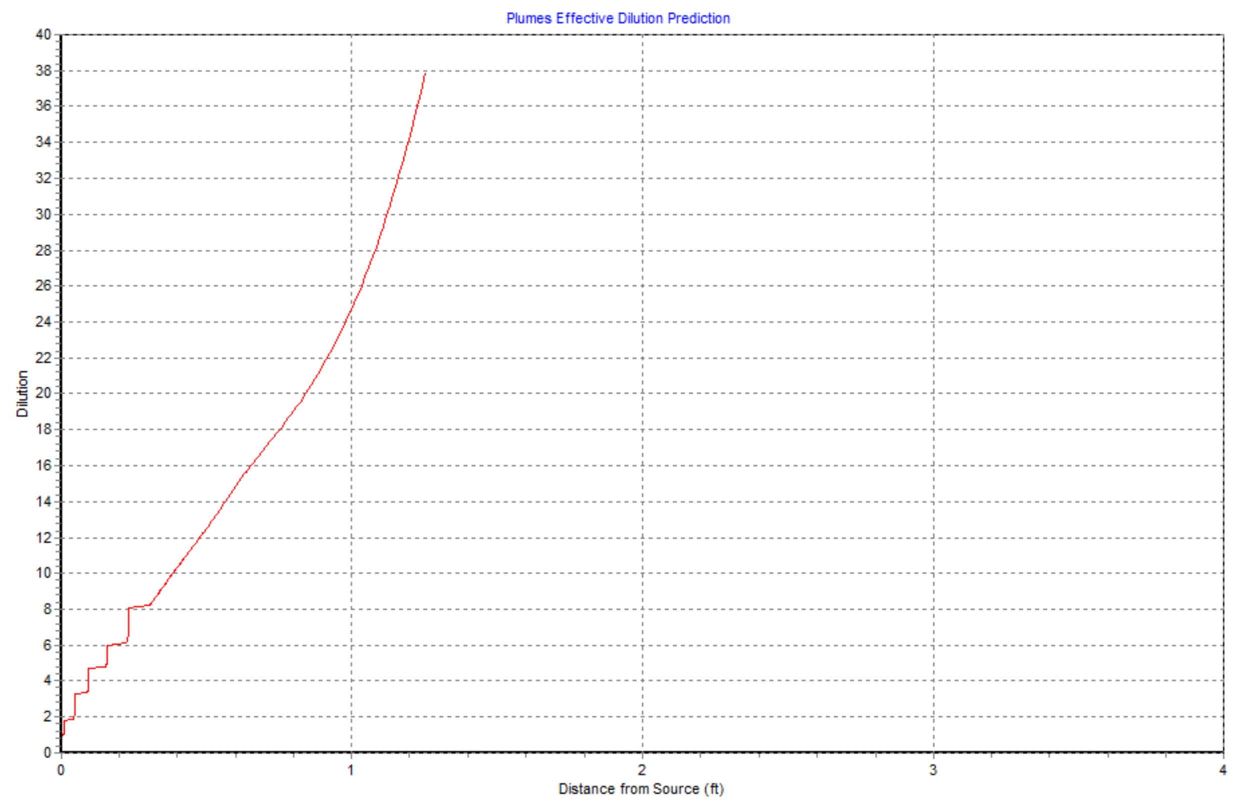

Figure C.5.3: Plumes 18b solution of effective (bulk average) dilution as a function of vertical distance from the point of discharge for discharges of 0 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt. and 5 mgd of brine from the Doheny Desalination Project with a brine salinity of 67 ppt. Effective dilution is S_a = 12.09 at the maximum rise of the plume at Xa = 6.038 ft. from the point of discharge. As the plume begins to impact the bottom, the plume centerline is at a distance of Xb = 10.14 ft from the point of discharge, where the effective dilution reaches S_a = 21.26.

C.6: Plumes 18b Results for SJCOO discharges of 0 mgd Wastewater and 3 mgd Brine:

Contents of the memo box (may not be current and must be updated manually)
Project "C:\Plumes18\SJCOO_WW0mgd_b3mgd_D-1"
memo
SJCOO discharging 0 mgd wastewater and 3 mgd brine

Model configuration items checked:
Channel width (m) 100
Start case for graphs 1
Max detailed graphs 10 (limits plots that can overflow memory)
Elevation Projection Plane (deg) 0
Shore vector (m,deg) not checked
Bacteria model : Mancini (1978) coliform model
PDS sfc. model heat transfer : Medium
Equation of State : S, T
Similarity Profile : Default profile (k=2.0, ...)
Diffuser port contraction coefficient 1
Light absorption coefficient 0.16
Farfield increment (m) 200
UM3 aspiration coefficient 0.1
Output file: text output tab
Output each ?? steps 1
Maximum dilution reported 1000
Text output format : Standard
Max vertical reversals : to max rise or fall

/ UM3. 1/14/2019 2:23:04 PM
Case 1; ambient file C:\Plumes18\SJCOO_WW0mgd_b3mgd_D-1.001.db; Diffuser table record 1: ---------
----

Ambient Table:

Depth	Amb-cur	Amb-dir	Amb-sal	Amb-tem	Amb-pol	Decay	Far-spd	Far-dir	Disprsn	Density
m	m/s	deg	psu	C	kg/kg	s-1	m/s	deg	m0.67/s2	sigma-T
0.0	0.0	0.0	33.38	19.97	0.0	0.0	0.0	0.0	0.0003	23.55719
2.000	0.0	0.0	33.39	19.87	0.0	0.0	0.0	0.0	0.0003	23.59288
4.000	0.0	0.0	33.40	19.81	0.0	0.0	0.0	0.0	0.0003	23.61484
7.000	0.0	0.0	33.41	19.64	0.0	0.0	0.0	0.0	0.0003	23.67096
10.00	0.0	0.0	33.40	17.76	0.0	0.0	0.0	0.0	0.0003	24.12888
12.00	0.0	0.0	33.38	17.34	0.0	0.0	0.0	0.0	0.0003	24.21549
14.00	0.0	0.0	33.38	16.73	0.0	0.0	0.0	0.0	0.0003	24.35932
16.00	0.0	0.0	33.38	15.17	0.0	0.0	0.0	0.0	0.0003	24.71340
18.00	0.0	0.0	33.37	14.37	0.0	0.0	0.0	0.0	0.0003	24.87044
20.00	0.0	0.0	33.35	14.18	0.0	0.0	0.0	0.0	0.0003	24.89660
22.00	0.0	0.0	33.33	13.83	0.0	0.0	0.0	0.0	0.0003	24.95172
24.00	0.0	0.0	33.32	13.75	0.0	0.0	0.0	0.0	0.0003	24.96707
26.00	0.0	0.0	33.36	13.42	0.0	0.0	0.0	0.0	0.0003	25.06459
31.85	0.0	0.0	33.36	13.39	0.0	0.0	0.0	0.0	0.0003	25.07201

Diffuser table:

P-dia	Ver angl	H-Angle	SourceX	SourceY	Ports	MZ-dis	Isoplth	P-depth	Ttl-flo	Eff-sal	Temp	Polutnt
(in)	(deg)	(deg)	(ft)	(ft)	()	(ft)	(concent)	(ft)	(MGD)	(psu)	(C)	(ppm)
3.0500	0.0	0.0	0.0	0.0	125.00	1000.0	0.0	100.00	3.0000	67.000	20.660	67000.0

Simulation:
Froude No: -1.678; Strat No:-1.57E-4; Spcg No: 14.39; k: 22307.6; eff den (sigmaT) 49.48870; eff vel 0.223(m/s);
Current is very small, flow regime may be transient.

Step	Depth (ft)	Amb-cur (m/s)	P-dia (in)	Eff-sal (psu)	Polutnt (ppm)	Dilutn ()	x-posn (ft)	y-posn (ft)	Iso dia (m)
0	100.0	1.000E-5	3.050	67.00	67000.0	1.000	0.0	0.0	0.07747;
1	100.0	0.0	3.055	66.89	66769.4	1.003	0.00219	0.0	0.0776;
2	100.0	0.0	3.092	66.23	65430.1	1.024	0.00299	0.0	0.07853;
3	100.0	0.0	3.154	65.59	64118.2	1.045	0.0033	0.0	0.08011;
4	100.0	0.0	3.218	64.95	62833.3	1.066	0.00342	0.0	0.08173;
5	100.0	0.0	3.283	64.33	61574.6	1.088	0.00347	0.0	0.08339;
6	100.0	0.0	3.349	63.73	60341.7	1.110	0.00349	0.0	0.08507;
7	100.0	0.0	3.417	63.13	59133.9	1.133	0.00349	0.0	0.08679;
8	100.0	0.0	3.486	62.55	57950.9	1.156	0.0035	0.0	0.08855;
9	100.0	0.0	3.557	61.98	56791.9	1.180	0.0035	0.0	0.09034;
10	100.0	0.0	3.628	61.41	55656.6	1.204	0.0035	0.0	0.09216;
11	100.0	0.0	3.702	60.86	54544.3	1.228	0.0035	0.0	0.09402;
12	100.0	0.0	3.776	60.33	53454.7	1.253	0.0035	0.0	0.09592;
13	100.0	0.0	3.853	59.80	52387.3	1.279	0.0035	0.0	0.09786;
14	100.0	0.0	3.930	59.28	51341.6	1.305	0.0035	0.0	0.09983;
15	100.0	0.0	4.010	58.77	50317.1	1.332	0.0035	0.0	0.1018;
16	100.0	0.0	4.091	58.27	49313.3	1.359	0.0205	0.0	0.1039;
17	100.0	0.0	4.153	58.00	48774.2	1.374	0.0231	0.0	0.1055;
18	100.0	0.0	4.217	57.52	47801.7	1.402	0.024	0.0	0.1071;
19	100.0	0.0	4.302	57.05	46849.0	1.430	0.0244	0.0	0.1093;
20	100.0	0.0	4.389	56.58	45915.5	1.459	0.0246	0.0	0.1115;
21	100.0	0.0	4.477	56.13	45000.9	1.489	0.0246	0.0	0.1137;
22	100.0	0.0	4.567	55.68	44104.9	1.519	0.0246	0.0	0.1160;
23	100.0	0.0	4.659	55.24	43226.9	1.550	0.0246	0.0	0.1183;
24	100.0	0.0	4.753	54.81	42366.6	1.581	0.0246	0.0	0.1207;
25	100.0	0.0	4.849	54.39	41523.7	1.614	0.0246	0.0	0.1232;
26	100.0	0.0	4.947	53.98	40697.9	1.646	0.0246	0.0	0.1256;
27	100.0	0.0	5.046	53.58	39888.6	1.680	0.0246	0.0	0.1282;
28	100.0	0.0	5.148	53.18	39095.7	1.714	0.0246	0.0	0.1308;
29	100.0	0.0	5.250	52.80	38341.3	1.747	0.0246	0.0	0.1334; begin overlap;
30	100.0	0.0	5.351	52.44	37623.1	1.781	0.0246	0.0	0.1359;
31	100.0	0.0	5.445	52.14	37022.9	1.810	0.0246	0.0	0.1383;
32	100.0	0.0	5.544	51.78	36287.6	1.846	0.0472	0.0	0.1408;
33	100.0	0.0	5.601	51.68	36098.3	1.856	0.0491	0.0	0.1423;
34	100.0	0.0	5.662	51.38	35493.1	1.888	0.0499	0.0	0.1438;
35	100.0	0.0	5.756	51.09	34913.6	1.919	0.0501	0.0	0.1462;
36	100.0	0.0	5.849	50.81	34361.7	1.950	0.0503	0.0	0.1486;
37	100.0	0.0	5.941	50.55	33835.4	1.980	0.0503	0.0	0.1509;
38	100.0	0.0	6.031	50.29	33332.8	2.010	0.0503	0.0	0.1532;
39	100.0	0.0	6.120	50.05	32852.2	2.039	0.0503	0.0	0.1555;
40	100.0	0.0	6.208	49.82	32392.0	2.068	0.0503	0.0	0.1577;
41	100.0	0.0	6.295	49.60	31950.9	2.097	0.0503	0.0	0.1599;
42	100.0	0.0	6.380	49.39	31527.5	2.125	0.0503	0.0	0.1620;
43	100.0	0.0	6.464	49.18	31120.7	2.153	0.0503	0.0	0.1642;
44	100.0	0.0	6.547	48.99	30729.4	2.180	0.0503	0.0	0.1663;
45	100.0	0.0	6.629	48.80	30353.0	2.207	0.0503	0.0	0.1684;
46	100.0	0.0	6.710	48.61	29980.9	2.235	0.0503	0.0	0.1704;
47	100.0	0.0	6.796	48.41	29586.9	2.265	0.0503	0.0	0.1726;
48	100.0	0.0	6.910	48.12	29000.8	2.310	0.0776	0.0	0.1755;
49	100.0	0.0	6.940	48.08	28933.6	2.316	0.0792	0.0	0.1763;

50	100.0	0.0	6.985	47.92	28604.9	2.342	0.0798	0.0	0.1774;
51	100.0	0.0	7.062	47.76	28285.2	2.369	0.0801	0.0	0.1794;
52	100.0	0.0	7.140	47.60	27975.7	2.395	0.0802	0.0	0.1814;
53	100.0	0.0	7.218	47.45	27676.2	2.421	0.0802	0.0	0.1833;
54	100.0	0.0	7.295	47.30	27386.1	2.446	0.0802	0.0	0.1853;
55	100.0	0.0	7.371	47.16	27105.0	2.472	0.0802	0.0	0.1872;
56	100.0	0.0	7.446	47.02	26832.5	2.497	0.0802	0.0	0.1891;
57	100.0	0.0	7.520	46.89	26568.1	2.522	0.0802	0.0	0.1910;
58	100.0	0.0	7.594	46.76	26311.5	2.546	0.0802	0.0	0.1929;
59	100.0	0.0	7.667	46.63	26062.1	2.571	0.0802	0.0	0.1947;
60	100.0	0.0	7.739	46.51	25819.9	2.595	0.0802	0.0	0.1966;
61	100.0	0.0	7.811	46.39	25584.2	2.619	0.0802	0.0	0.1984;
62	100.0	0.0	7.882	46.28	25353.0	2.643	0.0802	0.0	0.2002;
63	100.0	0.0	7.953	46.16	25127.0	2.666	0.0802	0.0	0.2020;
64	100.0	0.0	8.069	45.91	24630.0	2.720	0.111	0.0	0.2050;
65	100.0	0.0	8.039	45.89	24595.0	2.724	0.112	0.0	0.2042;
66	100.0	0.0	8.076	45.78	24370.2	2.749	0.113	0.0	0.2051;
67	100.0	0.0	8.147	45.67	24149.9	2.774	0.113	0.0	0.2069;
68	100.0	0.0	8.219	45.56	23935.1	2.799	0.113	0.0	0.2088;
69	100.0	0.0	8.291	45.45	23725.9	2.824	0.113	0.0	0.2106;
70	100.0	0.0	8.363	45.35	23522.0	2.848	0.113	0.0	0.2124;
71	100.0	0.0	8.435	45.25	23323.3	2.873	0.113	0.0	0.2142;
72	100.0	0.0	8.506	45.15	23129.6	2.897	0.113	0.0	0.2160;
73	100.0	0.0	8.576	45.06	22940.7	2.921	0.113	0.0	0.2178;
74	100.0	0.0	8.646	44.96	22756.3	2.944	0.113	0.0	0.2196;
75	100.0	0.0	8.715	44.87	22576.4	2.968	0.113	0.0	0.2214;
76	100.0	0.0	8.784	44.78	22400.7	2.991	0.113	0.0	0.2231;
77	100.0	0.0	8.852	44.70	22229.0	3.014	0.113	0.0	0.2248;
78	100.0	0.0	8.919	44.61	22061.3	3.037	0.113	0.0	0.2265;
79	100.0	0.0	8.987	44.53	21894.6	3.060	0.113	0.0	0.2283;
80	100.0	0.0	9.050	44.46	21753.6	3.080	0.113	0.0	0.2299;
81	100.0	0.0	9.097	44.41	21667.9	3.092	0.113	0.0	0.2311;
82	100.0	0.0	9.206	44.20	21239.8	3.154	0.145	0.0	0.2338;
83	100.0	0.0	9.076	44.19	21217.9	3.158	0.147	0.0	0.2305;
84	100.0	0.0	9.107	44.10	21044.5	3.184	0.147	0.0	0.2313;
85	100.0	0.0	9.177	44.01	20873.7	3.210	0.148	0.0	0.2331;
86	100.0	0.0	9.249	43.93	20706.6	3.236	0.148	0.0	0.2349;
87	100.0	0.0	9.321	43.84	20543.3	3.261	0.148	0.0	0.2368;
88	100.0	0.0	9.394	43.76	20383.7	3.287	0.148	0.0	0.2386;
89	100.0	0.0	9.467	43.68	20227.8	3.312	0.148	0.0	0.2405;
90	100.0	0.0	9.539	43.61	20075.4	3.337	0.148	0.0	0.2423;
91	100.0	0.0	9.610	43.53	19926.4	3.362	0.148	0.0	0.2441;
92	100.0	0.0	9.681	43.46	19780.7	3.387	0.148	0.0	0.2459;
93	100.0	0.0	9.752	43.39	19638.2	3.412	0.148	0.0	0.2477;
94	100.0	0.0	9.822	43.32	19498.7	3.436	0.148	0.0	0.2495;
95	100.0	0.0	9.891	43.25	19362.2	3.460	0.148	0.0	0.2512;
96	100.0	0.0	9.960	43.18	19228.5	3.484	0.148	0.0	0.2530;
97	100.0	0.0	10.03	43.11	19097.7	3.508	0.148	0.0	0.2547;
98	100.0	0.0	10.10	43.05	18970.7	3.532	0.148	0.0	0.2564;
99	100.0	0.0	10.16	42.99	18850.8	3.554	0.148	0.0	0.2581;
100	100.0	0.0	10.22	42.93	18738.9	3.575	0.148	0.0	0.2597;
101	100.0	0.0	10.28	42.89	18656.2	3.591	0.148	0.0	0.2610;
102	100.1	0.0	10.40	42.70	18288.0	3.664	0.181	0.0	0.2642;
103	100.1	0.0	10.13	42.69	18272.3	3.667	0.183	0.0	0.2573;
104	100.1	0.0	10.16	42.62	18126.7	3.696	0.183	0.0	0.2581;
105	100.1	0.0	10.23	42.55	17982.9	3.726	0.184	0.0	0.2599;

106	100.1	0.0	10.31	42.48	17841.9	3.755	0.184	0.0	0.2619;
107	100.1	0.0	10.39	42.41	17703.8	3.784	0.184	0.0	0.2639;
108	100.1	0.0	10.47	42.34	17568.8	3.814	0.184	0.0	0.2659;
109	100.1	0.0	10.55	42.27	17436.7	3.842	0.184	0.0	0.2679;
110	100.1	0.0	10.62	42.20	17307.5	3.871	0.184	0.0	0.2699;
111	100.1	0.0	10.70	42.14	17181.1	3.900	0.184	0.0	0.2718;
112	100.1	0.0	10.78	42.08	17057.5	3.928	0.184	0.0	0.2738;
113	100.1	0.0	10.86	42.02	16936.5	3.956	0.184	0.0	0.2758;
114	100.1	0.0	10.93	41.96	16818.1	3.984	0.184	0.0	0.2777;
115	100.1	0.0	11.01	41.90	16702.1	4.011	0.184	0.0	0.2797;
116	100.1	0.0	11.09	41.84	16588.5	4.039	0.184	0.0	0.2816;
117	100.1	0.0	11.16	41.78	16477.2	4.066	0.184	0.0	0.2835;
118	100.1	0.0	11.24	41.73	16368.1	4.093	0.184	0.0	0.2854;
119	100.1	0.0	11.31	41.67	16261.2	4.120	0.184	0.0	0.2873;
120	100.1	0.0	11.38	41.62	16156.2	4.147	0.184	0.0	0.2891;
121	100.1	0.0	11.46	41.57	16053.4	4.174	0.184	0.0	0.2910;
122	100.1	0.0	11.53	41.52	15952.6	4.200	0.184	0.0	0.2928;
123	100.1	0.0	11.60	41.47	15855.9	4.226	0.184	0.0	0.2947;
124	100.1	0.0	11.67	41.42	15755.6	4.252	0.184	0.0	0.2965;
125	100.1	0.0	11.75	41.36	15649.0	4.281	0.184	0.0	0.2984;
126	100.1	0.0	11.91	41.20	15340.5	4.368	0.218	0.0	0.3024;
127	100.1	0.0	11.44	41.20	15328.8	4.371	0.220	0.0	0.2906;
128	100.1	0.0	11.47	41.14	15203.5	4.407	0.221	0.0	0.2913;
129	100.1	0.0	11.55	41.07	15079.3	4.443	0.221	0.0	0.2933;
130	100.1	0.0	11.63	41.01	14957.2	4.479	0.222	0.0	0.2954;
131	100.1	0.0	11.72	40.95	14837.5	4.516	0.222	0.0	0.2976;
132	100.1	0.0	11.81	40.89	14720.2	4.552	0.222	0.0	0.2999;
133	100.1	0.0	11.90	40.83	14605.3	4.587	0.222	0.0	0.3021;
134	100.1	0.0	11.99	40.77	14493.0	4.623	0.222	0.0	0.3044;
135	100.1	0.0	12.08	40.72	14383.0	4.658	0.222	0.0	0.3067;
136	100.1	0.0	12.17	40.66	14275.5	4.693	0.222	0.0	0.3090;
137	100.1	0.0	12.26	40.61	14170.3	4.728	0.222	0.0	0.3113;
138	100.1	0.0	12.35	40.56	14067.3	4.763	0.223	0.0	0.3136;
139	100.1	0.0	12.43	40.51	13966.6	4.797	0.223	0.0	0.3158;
140	100.1	0.0	12.52	40.46	13868.0	4.831	0.223	0.0	0.3181;
141	100.1	0.0	12.61	40.41	13771.4	4.865	0.223	0.0	0.3203;
142	100.1	0.0	12.70	40.36	13676.8	4.899	0.223	0.0	0.3225;
143	100.1	0.0	12.79	40.31	13584.2	4.932	0.223	0.0	0.3247;
144	100.1	0.0	12.87	40.27	13493.4	4.965	0.223	0.0	0.3269;
145	100.1	0.0	12.96	40.22	13404.4	4.998	0.223	0.0	0.3291;
146	100.1	0.0	13.04	40.18	13317.2	5.031	0.223	0.0	0.3313;
147	100.1	0.0	13.13	40.13	13231.6	5.064	0.223	0.0	0.3334;
148	100.1	0.0	13.21	40.09	13147.7	5.096	0.223	0.0	0.3356;
149	100.1	0.0	13.29	40.05	13065.4	5.128	0.223	0.0	0.3377;
150	100.1	0.0	13.38	40.01	12984.6	5.160	0.223	0.0	0.3398;
151	100.1	0.0	13.46	39.97	12905.3	5.192	0.223	0.0	0.3419;
152	100.1	0.0	13.54	39.93	12827.5	5.223	0.223	0.0	0.3440;
153	100.1	0.0	13.62	39.89	12751.0	5.254	0.223	0.0	0.3460;
154	100.1	0.0	13.70	39.85	12676.0	5.286	0.223	0.0	0.3481;
155	100.1	0.0	13.79	39.81	12602.2	5.317	0.223	0.0	0.3501;
156	100.1	0.0	13.87	39.77	12529.6	5.347	0.223	0.0	0.3522;
157	100.1	0.0	13.95	39.74	12458.4	5.378	0.223	0.0	0.3542;
158	100.1	0.0	14.02	39.70	12388.7	5.408	0.223	0.0	0.3562;
159	100.1	0.0	14.10	39.67	12318.2	5.439	0.223	0.0	0.3582;
160	100.1	0.0	14.18	39.63	12249.7	5.470	0.223	0.0	0.3602;
161	100.1	0.0	14.27	39.59	12174.1	5.503	0.223	0.0	0.3624;

162	100.1	0.0	14.38	39.53	12056.1	5.557	0.223	0.0	0.3653;
163	100.1	0.0	14.49	39.50	11990.5	5.588	0.223	0.0	0.3680;
164	100.1	0.0	14.56	39.47	11938.4	5.612	0.223	0.0	0.3698;
165	100.1	0.0	14.61	39.46	11903.6	5.629	0.223	0.0	0.3712;
166	100.1	0.0	14.78	39.34	11669.2	5.742	0.259	0.0	0.3754;
167	100.1	0.0	13.88	39.33	11662.2	5.745	0.261	0.0	0.3525;
168	100.1	0.0	13.90	39.28	11565.7	5.793	0.262	0.0	0.3531;
169	100.1	0.0	13.99	39.23	11469.7	5.841	0.263	0.0	0.3553;
170	100.1	0.0	14.07	39.19	11374.5	5.890	0.264	0.0	0.3575;
171	100.1	0.0	14.16	39.14	11280.4	5.940	0.265	0.0	0.3597;
172	100.1	0.0	14.25	39.09	11187.3	5.989	0.266	0.0	0.3620;
173	100.1	0.0	14.35	39.04	11095.3	6.039	0.266	0.0	0.3644;
174	100.1	0.0	14.44	39.00	11004.5	6.088	0.267	0.0	0.3668;
175	100.1	0.0	14.53	38.95	10914.9	6.138	0.268	0.0	0.3692;
176	100.2	0.0	14.63	38.91	10826.5	6.189	0.269	0.0	0.3716;
177	100.2	0.0	14.73	38.86	10739.2	6.239	0.269	0.0	0.3740;
178	100.2	0.0	14.82	38.82	10653.1	6.289	0.270	0.0	0.3765;
179	100.2	0.0	14.92	38.77	10568.1	6.340	0.271	0.0	0.3789;
180	100.2	0.0	15.01	38.73	10484.2	6.391	0.272	0.0	0.3814;
181	100.2	0.0	15.11	38.69	10401.5	6.441	0.272	0.0	0.3838;
182	100.2	0.0	15.21	38.65	10319.8	6.492	0.273	0.0	0.3863;
183	100.2	0.0	15.30	38.61	10239.2	6.544	0.274	0.0	0.3887;
184	100.2	0.0	15.40	38.57	10159.5	6.595	0.275	0.0	0.3911;
185	100.2	0.0	15.49	38.53	10080.8	6.646	0.275	0.0	0.3935;
186	100.2	0.0	15.59	38.49	10003.0	6.698	0.276	0.0	0.3959;
187	100.2	0.0	15.68	38.45	9926.0	6.750	0.277	0.0	0.3982;
188	100.2	0.0	15.77	38.41	9849.8	6.802	0.278	0.0	0.4006;
189	100.2	0.0	15.86	38.37	9774.4	6.855	0.279	0.0	0.4029;
190	100.2	0.0	15.95	38.33	9699.6	6.907	0.280	0.0	0.4051;
191	100.2	0.0	16.04	38.29	9625.4	6.961	0.281	0.0	0.4073;
192	100.2	0.0	16.12	38.26	9551.7	7.014	0.282	0.0	0.4095;
193	100.2	0.0	16.20	38.22	9478.4	7.069	0.283	0.0	0.4116;
194	100.2	0.0	16.28	38.18	9405.5	7.124	0.284	0.0	0.4136;
195	100.2	0.0	16.36	38.14	9332.7	7.179	0.286	0.0	0.4155;
196	100.2	0.0	16.43	38.11	9263.3	7.233	0.287	0.0	0.4173;
197	100.2	0.0	16.49	38.08	9197.6	7.285	0.289	0.0	0.4189;
198	100.2	0.0	16.55	38.04	9132.4	7.337	0.290	0.0	0.4203;
199	100.2	0.0	16.61	38.01	9067.8	7.389	0.292	0.0	0.4218;
200	100.2	0.0	16.66	37.98	9003.7	7.441	0.293	0.0	0.4232;
201	100.2	0.0	16.72	37.94	8940.2	7.494	0.295	0.0	0.4246;
202	100.2	0.0	16.77	37.91	8877.2	7.547	0.297	0.0	0.4260;
203	100.2	0.0	16.83	37.88	8814.7	7.601	0.298	0.0	0.4274;
204	100.2	0.0	16.88	37.85	8752.7	7.655	0.300	0.0	0.4288;
205	100.2	0.0	16.94	37.82	8691.1	7.709	0.301	0.0	0.4302;
206	100.2	0.0	16.99	37.79	8630.0	7.764	0.303	0.0	0.4315;
207	100.2	0.0	17.04	37.75	8569.3	7.819	0.305	0.0	0.4328;
208	100.2	0.0	17.09	37.72	8509.0	7.874	0.306	0.0	0.4341;
209	100.2	0.0	17.14	37.69	8449.1	7.930	0.308	0.0	0.4354;
210	100.2	0.0	17.19	37.66	8389.5	7.986	0.310	0.0	0.4367;
211	100.2	0.0	17.24	37.63	8330.4	8.043	0.311	0.0	0.4380;
212	100.2	0.0	17.29	37.60	8271.6	8.100	0.313	0.0	0.4393;
213	100.2	0.0	17.34	37.57	8213.1	8.158	0.315	0.0	0.4405;
214	100.2	0.0	17.39	37.54	8154.9	8.216	0.317	0.0	0.4418;
215	100.2	0.0	17.44	37.51	8097.0	8.275	0.318	0.0	0.4430;
216	100.2	0.0	17.49	37.48	8039.5	8.334	0.320	0.0	0.4442;
217	100.2	0.0	17.54	37.45	7982.1	8.394	0.322	0.0	0.4454;

218	100.2	0.0	17.58	37.42	7925.1	8.454	0.324	0.0	0.4466;
219	100.2	0.0	17.63	37.40	7868.2	8.515	0.326	0.0	0.4478;
220	100.2	0.0	17.67	37.37	7811.6	8.577	0.328	0.0	0.4489;
221	100.2	0.0	17.72	37.34	7755.2	8.639	0.329	0.0	0.4501;
222	100.3	0.0	17.77	37.31	7699.0	8.702	0.331	0.0	0.4512;
223	100.3	0.0	17.81	37.28	7642.9	8.766	0.333	0.0	0.4524;
224	100.3	0.0	17.86	37.25	7587.0	8.831	0.335	0.0	0.4535;
225	100.3	0.0	17.90	37.22	7531.3	8.896	0.337	0.0	0.4546;
226	100.3	0.0	17.94	37.20	7475.6	8.962	0.339	0.0	0.4558;
227	100.3	0.0	17.99	37.17	7420.1	9.030	0.341	0.0	0.4569;
228	100.3	0.0	18.03	37.14	7364.6	9.098	0.343	0.0	0.4580;
229	100.3	0.0	18.07	37.11	7309.2	9.167	0.345	0.0	0.4591;
230	100.3	0.0	18.12	37.08	7253.8	9.237	0.348	0.0	0.4602;
231	100.3	0.0	18.16	37.05	7198.5	9.308	0.350	0.0	0.4613;
232	100.3	0.0	18.20	37.03	7143.2	9.380	0.352	0.0	0.4624;
233	100.3	0.0	18.25	37.00	7087.8	9.453	0.354	0.0	0.4635;
234	100.3	0.0	18.29	36.97	7032.4	9.527	0.356	0.0	0.4646;
235	100.3	0.0	18.33	36.94	6977.0	9.603	0.359	0.0	0.4657;
236	100.3	0.0	18.38	36.91	6921.4	9.680	0.361	0.0	0.4668;
237	100.3	0.0	18.42	36.88	6865.8	9.759	0.363	0.0	0.4679;
238	100.3	0.0	18.46	36.85	6810.0	9.838	0.366	0.0	0.4690;
239	100.3	0.0	18.51	36.83	6754.1	9.920	0.368	0.0	0.4701;
240	100.3	0.0	18.55	36.80	6698.0	10.00	0.370	0.0	0.4712;
241	100.3	0.0	18.60	36.77	6641.6	10.09	0.373	0.0	0.4724;
242	100.3	0.0	18.64	36.74	6585.1	10.17	0.375	0.0	0.4735;
243	100.4	0.0	18.69	36.71	6528.2	10.26	0.378	0.0	0.4747;
244	100.4	0.0	18.74	36.68	6471.0	10.35	0.380	0.0	0.4759;
245	100.4	0.0	18.78	36.65	6413.5	10.45	0.383	0.0	0.4771;
246	100.4	0.0	18.83	36.62	6355.6	10.54	0.386	0.0	0.4783;
247	100.4	0.0	18.88	36.59	6297.4	10.64	0.389	0.0	0.4796;
248	100.4	0.0	18.93	36.56	6240.4	10.74	0.391	0.0	0.4808; end overlap;
249	100.4	0.0	18.97	36.53	6184.4	10.83	0.394	0.0	0.4819;
250	100.4	0.0	19.02	36.51	6129.4	10.93	0.397	0.0	0.4830;
251	100.4	0.0	19.05	36.48	6075.3	11.03	0.400	0.0	0.4840;
252	100.4	0.0	19.09	36.45	6022.1	11.13	0.403	0.0	0.4848;
253	100.4	0.0	19.12	36.43	5969.8	11.22	0.406	0.0	0.4857;
254	100.4	0.0	19.15	36.40	5918.2	11.32	0.409	0.0	0.4864;
255	100.5	0.0	19.17	36.37	5867.2	11.42	0.412	0.0	0.4870;
256	100.5	0.0	19.20	36.35	5816.9	11.52	0.415	0.0	0.4876;
257	100.5	0.0	19.22	36.32	5767.2	11.62	0.419	0.0	0.4881;
258	100.5	0.0	19.23	36.30	5718.0	11.72	0.422	0.0	0.4885;
259	100.5	0.0	19.25	36.27	5669.2	11.82	0.425	0.0	0.4889;
260	100.5	0.0	19.26	36.25	5620.8	11.92	0.429	0.0	0.4892;
261	100.5	0.0	19.27	36.22	5572.7	12.02	0.433	0.0	0.4894;
262	100.5	0.0	19.28	36.20	5524.8	12.13	0.436	0.0	0.4896;
263	100.5	0.0	19.28	36.17	5477.0	12.23	0.440	0.0	0.4897;
264	100.6	0.0	19.28	36.15	5429.4	12.34	0.444	0.0	0.4898;
265	100.6	0.0	19.29	36.12	5381.8	12.45	0.448	0.0	0.4899;
266	100.6	0.0	19.29	36.10	5334.1	12.56	0.452	0.0	0.4899;
267	100.6	0.0	19.29	36.08	5286.2	12.67	0.456	0.0	0.4899;
268	100.6	0.0	19.28	36.05	5238.1	12.79	0.461	0.0	0.4898;
269	100.6	0.0	19.28	36.03	5189.7	12.91	0.465	0.0	0.4897;
270	100.7	0.0	19.28	36.00	5140.9	13.03	0.470	0.0	0.4897;
271	100.7	0.0	19.27	35.98	5091.5	13.16	0.474	0.0	0.4896;
272	100.7	0.0	19.27	35.95	5041.6	13.29	0.479	0.0	0.4895;
273	100.7	0.0	19.27	35.92	4990.9	13.42	0.484	0.0	0.4894;

274	100.7	0.0	19.27	35.90	4939.4	13.56	0.489	0.0	0.4894;	
275	100.8	0.0	19.27	35.87	4886.9	13.71	0.495	0.0	0.4893;	
276	100.8	0.0	19.27	35.84	4833.4	13.86	0.500	0.0	0.4894;	
277	100.8	0.0	19.27	35.82	4778.8	14.02	0.506	0.0	0.4895;	
278	100.9	0.0	19.28	35.79	4722.8	14.19	0.512	0.0	0.4896;	
279	100.9	0.0	19.28	35.76	4665.4	14.36	0.518	0.0	0.4898;	
280	100.9	0.0	19.30	35.73	4606.4	14.54	0.524	0.0	0.4902;	
281	100.9	0.0	19.31	35.70	4545.7	14.74	0.530	0.0	0.4906;	
282	101.0	0.0	19.34	35.66	4483.2	14.94	0.537	0.0	0.4912;	
283	101.0	0.0	19.37	35.63	4418.7	15.16	0.544	0.0	0.4919;	
284	101.1	0.0	19.40	35.60	4352.0	15.40	0.551	0.0	0.4928;	
285	101.1	0.0	19.44	35.56	4283.0	15.64	0.559	0.0	0.4939;	
286	101.2	0.0	19.50	35.52	4211.5	15.91	0.566	0.0	0.4952;	
287	101.2	0.0	19.56	35.49	4137.4	16.19	0.574	0.0	0.4968;	
288	101.3	0.0	19.63	35.45	4060.6	16.50	0.583	0.0	0.4987;	
289	101.3	0.0	19.72	35.41	3980.8	16.83	0.592	0.0	0.5008;	
290	101.4	0.0	19.81	35.37	3902.7	17.17	0.600	0.0	0.5031;	
291	101.4	0.0	19.90	35.33	3826.0	17.51	0.608	0.0	0.5056;	
292	101.5	0.0	20.01	35.29	3750.9	17.86	0.617	0.0	0.5083;	
293	101.6	0.0	20.13	35.25	3677.3	18.22	0.625	0.0	0.5112;	
294	101.6	0.0	20.25	35.21	3605.1	18.58	0.633	0.0	0.5143;	
295	101.7	0.0	20.38	35.18	3534.3	18.96	0.640	0.0	0.5177;	
296	101.7	0.0	20.52	35.14	3464.9	19.34	0.648	0.0	0.5212;	
297	101.8	0.0	20.66	35.11	3396.9	19.72	0.655	0.0	0.5249;	
298	101.9	0.0	20.82	35.07	3330.2	20.12	0.663	0.0	0.5287;	
299	101.9	0.0	20.97	35.04	3264.8	20.52	0.670	0.0	0.5328;	
300	102.0	0.0	21.14	35.01	3200.7	20.93	0.677	0.0	0.5369;	
301	102.1	0.0	21.31	34.97	3137.9	21.35	0.684	0.0	0.5413;	
302	102.1	0.0	21.49	34.94	3076.3	21.78	0.691	0.0	0.5457;	
303	102.2	0.0	21.67	34.91	3015.9	22.22	0.698	0.0	0.5503;	
304	102.2	0.0	21.85	34.88	2956.7	22.66	0.705	0.0	0.5551;	
305	102.3	0.0	22.05	34.85	2898.7	23.11	0.712	0.0	0.5600;	
306	102.4	0.0	22.24	34.82	2841.8	23.58	0.718	0.0	0.5650;	
307	102.5	0.0	22.44	34.79	2786.0	24.05	0.725	0.0	0.5701;	
308	102.5	0.0	22.65	34.77	2731.3	24.53	0.731	0.0	0.5754;	
309	102.6	0.0	22.86	34.74	2677.7	25.02	0.738	0.0	0.5807;	
310	102.7	0.0	23.08	34.71	2625.2	25.52	0.744	0.0	0.5862;	
311	102.7	0.0	23.30	34.68	2573.6	26.03	0.750	0.0	0.5919;	
312	102.8	0.0	23.53	34.66	2523.1	26.55	0.757	0.0	0.5976;	
313	102.9	0.0	23.76	34.63	2473.6	27.09	0.763	0.0	0.6034;	
314	102.9	0.0	23.99	34.61	2425.1	27.63	0.769	0.0	0.6094;	
315	103.0	0.0	24.23	34.58	2377.5	28.18	0.775	0.0	0.6155;	
316	103.1	0.0	24.48	34.56	2330.8	28.75	0.781	0.0	0.6217;	
317	103.2	0.0	24.73	34.54	2285.1	29.32	0.787	0.0	0.6280;	
318	103.3	0.0	24.98	34.51	2240.2	29.91	0.793	0.0	0.6345;	
319	103.3	0.0	25.24	34.49	2196.3	30.51	0.799	0.0	0.6410;	
320	103.4	0.0	25.50	34.47	2153.2	31.12	0.805	0.0	0.6476;	
321	103.5	0.0	25.76	34.45	2110.9	31.74	0.810	0.0	0.6544;	
322	103.6	0.0	26.04	34.43	2069.5	32.37	0.816	0.0	0.6613;	
323	103.6	0.0	26.31	34.41	2028.9	33.02	0.822	0.0	0.6683;	
324	103.7	0.0	26.59	34.38	1989.1	33.68	0.827	0.0	0.6754;	
325	103.8	0.0	26.87	34.36	1950.1	34.36	0.833	0.0	0.6826;	
326	103.9	0.0	27.16	34.35	1911.8	35.05	0.838	0.0	0.6899;	
327	104.0	0.0	27.46	34.33	1874.3	35.75	0.844	0.0	0.6974;	
328	104.1	0.0	27.75	34.31	1837.5	36.46	0.849	0.0	0.7049;	
329	104.2	0.0	28.06	34.29	1801.4	37.19	0.855	0.0	0.7126;	

330	104.2	0.0	28.36	34.27	1766.1	37.94	0.860	0.0	0.7204;
331	104.3	0.0	28.67	34.25	1731.5	38.70	0.866	0.0	0.7283;
332	104.4	0.0	28.99	34.24	1697.5	39.47	0.871	0.0	0.7363;
333	104.5	0.0	29.31	34.22	1664.2	40.26	0.876	0.0	0.7444; bottom hit;

Horiz plane projections in effluent direction: radius(m): 0.0; CL(m): 0.2671
Lmz(m): 0.2671
forced entrain 1 0.0 -1.375 0.744 0.0595
Rate sec-1 0.0 dy-1 0.0 kt: 0.0 Amb Sal 33.3632
;
2:23:05 PM. amb fills: 4

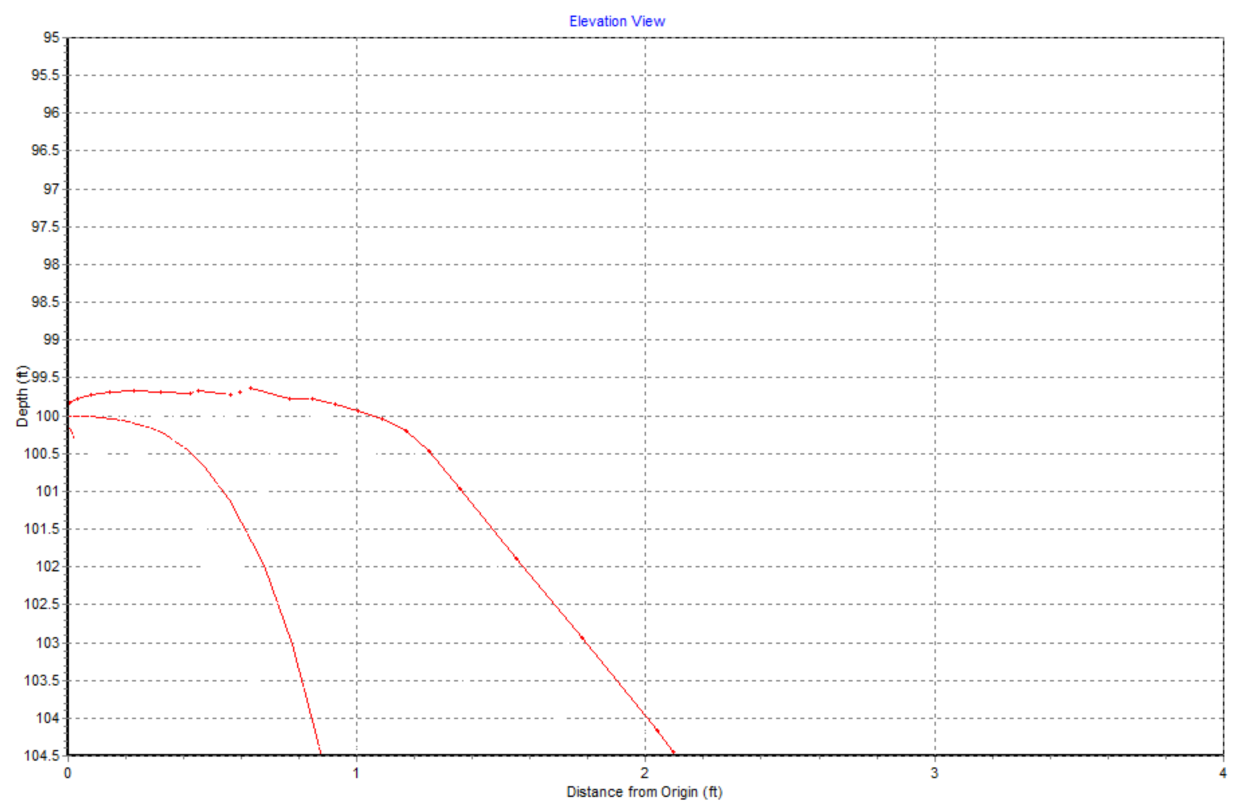

Figure C.6.1: Plumes 18b solution of discharge plume trajectories for discharges of 0 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt., and 3 mgd of brine from the Doheny Desalination Project with a brine salinity of 67 ppt. Depth of maximum rise of the plume is Z = 99.8 ft. at Xa = 0.750 ft from the point of discharge. The centerline of the plume is at an average distance of Xb = 0.876 ft from the point of discharge as the plume begins to impact the bottom at a depth of Z = 104.5 ft.

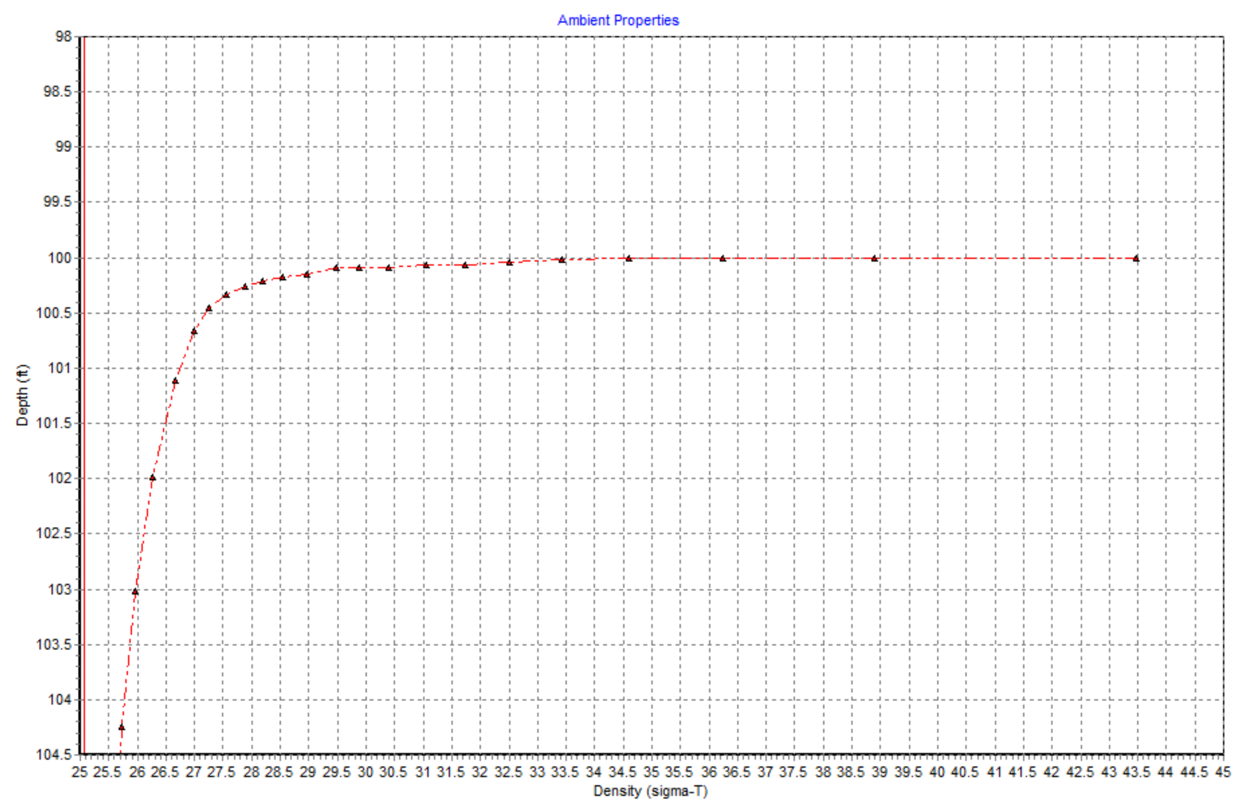

Figure C.6.2: Plumes 18b solution of vertical density profile for discharges of 0 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt. and 3 mgd of brine from the Doheny Desalination Project with a brine salinity of 67 ppt. Discharge effluent density shown as black triangles. Ambient water mass density profile shown as solid red line.

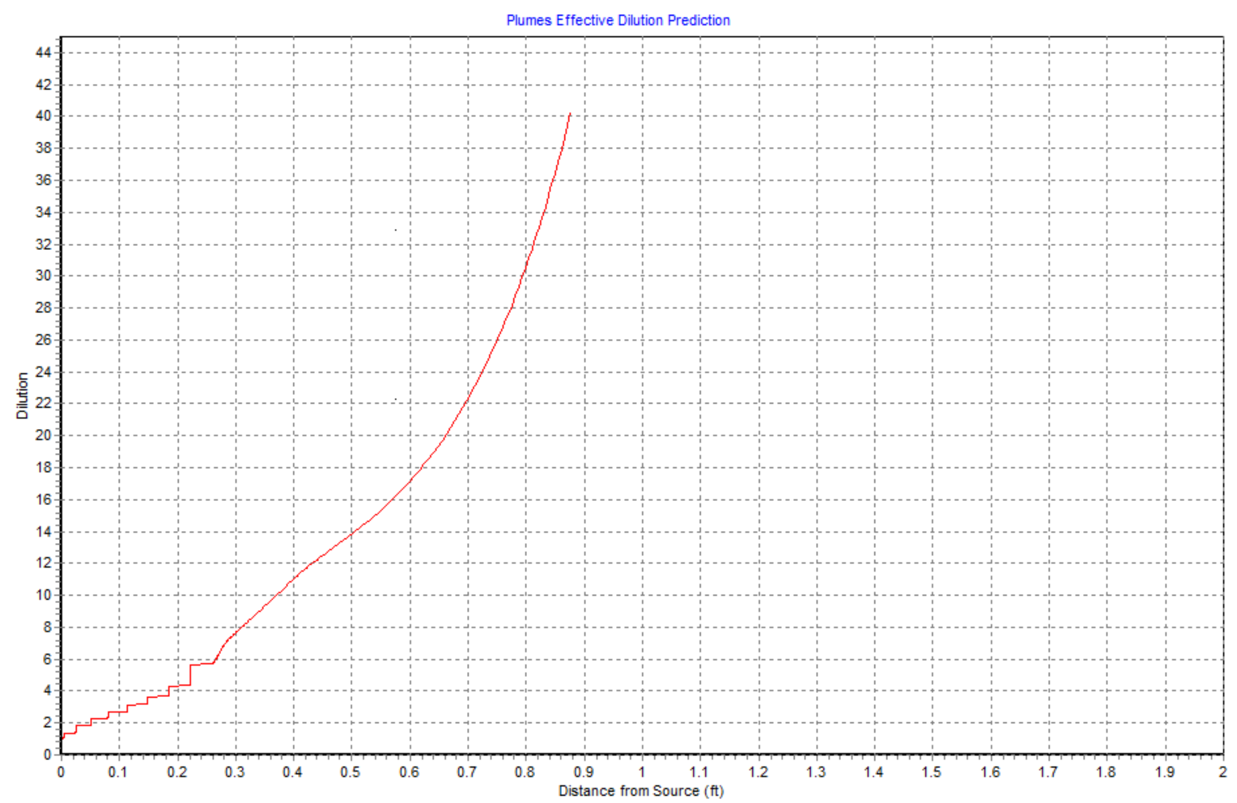

Figure C.6.3: Plumes 18b solution of effective (bulk average) dilution as a function of vertical distance from the point of discharge for discharges of 0 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt. and 3 mgd of brine from the Doheny Desalination Project with a brine salinity of 67 ppt. Effective dilution is S_a = 26.03 at the maximum rise of the plume at Xa = 0.750 ft. from the point of discharge. As the plume begins to impact the bottom, the plume centerline is at a distance of Xb = 0.876 ft from the point of discharge, where the effective dilution reaches S_a = 40.26.

C.7: Plumes 18b Results for SJCOO discharges of 0.35 mgd Wastewater and 5 mgd Brine:

Contents of the memo box (may not be current and must be updated manually)
Project "C:\Plumes18\SJCOO_WW0.35mgd_b5mgd_D-1"
memo
SJCOO discharging 0.35 mgd wastewater and 5 mgd brine

Model configuration items checked:
Channel width (m) 100
Start case for graphs 1
Max detailed graphs 10 (limits plots that can overflow memory)
Elevation Projection Plane (deg) 0
Shore vector (m,deg) not checked
Bacteria model : Mancini (1978) coliform model
PDS sfc. model heat transfer : Medium
Equation of State : S, T
Similarity Profile : Default profile (k=2.0, ...)
Diffuser port contraction coefficient 1
Light absorption coefficient 0.16
Farfield increment (m) 200
UM3 aspiration coefficient 0.1
Output file: text output tab
Output each ?? steps 1
Maximum dilution reported 1000
Text output format : Standard
Max vertical reversals : to max rise or fall

/ UM3. 1/14/2019 2:50:38 PM
Case 1; ambient file C:\Plumes18\SJCOO_WW0.35mgd_b5mgd_D-1.001.db; Diffuser table record 1: ----------------------------------

Ambient Table:

Depth	Amb-cur	Amb-dir	Amb-sal	Amb-tem	Amb-pol	Decay	Far-spd	Far-dir	Disprsn	Density
m	m/s	deg	psu	C	kg/kg	s-1	m/s	deg	m0.67/s2	sigma-T
0.0	0.0	0.0	33.38	19.97	0.0	0.0	0.0	0.0	0.0003	23.55719
2.000	0.0	0.0	33.39	19.87	0.0	0.0	0.0	0.0	0.0003	23.59288
4.000	0.0	0.0	33.40	19.81	0.0	0.0	0.0	0.0	0.0003	23.61484
7.000	0.0	0.0	33.41	19.64	0.0	0.0	0.0	0.0	0.0003	23.67096
10.00	0.0	0.0	33.40	17.76	0.0	0.0	0.0	0.0	0.0003	24.12888
12.00	0.0	0.0	33.38	17.34	0.0	0.0	0.0	0.0	0.0003	24.21549
14.00	0.0	0.0	33.38	16.73	0.0	0.0	0.0	0.0	0.0003	24.35932
16.00	0.0	0.0	33.38	15.17	0.0	0.0	0.0	0.0	0.0003	24.71340
18.00	0.0	0.0	33.37	14.37	0.0	0.0	0.0	0.0	0.0003	24.87044
20.00	0.0	0.0	33.35	14.18	0.0	0.0	0.0	0.0	0.0003	24.89660
22.00	0.0	0.0	33.33	13.83	0.0	0.0	0.0	0.0	0.0003	24.95172
24.00	0.0	0.0	33.32	13.75	0.0	0.0	0.0	0.0	0.0003	24.96707
26.00	0.0	0.0	33.36	13.42	0.0	0.0	0.0	0.0	0.0003	25.06459
31.85	0.0	0.0	33.36	13.39	0.0	0.0	0.0	0.0	0.0003	25.07201

Diffuser table:

P-dia	Ver angl	H-Angle	SourceX	SourceY	Ports	MZ-dis	Isoplth	P-depth	Ttl-flo	Eff-sal	Temp	Polutnt
(in)	(deg)	(deg)	(ft)	(ft)	()	(ft)	(concent)	(ft)	(MGD)	(psu)	(C)	(ppm)
3.0500	0.0	0.0	0.0	0.0	125.00	1000.0	0.0	100.00	5.3500	62.630	20.660	62630.0

Simulation:
Froude No: -3.226; Strat No:-1.84E-4; Spcg No: 14.39; k: 39781.8; eff den (sigmaT) 46.00388; eff vel 0.398(m/s);
Current is very small, flow regime may be transient.

Step	Depth (ft)	Amb-cur (m/s)	P-dia (in)	Eff-sal (psu)	Polutnt (ppm)	Dilutn ()	x-posn (ft)	y-posn (ft)	Iso dia (m)
0	100.0	1.000E-5	3.050	62.63	62630.0	1.000	0.0	0.0	0.07747;
1	100.0	0.0	3.059	62.45	62246.6	1.006	0.00389	0.0	0.07771;
2	100.0	0.0	3.100	61.88	61002.2	1.027	0.00642	0.0	0.07873;
3	100.0	0.0	3.162	61.32	59783.1	1.048	0.0082	0.0	0.08032;
4	100.0	0.0	3.226	60.78	58588.9	1.069	0.0095	0.0	0.08195;
5	100.0	0.0	3.291	60.24	57418.9	1.091	0.0105	0.0	0.0836;
6	100.0	0.0	3.358	59.71	56272.7	1.113	0.0112	0.0	0.08529;
7	100.0	0.0	3.426	59.20	55149.8	1.136	0.0118	0.0	0.08701;
8	100.0	0.0	3.495	58.69	54049.7	1.159	0.0123	0.0	0.08876;
9	100.0	0.0	3.565	58.19	52971.8	1.182	0.0126	0.0	0.09055;
10	100.0	0.0	3.637	57.71	51915.9	1.206	0.0129	0.0	0.09238;
11	100.0	0.0	3.710	57.23	50881.3	1.231	0.0131	0.0	0.09424;
12	100.0	0.0	3.785	56.76	49867.6	1.256	0.0133	0.0	0.09614;
13	100.0	0.0	3.861	56.30	48874.4	1.281	0.0134	0.0	0.09808;
14	100.0	0.0	3.939	55.85	47901.3	1.307	0.0136	0.0	0.1001;
15	100.0	0.0	4.019	55.41	46947.9	1.334	0.0136	0.0	0.1021;
16	100.0	0.0	4.100	54.98	46013.7	1.361	0.0137	0.0	0.1041;
17	100.0	0.0	4.182	54.55	45098.4	1.389	0.0138	0.0	0.1062;
18	100.0	0.0	4.267	54.14	44201.5	1.417	0.0138	0.0	0.1084;
19	100.0	0.0	4.353	53.73	43322.7	1.446	0.0139	0.0	0.1106;
20	100.0	0.0	4.440	53.33	42461.6	1.475	0.0139	0.0	0.1128;
21	100.0	0.0	4.530	52.94	41617.8	1.505	0.0139	0.0	0.1151;
22	100.0	0.0	4.621	52.56	40791.0	1.535	0.0139	0.0	0.1174;
23	100.0	0.0	4.714	52.18	39980.9	1.566	0.0139	0.0	0.1197;
24	100.0	0.0	4.809	51.81	39187.0	1.598	0.014	0.0	0.1221;
25	100.0	0.0	4.905	51.45	38409.1	1.631	0.014	0.0	0.1246;
26	100.0	0.0	5.004	51.10	37646.8	1.664	0.014	0.0	0.1271;
27	100.0	0.0	5.105	50.75	36899.8	1.697	0.014	0.0	0.1297;
28	100.0	0.0	5.208	50.41	36167.8	1.732	0.014	0.0	0.1323;
29	100.0	0.0	5.312	50.07	35450.5	1.767	0.014	0.0	0.1349;
30	100.0	0.0	5.419	49.74	34747.5	1.802	0.014	0.0	0.1376;
31	100.0	0.0	5.528	49.42	34058.7	1.839	0.014	0.0	0.1404;
32	100.0	0.0	5.639	49.11	33383.6	1.876	0.014	0.0	0.1432;
33	100.0	0.0	5.753	48.80	32722.1	1.914	0.014	0.0	0.1461;
34	100.0	0.0	5.868	48.50	32073.8	1.953	0.014	0.0	0.1491;
35	100.0	0.0	5.986	48.20	31438.5	1.992	0.014	0.0	0.1521;
36	100.0	0.0	6.107	47.91	30815.9	2.032	0.014	0.0	0.1551;
37	100.0	0.0	6.230	47.62	30205.7	2.073	0.014	0.0	0.1582;
38	100.0	0.0	6.355	47.34	29607.7	2.115	0.014	0.0	0.1614;
39	100.0	0.0	6.482	47.07	29021.7	2.158	0.014	0.0	0.1647;
40	100.0	0.0	6.613	46.80	28447.4	2.202	0.014	0.0	0.1680;
41	100.0	0.0	6.746	46.54	27884.5	2.246	0.014	0.0	0.1713;
42	100.0	0.0	6.881	46.28	27332.9	2.291	0.014	0.0	0.1748;
43	100.0	0.0	7.019	46.03	26792.3	2.338	0.014	0.0	0.1783;
44	100.0	0.0	7.160	45.78	26262.4	2.385	0.014	0.0	0.1819;
45	100.0	0.0	7.304	45.53	25743.2	2.433	0.014	0.0	0.1855;
46	100.0	0.0	7.451	45.30	25234.2	2.482	0.014	0.0	0.1893;
47	100.0	0.0	7.600	45.06	24735.5	2.532	0.014	0.0	0.1931;
48	100.0	0.0	7.753	44.83	24246.6	2.583	0.014	0.0	0.1969;

49	100.0	0.0	7.909	44.61	23767.5	2.635	0.014	0.0	0.2009;	
50	100.0	0.0	8.068	44.39	23297.9	2.688	0.014	0.0	0.2049;	
51	100.0	0.0	8.229	44.17	22837.7	2.742	0.014	0.0	0.2090;	
52	100.0	0.0	8.395	43.96	22386.6	2.798	0.014	0.0	0.2132;	
53	100.0	0.0	8.563	43.75	21944.5	2.854	0.014	0.0	0.2175;	
54	100.0	0.0	8.729	43.56	21542.8	2.907	0.014	0.0	0.2217;	begin overlap;
55	100.0	0.0	8.865	43.44	21273.9	2.944	0.014	0.0	0.2252;	
56	100.0	0.0	9.009	43.24	20853.9	3.003	0.0509	0.0	0.2288;	
57	100.0	0.0	9.118	43.19	20743.5	3.019	0.0552	0.0	0.2316;	end overlap;
58	100.0	0.0	9.233	42.99	20334.1	3.080	0.0584	0.0	0.2345;	
59	100.0	0.0	9.417	42.81	19932.8	3.142	0.0607	0.0	0.2392;	
60	100.0	0.0	9.605	42.62	19539.4	3.205	0.0625	0.0	0.2440;	
61	100.0	0.0	9.797	42.44	19153.9	3.270	0.0638	0.0	0.2489;	
62	100.0	0.0	9.993	42.26	18778.5	3.335	0.0649	0.0	0.2538;	begin overlap;
63	100.0	0.0	10.19	42.10	18424.5	3.399	0.0657	0.0	0.2588;	
64	100.0	0.0	10.38	41.94	18090.0	3.462	0.0663	0.0	0.2636;	
65	100.0	0.0	10.57	41.79	17773.2	3.524	0.0668	0.0	0.2684;	
66	100.0	0.0	10.75	41.65	17472.7	3.584	0.0671	0.0	0.2731;	
67	100.0	0.0	10.93	41.51	17187.1	3.644	0.0674	0.0	0.2777;	
68	100.0	0.0	11.11	41.38	16915.2	3.703	0.0677	0.0	0.2822;	
69	100.0	0.0	11.28	41.26	16656.0	3.760	0.0679	0.0	0.2866;	
70	100.0	0.0	11.46	41.14	16408.4	3.817	0.068	0.0	0.2910;	
71	100.0	0.0	11.63	41.03	16171.7	3.873	0.0681	0.0	0.2953;	
72	100.0	0.0	11.79	40.93	15945.1	3.928	0.0682	0.0	0.2996;	
73	100.0	0.0	11.96	40.82	15727.8	3.982	0.0683	0.0	0.3037;	
74	100.0	0.0	12.12	40.73	15519.2	4.036	0.0683	0.0	0.3079;	
75	100.0	0.0	12.28	40.63	15318.8	4.088	0.0684	0.0	0.3119;	
76	100.0	0.0	12.44	40.54	15126.1	4.141	0.0684	0.0	0.3160;	
77	100.0	0.0	12.60	40.45	14940.5	4.192	0.0685	0.0	0.3199;	
78	100.0	0.0	12.75	40.37	14761.7	4.243	0.0685	0.0	0.3238;	
79	100.0	0.0	12.90	40.29	14589.2	4.293	0.0685	0.0	0.3277;	
80	100.0	0.0	13.05	40.21	14422.6	4.342	0.0685	0.0	0.3315;	
81	100.0	0.0	13.20	40.13	14261.7	4.391	0.0685	0.0	0.3353;	
82	100.0	0.0	13.35	40.06	14106.1	4.440	0.0685	0.0	0.3390;	
83	100.0	0.0	13.49	39.99	13955.5	4.488	0.0685	0.0	0.3427;	
84	100.0	0.0	13.64	39.92	13809.7	4.535	0.0685	0.0	0.3464;	
85	100.0	0.0	13.78	39.85	13668.4	4.582	0.0685	0.0	0.3500;	
86	100.0	0.0	13.92	39.79	13531.4	4.628	0.0686	0.0	0.3536;	
87	100.0	0.0	14.06	39.72	13398.5	4.674	0.0686	0.0	0.3571;	
88	100.0	0.0	14.20	39.66	13269.4	4.720	0.0686	0.0	0.3606;	
89	100.0	0.0	14.33	39.60	13144.1	4.765	0.0686	0.0	0.3641;	
90	100.0	0.0	14.47	39.55	13022.2	4.809	0.0686	0.0	0.3675;	
91	100.0	0.0	14.60	39.49	12903.7	4.854	0.0686	0.0	0.3709;	
92	100.0	0.0	14.73	39.44	12788.4	4.897	0.0686	0.0	0.3743;	
93	100.0	0.0	14.87	39.38	12676.2	4.941	0.0686	0.0	0.3776;	
94	100.0	0.0	15.00	39.33	12566.9	4.984	0.0686	0.0	0.3809;	
95	100.0	0.0	15.13	39.28	12460.3	5.026	0.0686	0.0	0.3842;	
96	100.0	0.0	15.25	39.23	12356.5	5.069	0.0686	0.0	0.3874;	
97	100.0	0.0	15.38	39.18	12255.2	5.110	0.0686	0.0	0.3907;	
98	100.0	0.0	15.51	39.14	12156.4	5.152	0.0686	0.0	0.3938;	
99	100.0	0.0	15.63	39.09	12060.0	5.193	0.0686	0.0	0.3970;	
100	100.0	0.0	15.75	39.05	11965.9	5.234	0.0686	0.0	0.4002;	
101	100.0	0.0	15.88	39.00	11873.9	5.275	0.0686	0.0	0.4033;	
102	100.0	0.0	16.00	38.96	11784.0	5.315	0.0686	0.0	0.4064;	
103	100.0	0.0	16.12	38.92	11696.0	5.355	0.0686	0.0	0.4094;	
104	100.0	0.0	16.24	38.88	11610.1	5.394	0.0686	0.0	0.4125;	

105	100.0	0.0	16.36	38.84	11525.2	5.434	0.0686	0.0	0.4155;
106	100.0	0.0	16.48	38.80	11443.4	5.473	0.0686	0.0	0.4185;
107	100.0	0.0	16.60	38.76	11362.1	5.512	0.0686	0.0	0.4215;
108	100.0	0.0	16.72	38.72	11277.9	5.553	0.0686	0.0	0.4246;
109	100.0	0.0	16.84	38.68	11194.4	5.595	0.0686	0.0	0.4278;
110	100.0	0.0	16.96	38.65	11119.8	5.632	0.0686	0.0	0.4308;
111	100.0	0.0	17.07	38.61	11048.7	5.669	0.0686	0.0	0.4336;
112	100.0	0.0	17.19	38.58	10972.9	5.708	0.0686	0.0	0.4365;
113	100.0	0.0	17.29	38.55	10910.6	5.740	0.0686	0.0	0.4393;
114	100.0	0.0	17.38	38.53	10867.6	5.763	0.0686	0.0	0.4414;
115	100.0	0.0	17.43	38.52	10846.7	5.774	0.0686	0.0	0.4427;
116	100.0	0.0	17.62	38.42	10633.2	5.890	0.138	0.0	0.4475;
117	100.0	0.0	17.60	38.41	10628.2	5.893	0.141	0.0	0.4470;
118	100.0	0.0	17.65	38.38	10558.5	5.932	0.143	0.0	0.4484;
119	100.0	0.0	17.76	38.35	10489.8	5.971	0.144	0.0	0.4511;
120	100.0	0.0	17.87	38.32	10422.2	6.009	0.145	0.0	0.4539;
121	100.0	0.0	17.98	38.28	10355.7	6.048	0.146	0.0	0.4566;
122	100.0	0.0	18.09	38.25	10290.4	6.086	0.147	0.0	0.4594;
123	100.0	0.0	18.20	38.22	10226.2	6.124	0.148	0.0	0.4622;
124	100.0	0.0	18.31	38.19	10163.0	6.163	0.148	0.0	0.4650;
125	100.0	0.0	18.42	38.16	10101.0	6.200	0.149	0.0	0.4678;
126	100.0	0.0	18.53	38.13	10040.1	6.238	0.149	0.0	0.4706;
127	100.0	0.0	18.64	38.11	9980.1	6.275	0.150	0.0	0.4734;
128	100.0	0.0	18.75	38.08	9921.3	6.313	0.150	0.0	0.4761;
129	100.0	0.0	18.85	38.05	9863.4	6.350	0.150	0.0	0.4789;
130	100.0	0.0	18.96	38.02	9806.5	6.387	0.150	0.0	0.4817;
131	100.0	0.0	19.07	38.00	9750.5	6.423	0.151	0.0	0.4844;
132	100.0	0.0	19.18	37.97	9695.5	6.460	0.151	0.0	0.4872;
133	100.0	0.0	19.29	37.95	9641.4	6.496	0.151	0.0	0.4899;
134	100.0	0.0	19.39	37.92	9588.1	6.532	0.151	0.0	0.4926;
135	100.0	0.0	19.50	37.90	9535.8	6.568	0.151	0.0	0.4953;
136	100.0	0.0	19.61	37.87	9484.3	6.604	0.151	0.0	0.4980;
137	100.0	0.0	19.71	37.85	9433.6	6.639	0.151	0.0	0.5007;
138	100.0	0.0	19.82	37.82	9383.7	6.674	0.151	0.0	0.5033;
139	100.0	0.0	19.92	37.80	9334.6	6.709	0.151	0.0	0.5060;
140	100.0	0.0	20.02	37.78	9286.2	6.744	0.151	0.0	0.5086;
141	100.0	0.0	20.13	37.75	9238.6	6.779	0.151	0.0	0.5112;
142	100.0	0.0	20.23	37.73	9191.7	6.814	0.151	0.0	0.5138;
143	100.0	0.0	20.33	37.71	9145.5	6.848	0.151	0.0	0.5164;
144	100.0	0.0	20.43	37.69	9100.1	6.882	0.151	0.0	0.5190;
145	100.0	0.0	20.54	37.67	9055.2	6.916	0.151	0.0	0.5216;
146	100.0	0.0	20.64	37.65	9011.1	6.950	0.151	0.0	0.5242;
147	100.0	0.0	20.74	37.63	8967.6	6.984	0.151	0.0	0.5267;
148	100.0	0.0	20.84	37.61	8924.7	7.018	0.151	0.0	0.5293;
149	100.0	0.0	20.94	37.59	8882.4	7.051	0.151	0.0	0.5318;
150	100.0	0.0	21.04	37.57	8840.7	7.084	0.151	0.0	0.5343;
151	100.0	0.0	21.13	37.55	8799.6	7.117	0.151	0.0	0.5368;
152	100.0	0.0	21.23	37.53	8759.1	7.150	0.151	0.0	0.5393;
153	100.0	0.0	21.33	37.51	8719.1	7.183	0.151	0.0	0.5418;
154	100.0	0.0	21.43	37.49	8679.7	7.216	0.151	0.0	0.5442;
155	100.0	0.0	21.52	37.47	8640.8	7.248	0.151	0.0	0.5467;
156	100.0	0.0	21.62	37.45	8602.4	7.280	0.151	0.0	0.5491;
157	100.0	0.0	21.72	37.44	8564.6	7.313	0.151	0.0	0.5516;
158	100.0	0.0	21.81	37.42	8527.2	7.345	0.151	0.0	0.5540;
159	100.0	0.0	21.91	37.40	8490.3	7.377	0.151	0.0	0.5564;
160	100.0	0.0	22.00	37.38	8453.9	7.408	0.151	0.0	0.5588;

161	100.0	0.0	22.09	37.37	8418.0	7.440	0.151	0.0	0.5612;
162	100.0	0.0	22.19	37.35	8382.5	7.472	0.151	0.0	0.5636;
163	100.0	0.0	22.28	37.33	8347.5	7.503	0.151	0.0	0.5660;
164	100.0	0.0	22.37	37.32	8312.9	7.534	0.151	0.0	0.5683;
165	100.0	0.0	22.47	37.30	8278.7	7.565	0.151	0.0	0.5707;
166	100.0	0.0	22.56	37.28	8245.0	7.596	0.151	0.0	0.5730;
167	100.0	0.0	22.65	37.27	8211.6	7.627	0.151	0.0	0.5754;
168	100.0	0.0	22.74	37.25	8178.7	7.658	0.151	0.0	0.5777;
169	100.0	0.0	22.83	37.24	8146.1	7.688	0.151	0.0	0.5800;
170	100.0	0.0	22.92	37.22	8114.0	7.719	0.151	0.0	0.5823;
171	100.0	0.0	23.02	37.21	8082.2	7.749	0.151	0.0	0.5846;
172	100.0	0.0	23.11	37.19	8050.8	7.779	0.151	0.0	0.5869;
173	100.0	0.0	23.19	37.18	8019.7	7.809	0.151	0.0	0.5892;
174	100.0	0.0	23.28	37.16	7989.1	7.839	0.151	0.0	0.5914;
175	100.0	0.0	23.37	37.15	7958.7	7.869	0.151	0.0	0.5937;
176	100.0	0.0	23.46	37.13	7928.7	7.899	0.151	0.0	0.5959;
177	100.0	0.0	23.55	37.12	7899.1	7.929	0.151	0.0	0.5982;
178	100.0	0.0	23.64	37.11	7869.8	7.958	0.151	0.0	0.6004;
179	100.0	0.0	23.73	37.09	7840.7	7.988	0.151	0.0	0.6026;
180	100.0	0.0	23.81	37.08	7811.9	8.017	0.151	0.0	0.6049;
181	100.0	0.0	23.90	37.07	7783.5	8.047	0.151	0.0	0.6071;
182	100.0	0.0	23.99	37.05	7755.6	8.075	0.151	0.0	0.6093;
183	100.0	0.0	24.07	37.04	7727.6	8.105	0.151	0.0	0.6115;
184	100.0	0.0	24.16	37.03	7699.9	8.134	0.151	0.0	0.6137;
185	100.0	0.0	24.24	37.01	7674.2	8.161	0.151	0.0	0.6158;
186	100.0	0.0	24.33	37.00	7644.5	8.193	0.151	0.0	0.6180;
187	100.0	0.0	24.42	36.99	7617.1	8.222	0.151	0.0	0.6203;
188	100.0	0.0	24.51	36.97	7591.5	8.250	0.151	0.0	0.6225;
189	100.0	0.0	24.59	36.96	7568.6	8.275	0.151	0.0	0.6245;
190	100.0	0.0	24.66	36.95	7547.0	8.299	0.151	0.0	0.6263;
191	100.0	0.0	24.72	36.94	7528.2	8.319	0.151	0.0	0.6280;
192	100.0	0.0	24.77	36.94	7516.7	8.332	0.151	0.0	0.6293;
193	100.1	0.0	25.04	36.87	7369.0	8.499	0.243	0.0	0.6360;
194	100.1	0.0	24.05	36.87	7367.6	8.501	0.244	0.0	0.6108;
195	100.1	0.0	24.08	36.85	7333.7	8.540	0.246	0.0	0.6115;
196	100.1	0.0	24.16	36.84	7299.9	8.580	0.248	0.0	0.6136;
197	100.1	0.0	24.24	36.82	7266.2	8.619	0.250	0.0	0.6156;
198	100.1	0.0	24.32	36.80	7232.7	8.659	0.252	0.0	0.6177;
199	100.1	0.0	24.40	36.79	7199.4	8.699	0.254	0.0	0.6197;
200	100.1	0.0	24.48	36.77	7166.2	8.740	0.256	0.0	0.6217;
201	100.1	0.0	24.55	36.76	7134.2	8.779	0.258	0.0	0.6236;
202	100.1	0.0	24.62	36.74	7102.5	8.818	0.260	0.0	0.6255;
203	100.1	0.0	24.70	36.73	7071.0	8.857	0.262	0.0	0.6273;
204	100.1	0.0	24.77	36.71	7039.8	8.897	0.264	0.0	0.6291;
205	100.1	0.0	24.84	36.70	7008.9	8.936	0.266	0.0	0.6309;
206	100.1	0.0	24.91	36.68	6978.2	8.975	0.268	0.0	0.6327;
207	100.1	0.0	24.98	36.67	6947.8	9.014	0.270	0.0	0.6345;
208	100.1	0.0	25.05	36.65	6917.6	9.054	0.272	0.0	0.6363;
209	100.1	0.0	25.12	36.64	6887.7	9.093	0.274	0.0	0.6380;
210	100.1	0.0	25.19	36.63	6858.0	9.132	0.276	0.0	0.6397;
211	100.1	0.0	25.25	36.61	6828.5	9.172	0.278	0.0	0.6414;
212	100.1	0.0	25.32	36.60	6799.2	9.211	0.280	0.0	0.6431;
213	100.1	0.0	25.39	36.58	6770.2	9.251	0.282	0.0	0.6448;
214	100.1	0.0	25.45	36.57	6741.3	9.290	0.284	0.0	0.6465;
215	100.1	0.0	25.52	36.56	6712.7	9.330	0.287	0.0	0.6481;
216	100.1	0.0	25.58	36.54	6684.3	9.370	0.289	0.0	0.6498;

217	100.1	0.0	25.64	36.53	6656.1	9.409	0.291	0.0	0.6514;
218	100.1	0.0	25.71	36.52	6628.1	9.449	0.293	0.0	0.6530;
219	100.1	0.0	25.77	36.50	6600.2	9.489	0.295	0.0	0.6546;
220	100.1	0.0	25.83	36.49	6572.6	9.529	0.297	0.0	0.6561;
221	100.1	0.0	25.89	36.48	6545.1	9.569	0.299	0.0	0.6577;
222	100.1	0.0	25.95	36.46	6517.8	9.609	0.301	0.0	0.6592;
223	100.1	0.0	26.01	36.45	6490.7	9.649	0.303	0.0	0.6608;
224	100.1	0.0	26.07	36.44	6463.8	9.689	0.305	0.0	0.6623;
225	100.1	0.0	26.13	36.43	6437.0	9.730	0.308	0.0	0.6638;
226	100.1	0.0	26.19	36.41	6410.4	9.770	0.310	0.0	0.6652;
227	100.1	0.0	26.25	36.40	6383.9	9.811	0.312	0.0	0.6667;
228	100.1	0.0	26.31	36.39	6357.6	9.851	0.314	0.0	0.6682;
229	100.1	0.0	26.36	36.38	6331.5	9.892	0.316	0.0	0.6696;
230	100.1	0.0	26.42	36.36	6305.5	9.933	0.318	0.0	0.6710;
231	100.1	0.0	26.47	36.35	6279.7	9.973	0.321	0.0	0.6724;
232	100.1	0.0	26.53	36.34	6254.0	10.01	0.323	0.0	0.6738;
233	100.1	0.0	26.58	36.33	6228.4	10.06	0.325	0.0	0.6752;
234	100.1	0.0	26.63	36.31	6203.0	10.10	0.327	0.0	0.6765;
235	100.1	0.0	26.69	36.30	6177.7	10.14	0.329	0.0	0.6779;
236	100.1	0.0	26.74	36.29	6152.5	10.18	0.332	0.0	0.6792;
237	100.1	0.0	26.79	36.28	6127.5	10.22	0.334	0.0	0.6805;
238	100.1	0.0	26.84	36.27	6102.6	10.26	0.336	0.0	0.6818;
239	100.1	0.0	26.89	36.26	6077.8	10.30	0.338	0.0	0.6831;
240	100.1	0.0	26.94	36.24	6053.1	10.35	0.341	0.0	0.6844;
241	100.1	0.0	26.99	36.23	6028.5	10.39	0.343	0.0	0.6856;
242	100.1	0.0	27.04	36.22	6004.1	10.43	0.345	0.0	0.6869;
243	100.1	0.0	27.09	36.21	5979.8	10.47	0.348	0.0	0.6881;
244	100.2	0.0	27.14	36.20	5955.5	10.52	0.350	0.0	0.6893;
245	100.2	0.0	27.19	36.19	5931.4	10.56	0.352	0.0	0.6905;
246	100.2	0.0	27.23	36.17	5907.4	10.60	0.355	0.0	0.6917;
247	100.2	0.0	27.28	36.16	5883.5	10.65	0.357	0.0	0.6929;
248	100.2	0.0	27.32	36.15	5859.6	10.69	0.359	0.0	0.6940;
249	100.2	0.0	27.37	36.14	5835.9	10.73	0.362	0.0	0.6952;
250	100.2	0.0	27.41	36.13	5812.2	10.78	0.364	0.0	0.6963;
251	100.2	0.0	27.46	36.12	5788.7	10.82	0.367	0.0	0.6974;
252	100.2	0.0	27.50	36.11	5765.2	10.86	0.369	0.0	0.6985;
253	100.2	0.0	27.54	36.10	5741.8	10.91	0.371	0.0	0.6996;
254	100.2	0.0	27.59	36.08	5718.5	10.95	0.374	0.0	0.7007;
255	100.2	0.0	27.63	36.07	5695.2	11.00	0.376	0.0	0.7017;
256	100.2	0.0	27.67	36.06	5672.1	11.04	0.379	0.0	0.7028;
257	100.2	0.0	27.71	36.05	5649.0	11.09	0.381	0.0	0.7038;
258	100.2	0.0	27.75	36.04	5625.9	11.13	0.384	0.0	0.7048;
259	100.2	0.0	27.79	36.03	5603.0	11.18	0.386	0.0	0.7058;
260	100.2	0.0	27.83	36.02	5580.1	11.22	0.389	0.0	0.7068;
261	100.2	0.0	27.87	36.01	5557.2	11.27	0.391	0.0	0.7078;
262	100.2	0.0	27.90	36.00	5534.4	11.32	0.394	0.0	0.7088;
263	100.2	0.0	27.94	35.99	5511.7	11.36	0.397	0.0	0.7097;
264	100.2	0.0	27.98	35.98	5489.0	11.41	0.399	0.0	0.7107;
265	100.2	0.0	28.02	35.97	5466.4	11.46	0.402	0.0	0.7116;
266	100.2	0.0	28.05	35.95	5443.8	11.50	0.405	0.0	0.7125;
267	100.2	0.0	28.09	35.94	5421.3	11.55	0.407	0.0	0.7134;
268	100.2	0.0	28.12	35.93	5398.8	11.60	0.410	0.0	0.7143;
269	100.2	0.0	28.16	35.92	5376.3	11.65	0.413	0.0	0.7152;
270	100.2	0.0	28.19	35.91	5353.9	11.70	0.415	0.0	0.7160;
271	100.2	0.0	28.22	35.90	5331.5	11.75	0.418	0.0	0.7169;
272	100.2	0.0	28.26	35.89	5309.1	11.80	0.421	0.0	0.7177;

273	100.2	0.0	28.29	35.88	5286.8	11.85	0.424	0.0	0.7186;
274	100.2	0.0	28.32	35.87	5264.5	11.90	0.427	0.0	0.7194;
275	100.2	0.0	28.35	35.86	5242.2	11.95	0.429	0.0	0.7202;
276	100.2	0.0	28.39	35.85	5219.9	12.00	0.432	0.0	0.7210;
277	100.2	0.0	28.42	35.84	5197.6	12.05	0.435	0.0	0.7218;
278	100.3	0.0	28.45	35.83	5175.4	12.10	0.438	0.0	0.7225;
279	100.3	0.0	28.48	35.82	5153.1	12.15	0.441	0.0	0.7233;
280	100.3	0.0	28.51	35.81	5130.9	12.21	0.444	0.0	0.7241;
281	100.3	0.0	28.54	35.80	5108.6	12.26	0.447	0.0	0.7248;
282	100.3	0.0	28.56	35.78	5086.4	12.31	0.450	0.0	0.7255;
283	100.3	0.0	28.59	35.77	5064.1	12.37	0.453	0.0	0.7263;
284	100.3	0.0	28.62	35.76	5041.9	12.42	0.456	0.0	0.7270;
285	100.3	0.0	28.65	35.75	5019.6	12.48	0.459	0.0	0.7277;
286	100.3	0.0	28.68	35.74	4997.3	12.53	0.462	0.0	0.7284;
287	100.3	0.0	28.70	35.73	4975.0	12.59	0.466	0.0	0.7290;
288	100.3	0.0	28.73	35.72	4952.7	12.65	0.469	0.0	0.7297;
289	100.3	0.0	28.76	35.71	4930.3	12.70	0.472	0.0	0.7304;
290	100.3	0.0	28.78	35.70	4907.9	12.76	0.475	0.0	0.7311;
291	100.3	0.0	28.81	35.69	4885.5	12.82	0.479	0.0	0.7317;
292	100.3	0.0	28.83	35.68	4863.0	12.88	0.482	0.0	0.7324;
293	100.3	0.0	28.86	35.67	4840.4	12.94	0.485	0.0	0.7330;
294	100.3	0.0	28.88	35.66	4817.9	13.00	0.489	0.0	0.7336;
295	100.3	0.0	28.91	35.65	4795.2	13.06	0.492	0.0	0.7343;
296	100.3	0.0	28.93	35.64	4772.5	13.12	0.496	0.0	0.7349;
297	100.3	0.0	28.96	35.62	4749.7	13.19	0.499	0.0	0.7355;
298	100.3	0.0	28.98	35.61	4726.9	13.25	0.503	0.0	0.7362;
299	100.4	0.0	29.01	35.60	4704.0	13.31	0.506	0.0	0.7368;
300	100.4	0.0	29.03	35.59	4681.0	13.38	0.510	0.0	0.7374;
301	100.4	0.0	29.06	35.58	4657.9	13.45	0.513	0.0	0.7380;
302	100.4	0.0	29.08	35.57	4634.7	13.51	0.517	0.0	0.7386;
303	100.4	0.0	29.10	35.56	4611.4	13.58	0.521	0.0	0.7392;
304	100.4	0.0	29.13	35.55	4588.0	13.65	0.525	0.0	0.7399;
305	100.4	0.0	29.15	35.54	4564.4	13.72	0.529	0.0	0.7405;
306	100.4	0.0	29.18	35.53	4540.8	13.79	0.532	0.0	0.7411;
307	100.4	0.0	29.20	35.51	4517.0	13.87	0.536	0.0	0.7418;
308	100.4	0.0	29.23	35.50	4493.0	13.94	0.540	0.0	0.7424;
309	100.4	0.0	29.25	35.49	4469.0	14.01	0.544	0.0	0.7431;
310	100.4	0.0	29.28	35.48	4444.7	14.09	0.548	0.0	0.7437;
311	100.4	0.0	29.31	35.47	4420.3	14.17	0.553	0.0	0.7444;
312	100.4	0.0	29.33	35.46	4395.7	14.25	0.557	0.0	0.7451;
313	100.5	0.0	29.36	35.44	4370.9	14.33	0.561	0.0	0.7458;
314	100.5	0.0	29.39	35.43	4345.9	14.41	0.565	0.0	0.7465;
315	100.5	0.0	29.42	35.42	4320.7	14.50	0.570	0.0	0.7472;
316	100.5	0.0	29.45	35.41	4295.3	14.58	0.574	0.0	0.7480;
317	100.5	0.0	29.48	35.40	4269.6	14.67	0.579	0.0	0.7487;
318	100.5	0.0	29.51	35.38	4243.7	14.76	0.583	0.0	0.7496;
319	100.5	0.0	29.54	35.37	4217.5	14.85	0.588	0.0	0.7504;
320	100.5	0.0	29.58	35.36	4191.1	14.94	0.592	0.0	0.7512;
321	100.5	0.0	29.61	35.35	4164.3	15.04	0.597	0.0	0.7521;
322	100.5	0.0	29.65	35.33	4137.3	15.14	0.602	0.0	0.7531;
323	100.5	0.0	29.69	35.32	4109.9	15.24	0.607	0.0	0.7541;
324	100.6	0.0	29.73	35.31	4082.2	15.34	0.612	0.0	0.7551;
325	100.6	0.0	29.77	35.29	4054.1	15.45	0.617	0.0	0.7561;
326	100.6	0.0	29.81	35.28	4026.2	15.56	0.622	0.0	0.7572;
327	100.6	0.0	29.85	35.27	3998.5	15.66	0.627	0.0	0.7582; end overlap;
328	100.6	0.0	29.89	35.25	3971.2	15.77	0.632	0.0	0.7592;

329	100.6	0.0	29.92	35.24	3944.1	15.88	0.638	0.0	0.7601;
330	100.6	0.0	29.96	35.23	3917.2	15.99	0.643	0.0	0.7609;
331	100.6	0.0	29.98	35.22	3890.5	16.10	0.649	0.0	0.7616;
332	100.7	0.0	30.01	35.20	3864.0	16.21	0.654	0.0	0.7623;
333	100.7	0.0	30.04	35.19	3837.7	16.32	0.660	0.0	0.7629;
334	100.7	0.0	30.06	35.18	3811.5	16.43	0.666	0.0	0.7635;
335	100.7	0.0	30.08	35.17	3785.5	16.54	0.671	0.0	0.7640;
336	100.7	0.0	30.09	35.15	3759.6	16.66	0.677	0.0	0.7644;
337	100.7	0.0	30.11	35.14	3733.7	16.77	0.684	0.0	0.7648;
338	100.8	0.0	30.12	35.13	3707.9	16.89	0.690	0.0	0.7652;
339	100.8	0.0	30.14	35.12	3682.1	17.01	0.696	0.0	0.7655;
340	100.8	0.0	30.15	35.10	3656.4	17.13	0.703	0.0	0.7657;
341	100.8	0.0	30.16	35.09	3630.6	17.25	0.709	0.0	0.7660;
342	100.8	0.0	30.16	35.08	3604.8	17.37	0.716	0.0	0.7662;
343	100.8	0.0	30.17	35.07	3578.9	17.50	0.723	0.0	0.7663;
344	100.9	0.0	30.18	35.06	3552.9	17.63	0.730	0.0	0.7665;
345	100.9	0.0	30.18	35.04	3526.8	17.76	0.737	0.0	0.7666;
346	100.9	0.0	30.19	35.03	3500.5	17.89	0.744	0.0	0.7667;
347	100.9	0.0	30.19	35.02	3474.1	18.03	0.752	0.0	0.7668;
348	101.0	0.0	30.19	35.01	3447.4	18.17	0.760	0.0	0.7669;
349	101.0	0.0	30.20	34.99	3420.5	18.31	0.767	0.0	0.7670;
350	101.0	0.0	30.20	34.98	3393.3	18.46	0.775	0.0	0.7671;
351	101.0	0.0	30.21	34.97	3365.8	18.61	0.784	0.0	0.7672;
352	101.1	0.0	30.21	34.95	3337.9	18.76	0.792	0.0	0.7674;
353	101.1	0.0	30.22	34.94	3309.7	18.92	0.801	0.0	0.7675;
354	101.1	0.0	30.23	34.93	3281.0	19.09	0.809	0.0	0.7678;
355	101.2	0.0	30.24	34.91	3251.9	19.26	0.818	0.0	0.7680;
356	101.2	0.0	30.25	34.90	3222.3	19.44	0.828	0.0	0.7683;
357	101.2	0.0	30.27	34.88	3192.1	19.62	0.837	0.0	0.7687;
358	101.3	0.0	30.28	34.87	3161.4	19.81	0.847	0.0	0.7692;
359	101.3	0.0	30.31	34.85	3130.0	20.01	0.857	0.0	0.7698;
360	101.3	0.0	30.33	34.84	3098.0	20.22	0.867	0.0	0.7704;
361	101.4	0.0	30.36	34.82	3065.2	20.43	0.878	0.0	0.7712;
362	101.4	0.0	30.40	34.81	3031.7	20.66	0.889	0.0	0.7721;
363	101.5	0.0	30.44	34.79	2997.4	20.89	0.900	0.0	0.7731;
364	101.5	0.0	30.49	34.77	2962.2	21.14	0.911	0.0	0.7743;
365	101.6	0.0	30.54	34.76	2926.1	21.40	0.923	0.0	0.7757;
366	101.6	0.0	30.60	34.74	2889.0	21.68	0.935	0.0	0.7773;
367	101.7	0.0	30.67	34.72	2850.9	21.97	0.947	0.0	0.7791;
368	101.8	0.0	30.75	34.70	2811.8	22.27	0.960	0.0	0.7811;
369	101.8	0.0	30.84	34.68	2771.6	22.60	0.974	0.0	0.7835;
370	101.9	0.0	30.95	34.66	2730.2	22.94	0.987	0.0	0.7861;
371	102.0	0.0	31.06	34.64	2687.5	23.30	1.001	0.0	0.7890;
372	102.0	0.0	31.19	34.62	2643.6	23.69	1.016	0.0	0.7922;
373	102.1	0.0	31.33	34.60	2598.4	24.10	1.031	0.0	0.7959;
374	102.2	0.0	31.49	34.58	2551.9	24.54	1.046	0.0	0.7999;
375	102.3	0.0	31.67	34.56	2503.9	25.01	1.062	0.0	0.8045;
376	102.4	0.0	31.87	34.53	2454.8	25.51	1.079	0.0	0.8094;
377	102.5	0.0	32.07	34.51	2406.6	26.02	1.095	0.0	0.8146;
378	102.6	0.0	32.28	34.49	2359.4	26.54	1.111	0.0	0.8200;
379	102.7	0.0	32.51	34.47	2313.1	27.08	1.126	0.0	0.8257;
380	102.8	0.0	32.74	34.44	2267.7	27.62	1.141	0.0	0.8317;
381	102.9	0.0	32.99	34.42	2223.2	28.17	1.156	0.0	0.8379;
382	103.0	0.0	33.24	34.40	2179.6	28.73	1.171	0.0	0.8444;
383	103.1	0.0	33.51	34.38	2136.8	29.31	1.186	0.0	0.8511;
384	103.2	0.0	33.78	34.36	2094.9	29.90	1.200	0.0	0.8580;

385	103.3	0.0	34.06	34.34	2053.8	30.49	1.215	0.0	0.8652;
386	103.4	0.0	34.36	34.32	2013.5	31.11	1.229	0.0	0.8726;
387	103.5	0.0	34.65	34.30	1974.0	31.73	1.243	0.0	0.8802;
388	103.6	0.0	34.96	34.29	1935.3	32.36	1.256	0.0	0.8881;
389	103.7	0.0	35.28	34.27	1897.3	33.01	1.270	0.0	0.8961;
390	103.8	0.0	35.60	34.25	1860.1	33.67	1.283	0.0	0.9043;
391	103.9	0.0	35.93	34.23	1823.6	34.34	1.297	0.0	0.9127;
392	104.0	0.0	36.27	34.22	1787.8	35.03	1.310	0.0	0.9213;
393	104.1	0.0	36.62	34.20	1752.7	35.73	1.323	0.0	0.9301;
394	104.3	0.0	36.97	34.18	1718.3	36.45	1.335	0.0	0.9391;
395	104.4	0.0	37.33	34.17	1684.6	37.18	1.348	0.0	0.9483; bottom hit;

Horiz plane projections in effluent direction: radius(m): 0.0; CL(m): 0.4109
Lmz(m): 0.4109
forced entrain 1 0.0 -1.333 0.948 0.113
Rate sec-1 0.0 dy-1 0.0 kt: 0.0 Amb Sal 33.3632
;
2:50:38 PM. amb fills: 4

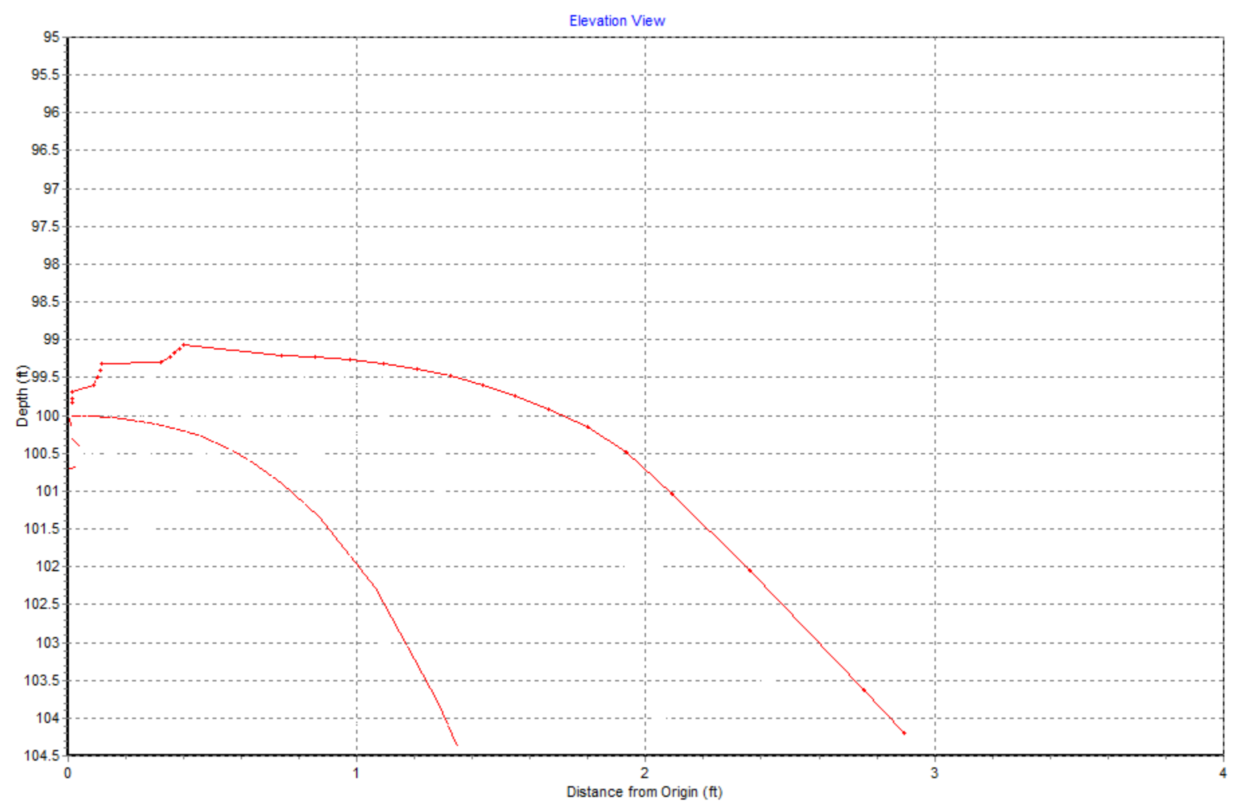

Figure C.7.1: Plumes 18b solution of discharge plume trajectories for discharges of 0.35 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt., and 5 mgd of brine from the Doheny Desalination Project with a brine salinity of 67 ppt. Depth of maximum rise of the plume is Z = 99.1 ft. at Xa = 0.462 ft from the point of discharge. The centerline of the plume is at an average distance of Xb = 1.348 ft from the point of discharge as the plume begins to impact the bottom at a depth of Z = 104.5 ft.

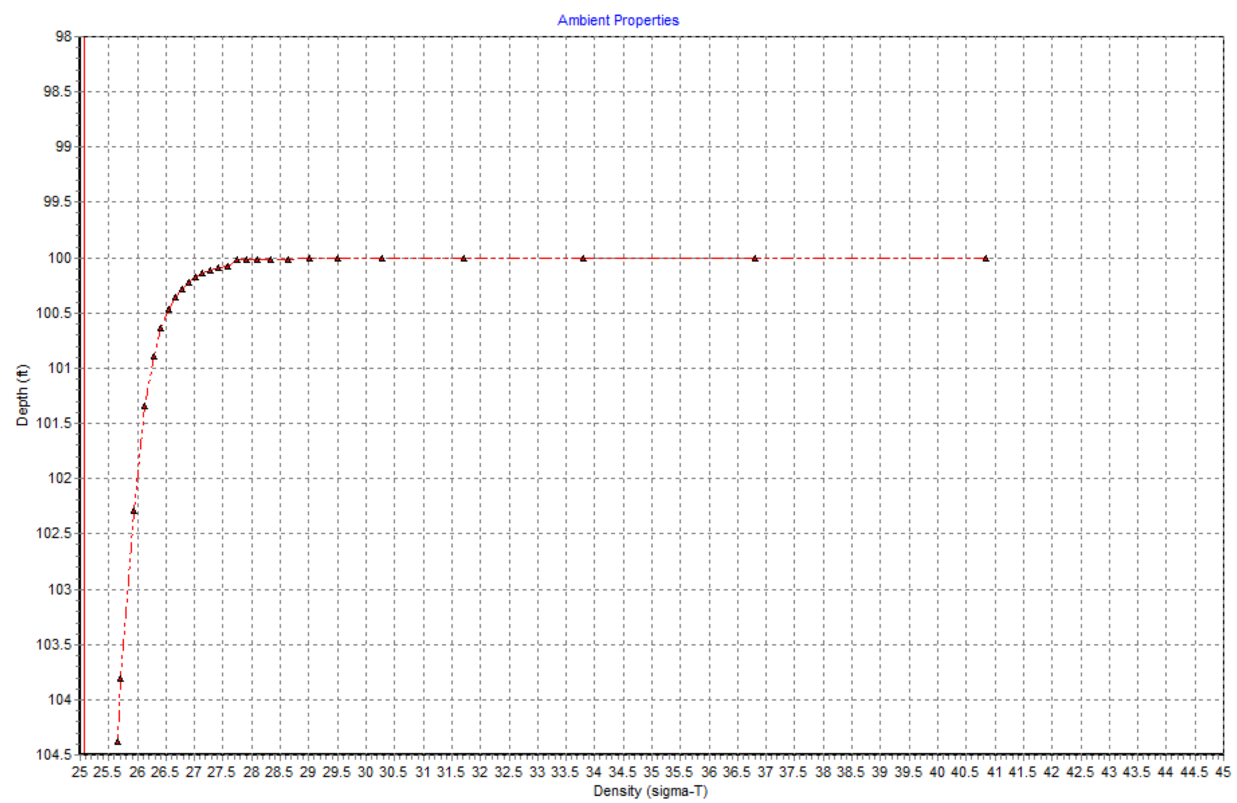

Figure C.7.2: Plumes 18b solution of vertical density profile for discharges of 0.35 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt. and 5 mgd of brine from the Doheny Desalination Project with a brine salinity of 67 ppt. Discharge effluent density shown as black triangles. Ambient water mass density profile shown as solid red line.

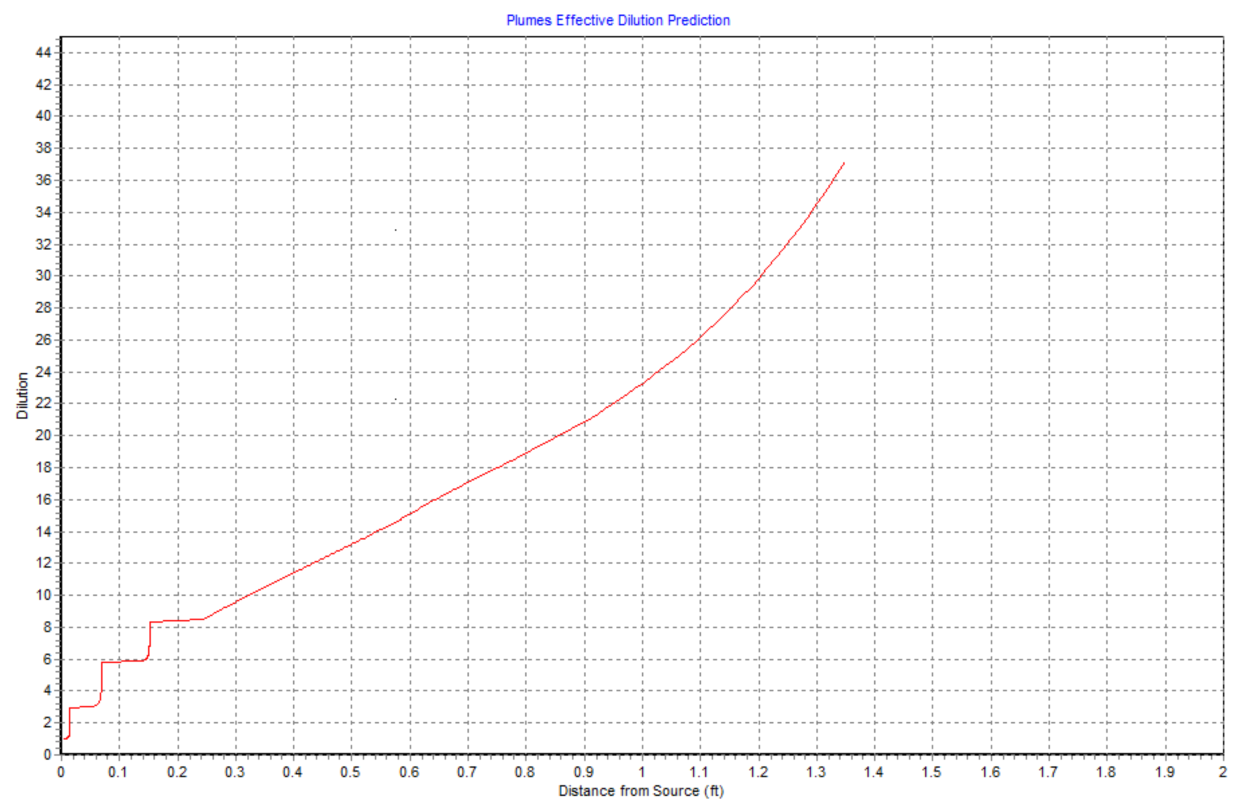

Figure C.7.3: Plumes 18b solution of effective (bulk average) dilution as a function of vertical distance from the point of discharge for discharges of 0.35 mgd of SOCWA wastewater at average annual TDS = 1.25 ppt. and 5 mgd of brine from the Doheny Desalination Project with a brine salinity of 67 ppt. Effective dilution is S_a = 12.53 at the maximum rise of the plume at Xa = 0.462 ft. from the point of discharge. As the plume begins to impact the bottom, the plume centerline is at a distance of Xb = 1.348 ft from the point of discharge, where the effective dilution reaches S_a = 37.18.

C.8: Plumes 18b Results for SJCOO discharges of 1.8 mgd Well Water and 3 mgd Brine:

ontents of the memo box (may not be current and must be updated manually)
Project "C:\Plumes18\SJCOO_WellW1.8mgd_b3mgd_D-1"
memo
SJCOO discharging 1.8 mgd well water and 3 mgd brine

Model configuration items checked:
Channel width (m) 100
Start case for graphs 1
Max detailed graphs 10 (limits plots that can overflow memory)
Elevation Projection Plane (deg) 0
Shore vector (m,deg) not checked
Bacteria model : Mancini (1978) coliform model
PDS sfc. model heat transfer : Medium
Equation of State : S, T
Similarity Profile : Default profile (k=2.0, ...)
Diffuser port contraction coefficient 1
Light absorption coefficient 0.16
Farfield increment (m) 200
UM3 aspiration coefficient 0.1
Output file: text output tab
Output each ?? steps 1
Maximum dilution reported 1000
Text output format : Standard
Max vertical reversals : to max rise or fall

/ UM3. 1/14/2019 3:07:41 PM
Case 1; ambient file C:\Plumes18\SJCOO_WellW1.8mgd_b3mgd_D-1.001.db; Diffuser table record 1: ----
----

Ambient Table:

Depth	Amb-cur	Amb-dir	Amb-sal	Amb-tem	Amb-pol	Decay	Far-spd	Far-dir	Disprsn	Density
m	m/s	deg	psu	C	kg/kg	s-1	m/s	deg	m0.67/s2	sigma-T
0.0	0.0	0.0	33.38	19.97	0.0	0.0	0.0	0.0	0.0003	23.55719
2.000	0.0	0.0	33.39	19.87	0.0	0.0	0.0	0.0	0.0003	23.59288
4.000	0.0	0.0	33.40	19.81	0.0	0.0	0.0	0.0	0.0003	23.61484
7.000	0.0	0.0	33.41	19.64	0.0	0.0	0.0	0.0	0.0003	23.67096
10.00	0.0	0.0	33.40	17.76	0.0	0.0	0.0	0.0	0.0003	24.12888
12.00	0.0	0.0	33.38	17.34	0.0	0.0	0.0	0.0	0.0003	24.21549
14.00	0.0	0.0	33.38	16.73	0.0	0.0	0.0	0.0	0.0003	24.35932
16.00	0.0	0.0	33.38	15.17	0.0	0.0	0.0	0.0	0.0003	24.71340
18.00	0.0	0.0	33.37	14.37	0.0	0.0	0.0	0.0	0.0003	24.87044
20.00	0.0	0.0	33.35	14.18	0.0	0.0	0.0	0.0	0.0003	24.89660
22.00	0.0	0.0	33.33	13.83	0.0	0.0	0.0	0.0	0.0003	24.95172
24.00	0.0	0.0	33.32	13.75	0.0	0.0	0.0	0.0	0.0003	24.96707
26.00	0.0	0.0	33.36	13.42	0.0	0.0	0.0	0.0	0.0003	25.06459
31.85	0.0	0.0	33.36	13.39	0.0	0.0	0.0	0.0	0.0003	25.07201

Diffuser table:

P-dia	Ver angl	H-Angle	SourceX	SourceY	Ports	MZ-dis	Isoplth	P-depth	Ttl-flo	Eff-sal	Temp	Polutnt
(in)	(deg)	(deg)	(ft)	(ft)	()	(ft)	(concent)	(ft)	(MGD)	(psu)	(C)	(ppm)

3.0500 0.0 0.0 0.0 0.0 125.00 1000.0 0.0 100.00 4.8000 54.440 20.660 54440.0

Simulation:
Froude No: -3.468; Strat No:-2.65E-4; Spcg No: 14.39; k: 35692.1; eff den (sigmaT) 39.56029; eff vel 0.357(m/s);
Current is very small, flow regime may be transient.

Step	Depth (ft)	Amb-cur (m/s)	P-dia (in)	Eff-sal (psu)	Polutnt (ppm)	Dilutn ()	x-posn (ft)	y-posn (ft)	Iso dia (m)
0	100.0	1.000E-5	3.050	54.44	54440.0	1.000	0.0	0.0	0.07747;
1	100.0	0.0	3.058	54.33	54140.8	1.006	0.00349	0.0	0.07768;
2	100.0	0.0	3.098	53.91	53065.0	1.026	0.00632	0.0	0.07868;
3	100.0	0.0	3.160	53.51	52010.8	1.047	0.00871	0.0	0.08027;
4	100.0	0.0	3.224	53.12	50977.8	1.068	0.0108	0.0	0.08188;
5	100.0	0.0	3.289	52.73	49965.6	1.090	0.0127	0.0	0.08353;
6	100.0	0.0	3.355	52.35	48973.7	1.112	0.0144	0.0	0.08521;
7	100.0	0.0	3.422	51.98	48001.7	1.134	0.0159	0.0	0.08692;
8	100.0	0.0	3.491	51.61	47049.3	1.157	0.0174	0.0	0.08867;
9	100.0	0.0	3.561	51.25	46115.9	1.181	0.0187	0.0	0.09046;
10	100.0	0.0	3.633	50.90	45201.3	1.204	0.020	0.0	0.09228;
11	100.0	0.0	3.706	50.56	44305.0	1.229	0.0212	0.0	0.09413;
12	100.0	0.0	3.781	50.22	43426.7	1.254	0.0224	0.0	0.09603;
13	100.0	0.0	3.857	49.89	42565.9	1.279	0.0235	0.0	0.09796;
14	100.0	0.0	3.934	49.57	41722.4	1.305	0.0246	0.0	0.09993;
15	100.0	0.0	4.013	49.25	40895.8	1.331	0.0256	0.0	0.1019;
16	100.0	0.0	4.094	48.94	40085.7	1.358	0.0266	0.0	0.1040;
17	100.0	0.0	4.176	48.63	39291.8	1.386	0.0276	0.0	0.1061;
18	100.0	0.0	4.260	48.33	38513.8	1.414	0.0285	0.0	0.1082;
19	100.0	0.0	4.346	48.04	37751.3	1.442	0.0294	0.0	0.1104;
20	100.0	0.0	4.433	47.75	37004.1	1.471	0.0303	0.0	0.1126;
21	100.0	0.0	4.522	47.47	36271.7	1.501	0.0312	0.0	0.1149;
22	100.0	0.0	4.613	47.19	35554.0	1.531	0.032	0.0	0.1172;
23	100.0	0.0	4.706	46.92	34850.7	1.562	0.0329	0.0	0.1195;
24	100.0	0.0	4.800	46.66	34161.3	1.594	0.0337	0.0	0.1219;
25	100.0	0.0	4.897	46.40	33485.7	1.626	0.0345	0.0	0.1244;
26	100.0	0.0	4.995	46.14	32823.6	1.659	0.0353	0.0	0.1269;
27	100.0	0.0	5.095	45.89	32174.7	1.692	0.036	0.0	0.1294;
28	100.0	0.0	5.198	45.64	31538.7	1.726	0.0368	0.0	0.1320;
29	100.0	0.0	5.302	45.40	30915.3	1.761	0.0375	0.0	0.1347;
30	100.0	0.0	5.408	45.17	30304.4	1.796	0.0383	0.0	0.1374;
31	100.0	0.0	5.517	44.94	29705.6	1.833	0.039	0.0	0.1401;
32	100.0	0.0	5.628	44.71	29118.8	1.870	0.0397	0.0	0.1429;
33	100.0	0.0	5.741	44.49	28543.6	1.907	0.0404	0.0	0.1458;
34	100.0	0.0	5.856	44.27	27979.9	1.946	0.0411	0.0	0.1487;
35	100.0	0.0	5.973	44.05	27427.4	1.985	0.0418	0.0	0.1517;
36	100.0	0.0	6.093	43.85	26885.8	2.025	0.0425	0.0	0.1548;
37	100.0	0.0	6.215	43.64	26355.1	2.066	0.0432	0.0	0.1579;
38	100.0	0.0	6.340	43.44	25834.8	2.107	0.0438	0.0	0.1610;
39	100.0	0.0	6.467	43.24	25324.9	2.150	0.0445	0.0	0.1643;
40	100.0	0.0	6.597	43.05	24825.2	2.193	0.0452	0.0	0.1676;
41	100.0	0.0	6.729	42.86	24335.3	2.237	0.0458	0.0	0.1709;
42	100.0	0.0	6.864	42.67	23855.2	2.282	0.0464	0.0	0.1744;
43	100.0	0.0	7.002	42.49	23384.6	2.328	0.0471	0.0	0.1779;
44	100.0	0.0	7.143	42.31	22923.4	2.375	0.0477	0.0	0.1814;
45	100.0	0.0	7.286	42.13	22471.2	2.423	0.0483	0.0	0.1851;
46	100.0	0.0	7.432	41.96	22028.1	2.471	0.0489	0.0	0.1888;
47	100.0	0.0	7.581	41.79	21593.7	2.521	0.0496	0.0	0.1926;

48	100.0	0.0	7.733	41.63	21168.0	2.572	0.0502	0.0	0.1964;
49	100.0	0.0	7.888	41.47	20750.7	2.624	0.0508	0.0	0.2004;
50	100.0	0.0	8.046	41.31	20341.7	2.676	0.0514	0.0	0.2044;
51	100.0	0.0	8.207	41.15	19940.7	2.730	0.052	0.0	0.2085;
52	100.0	0.0	8.372	41.00	19547.7	2.785	0.0525	0.0	0.2126;
53	100.0	0.0	8.540	40.85	19162.5	2.841	0.0531	0.0	0.2169;
54	100.0	0.0	8.711	40.70	18784.9	2.898	0.0537	0.0	0.2213;
55	100.0	0.0	8.885	40.56	18414.8	2.956	0.0543	0.0	0.2257;
56	100.0	0.0	9.063	40.42	18052.1	3.016	0.0548	0.0	0.2302;
57	100.0	0.0	9.245	40.28	17696.4	3.076	0.0554	0.0	0.2348;
58	100.0	0.0	9.430	40.14	17347.9	3.138	0.056	0.0	0.2395;
59	100.0	0.0	9.619	40.01	17006.2	3.201	0.0565	0.0	0.2443;
60	100.0	0.0	9.812	39.88	16671.3	3.265	0.0571	0.0	0.2492;
61	100.0	0.0	10.01	39.75	16343.0	3.331	0.0576	0.0	0.2542;
62	100.0	0.0	10.21	39.63	16021.2	3.398	0.0582	0.0	0.2593;
63	100.0	0.0	10.41	39.50	15705.8	3.466	0.0587	0.0	0.2645;
64	100.0	0.0	10.62	39.38	15396.6	3.536	0.0592	0.0	0.2698;
65	100.0	0.0	10.83	39.27	15103.8	3.604	0.0598	0.0	0.2751; begin overlap;
66	100.0	0.0	11.04	39.16	14827.3	3.672	0.0603	0.0	0.2803;
67	100.0	0.0	11.24	39.06	14565.6	3.738	0.0608	0.0	0.2855;
68	100.0	0.0	11.44	38.96	14317.5	3.802	0.0614	0.0	0.2905;
69	100.0	0.0	11.63	38.87	14081.8	3.866	0.0619	0.0	0.2954;
70	100.0	0.0	11.82	38.78	13857.6	3.929	0.0624	0.0	0.3003;
71	100.0	0.0	12.01	38.70	13643.8	3.990	0.0629	0.0	0.3050;
72	100.0	0.0	12.19	38.62	13439.7	4.051	0.0635	0.0	0.3097;
73	100.0	0.0	12.38	38.55	13244.6	4.110	0.064	0.0	0.3143;
74	100.0	0.0	12.55	38.47	13057.9	4.169	0.0645	0.0	0.3189;
75	100.0	0.0	12.73	38.40	12878.9	4.227	0.065	0.0	0.3234;
76	100.0	0.0	12.91	38.34	12707.1	4.284	0.0655	0.0	0.3278;
77	100.0	0.0	13.08	38.27	12542.1	4.341	0.0661	0.0	0.3322;
78	100.0	0.0	13.25	38.21	12383.4	4.396	0.0666	0.0	0.3365;
79	100.0	0.0	13.41	38.15	12230.7	4.451	0.0671	0.0	0.3407;
80	100.0	0.0	13.58	38.09	12083.5	4.505	0.0676	0.0	0.3449;
81	100.0	0.0	13.74	38.04	11941.5	4.559	0.0681	0.0	0.3490;
82	100.0	0.0	13.90	37.98	11804.4	4.612	0.0687	0.0	0.3531;
83	100.0	0.0	14.06	37.93	11672.0	4.664	0.0692	0.0	0.3572;
84	100.0	0.0	14.22	37.88	11544.0	4.716	0.0697	0.0	0.3611;
85	100.0	0.0	14.37	37.83	11420.1	4.767	0.0702	0.0	0.3651;
86	100.0	0.0	14.53	37.79	11300.1	4.818	0.0708	0.0	0.3690;
87	100.0	0.0	14.68	37.74	11183.9	4.868	0.0713	0.0	0.3729;
88	100.0	0.0	14.83	37.70	11071.2	4.917	0.0718	0.0	0.3767;
89	100.0	0.0	14.98	37.65	10961.8	4.966	0.0724	0.0	0.3805;
90	100.0	0.0	15.13	37.61	10855.7	5.015	0.0729	0.0	0.3842;
91	100.0	0.0	15.27	37.57	10752.5	5.063	0.0734	0.0	0.3879;
92	100.0	0.0	15.42	37.53	10652.3	5.111	0.074	0.0	0.3916;
93	100.0	0.0	15.56	37.50	10554.8	5.158	0.0745	0.0	0.3952;
94	100.0	0.0	15.70	37.46	10460.0	5.205	0.0751	0.0	0.3988;
95	100.0	0.0	15.84	37.42	10367.6	5.251	0.0756	0.0	0.4024;
96	100.0	0.0	15.98	37.39	10277.7	5.297	0.0762	0.0	0.4059;
97	100.0	0.0	16.12	37.35	10190.1	5.342	0.0767	0.0	0.4094;
98	100.0	0.0	16.26	37.32	10104.7	5.388	0.0773	0.0	0.4129;
99	100.0	0.0	16.39	37.29	10021.3	5.432	0.0778	0.0	0.4164;
100	100.0	0.0	16.53	37.26	9940.1	5.477	0.0784	0.0	0.4198;
101	100.0	0.0	16.66	37.22	9860.7	5.521	0.079	0.0	0.4232;
102	100.0	0.0	16.79	37.19	9783.2	5.565	0.0795	0.0	0.4265;
103	100.0	0.0	16.92	37.16	9707.6	5.608	0.0801	0.0	0.4298;

104	100.0	0.0	17.05	37.14	9633.6	5.651	0.0807	0.0	0.4332;
105	100.0	0.0	17.18	37.11	9561.3	5.694	0.0813	0.0	0.4364;
106	100.0	0.0	17.31	37.08	9490.6	5.736	0.0819	0.0	0.4397;
107	100.0	0.0	17.44	37.05	9421.4	5.778	0.0825	0.0	0.4429;
108	100.0	0.0	17.56	37.03	9353.7	5.820	0.0831	0.0	0.4461;
109	100.0	0.0	17.69	37.00	9287.5	5.862	0.0837	0.0	0.4493;
110	100.0	0.0	17.81	36.98	9222.6	5.903	0.0843	0.0	0.4525;
111	100.0	0.0	17.94	36.95	9159.1	5.944	0.0849	0.0	0.4556;
112	100.0	0.0	18.06	36.93	9096.8	5.985	0.0855	0.0	0.4587;
113	100.0	0.0	18.18	36.90	9035.8	6.025	0.0861	0.0	0.4618;
114	100.0	0.0	18.30	36.88	8976.0	6.065	0.0867	0.0	0.4649;
115	100.0	0.0	18.42	36.86	8917.3	6.105	0.0874	0.0	0.4679;
116	100.0	0.0	18.54	36.83	8859.8	6.145	0.088	0.0	0.4710;
117	100.0	0.0	18.66	36.81	8803.3	6.184	0.0886	0.0	0.4740;
118	100.0	0.0	18.78	36.79	8747.9	6.223	0.0893	0.0	0.4770;
119	100.0	0.0	18.90	36.77	8693.5	6.262	0.0899	0.0	0.4799;
120	100.0	0.0	19.01	36.75	8640.1	6.301	0.0906	0.0	0.4829;
121	100.0	0.0	19.13	36.73	8587.7	6.339	0.0913	0.0	0.4858;
122	100.0	0.0	19.24	36.71	8536.2	6.378	0.092	0.0	0.4887;
123	100.0	0.0	19.36	36.69	8485.6	6.416	0.0926	0.0	0.4916;
124	100.0	0.0	19.47	36.67	8435.8	6.453	0.0933	0.0	0.4945;
125	100.0	0.0	19.58	36.65	8386.9	6.491	0.094	0.0	0.4974;
126	100.0	0.0	19.69	36.63	8338.9	6.528	0.0947	0.0	0.5002;
127	100.0	0.0	19.81	36.61	8291.6	6.566	0.0954	0.0	0.5031;
128	100.0	0.0	19.92	36.59	8245.1	6.603	0.0962	0.0	0.5059;
129	100.0	0.0	20.03	36.58	8199.4	6.640	0.0969	0.0	0.5087;
130	100.0	0.0	20.14	36.56	8154.4	6.676	0.0976	0.0	0.5114;
131	100.0	0.0	20.24	36.54	8110.1	6.713	0.0984	0.0	0.5142;
132	100.0	0.0	20.35	36.52	8066.4	6.749	0.0991	0.0	0.5170;
133	100.0	0.0	20.46	36.51	8023.5	6.785	0.0999	0.0	0.5197;
134	100.0	0.0	20.57	36.49	7981.2	6.821	0.101	0.0	0.5224;
135	100.0	0.0	20.67	36.47	7939.6	6.857	0.101	0.0	0.5251;
136	100.0	0.0	20.78	36.46	7898.6	6.892	0.102	0.0	0.5278;
137	100.0	0.0	20.88	36.44	7858.2	6.928	0.103	0.0	0.5305;
138	100.0	0.0	20.99	36.43	7818.3	6.963	0.104	0.0	0.5331;
139	100.0	0.0	21.09	36.41	7779.1	6.998	0.105	0.0	0.5358;
140	100.0	0.0	21.20	36.40	7740.4	7.033	0.106	0.0	0.5384;
141	100.0	0.0	21.30	36.38	7702.2	7.068	0.106	0.0	0.5410;
142	100.0	0.0	21.40	36.37	7664.6	7.103	0.107	0.0	0.5436;
143	100.0	0.0	21.50	36.35	7627.5	7.137	0.108	0.0	0.5462;
144	100.0	0.0	21.60	36.34	7590.9	7.172	0.109	0.0	0.5488;
145	100.0	0.0	21.71	36.32	7554.8	7.206	0.110	0.0	0.5513;
146	100.0	0.0	21.81	36.31	7519.2	7.240	0.111	0.0	0.5539;
147	100.0	0.0	21.91	36.30	7484.0	7.274	0.112	0.0	0.5564;
148	100.0	0.0	22.00	36.28	7449.3	7.308	0.113	0.0	0.5589;
149	100.0	0.0	22.10	36.27	7415.0	7.342	0.114	0.0	0.5614;
150	100.0	0.0	22.20	36.26	7381.2	7.376	0.115	0.0	0.5639;
151	100.0	0.0	22.30	36.24	7347.8	7.409	0.116	0.0	0.5664;
152	100.0	0.0	22.39	36.23	7314.8	7.442	0.117	0.0	0.5688;
153	100.0	0.0	22.49	36.22	7282.2	7.476	0.118	0.0	0.5713;
154	100.0	0.0	22.59	36.20	7250.0	7.509	0.119	0.0	0.5737;
155	100.0	0.0	22.68	36.19	7218.1	7.542	0.120	0.0	0.5761;
156	100.0	0.0	22.78	36.18	7186.7	7.575	0.121	0.0	0.5785;
157	100.0	0.0	22.87	36.17	7155.6	7.608	0.123	0.0	0.5809;
158	100.0	0.0	22.97	36.16	7124.9	7.641	0.124	0.0	0.5833;
159	100.0	0.0	23.06	36.14	7094.5	7.674	0.125	0.0	0.5857;

160	100.0	0.0	23.15	36.13	7064.4	7.706	0.126	0.0	0.5880;
161	100.0	0.0	23.24	36.12	7034.7	7.739	0.127	0.0	0.5904;
162	100.0	0.0	23.33	36.11	7005.3	7.771	0.129	0.0	0.5927;
163	100.0	0.0	23.42	36.10	6976.1	7.804	0.130	0.0	0.5950;
164	100.0	0.0	23.51	36.09	6947.3	7.836	0.132	0.0	0.5973;
165	100.0	0.0	23.60	36.07	6918.8	7.868	0.133	0.0	0.5995;
166	100.0	0.0	23.69	36.06	6890.6	7.901	0.134	0.0	0.6018;
167	100.0	0.0	23.78	36.05	6862.6	7.933	0.136	0.0	0.6040;
168	100.0	0.0	23.87	36.04	6835.0	7.965	0.138	0.0	0.6063;
169	100.0	0.0	23.96	36.03	6807.5	7.997	0.139	0.0	0.6085;
170	100.0	0.0	24.04	36.02	6780.4	8.029	0.141	0.0	0.6107;
171	100.0	0.0	24.13	36.01	6753.4	8.061	0.143	0.0	0.6128;
172	100.0	0.0	24.21	36.00	6726.7	8.093	0.144	0.0	0.6150;
173	100.0	0.0	24.29	35.99	6700.3	8.125	0.146	0.0	0.6171;
174	100.0	0.0	24.38	35.98	6674.1	8.157	0.148	0.0	0.6192;
175	100.0	0.0	24.46	35.97	6649.1	8.188	0.150	0.0	0.6212;
176	100.0	0.0	24.53	35.96	6624.4	8.218	0.152	0.0	0.6232;
177	100.0	0.0	24.61	35.95	6599.9	8.249	0.154	0.0	0.6251;
178	100.0	0.0	24.69	35.94	6575.5	8.279	0.156	0.0	0.6271;
179	100.0	0.0	24.76	35.93	6551.5	8.310	0.158	0.0	0.6290;
180	100.0	0.0	24.84	35.92	6527.6	8.340	0.160	0.0	0.6309;
181	100.0	0.0	24.91	35.91	6503.9	8.370	0.162	0.0	0.6328;
182	100.0	0.0	24.99	35.90	6480.5	8.401	0.164	0.0	0.6346;
183	100.0	0.0	25.06	35.89	6457.2	8.431	0.165	0.0	0.6365;
184	100.0	0.0	25.13	35.89	6434.2	8.461	0.167	0.0	0.6384;
185	100.0	0.0	25.20	35.88	6411.3	8.491	0.169	0.0	0.6402;
186	100.0	0.0	25.28	35.87	6388.7	8.521	0.171	0.0	0.6420;
187	100.0	0.0	25.35	35.86	6366.2	8.551	0.173	0.0	0.6438;
188	100.0	0.0	25.42	35.85	6343.9	8.581	0.175	0.0	0.6456;
189	100.0	0.0	25.49	35.84	6321.8	8.611	0.177	0.0	0.6474;
190	100.0	0.0	25.56	35.83	6299.9	8.641	0.179	0.0	0.6491;
191	100.0	0.0	25.62	35.82	6278.1	8.671	0.181	0.0	0.6509;
192	100.0	0.0	25.69	35.82	6256.6	8.701	0.183	0.0	0.6526;
193	100.0	0.0	25.76	35.81	6235.1	8.731	0.185	0.0	0.6543;
194	100.0	0.0	25.83	35.80	6213.9	8.761	0.186	0.0	0.6560;
195	100.0	0.0	25.89	35.79	6192.8	8.791	0.188	0.0	0.6577;
196	100.0	0.0	25.96	35.78	6171.9	8.821	0.190	0.0	0.6594;
197	100.0	0.0	26.03	35.77	6151.1	8.850	0.192	0.0	0.6610;
198	100.0	0.0	26.09	35.77	6130.5	8.880	0.194	0.0	0.6627;
199	100.0	0.0	26.15	35.76	6110.0	8.910	0.196	0.0	0.6643;
200	100.0	0.0	26.22	35.75	6089.7	8.940	0.198	0.0	0.6659;
201	100.0	0.0	26.28	35.74	6069.5	8.969	0.200	0.0	0.6675;
202	100.0	0.0	26.34	35.73	6049.4	8.999	0.202	0.0	0.6691;
203	100.0	0.0	26.41	35.73	6029.5	9.029	0.204	0.0	0.6707;
204	100.0	0.0	26.47	35.72	6009.7	9.059	0.205	0.0	0.6723;
205	100.0	0.0	26.53	35.71	5990.1	9.088	0.207	0.0	0.6738;
206	100.0	0.0	26.59	35.70	5970.6	9.118	0.209	0.0	0.6754;
207	100.0	0.0	26.65	35.70	5951.2	9.148	0.211	0.0	0.6769;
208	100.0	0.0	26.71	35.69	5932.0	9.177	0.213	0.0	0.6784;
209	100.0	0.0	26.77	35.68	5912.8	9.207	0.215	0.0	0.6799;
210	100.0	0.0	26.83	35.67	5893.8	9.237	0.217	0.0	0.6814;
211	100.0	0.0	26.88	35.67	5874.9	9.267	0.219	0.0	0.6828;
212	100.0	0.0	26.94	35.66	5856.1	9.296	0.221	0.0	0.6843;
213	100.0	0.0	27.00	35.65	5837.5	9.326	0.223	0.0	0.6857;
214	100.0	0.0	27.05	35.64	5818.9	9.356	0.225	0.0	0.6872;
215	100.0	0.0	27.11	35.64	5800.5	9.385	0.226	0.0	0.6886;

216	100.0	0.0	27.16	35.63	5782.2	9.415	0.228	0.0	0.6900;
217	100.0	0.0	27.22	35.62	5763.9	9.445	0.230	0.0	0.6914;
218	100.0	0.0	27.27	35.62	5745.8	9.475	0.232	0.0	0.6927;
219	100.0	0.0	27.33	35.61	5727.8	9.505	0.234	0.0	0.6941;
220	100.0	0.0	27.38	35.60	5709.9	9.534	0.236	0.0	0.6955;
221	100.0	0.0	27.43	35.59	5692.1	9.564	0.238	0.0	0.6968;
222	100.0	0.0	27.48	35.59	5674.3	9.594	0.240	0.0	0.6981;
223	100.0	0.0	27.54	35.58	5656.7	9.624	0.242	0.0	0.6994;
224	100.0	0.0	27.59	35.57	5639.2	9.654	0.244	0.0	0.7007;
225	100.0	0.0	27.64	35.57	5621.7	9.684	0.246	0.0	0.7020;
226	100.0	0.0	27.69	35.56	5604.4	9.714	0.248	0.0	0.7033;
227	100.1	0.0	27.74	35.55	5587.1	9.744	0.250	0.0	0.7045;
228	100.1	0.0	27.79	35.55	5569.9	9.774	0.252	0.0	0.7058;
229	100.1	0.0	27.83	35.54	5552.8	9.804	0.254	0.0	0.7070;
230	100.1	0.0	27.88	35.53	5535.8	9.834	0.256	0.0	0.7082;
231	100.1	0.0	27.93	35.53	5518.9	9.864	0.258	0.0	0.7094;
232	100.1	0.0	27.98	35.52	5502.0	9.895	0.260	0.0	0.7106;
233	100.1	0.0	28.02	35.51	5485.3	9.925	0.262	0.0	0.7118;
234	100.1	0.0	28.07	35.51	5468.6	9.955	0.264	0.0	0.7129;
235	100.1	0.0	28.11	35.50	5451.9	9.985	0.266	0.0	0.7141;
236	100.1	0.0	28.16	35.49	5435.4	10.02	0.268	0.0	0.7152;
237	100.1	0.0	28.20	35.49	5418.9	10.05	0.270	0.0	0.7164;
238	100.1	0.0	28.25	35.48	5402.5	10.08	0.272	0.0	0.7175;
239	100.1	0.0	28.29	35.47	5386.1	10.11	0.274	0.0	0.7186;
240	100.1	0.0	28.33	35.47	5369.9	10.14	0.276	0.0	0.7197;
241	100.1	0.0	28.38	35.46	5353.7	10.17	0.278	0.0	0.7207;
242	100.1	0.0	28.42	35.46	5337.5	10.20	0.280	0.0	0.7218;
243	100.1	0.0	28.46	35.45	5321.4	10.23	0.282	0.0	0.7229;
244	100.1	0.0	28.50	35.44	5305.4	10.26	0.284	0.0	0.7239;
245	100.1	0.0	28.54	35.44	5289.4	10.29	0.286	0.0	0.7249;
246	100.1	0.0	28.58	35.43	5273.5	10.32	0.288	0.0	0.7259;
247	100.1	0.0	28.62	35.42	5257.7	10.35	0.290	0.0	0.7269;
248	100.1	0.0	28.66	35.42	5241.9	10.39	0.292	0.0	0.7279;
249	100.1	0.0	28.70	35.41	5226.2	10.42	0.294	0.0	0.7289;
250	100.1	0.0	28.73	35.41	5210.5	10.45	0.296	0.0	0.7298;
251	100.1	0.0	28.77	35.40	5194.9	10.48	0.299	0.0	0.7308;
252	100.1	0.0	28.81	35.39	5179.3	10.51	0.301	0.0	0.7317;
253	100.1	0.0	28.84	35.39	5163.8	10.54	0.303	0.0	0.7326;
254	100.1	0.0	28.88	35.38	5148.3	10.57	0.305	0.0	0.7336;
255	100.1	0.0	28.92	35.38	5132.9	10.61	0.307	0.0	0.7345;
256	100.1	0.0	28.95	35.37	5117.5	10.64	0.309	0.0	0.7353;
257	100.1	0.0	28.98	35.36	5102.2	10.67	0.311	0.0	0.7362;
258	100.1	0.0	29.02	35.36	5086.9	10.70	0.313	0.0	0.7371;
259	100.1	0.0	29.05	35.35	5071.6	10.73	0.316	0.0	0.7379;
260	100.1	0.0	29.08	35.35	5056.4	10.77	0.318	0.0	0.7387;
261	100.1	0.0	29.12	35.34	5041.2	10.80	0.320	0.0	0.7396;
262	100.1	0.0	29.15	35.33	5026.1	10.83	0.322	0.0	0.7404;
263	100.1	0.0	29.18	35.33	5011.0	10.86	0.324	0.0	0.7412;
264	100.1	0.0	29.21	35.32	4995.9	10.90	0.327	0.0	0.7420;
265	100.1	0.0	29.24	35.32	4980.9	10.93	0.329	0.0	0.7427;
266	100.1	0.0	29.27	35.31	4965.9	10.96	0.331	0.0	0.7435;
267	100.1	0.0	29.30	35.30	4951.0	11.00	0.333	0.0	0.7442;
268	100.1	0.0	29.33	35.30	4936.1	11.03	0.336	0.0	0.7450;
269	100.1	0.0	29.36	35.29	4921.2	11.06	0.338	0.0	0.7457;
270	100.1	0.0	29.39	35.29	4906.3	11.10	0.340	0.0	0.7464;
271	100.1	0.0	29.41	35.28	4891.5	11.13	0.343	0.0	0.7471;

272	100.1	0.0	29.44	35.28	4876.6	11.16	0.345	0.0	0.7478;
273	100.1	0.0	29.47	35.27	4861.9	11.20	0.347	0.0	0.7485;
274	100.1	0.0	29.49	35.26	4847.1	11.23	0.350	0.0	0.7491;
275	100.1	0.0	29.52	35.26	4832.4	11.27	0.352	0.0	0.7498;
276	100.1	0.0	29.54	35.25	4817.6	11.30	0.354	0.0	0.7504;
277	100.1	0.0	29.57	35.25	4802.9	11.33	0.357	0.0	0.7510;
278	100.1	0.0	29.59	35.24	4788.3	11.37	0.359	0.0	0.7516;
279	100.1	0.0	29.62	35.24	4773.6	11.40	0.362	0.0	0.7522;
280	100.1	0.0	29.64	35.23	4759.0	11.44	0.364	0.0	0.7528;
281	100.1	0.0	29.66	35.22	4744.3	11.47	0.366	0.0	0.7534;
282	100.1	0.0	29.68	35.22	4729.7	11.51	0.369	0.0	0.7540;
283	100.1	0.0	29.71	35.21	4715.1	11.55	0.371	0.0	0.7545;
284	100.1	0.0	29.73	35.21	4700.5	11.58	0.374	0.0	0.7551;
285	100.1	0.0	29.75	35.20	4685.9	11.62	0.376	0.0	0.7556;
286	100.1	0.0	29.77	35.20	4671.4	11.65	0.379	0.0	0.7561;
287	100.2	0.0	29.79	35.19	4656.8	11.69	0.381	0.0	0.7566;
288	100.2	0.0	29.81	35.18	4642.3	11.73	0.384	0.0	0.7571;
289	100.2	0.0	29.83	35.18	4627.7	11.76	0.387	0.0	0.7576;
290	100.2	0.0	29.84	35.17	4613.2	11.80	0.389	0.0	0.7580;
291	100.2	0.0	29.86	35.17	4598.6	11.84	0.392	0.0	0.7585;
292	100.2	0.0	29.88	35.16	4584.1	11.88	0.394	0.0	0.7589;
293	100.2	0.0	29.90	35.16	4569.5	11.91	0.397	0.0	0.7594;
294	100.2	0.0	29.91	35.15	4555.0	11.95	0.400	0.0	0.7598;
295	100.2	0.0	29.93	35.14	4540.4	11.99	0.402	0.0	0.7602;
296	100.2	0.0	29.95	35.14	4525.9	12.03	0.405	0.0	0.7606;
297	100.2	0.0	29.96	35.13	4511.3	12.07	0.408	0.0	0.7610;
298	100.2	0.0	29.98	35.13	4496.7	12.11	0.411	0.0	0.7614;
299	100.2	0.0	29.99	35.12	4482.2	12.15	0.413	0.0	0.7618;
300	100.2	0.0	30.00	35.12	4467.6	12.19	0.416	0.0	0.7621;
301	100.2	0.0	30.02	35.11	4453.0	12.23	0.419	0.0	0.7625;
302	100.2	0.0	30.03	35.10	4438.3	12.27	0.422	0.0	0.7628;
303	100.2	0.0	30.04	35.10	4423.7	12.31	0.425	0.0	0.7631;
304	100.2	0.0	30.06	35.09	4409.0	12.35	0.428	0.0	0.7634;
305	100.2	0.0	30.07	35.09	4394.3	12.39	0.430	0.0	0.7637;
306	100.2	0.0	30.08	35.08	4379.6	12.43	0.433	0.0	0.7640;
307	100.2	0.0	30.09	35.07	4364.9	12.47	0.436	0.0	0.7643;
308	100.2	0.0	30.10	35.07	4350.2	12.51	0.439	0.0	0.7646;
309	100.2	0.0	30.11	35.06	4335.4	12.56	0.442	0.0	0.7649;
310	100.2	0.0	30.12	35.06	4320.6	12.60	0.445	0.0	0.7651;
311	100.2	0.0	30.13	35.05	4305.7	12.64	0.448	0.0	0.7654;
312	100.2	0.0	30.14	35.05	4290.8	12.69	0.451	0.0	0.7656;
313	100.2	0.0	30.15	35.04	4275.9	12.73	0.455	0.0	0.7659;
314	100.2	0.0	30.16	35.03	4261.0	12.78	0.458	0.0	0.7661;
315	100.2	0.0	30.17	35.03	4246.0	12.82	0.461	0.0	0.7663;
316	100.2	0.0	30.18	35.02	4230.9	12.87	0.464	0.0	0.7665;
317	100.3	0.0	30.19	35.02	4215.8	12.91	0.467	0.0	0.7667;
318	100.3	0.0	30.19	35.01	4200.7	12.96	0.471	0.0	0.7669;
319	100.3	0.0	30.20	35.00	4185.5	13.01	0.474	0.0	0.7671;
320	100.3	0.0	30.21	35.00	4170.2	13.05	0.477	0.0	0.7673;
321	100.3	0.0	30.21	34.99	4154.9	13.10	0.481	0.0	0.7675;
322	100.3	0.0	30.22	34.99	4139.5	13.15	0.484	0.0	0.7676;
323	100.3	0.0	30.23	34.98	4124.1	13.20	0.487	0.0	0.7678;
324	100.3	0.0	30.23	34.97	4108.5	13.25	0.491	0.0	0.7679;
325	100.3	0.0	30.24	34.97	4092.9	13.30	0.494	0.0	0.7681;
326	100.3	0.0	30.25	34.96	4077.3	13.35	0.498	0.0	0.7683;
327	100.3	0.0	30.25	34.96	4061.5	13.40	0.502	0.0	0.7684;

328	100.3	0.0	30.26	34.95	4045.7	13.46	0.505	0.0	0.7686;
329	100.3	0.0	30.26	34.94	4029.8	13.51	0.509	0.0	0.7687;
330	100.3	0.0	30.27	34.94	4013.8	13.56	0.513	0.0	0.7688;
331	100.3	0.0	30.28	34.93	3997.7	13.62	0.516	0.0	0.7690;
332	100.3	0.0	30.28	34.92	3981.4	13.67	0.520	0.0	0.7691;
333	100.3	0.0	30.29	34.92	3965.1	13.73	0.524	0.0	0.7693;
334	100.3	0.0	30.29	34.91	3948.7	13.79	0.528	0.0	0.7694;
335	100.3	0.0	30.30	34.91	3932.1	13.84	0.532	0.0	0.7696;
336	100.4	0.0	30.30	34.90	3915.5	13.90	0.536	0.0	0.7697;
337	100.4	0.0	30.31	34.89	3898.7	13.96	0.540	0.0	0.7699;
338	100.4	0.0	30.32	34.89	3881.7	14.02	0.544	0.0	0.7700;
339	100.4	0.0	30.32	34.88	3864.6	14.09	0.548	0.0	0.7702;
340	100.4	0.0	30.33	34.87	3847.4	14.15	0.552	0.0	0.7704;
341	100.4	0.0	30.34	34.87	3830.0	14.21	0.556	0.0	0.7706;
342	100.4	0.0	30.35	34.86	3812.4	14.28	0.560	0.0	0.7708;
343	100.4	0.0	30.35	34.85	3794.7	14.35	0.565	0.0	0.7710;
344	100.4	0.0	30.36	34.84	3776.8	14.41	0.569	0.0	0.7712;
345	100.4	0.0	30.37	34.84	3758.7	14.48	0.574	0.0	0.7715;
346	100.4	0.0	30.38	34.83	3740.4	14.55	0.578	0.0	0.7717;
347	100.4	0.0	30.39	34.82	3721.9	14.63	0.583	0.0	0.7720;
348	100.4	0.0	30.41	34.82	3703.2	14.70	0.587	0.0	0.7723;
349	100.5	0.0	30.42	34.81	3684.3	14.78	0.592	0.0	0.7726;
350	100.5	0.0	30.43	34.80	3665.1	14.85	0.597	0.0	0.7730;
351	100.5	0.0	30.45	34.79	3645.7	14.93	0.601	0.0	0.7734;
352	100.5	0.0	30.46	34.79	3626.0	15.01	0.606	0.0	0.7738;
353	100.5	0.0	30.48	34.78	3606.1	15.10	0.611	0.0	0.7742;
354	100.5	0.0	30.50	34.77	3585.9	15.18	0.616	0.0	0.7747;
355	100.5	0.0	30.52	34.76	3565.3	15.27	0.621	0.0	0.7752;
356	100.5	0.0	30.54	34.75	3544.5	15.36	0.627	0.0	0.7758;
357	100.5	0.0	30.57	34.75	3523.3	15.45	0.632	0.0	0.7764;
358	100.6	0.0	30.59	34.74	3501.8	15.55	0.637	0.0	0.7771;
359	100.6	0.0	30.62	34.73	3480.2	15.64	0.642	0.0	0.7778;
360	100.6	0.0	30.65	34.72	3458.7	15.74	0.648	0.0	0.7785; end overlap;
361	100.6	0.0	30.67	34.71	3437.4	15.84	0.653	0.0	0.7791;
362	100.6	0.0	30.69	34.70	3416.2	15.94	0.659	0.0	0.7796;
363	100.6	0.0	30.71	34.70	3395.1	16.03	0.665	0.0	0.7802;
364	100.6	0.0	30.73	34.69	3374.1	16.13	0.671	0.0	0.7806;
365	100.6	0.0	30.75	34.68	3353.3	16.23	0.677	0.0	0.7810;
366	100.7	0.0	30.76	34.67	3332.5	16.34	0.683	0.0	0.7814;
367	100.7	0.0	30.78	34.66	3311.7	16.44	0.689	0.0	0.7817;
368	100.7	0.0	30.79	34.65	3291.0	16.54	0.695	0.0	0.7820;
369	100.7	0.0	30.80	34.65	3270.3	16.65	0.701	0.0	0.7822;
370	100.7	0.0	30.80	34.64	3249.6	16.75	0.708	0.0	0.7824;
371	100.7	0.0	30.81	34.63	3228.9	16.86	0.714	0.0	0.7826;
372	100.8	0.0	30.81	34.62	3208.2	16.97	0.721	0.0	0.7827;
373	100.8	0.0	30.82	34.61	3187.4	17.08	0.728	0.0	0.7828;
374	100.8	0.0	30.82	34.61	3166.6	17.19	0.735	0.0	0.7829;
375	100.8	0.0	30.82	34.60	3145.7	17.31	0.742	0.0	0.7829;
376	100.8	0.0	30.82	34.59	3124.7	17.42	0.749	0.0	0.7829;
377	100.8	0.0	30.83	34.58	3103.6	17.54	0.756	0.0	0.7830;
378	100.9	0.0	30.83	34.57	3082.3	17.66	0.764	0.0	0.7830;
379	100.9	0.0	30.83	34.56	3060.9	17.79	0.771	0.0	0.7830;
380	100.9	0.0	30.83	34.56	3039.2	17.91	0.779	0.0	0.7830;
381	100.9	0.0	30.83	34.55	3017.4	18.04	0.787	0.0	0.7830;
382	101.0	0.0	30.83	34.54	2995.3	18.17	0.795	0.0	0.7830;
383	101.0	0.0	30.83	34.53	2973.0	18.31	0.804	0.0	0.7831;

384	101.0	0.0	30.83	34.52	2950.4	18.45	0.812	0.0	0.7831;
385	101.0	0.0	30.84	34.51	2927.5	18.60	0.821	0.0	0.7832;
386	101.1	0.0	30.84	34.50	2904.3	18.74	0.830	0.0	0.7833;
387	101.1	0.0	30.85	34.49	2880.6	18.90	0.839	0.0	0.7835;
388	101.1	0.0	30.86	34.48	2856.6	19.06	0.848	0.0	0.7837;
389	101.2	0.0	30.87	34.47	2832.2	19.22	0.858	0.0	0.7840;
390	101.2	0.0	30.88	34.46	2807.4	19.39	0.868	0.0	0.7843;
391	101.2	0.0	30.90	34.45	2782.0	19.57	0.878	0.0	0.7848;
392	101.3	0.0	30.92	34.44	2756.2	19.75	0.888	0.0	0.7853;
393	101.3	0.0	30.94	34.43	2729.8	19.94	0.899	0.0	0.7859;
394	101.4	0.0	30.97	34.42	2702.8	20.14	0.909	0.0	0.7866;
395	101.4	0.0	31.00	34.41	2675.2	20.35	0.920	0.0	0.7874;
396	101.4	0.0	31.04	34.40	2646.9	20.57	0.932	0.0	0.7884;
397	101.5	0.0	31.08	34.39	2618.0	20.79	0.944	0.0	0.7895;
398	101.5	0.0	31.13	34.38	2588.4	21.03	0.956	0.0	0.7907;
399	101.6	0.0	31.19	34.37	2558.0	21.28	0.968	0.0	0.7922;
400	101.6	0.0	31.25	34.35	2526.8	21.54	0.981	0.0	0.7938;
401	101.7	0.0	31.33	34.34	2494.8	21.82	0.994	0.0	0.7957;
402	101.8	0.0	31.41	34.33	2461.9	22.11	1.007	0.0	0.7978;
403	101.8	0.0	31.50	34.32	2428.2	22.42	1.021	0.0	0.8001;
404	101.9	0.0	31.60	34.30	2393.4	22.75	1.035	0.0	0.8027;
405	102.0	0.0	31.72	34.29	2357.7	23.09	1.049	0.0	0.8056;
406	102.0	0.0	31.84	34.27	2321.0	23.46	1.064	0.0	0.8089;
407	102.1	0.0	31.99	34.26	2283.3	23.84	1.080	0.0	0.8125;
408	102.2	0.0	32.14	34.24	2244.5	24.26	1.096	0.0	0.8165;
409	102.3	0.0	32.32	34.23	2204.5	24.69	1.112	0.0	0.8209;
410	102.4	0.0	32.51	34.21	2163.4	25.16	1.129	0.0	0.8257;
411	102.5	0.0	32.72	34.20	2121.2	25.66	1.147	0.0	0.8311;
412	102.6	0.0	32.94	34.18	2079.6	26.18	1.164	0.0	0.8367;
413	102.7	0.0	33.17	34.16	2038.8	26.70	1.181	0.0	0.8425;
414	102.8	0.0	33.41	34.15	1998.8	27.24	1.198	0.0	0.8486;
415	102.9	0.0	33.66	34.13	1959.6	27.78	1.214	0.0	0.8549;
416	103.0	0.0	33.92	34.12	1921.1	28.34	1.230	0.0	0.8615;
417	103.1	0.0	34.19	34.10	1883.5	28.90	1.246	0.0	0.8684;
418	103.2	0.0	34.47	34.09	1846.5	29.48	1.262	0.0	0.8755;
419	103.3	0.0	34.76	34.07	1810.3	30.07	1.277	0.0	0.8828;
420	103.4	0.0	35.05	34.06	1774.8	30.67	1.293	0.0	0.8904;
421	103.5	0.0	35.36	34.05	1740.0	31.29	1.308	0.0	0.8982;
422	103.6	0.0	35.68	34.03	1705.8	31.91	1.323	0.0	0.9061;
423	103.7	0.0	36.00	34.02	1672.4	32.55	1.338	0.0	0.9143;
424	103.8	0.0	36.33	34.01	1639.6	33.20	1.352	0.0	0.9227;
425	104.0	0.0	36.67	33.99	1607.4	33.87	1.367	0.0	0.9313;
426	104.1	0.0	37.01	33.98	1575.9	34.55	1.381	0.0	0.9401;
427	104.2	0.0	37.37	33.97	1544.9	35.24	1.395	0.0	0.9491;
428	104.3	0.0	37.73	33.96	1514.6	35.94	1.409	0.0	0.9583;
429	104.4	0.0	38.10	33.95	1484.9	36.66	1.423	0.0	0.9677; bottom hit;

Horiz plane projections in effluent direction: radius(m): 0.0; CL(m): 0.4337
Lmz(m): 0.4337
forced entrain 1 0.0 -1.345 0.968 0.120
Rate sec-1 0.0 dy-1 0.0 kt: 0.0 Amb Sal 33.3632
;
3:07:41 PM. amb fills: 4

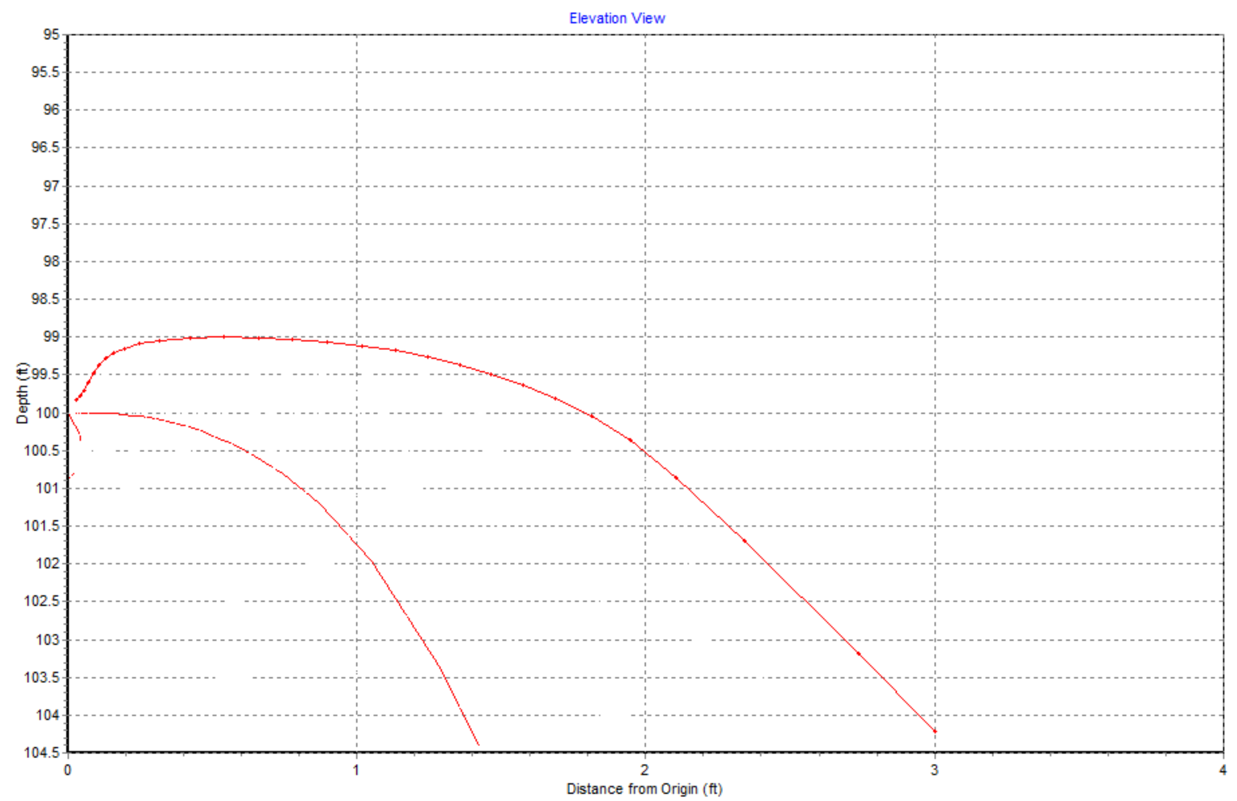

Figure C.8.1: Plumes 18b solution of discharge plume trajectories for discharges of 1.8 mgd of Doheny and Capistrano Beach well water average annual TDS = 33.5 ppt., and 3 mgd of brine from the Doheny Desalination Project with a brine salinity of 67 ppt. Depth of maximum rise of the plume is Z = 99.0 ft. at Xa = 0.735 ft from the point of discharge. The centerline of the plume is at an average distance of Xb = 1.423 ft from the point of discharge as the plume begins to impact the bottom at a depth of Z = 104.5 ft.

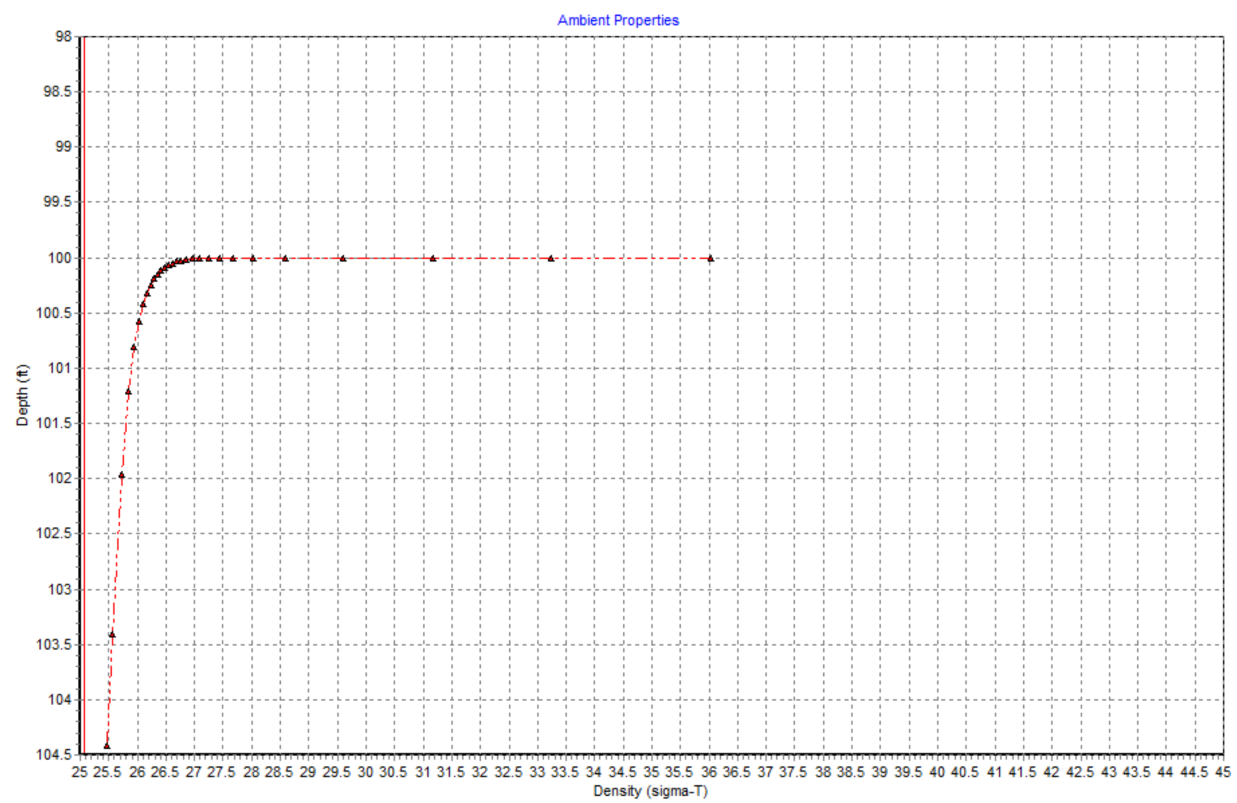

Figure C.8.2: Plumes 18b solution of vertical density profile for discharges of 1.8 mgd of Doheny and Capistrano Beach well water average annual TDS = 33.5 ppt. and 3 mgd of brine from the Doheny Desalination Project with a brine salinity of 67 ppt. Discharge effluent density shown as black triangles. Ambient water mass density profile shown as solid red line.

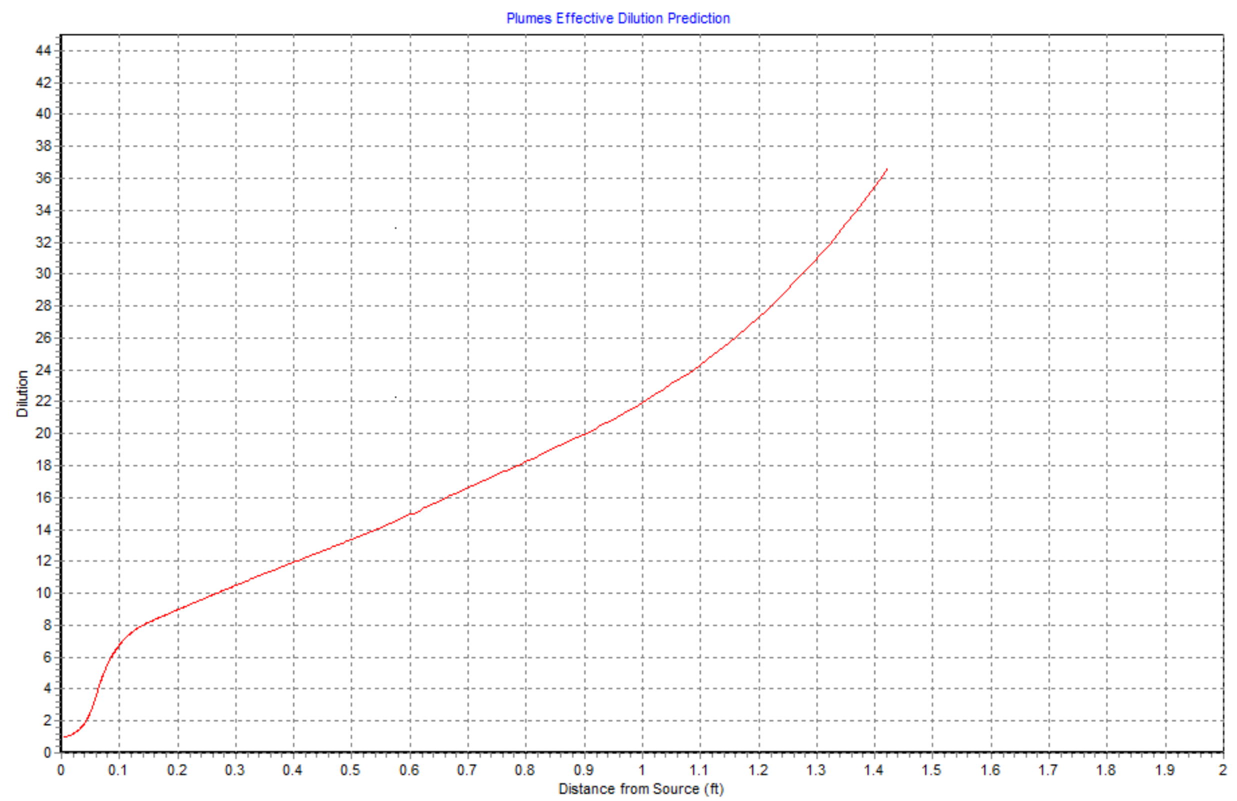

Figure C.8.3: Plumes 18b solution of effective (bulk average) dilution as a function of vertical distance from the point of discharge for discharges of 0.35 mgd of 1.8 mgd of Doheny and Capistrano Beach well water average annual TDS = 33.5 ppt and 3 mgd of brine from the Doheny Desalination Project with a brine salinity of 67 ppt. Effective dilution is S_a = 17.19 at the maximum rise of the plume at Xa = 0.735 ft. from the point of discharge. As the plume begins to impact the bottom, the plume centerline is at a distance of Xb = 1.423 ft from the point of discharge, where the effective dilution reaches S_a = 36.66.

APPENDIX 4.2.3

HYDROGEOLOGIC ANALYSES

APPENDIX 4.2.3.1

GROUNDWATER MODELING FOR FINAL EIR

Doheny Ocean Desalination Project

Hydrogeologic Analysis Related to Responses to Comments: Evaluate Project Impacts on San Juan Creek Surface Water Levels and Assessment of Project Impacts from Potential Upstream Bedrock “Barrier”

Prepared for: South Coast Water District / GHD

March 7, 2019

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THIS REPORT IS RENDERED TO GHD AND SOUTH COAST WATER DISTRICT AS OF THE DATE HEREOF, SOLELY FOR THEIR BENEFIT IN CONNECTION WITH ITS STATED PURPOSE AND MAY NOT BE RELIED ON BY ANY OTHER PERSON OR ENTITY OR BY THEM IN ANY OTHER CONTEXT. AS DATA IS UPDATED FROM TIME TO TIME, ANY RELIANCE ON THIS REPORT AT A FUTURE DATE SHOULD TAKE INTO ACCOUNT UPDATED DATA.

THIS DOCUMENT HAS BEEN CHECKED FOR COMPLETENESS, ACCURACY, AND CONSISTENCY BY THE FOLLOWING PROFESSIONALS:

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Principal
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EIR DOHENY OCEAN DESALINATION PROJECT

HYDROGEOLOGIC ANALYSIS RELATED TO RESPONSES TO COMMENTS
EVALUATE PROJECT IMPACTS ON SAN JUAN CREEK SURFACE WATER LEVELS AND ASSESSMENT OF PROJECT IMPACTS FROM POTENTIAL UPSTREAM BEDROCK "BARRIER"

Contents

FIGURES

FIGURES (continued)

FIGURES (continued)

TABLES

EIR DOHENY OCEAN DESALINATION PROJECT

HYDROGEOLOGIC ANALYSIS RELATED TO RESPONSES TO COMMENTS EVALUATE PROJECT IMPACTS ON SAN JUAN CREEK SURFACE WATER LEVELS AND ASSESSMENT OF PROJECT IMPACTS FROM POTENTIAL UPSTREAM BEDROCK "BARRIER"

1.0 INTRODUCTION

The Doheny Ocean Desalination Project Draft Environmental Impact Report (DEIR) was issued on May 17, 2018. GEOSCIENCE Support Services, Inc. (GEOSCIENCE) reviewed the DEIR comments related to project impacts to groundwater and surface water provided to us by the project team, including those provided by the National Oceanic and Atmospheric Administration National Marine Fisheries Service (NOAA-NMFS, San Juan Basin Authority, and Santa Margarita Water District). In response to these comments, GEOSCIENCE has conducted additional analysis regarding the influence of slant well pumping on San Juan Creek lagoon, surface and groundwater levels in the shallow aquifer, and potential changes due to a suspected bedrock barrier. This technical memorandum summarizes the results of our analysis, as outlined in the approved scope of work from our proposal dated August 29, 2018 for GHD and South Coast Water District (SCWD).

2.0 EVALUATION OF PROJECT CHANGES ON SAN JUAN CREEK SURFACE FLOW

The San Juan Basin (SJB) Regional Groundwater Model was used to determine San Juan Creek discharges to the ocean under "No Project" (i.e., Baseline) and "Project" (i.e., pumping) conditions. Current pumping is assumed to continue into the future since the groundwater basin is currently managed by basin stakeholders to avoid over pumping. Under No Pumping conditions, current production from existing pumping wells are considered. Evaluating the surface outflow component under No Project and Project conditions allows for the quantification of the potential reduction in surface flow, in cubic feet per second (cfs).

2.1 San Juan Basin Regional Groundwater Model

The SJB Regional Groundwater Model was originally developed in 2013 to evaluate the basin yield and groundwater level response from existing and planned groundwater development. It was also utilized to determine potential Changes in groundwater levels and pumping interference from the installation of sheet piling along the San Juan Creek flood control channel, and assess changes associated with the Doheny Ocean Desalination Project (GEOSCIENCE, 2013). Later, the model was updated for the Doheny Ocean Desalination Project during work for the Foundational Actions Funding Program – Advancement of Slant Well Technology and Groundwater Flow and Solute Transport Modeling (GEOSCIENCE, 2015) to better understand 1) feedwater quality produced over time from a slant well system, 2) drawdown effects and environmental strategies along coastal reaches, and 3) the behavior of seawater flow and intrusion control in a multi-layered aquifer system. Following onshore and offshore geophysical surveys and the drilling of a borehole in 2017-2018 to better define the geometry of coastal paleochannels, the SJB Regional Model was further refined by incorporating the newer hydrogeological data.

The SJB Regional Model is a three (3) layered MODFLOW[1] model covering the lower and middle SJB area of approximately 47.5 square miles (30,400 acres), including an offshore area to incorporate infiltration from the ocean (see Figure 1). The model consists of a finite-difference grid with 1,012 rows in the north to south direction and 524 columns in the west to east direction, for a total of 530,288 cells per layer, or 1,590,864 cells total. Each model cell of the SJB Regional Model represents an area of 50 ft x 50 ft. The active model area represents unconsolidated and semi-consolidated fluvial deposits interbedded with numerous fine-grained silt and clay deposits. Inactive model areas and the base of the groundwater model represent surrounding and underlying consolidated geologic formations (i.e., bedrock).

The regional model was calibrated for the period from January 2004 to December 2014 for purposes of analyzing the impacts of full-scale pumping. This period was selected due to the importance of recent stresses (dry hydrologic period) on the basin for predicting future performance. The calibration was based on 2,435 groundwater level measurements from 36 target wells and measured streamflow from the San Juan Creek at La Novia gaging station and Trabuco Creek at San Juan Capistrano gaging station (see Figure 1).

2.2 Model-Calculated San Juan Creek Lagoon Elevations

The SJB Focused Model was developed and calibrated as part of the Foundational Actions Funding Program (GEOSCIENCE, 2015) to more accurately predict slant well pumped water quality over time, injection water flow/water quality/reactants, and ocean water intrusion. The finer cell size (resolution) used for the focused model was also important to understand seasonal coastal lagoon drawdown effects. Lagoon levels were calculated by the Focused Model, which employs the Lake Package to simulate the surface water-groundwater interaction.

Changes to San Juan Creek lagoon levels from Project operations were evaluated for the hydrologic period from January 1947 through December 2010 for the following scenarios:

- Baseline (i.e., no Project pumping)

[1] MODFLOW is a block-centered, finite-difference groundwater flow code developed by the United States Geologic Survey (USGS) (McDonald and Harbaugh, 1988) for the purpose of modeling both saturated and unsaturated groundwater flow.

- Scenario 1: Project pumping of 10 million gallons per day (MGD) from three slant wells
- Scenario 2: Project pumping of 10 MGD from seven slant wells at Capistrano Beach
- Scenario 3: Project pumping of 30 MGD from sixteen slant wells (20 MGD from slant well pods at Doheny and 10 MGD from pods at Capistrano Beach).

Due to the uncertainties remaining as to the hydrogeologic conditions at Capistrano Beach, a 30MGD with pumping from only a Doheny Beach wellfield was modeled as Scenario 4 also the current analysis does not consider Scenarios 2 for the cumulative changes.

- Scenario 4: Project pumping of 30 MGD from twelve slant wells at Doheny Beach

As presented in Table 4-3 of "Model Update and Refinement Using Results from Onshore and Offshore Geophysical Surveys and Exploratory Borehole Data" Technical Memorandum, dated March 1, 2018, changes to lagoon water levels from Project pumping were reported as ranging from -0.14 to -0.26 ft under Scenario 1 conditions and -0.16 to -0.63 ft under Scenario 3 conditions for dry and wet hydrologic conditions. Scenario 4 conditions were not reported in Table 4-3 as Scenario 4 was modeled after the TM was issued (March 1, 2018). Under Scenario 4 conditions lagoon level changes will range from -0.15 to -0.74 ft. Groundwater levels in the shallow aquifer will range from -33.47 to -42.79 ft. To clarify the decreases in lagoon levels for dry and wet hydrologic conditions shown on Table 2-1 below summarize the model-calculated decreases and potential project impacts on lagoon surface levels. The "dry" and "wet" hydrologic cycle periods are determined when the cumulative departure from the mean is in a downward or upward trend (see Figure 2 and Section 2.6 of this memorandum).

Table 2-1. Change in Lagoon Surface Levels under Project Conditions

Hydrologic Cycle	Hydrologic Period	Change in Lagoon Level, ft		
		Scenario 1	Scenario 3	Scenario 4
Dry	1947-1976	-0.14	-0.16	-0.15
Wet	1978-1983	-0.26	-0.85	-0.86

It is important to note that the model-calculated decreases in San Juan Creek lagoon levels occur over the entire lagoon area of approximately 13.2 acres. The following section provides the model-calculated reduction in outflow that will occur as a result of Scenario 1, Scenario 3, and Scenario 4 pumping over the area influenced by Project pumping under dry and wet hydrologic conditions.

2.3 Model-Calculated San Juan Creek Outflow to the Ocean under No Project and Project Conditions

Surface flow in San Juan Creek was simulated in the 2016 SJB Regional Model using the Streamflow Routing Package (SFR Package) from MODLFOW, which accounts for the interaction between surface water and groundwater. This area is outside the Focused Model, so the Regional Model was used to evaluate changes in groundwater levels in this area. The SFR Package assigns recharge to stream cells that are sequentially numbered in the downstream direction. The downward leakage of streamflow, or streambed percolation, is calculated as a function of the hydraulic conductivity of the streambed, the wetted perimeter of the streambed, the length of the stream reach, the underlying groundwater head, stream stage, and streambed thickness. Model input for the routing package includes stream inflow, stream channel geometry, and streambed conductance (Niswonger and Prudic, 2006). Information on streamflow was available from observed measurements at the San Juan Creek at La Novia and Trabuco Creek at San Juan Capistrano gaging stations, which were also used for model calibration. These gaging stations are shown on Figure 1.

In order to evaluate the long-term outflow at the ocean, streamflow in San Juan Creek and surface outflow to the ocean was calculated by the SJB Regional Model for the hydrologic period from January 1947 through December 2010 for the same scenarios presented above. The model-calculated streamflow from San Juan Creek to the ocean is shown on Figures 3 through 6 under Baseline, Scenario 1, Scenario 3, and Scenario 4 conditions, respectively.

For baseline conditions, outflow at the ocean ranges from 15.91 cubic feet per second (cfs) under dry hydrologic conditions to 56.04 cfs under wet hydrologic conditions. Discharge under Scenario 1 Project pumping conditions ranges from 15.81 cfs under dry hydrologic conditions to 55.59 cfs under wet hydrologic conditions. Discharge under Scenario 3 ranges from 15.78 cfs under dry hydrologic conditions to 55.41 cfs under wet hydrologic conditions. Discharge under Scenario 4 ranges from 15.76 cfs under dry hydrologic conditions to 55.30 cfs under wet hydrologic conditions. The corresponding decrease in San Juan Creek streamflow under Project pumping conditions, as compared to baseline, is summarized in Table 2-2 below.

Table 2-2. Change in San Juan Creek Outflow to the Ocean under Project Conditions

Hydrologic Cycle	Hydrologic Period	Change in San Juan Creek Discharge, cfs		
		Scenario 1	Scenario 3	Scenario 4
Dry	1947-1976	-0.10	-0.13	-0.15
Wet	1978-1983	-0.45	-0.63	-0.74

The decreases in San Juan Creek streamflow from Project pumping correspond to approximately 0.6 to 0.8 percent of the baseline outflow under Scenario 1 conditions, 0.8 to 1.1 percent of the baseline outflow under Scenario 3 conditions, and 0.9 to 1.3 percent under Scenario 4 conditions.

2.4 Evaluation of Changes in Shallow Aquifer Groundwater Levels under Project Conditions

2.4.1 Groundwater Level Monitoring

Historical groundwater levels for the shallow aquifer in the vicinity of the San Juan Creek lagoon were collected both during and after the long-term slant well pumping test from transducers placed in District-owned nested monitoring wells MW-1 through MW-4 (12 total monitoring wells). MW-1 and MW-2, which are closest to the lagoon, are shown on Figure 2-1 (inset below).

Figure 2-1. Monitoring Well Locations in the Vicinity of the San Juan Creek Lagoon

During the initial field investigations conducted in 2005, boreholes B-2 and B-4 were completed as nested monitoring wells (MW-1 and MW-2, respectively). Each borehole contains three nested 2-inch PVC wells screened in the shallow, middle, or deep aquifer. A basic monitoring well construction diagram for

B-2/MW-1 and B-4/MW-2 is shown on the sketch below (inset Figure 2-2), which illustrates the general configuration of the nested monitoring wells in MW-1 and MW-2.

After the monitoring wells were constructed, they were developed and sampled. The depth to static groundwater level in each nested monitoring well was initially measured with an electronic sounder. Each nested monitoring well was later equipped with a pressure transducer to measure groundwater levels every 15 minutes. Transducer data were downloaded on a weekly basis during the long-term pumping test (June 2010 through April 2012) and on a monthly basis after the long-term pumping test to present. An on-site barometer was used to compensate the transducer data downloaded from each nested monitoring well. The compensated water level data were then converted to elevation (NAVD88) and plotted over time.

The nested well design allows for accurate water level data from each of the nearshore aquifers (shallow, middle, and deep). For example, the shallow screen in each nested monitoring well (MW-1S and MW-2S) provides water level data for the shallow aquifer, which is in direct hydraulic connection with San Juan Creek. Since the shallow aquifer is separated from the middle aquifer by an aquitard that is approximately 10 ft thick, water level measurements in the shallow aquifer are not influenced by the deeper systems. Therefore, water level data from MW-1S and MW-2S can be used to monitor and evaluate Project pumping impacts on the shallow aquifer. MW-2S was used to evaluate groundwater levels in the shallow aquifer under the lagoon due to its location in relation to both the Test Slant Well and the lagoon.

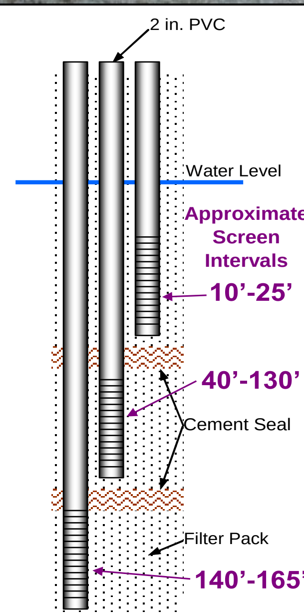

Figure 2-2. General Monitoring Well Construction

In addition, MW-2 is located at approximately the same location as one of the lagoon bottom profiles surveyed by the Chambers Group (2016) from Spring 2015 to Spring 2016 – providing a reference on the relative position of shallow aquifer water levels with respect to the lagoon bottom (see Figure 2-1). These profiles presented originally as Figure 2-14 of the Chambers Group report, are shown as attached Figure 7. The cross-sectional transects indicate most erosion taking place within the southwest corner of the lagoon where the sand berm is typically breached. Some accretion of sediment occurs on the eastern bank of the lagoon (see Chambers Group, 2016). An average bottom elevation of the lagoon was estimated to be approximately 4 ft NAVD88 from the Lagoon S cross-section (shown on Figure 2-1 as A-A').

2.5 Shallow Aquifer Groundwater Levels during Historical Hydrologic Cycles

Shallow aquifer groundwater levels from MW-2S are shown on Figure 8 from before the start of the long-term pumping test in June 2010, through present. The shallow aquifer groundwater levels are also shown in comparison to the average estimated lagoon bottom elevation of 4 ft NAVD88 and monthly precipitation from the Laguna Beach #2 precipitation gage. The water levels represent a combination of two sets of collected data from MW-2S that were combined to display shallow groundwater levels during the pumping test and post-pumping period. One dataset was collected and processed by GEOSCIENCE during slant well pumping test monitoring, and the other was collected and processed by South Coast Water District (SCWD) during post-pumping test monitoring.

As shown, groundwater levels in the shallow aquifer fluctuated above and below the average lagoon bottom elevation both during and after the long-term pumping test. Based on the observed fluctuations and local precipitation, changes in groundwater levels in MW-2S appear to correlate with periods of rainfall or no rainfall. At the start of the long-term pumping test, shallow aquifer water levels were near the elevation of the lagoon bottom. Just after the initiation of the test, water levels fall below the lagoon bottom elevation during the summer months. Water levels then rose above the lagoon bottom elevation in response to the increased precipitation that occurred from January 2011 through June 2011. Subsequently, water level in the shallow aquifer fell below the lagoon bottom elevation during the following dry season and continued to decline after the test slant well pump was shut off in April 2012 (2012 through 2014). Groundwater rose again following increased precipitation events in 2015 and 2016. However, despite the lower groundwater levels in the shallow aquifer during dry hydrologic conditions, review of aerial photos and land-based photos taken during monitoring events indicate surface water was still present in the lagoon during these times (Figure 9) which is likely due to low permeability of the materials (silt and clay) lining the lagoon bottom.

Based on the observed response of water levels to changes in precipitation, it appears that groundwater levels in the shallow aquifer in the vicinity of the San Juan Creek lagoon are primarily influenced by rainfall conditions.

2.6 Project Impacts during Wet, Dry, and Average Hydrologic Cycles

Figure 2 shows historical annual precipitation from 1928 through 2017, along with the calculated cumulative departure from the mean. The 70-year average precipitation is approximately 11.77 in/year during the period from 1928 through 2017. A downward slope on the cumulative departure curve indicates a less than average or “dry” conditions and upward slope shows periods during “wet” conditions. The long-term cumulative departure from the mean precipitation show that the study period during slant well pumping (June 2010 through April 2012) and post slant well pumping represented an overall dry hydrologic period which included the years 2005 - 2017.

A snapshot of the recent precipitation data, shown on Figure 2-3 (on the following page), suggests that the average precipitation over the past 13 years (which includes before and after test slant well pumping from 2005 – 2012) was only 6.84 in/year, which is nearly half of the long-term precipitation average of 11.77 in/year. The precipitation data shown on Figure 2-3 corresponds to the period for which groundwater level data was recorded, from 2005 to 2017, in the monitoring wells near the San Juan Creek lagoon.

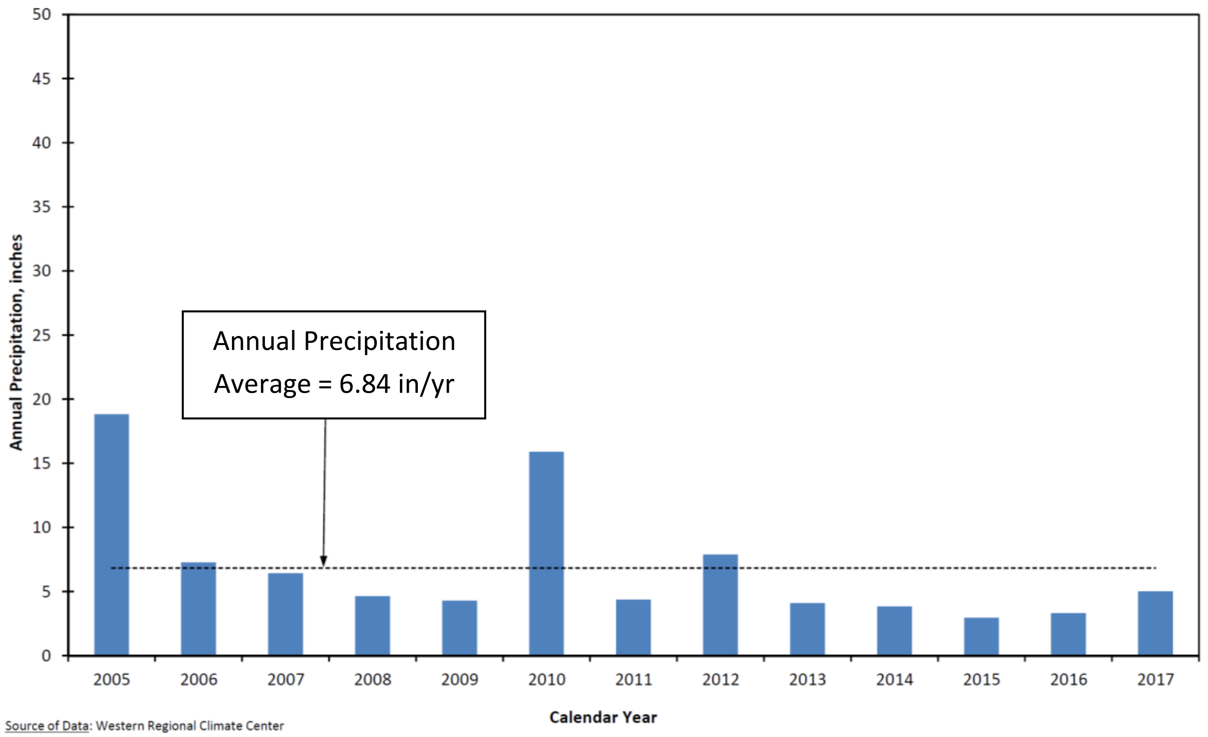

Figure 2-3. Annual Precipitation at Laguna Beach #2 Precipitation Gage (2005-2017)

Water levels in the shallow aquifer during the long-term pumping test and during the post-pumping period, which occurred during primarily dry hydrologic conditions (2005 to 2017), showed fluctuations that rose and fell above/below the average bottom elevation of the San Juan Creek lagoon. These fluctuations appear to be driven largely by local hydrologic cycles and precipitation patterns. However, even when shallow aquifer water levels fell below the lagoon bottom elevation in dry conditions, aerial imagery and field observations showed that water remained in the lagoon. When shallow groundwater levels are below the lagoon bottom, the water percolate from the lagoon in a "free fall" condition; that is, as water percolates into the subsurface it must percolate some distance before it reaches the water table. Therefore, the percolation rate is driven by the streambed hydraulic conductivity and not by depth to the groundwater elevation. As such, the degree of water level fluctuation expected under Project pumping conditions for the full-scale wellfield will not significantly affect surface outflow and lagoon levels as supported by the analyses reported in Table 2-1 and 2-2.

2.7 Changes in Annual Fish-Passage Days

2.7.1 San Juan Watershed Project

Environmental Science Associates (ESA) prepared a report entitled "San Juan Creek Fish Passage Assessment - Hydrologic Modeling Report (Three dam alternative)". The purpose of the analysis was to evaluate the impacts from the San Juan Watershed Project (SJWP) which will consist of the construction of three Rubber Dams on San Juan Creek between Stonehill Drive and just below the confluence of Trabuco Creek and San Juan Creek. ESA conducted an analysis with a surface water model using daily timesteps for hydrology between 1945 and 2014. ESA also used a hydraulic model of San Juan and Trabuco Creeks to identify flows at which average channel velocity and depths become barriers to adult upstream and downstream steelhead migration. The analysis assumed a minimum depth of 0.5 ft for swim-through cross-sections that do not require leaping. 7.9 feet/second was use for the average channel velocity parameter which is consistent with previous Trabuco Creek migration assessments (HDR, Inc., 2015).

ESA prepared a daily lagoon mouth closure model for the period 1945 to 2014. This model estimates when the mouth of San Juan Creek will open (from fluvial or wave overtopping) and close (from wave-caused beach buildup). Table 3 of the ESA analysis reports that the minimum flow to support a depth of 0.5 ft in the Reach 1 (between the ocean and halfway to Stonehill Drive) is 60 cfs. The same table reports that the minimum flow to support a depth of 0.5 ft in Reach 2 (Reach 1 to near Stonehill Drive is 70 cfs. Table 4 of the ESA report notes that the baseline modeled passage window for adult steelhead is 10.8 days for Reach 1 and 11.1 days for Reach 2. The SJWP will reduce the passage days to 9.7 for Reach 1 and 9.8 for Reach 2. However, the days for fish-passage for each reach must also consider conditions in all the other reaches. For Reaches 1 – 8 combined, the possible migration days from the ocean to Trabuco Creek above the dam pool for baseline and with project conditions is 8.7 days and 8.1 days, respectively. This is approximately an 8% reduction in passage days. The possible migration days from Reaches 1 – 9 combined, from the ocean to San Juan Creek above dam pool for baseline and with project is 9.1 days and 8.4 days, respectively This is also an 8% reduction in passage days.

2.7.2 Doheny Ocean Desalination Project

The impact to potential migration of steelhead is addressed using the data and methodology reported in the San Juan Creek Fish Passage Assessment (ESA, 2017). GEOSCIENCE used the model files from 2017 ESA report to assess the surface flow in Reaches 1 and 2 under the various conditions below:

- Baseline (no Project conditions),
- Baseline + Scenario 1 of the Doheny Ocean Desalination Project 10 MGD project,
- Baseline + Scenario 3 of the Doheny Ocean Desalination Project 20 MGD project,
- Baseline + Scenario 4 of the Doheny Ocean Desalination Project 30 MGD project,
- SJWP (conditions as reported in the 2017 ESA report),

- SJWP and Scenario 1 of the Doheny Ocean Desalination Project 10 MGD project,
- SJWP and Scenario 3 of the Doheny Ocean Desalination Project 20 MGD project, and
- SJWP and Scenario 4 of the Doheny Ocean Desalination Project 30 MGD project.

The surface outflow from the groundwater model was analyzed to determine the days that surface flow was equal to or exceeded 60 cfs in Reach 1 and 70 cfs in Reach 2 for all scenarios. The tables on the following pages summarizes surface flow in Reaches 1 and 2 under baseline conditions, Doheny Ocean Desalination Project conditions, SJWP conditions, and both project's cumulative conditions.

Table 2-3. Fish-Passage Days in Reaches 1 & 2 San Juan Creek

Scenario	Number of Days Daily Streamflow Exceeds 60 cfs[2]	Number of Days Daily Streamflow Exceeds 70 cfs[2]
	Reach 1	Reach 2
	Average for 1947-2014 (days/yr)	
Baseline	10.97	11.21
Scenario 1	10.76	11.13
Scenario 3	10.65	11.09
Scenario 4	10.63	11.04
SJWP	9.84	9.91
SJWP + Scenario 1	9.75	9.90
SJWP + Scenario 3	9.68	9.85
SJWP + Scenario 4	9.68	9.82

2 The ESA baseline numbers and baseline numbers generated for this analysis for Reaches 1 and 2 are slightly different because the ESA study considered all reaches (1 – 9) together in setting the baseline. This current study considers only the reaches affected by the Project (1 and 2).

Table 2-4. Reduction of Fish Passage Days from Baseline for the Doheny Ocean Desalination Project

Scenario	Reduced Number of Days Daily Streamflow Exceeds 60 cfs (Compared to Baseline)	Reduced Number of Days Daily Streamflow Exceeds 70 cfs (Compared to Baseline)
	Reach 1	Reach 2
	Average for 1947-2014 (days/yr)	
Scenario 1	0.21	0.08
Scenario 3	0.32	0.12
Scenario 4	0.34	0.17

Table 2-5. Reduction of Fish Passage Days with SJWP for the Doheny Ocean Desalination Project

Scenario	Reduced Number of Days Daily Streamflow Exceeds 60 cfs (Compared to SJWP)	Reduced Number of Days Daily Streamflow Exceeds 70 cfs (Compared to SJWP)
	Reach 1	Reach 2
	Average for 1947-2014 (days/yr)	
SJWP + Scenario 1	0.09	0.01
SJWP + Scenario 3	0.16	0.06
SJWP + Scenario 4	0.16	0.09

The results (Tables 2-3 through 2-5 above) show that the Doheny Ocean Desalination Project will reduce the potential fish-passage days from baseline in Reach 1 a maximum 0.34 days under Scenario 4 and 0.17 days in Reach 2 for the same Scenario. Table 2-5 shows the difference in fish passage days from that of the SJWP. For Reach 1 an additional 0.09 days for the 10 MGD project and 0.16 days for the 30 MGD project. For Reach 2 the difference in reduction in fish passage days form the SJWP ranges from 0.01 days for the 10 MGD project and 0.09 days for the 30 MGD project.

3.0 ASSESSMENT OF POTENTIAL IMPACTS FROM PROPOSED UPSTREAM BEDROCK BARRIER

During the hydrogeologic characterization of San Juan Creek, the elevation of bedrock beneath San Juan Creek was encountered in drill holes at a higher elevation than previously reported. The elevated bedrock area is below San Juan Creek in the area near Calle Jardin on the west and Naranja Road on the east. The existence of the elevated bedrock had been known historically as evidence by rising water at locations one to two miles from the coast (DWR, 1972). Historical gage data from the USGS surface water gage (11047350) at Stonehill Drive has shown additional flow in San Juan Creek with respect to upstream areas which has been attributed to rising water. As an example, Figure 3-1 below dated February 28, 2007 shows water rising to the surface in the area of elevated bedrock and infiltrating back into the aquifer downstream of Stonehill Drive. During very wet hydrologic seasons, rising water in San Juan Creek at this location can provide significant additional surface flow in San Juan Creek to the stream reaches downstream of Stonehill Drive.

Figure 3-1. Rising Water in San Juan Creek – Upstream of Stonehill Drive

An ancient landslide is mapped by the USGS (Tan, 1999) on the hillside east and north of San Juan Creek at Stonehill Drive and beneath the San Juan Creek. A geologic cross-section prepared by the USGS shows the landslide moved westward into San Juan Creek during a period when the base level of the creek was much lower due to lower sea level. However, the USGS mapping does not show the landslide extending to San Juan Creek. The landslide appears to have failed along bedding planes in the Capistrano Formation which would make it difficult to distinguish landslide deposits from the underlying in-place Capistrano

Formation in borings. Furthermore, with rising sea levels, San Juan Creek backfilled the valley with alluvial sediments effectively burying the elevated bedrock or landslide deposits.

The San Juan Basin Authority contracted WEI to perform an investigation to assess the extent of elevated bedrock and to determine whether elevation of the bedrock was sufficiently high to potentially act as a barrier to groundwater flow in the alluvial materials which compose the San Juan Basin aquifer. The investigation consisted of drilling sonic boreholes and collecting continuous core to accurately delineate the bedrock elevation, installing new monitoring wells at strategic locations to provide additional water level information, preparing geologic cross-sections to illustrate the distribution of bedrock, and conducting two pumping tests to assess whether the elevated bedrock would act as a barrier or boundary to pumping water levels. The results of the exploratory drilling and monitoring well installation completed as part of the “Bedrock Barrier Investigation” have validated the existence and better delineated the extent of elevated bedrock in the area. The cross-section below was prepared by WEI from the data collected during the barrier investigation. The results of the investigation show that elevated bedrock area is overlain by a minimum of 41 feet of alluvium in the cross-sectional area across San Juan Creek. Along this cross-section, the paleochannel is filled with aquifer material to depths ranging from 78 feet to 94 feet below ground surface. Therefore, the aquifer is continuous from upstream of the elevated bedrock area to the area downstream. The cross-section is located across the San Juan Creek along the alignment of Profile 2-2’ shown on Figure 10.

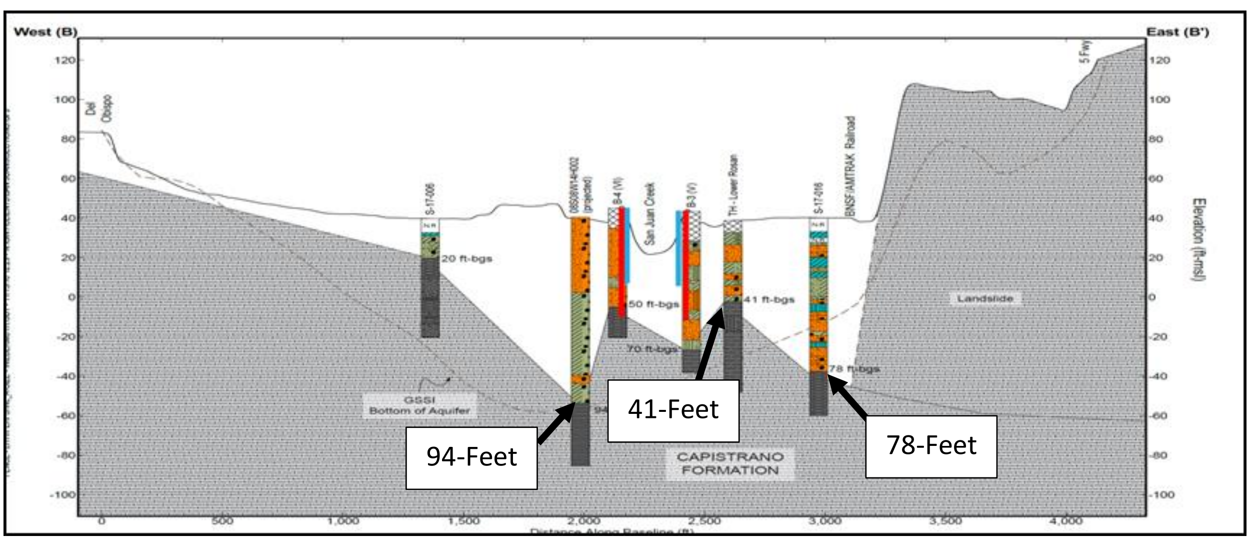

Figure 3-2. Cross-Section through the Elevated Bedrock Area - San Juan Creek near Calle Jardin and Naranja Road.

Pumping tests were conducted in wells located both upstream and downstream of the elevated bedrock area. Neither pumping test detected a barrier effect form the existence of elevated bedrock. It is likely that the pumping test, were not long enough, but more likely that the groundwater levels could not be lowered deep enough for the elevated bedrock to act as a barrier.

3.1 Assessment of Cumulative Impacts from the Potential Existence of Elevated Bedrock Beneath San Juan Creek

DEIR commenters have recommended an assessment of cumulative impacts from SJWP and Doheny Ocean Desalination Project due to a potential bedrock "barrier" located upstream of the slant well field located at Doheny Beach. To ensure that cumulative impacts from SJWP and Doheny Ocean Desalination Project were modeled correctly in combination, Wildermuth Environmental Inc., (WEI) provided the model files that they used to analyze groundwater changes for the SJWP EIR analysis. The cumulative impacts from the Doheny Ocean Desalination and SJW projects were evaluated using WEI model and model files. The model files provided by WEI included the results of Phase 1A and Phase 1C. For this analysis, GEOSCIENCE added the Doheny Ocean Desalination Project Scenarios 1, 3, and 4 operations to the WEI model. As stated, the WEI investigation has confirmed the existence of the elevated bedrock. However, the aquifer remains continuous over the elevated bedrock and as such groundwater is not isolated across the elevated bedrock. For the elevated bedrock area to act as a groundwater barrier, groundwater levels would have to be lowered below a depth of 94 feet to isolate groundwater upstream from groundwater downstream. Historical low groundwater levels have never been recorded below 35 feet above the elevated bedrock surface (See Figure 3-2), Never-the-less, impacts from the Doheny Ocean Desalination Project cumulative to those from the SJWP were assessed. The analysis was conducted using the WEI model output files which included SJWP Phase 1A (three rubber dams) and SJWP Phase 1C (three rubber dams and a new pumping well upstream of Stonehill Drive). GEOSCIENCE added the Doheny Ocean Desalination Project Scenarios 1, 3, and 4 to the SJWP to assess the cumulative impacts to groundwater levels beneath San Juan Creek along four cross-sectional profiles (see Figure 10). Profile 1 is near the SJBA Kinoshita Well. Profile 2 is through the elevated bedrock area. Profile 3 is through San Juan Creek at SCWD's GRF well located south of Stonehill Drive and Profile 4 is located at the coast.

Figures 11 through 22 show the predicted groundwater levels with and without the SJWP for Scenario 1, 3, and 4 under high (1998) and low (2014) groundwater level conditions. With respect to the elevated bedrock area, under maximum pumping conditions and under historically low groundwater conditions, groundwater levels remain 22 feet above the top of the elevated bedrock and adjacent paleochannel areas contain aquifer materials that maintain a saturated thickness of 42 ft. Therefore, the elevated bedrock does not affect the cumulative groundwater level responses from the SJWP in combination with Doheny Ocean Desalination Project.

4.0 SUMMARY AND CONCLUSIONS

- Both the creek outflow and the shallow aquifer near the lagoon are affected primarily by hydrologic conditions (i.e., precipitation patterns).

- During periods of low precipitation (dry hydrologic conditions), water levels in the shallow aquifer generally fall below the average estimated lagoon bottom elevation – both during pumping conditions and in the absence of slant well pumping.

- Even during dry conditions when groundwater levels in the shallow aquifer fall below the lagoon bottom during No Project (no pumping) and Project (pumping) conditions, water is still present in the lagoon. In other words, when groundwater levels are below the bottom of the lagoon as they are seasonally, the lagoon water levels become independent of the surface flow or standing water in the lagoon. The relationship of water in the lagoon and slant well pumping can be described in terms of volumes diminish until the groundwater levels are below the bottom of the lagoon.

- When groundwater levels in the shallow aquifer fall below the lagoon/river bottom, surface water level in the lagoon is controlled by the hydraulic conductivity of the underlying sediments and is independent of groundwater levels (i.e., “free fall” conditions).

- During periods of high precipitation (wet hydrologic conditions) groundwater levels in the shallow aquifer generally rise above the lagoon bottom.

- Additional seepage from the lagoon and streambed upgradient of the lagoon occurs under Project pumping conditions. However, decreases in San Juan Creek streamflow from Project pumping correspond to approximately 0.6 to 0.8 percent of the baseline outflow under Scenario 1, 0.8 to 1.1 percent under Scenario 3, and 0.9 to 1.3 percent under Scenario 4 conditions.

- The total change to the annual fish passage days for the SJWP from baseline conditions are 1.1 to 1.3 for Reaches 1 and 2 respectively.

- The combined (SJWP and Doheny Ocean Desalination Projects) changes to annual fish-passage days through Reach 1 is 9.75 days for the 10 MGD project and 9.68 days for the 30 MGD project. from a baseline of 10.97 days. For Reach 2 the combined reduction in fish passage days ranges from 9.9 days for the 10 MGD project and 9.82 days for the 30 MGD project from a baseline of 11.21 days.

- Investigations have concluded the existence of an elevated bedrock surface upstream of Stonehill Drive that is overlain with a minimum of 41 ft and a maximum 94 ft of aquifer material. The top of the elevated bedrock is at about -20 ft above mean sea level (amsl)

- The elevated bedrock within San Juan Creek may represent a landslide deposited when sea level was lower.

- Historical groundwater levels in San Juan Creek have never been recorded lower than 35 ft. The pumping tests conducted during the “Bedrock Barrier Investigation” did detect the influence of the elevated bedrock.

- Under maximum pumping and historically low groundwater conditions, groundwater levels remain 22 feet above the top of the elevated bedrock. The deepest portion of the paleochannel present on both sides of the elevated bedrock also remains saturated with a thickness of 42 ft. Therefore, the elevated bedrock does not affect the cumulative groundwater level responses from both the SJWP and Doheny Ocean Desalination Projects under the maximum conditions.

- The Doheny Ocean Desalination Project will result in changes to annual fish-passage days from a baseline of 10.97 days to 10.76 (0.21 days) for the 10 MGD project and 10.63 days (0.34 days) for the 30 MGD project. In Reach 2 fish passage days will be reduced from a baseline of 11.21 days to 11.13 days ((0.08 days) for the 10 MGD project and 11.04 days (0.17 days) for the 30 MGD project.

5.0 REFERENCES

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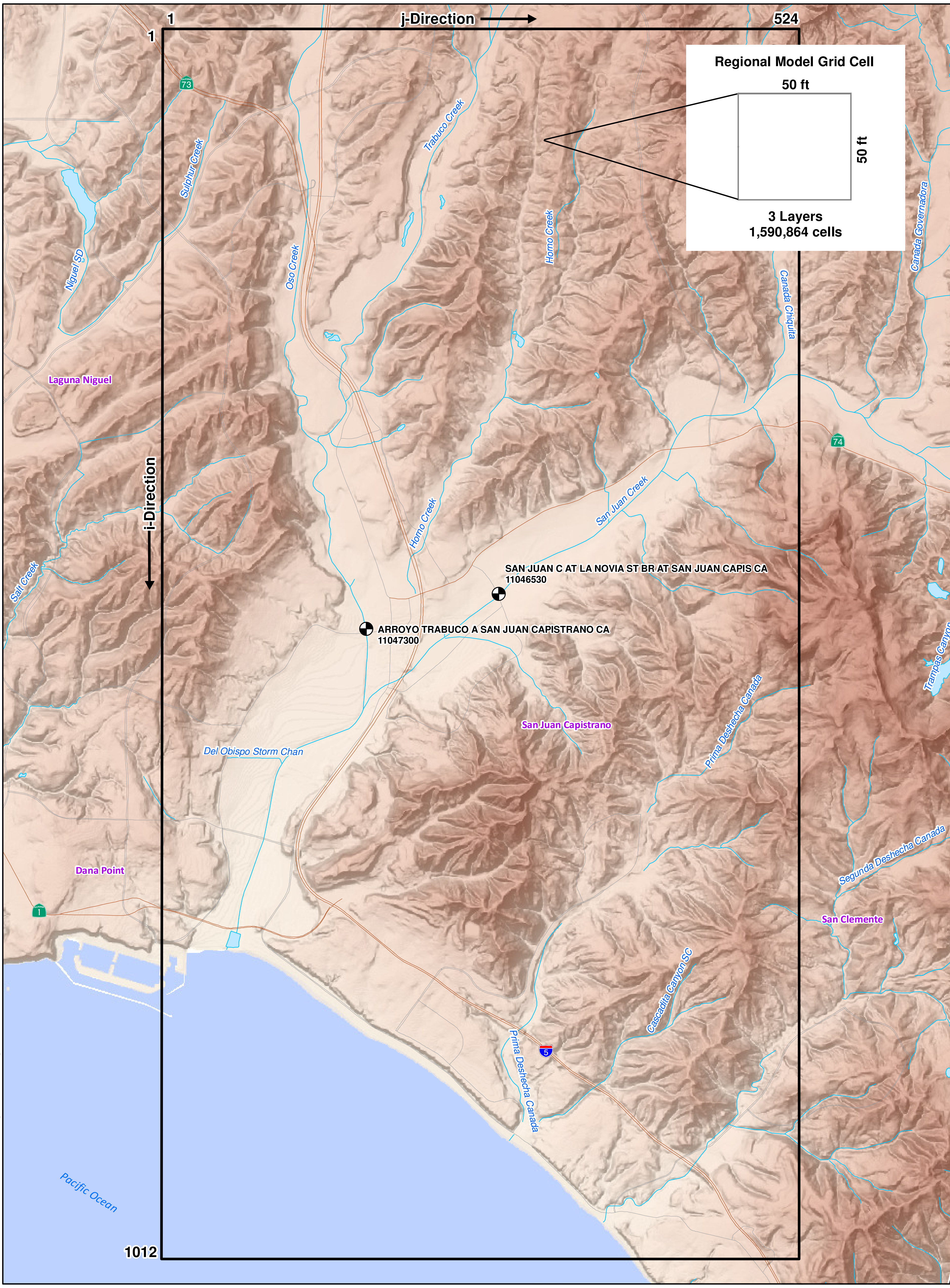

San Juan Basin Regional Groundwater Model Boundary

Gaging Station Name and Designation

SAN JUAN BASIN REGIONAL GROUNDWATER MODEL

19-Nov-18

Prepared by: DB. Map Projection: State Plane 1983, Zone VI.

NORTH

0 2,000 4,000 8,000 Feet

GEOSCIENCE
GEOSCIENCE Support Services, Inc.
P.O. Box 220, Claremont, CA 91711
Tel: (909) 451-6650 Fax: (909) 451-6638
www.gssiwater.com

FIGURE 1

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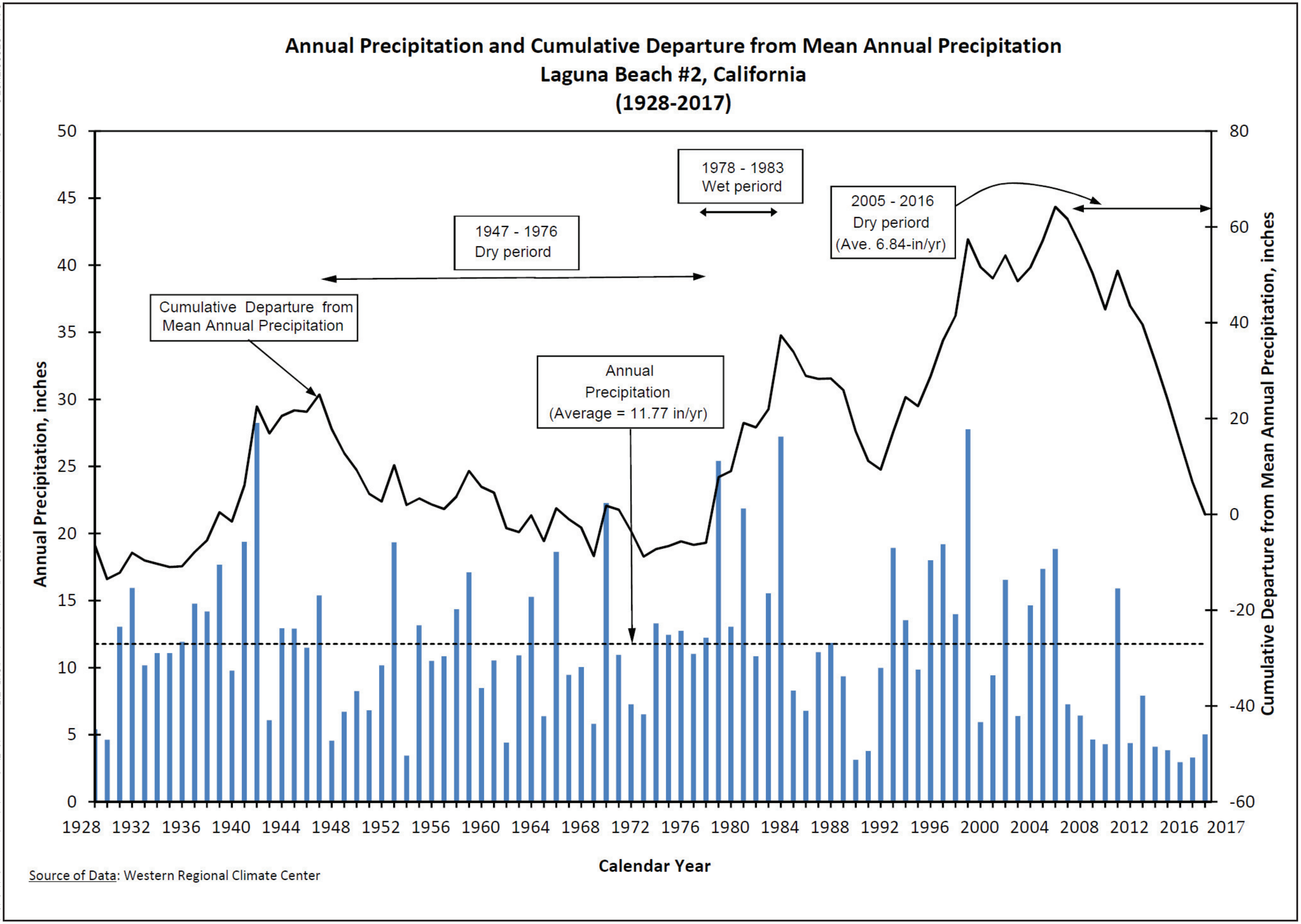


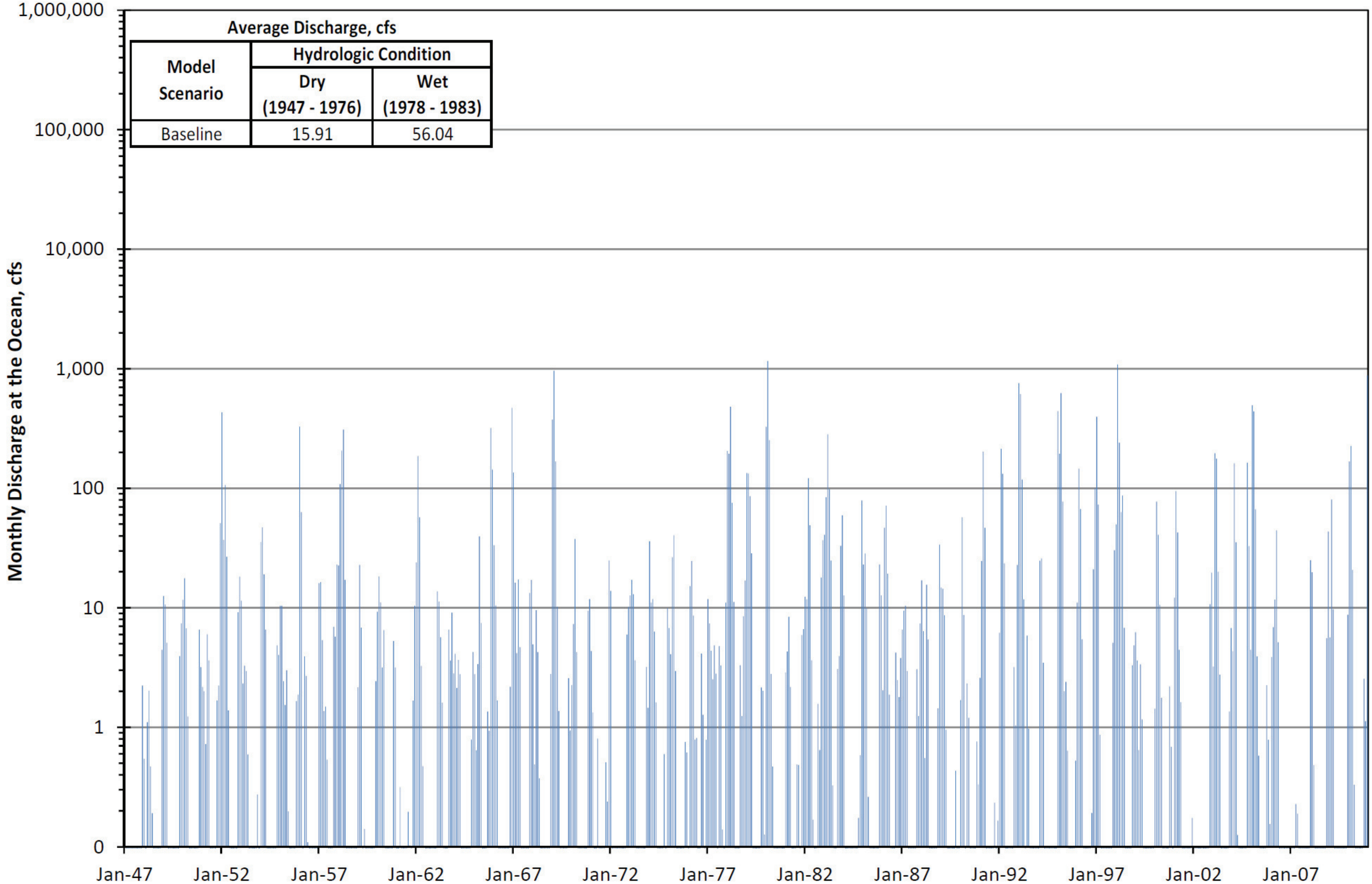
7-Mar-19

SOUTH COAST WATER DISTRICT

FIGURE 2

GEOSCIENCE

Model-Calculated Monthly Discharge at the Ocean
Baseline



Average Discharge, cfs

Model Scenario	Hydrologic Condition	
	Dry (1947 - 1976)	Wet (1978 - 1983)
Baseline	15.91	56.04

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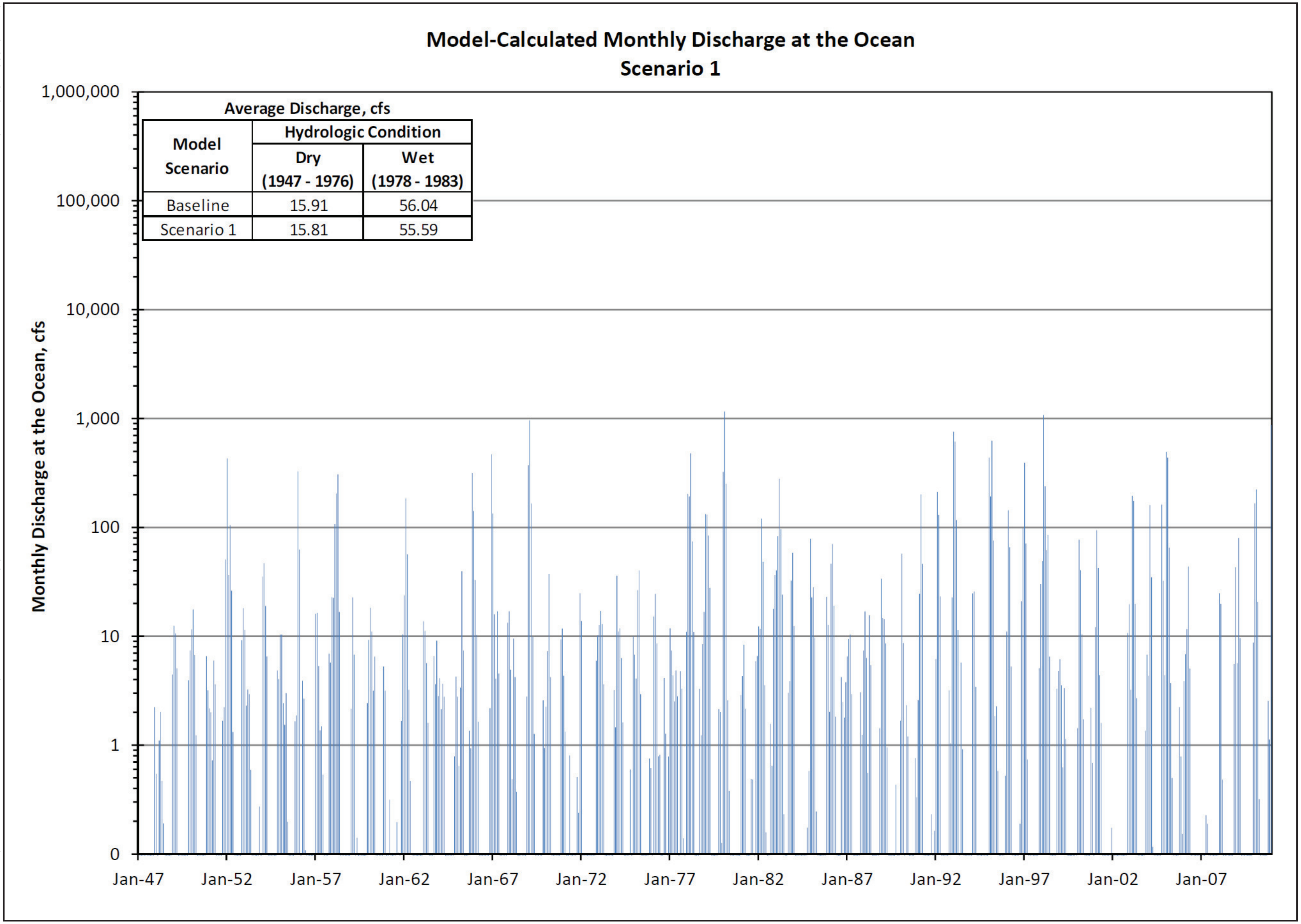
7-Mar-19



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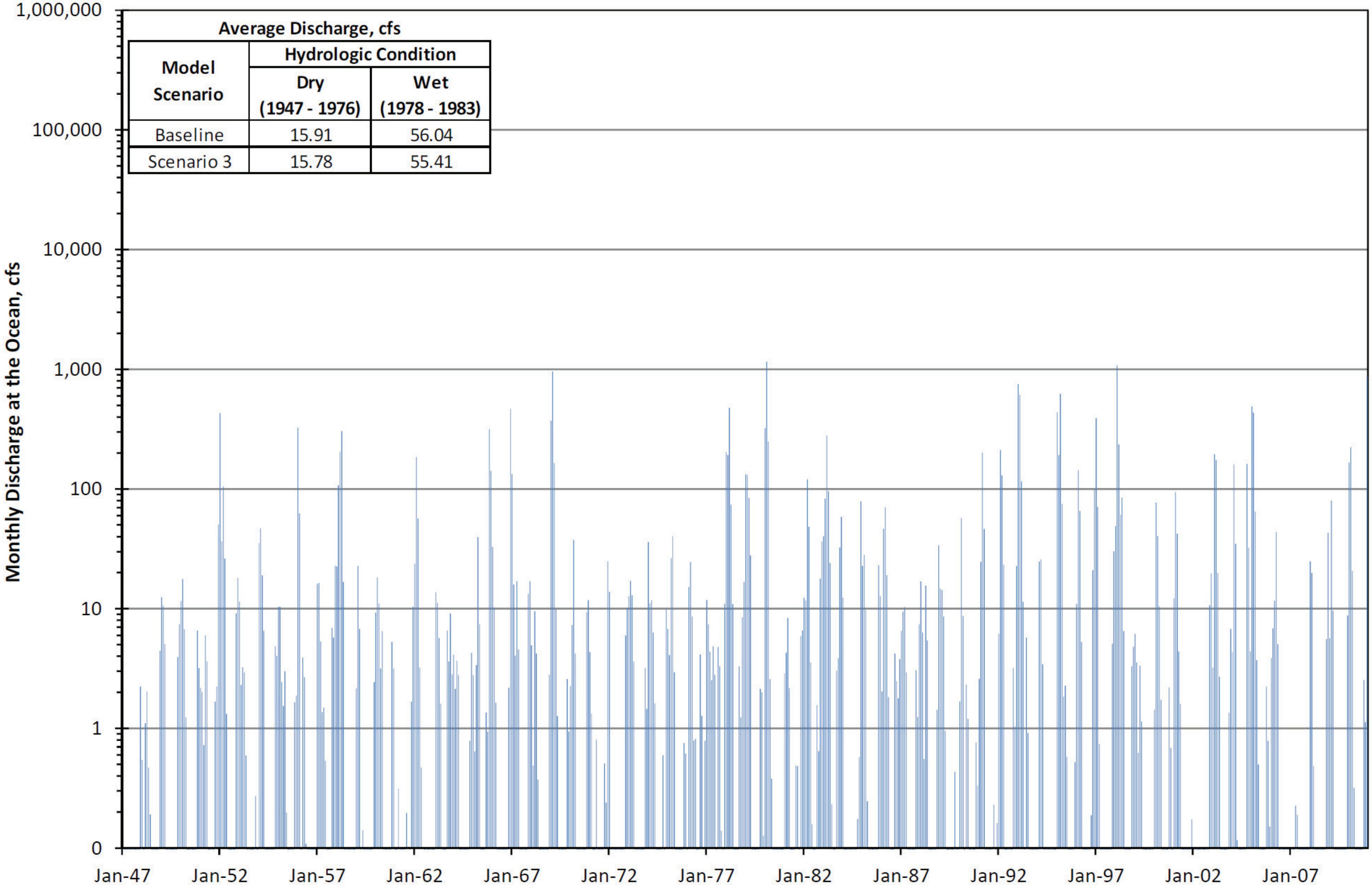
7-Mar-19

SOUTH COAST WATER DISTRICT

FIGURE 4

GEOSCIENCE

Model-Calculated Monthly Discharge at the Ocean
Scenario 3



Average Discharge, cfs

Model Scenario	Hydrologic Condition	
	Dry (1947 - 1976)	Wet (1978 - 1983)
Baseline	15.91	56.04
Scenario 3	15.78	55.41

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SOUTH COAST WATER DISTRICT

FIGURE 5

GEOSCIENCE

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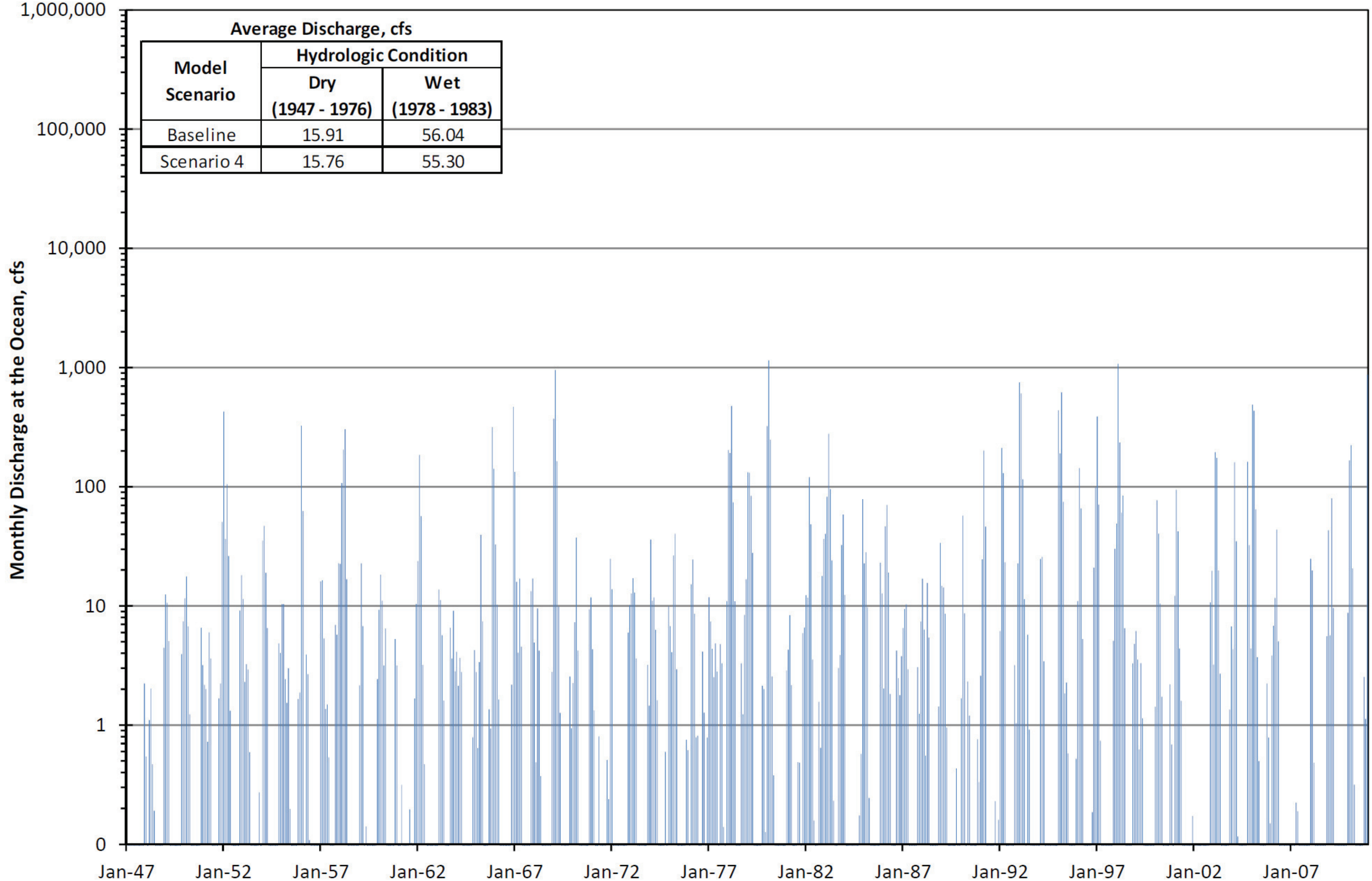
Model-Calculated Monthly Discharge at the Ocean
Scenario 4



Average Discharge, cfs

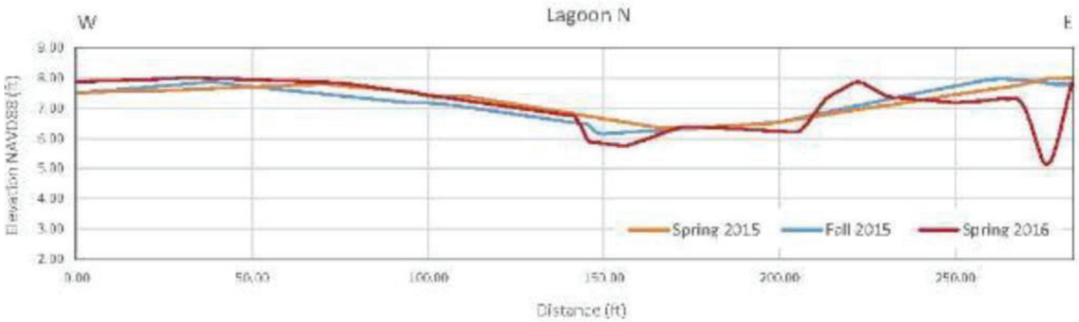
Model Scenario	Hydrologic Condition	
	Dry (1947 - 1976)	Wet (1978 - 1983)
Baseline	15.91	56.04
Scenario 4	15.76	55.30

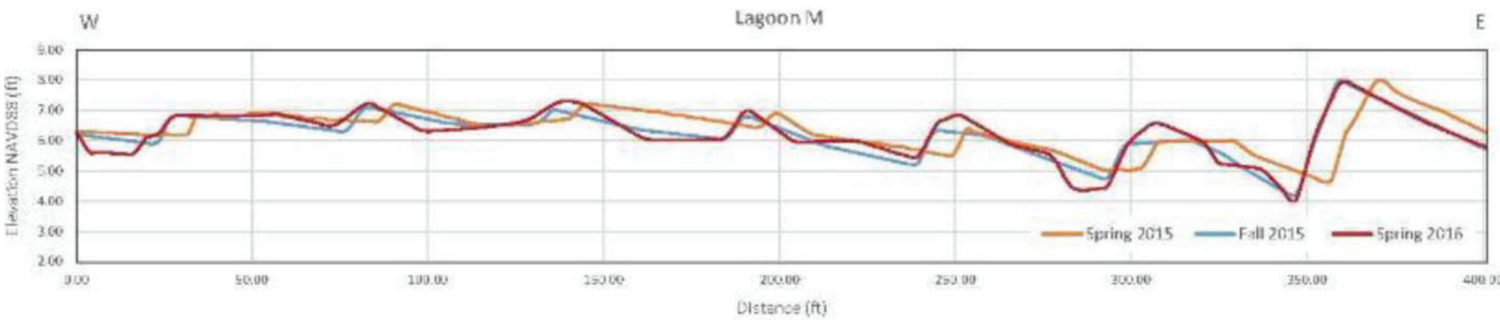
7-Mar-19

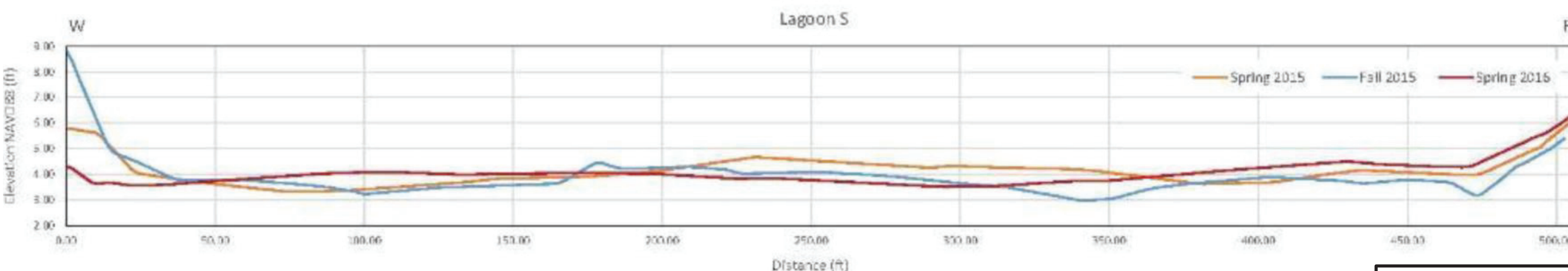
SOUTH COAST WATER DISTRICT

FIGURE 6

GEOSCIENCE







SOURCE: FIGURE 2-14 OF CHAMBERS GROUP, 2016

LAGOON BOTTOM PROFILES

7-Mar-19

SOUTH COAST WATER DISTRICT/GHD

FIGURE 7

GEOSCIENCE

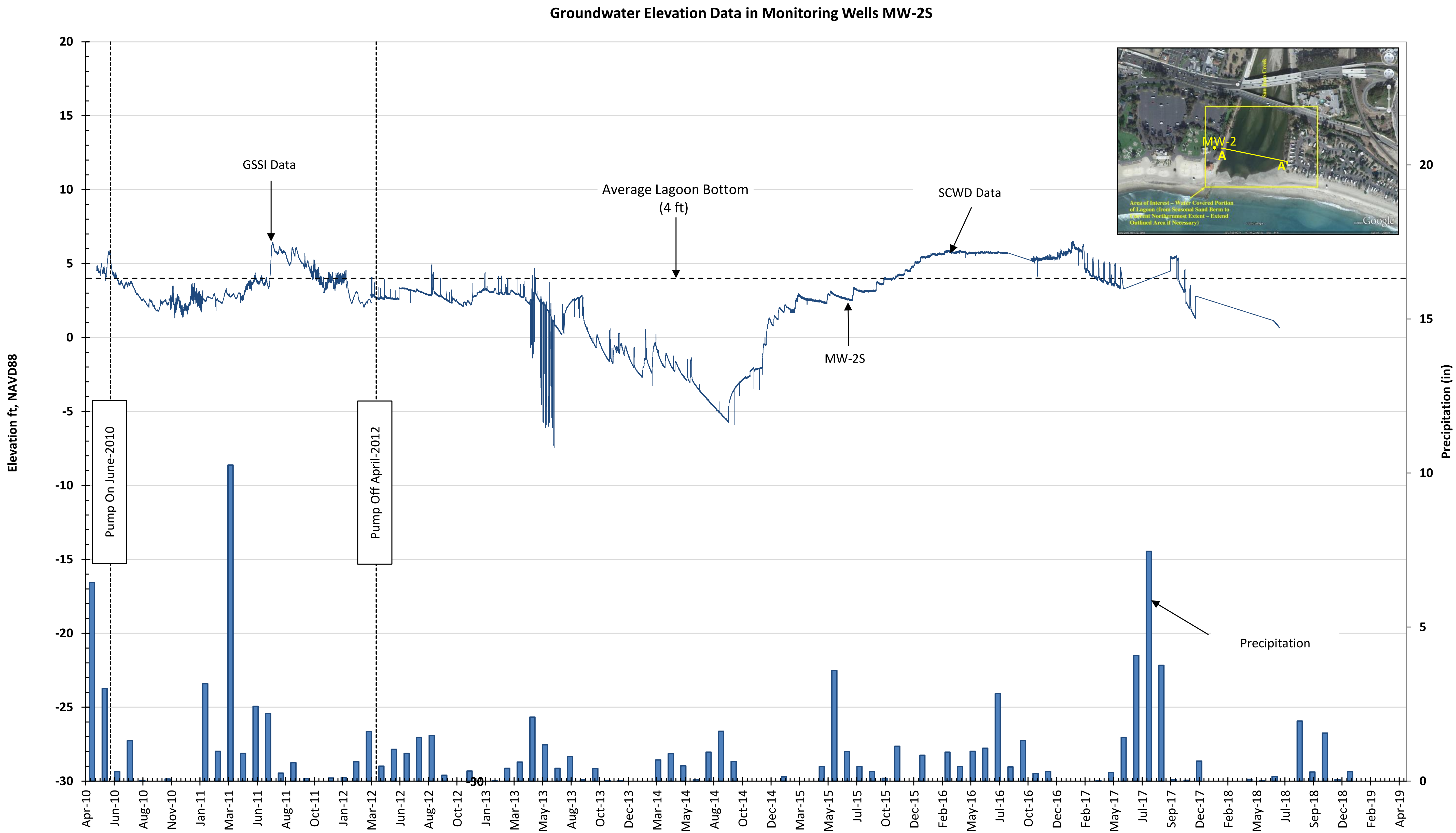

Groundwater Elevation Data in Monitoring Wells MW-2S
Elevation ft, NAVD88
Precipitation (in)
GSSI Data
Average Lagoon Bottom
(4 ft)
SCWD Data
MW-2S
Precipitation
Pump On June-2010
Pump Off April-2012
MW-2
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Figure 8

Groundwater Elevation Data in Monitoring Wells MW-2S
(Long-Term Pump Test)

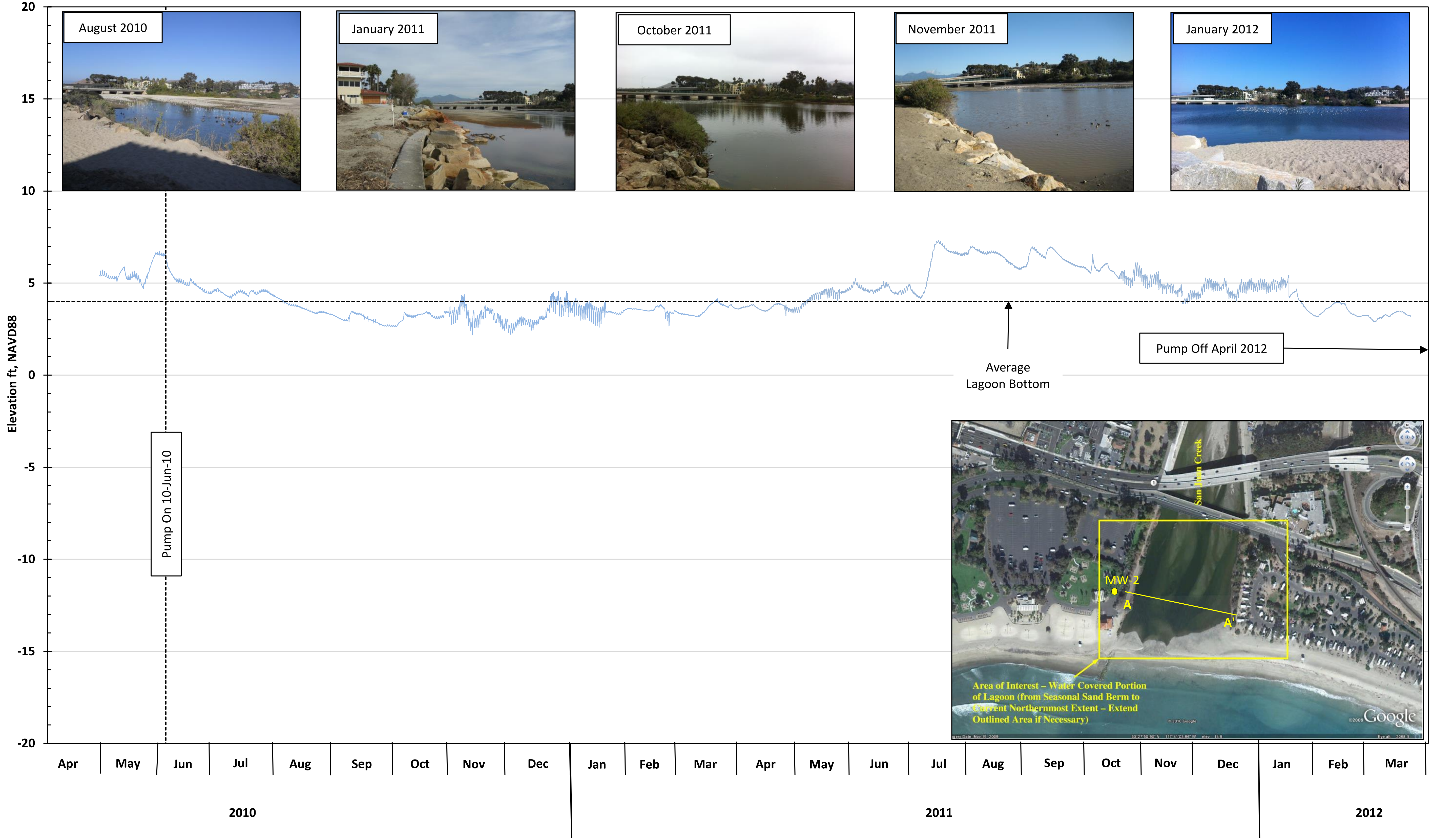

Figure 9

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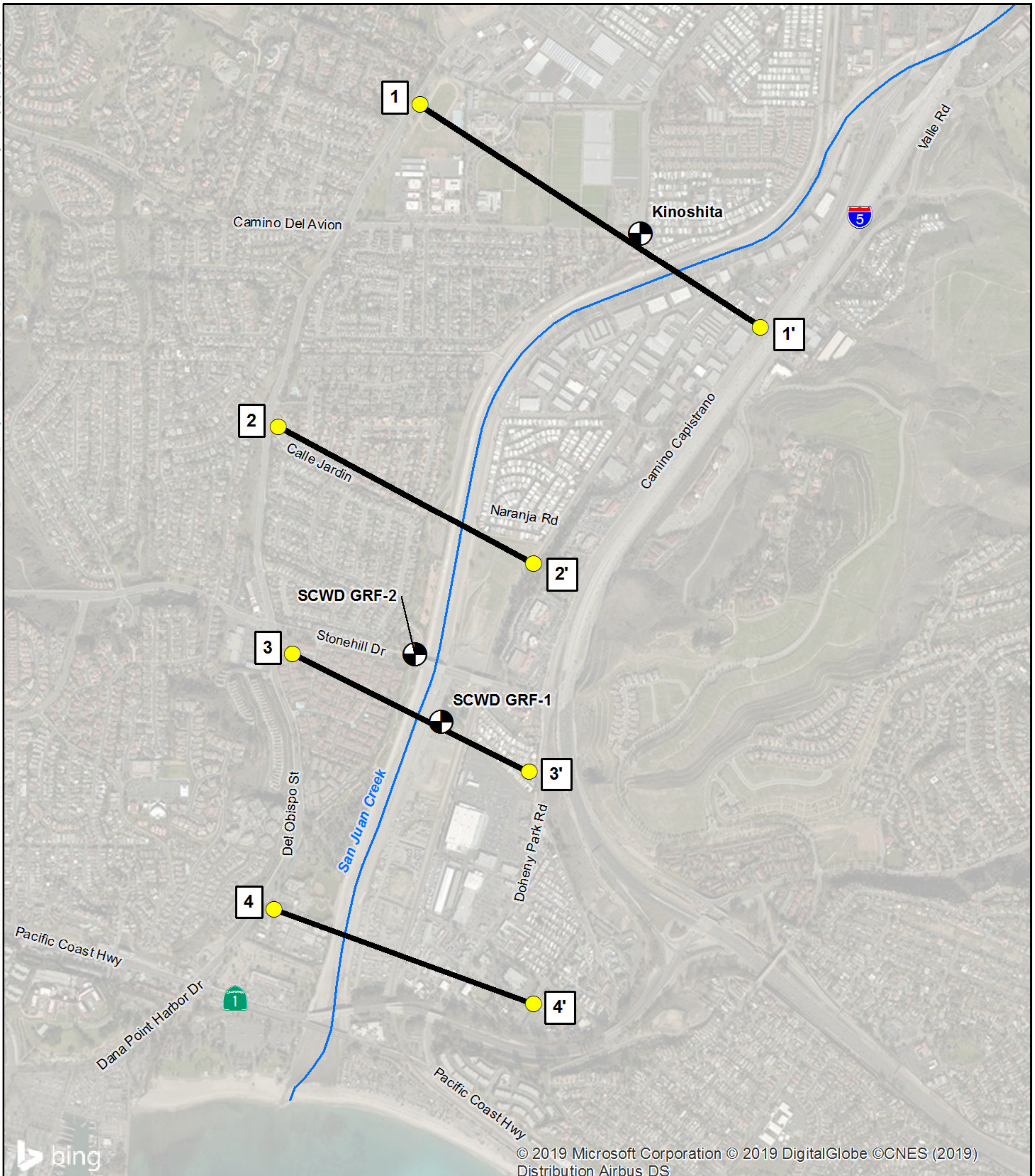


EXPLANATION

1 — 1' Profile Location

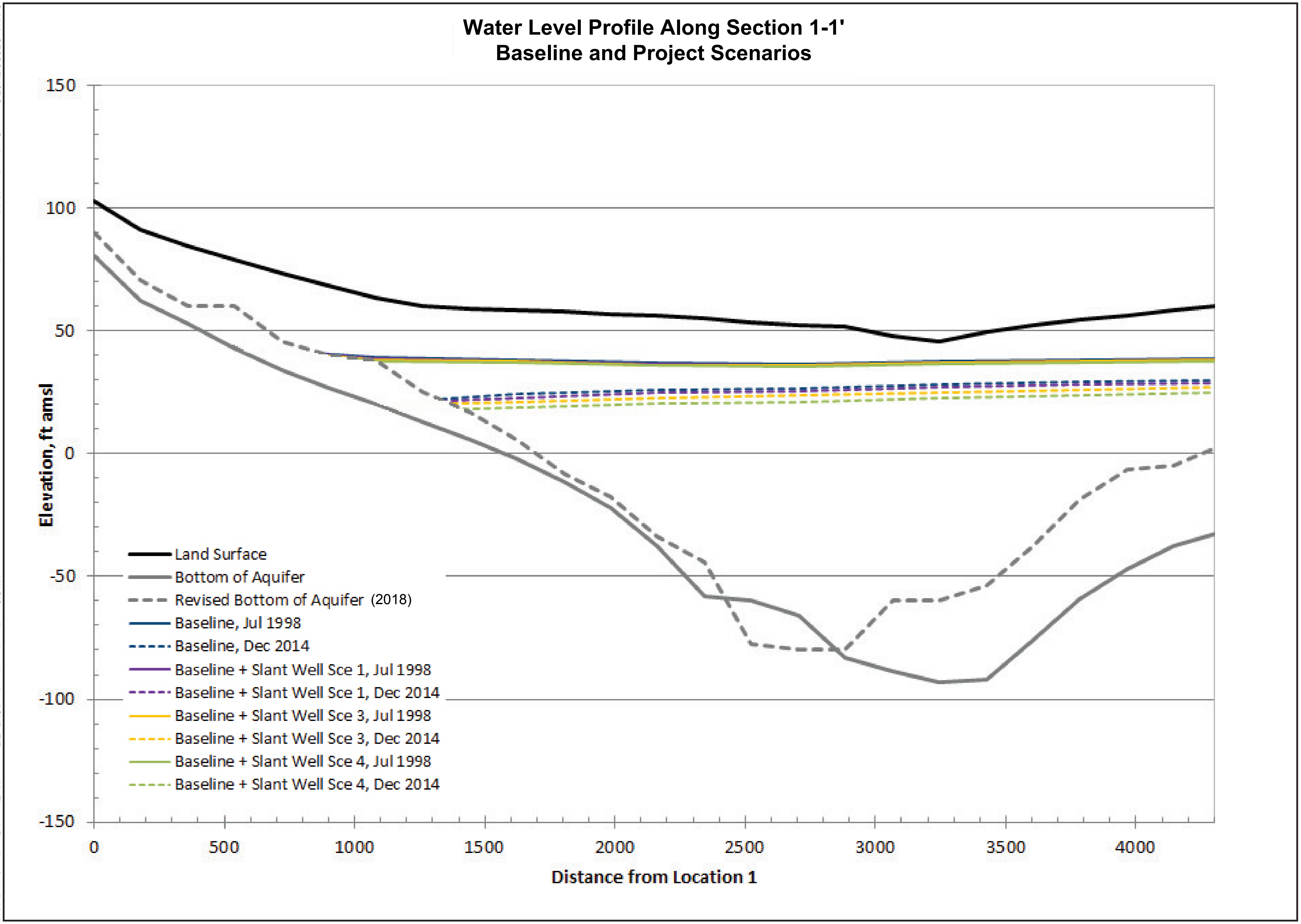
N

0 750 1,500

Feet

7-Mar-19

SAN JUAN CREEK PROFILE LOCATIONS



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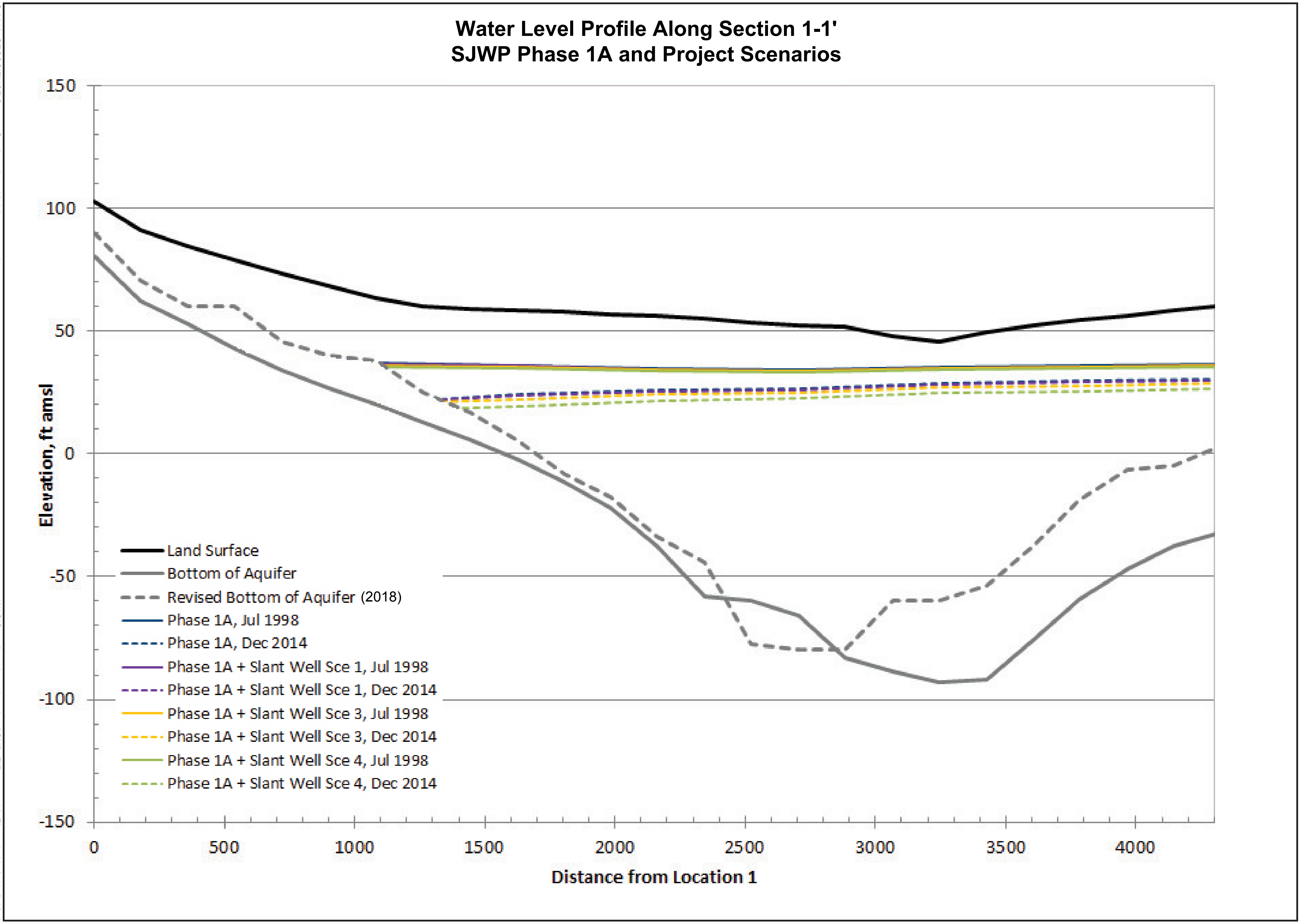
SOUTH COAST WATER DISTRICT

FIGURE 11

GEOSCIENCE



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SOUTH COAST WATER DISTRICT

FIGURE 12

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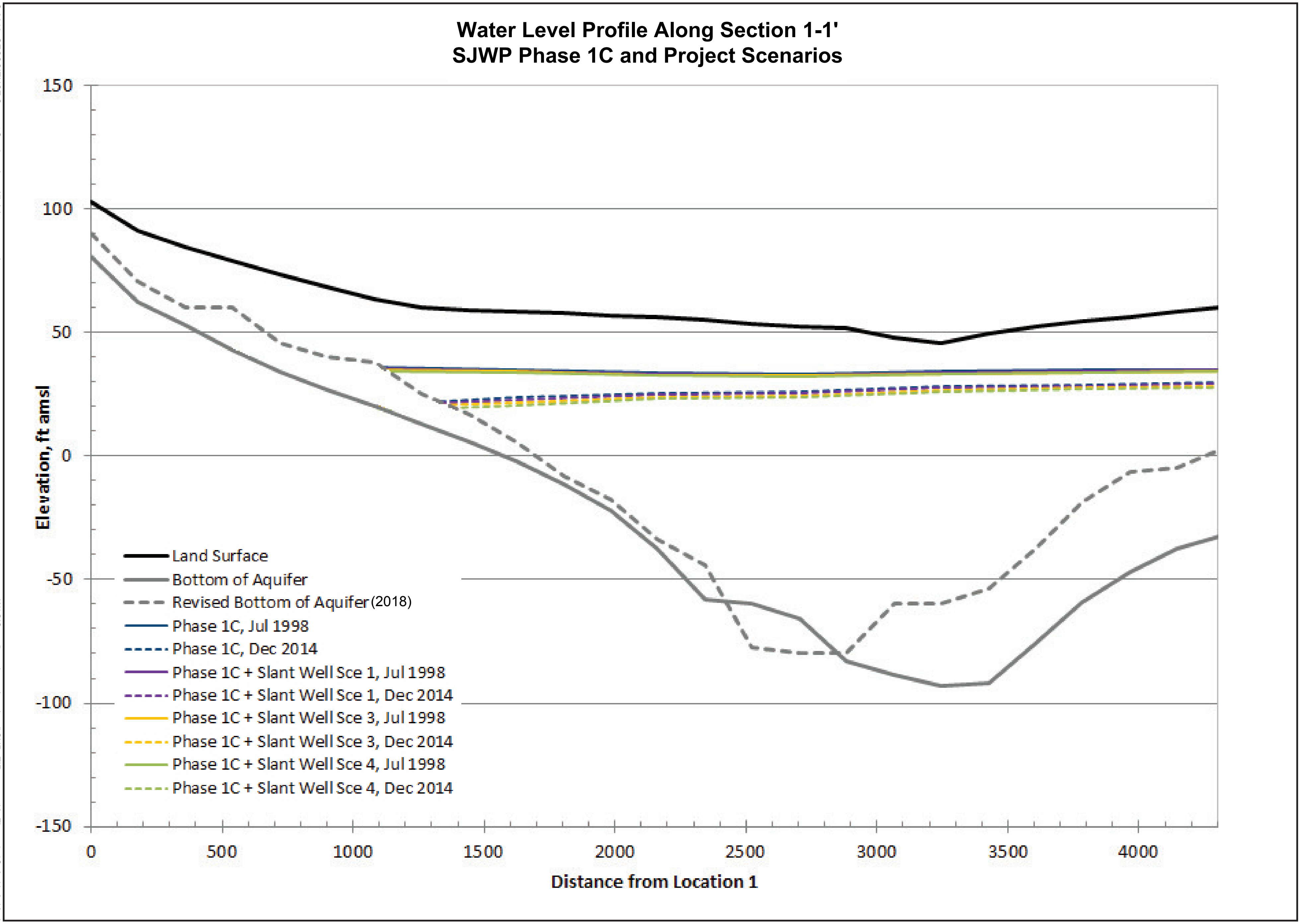

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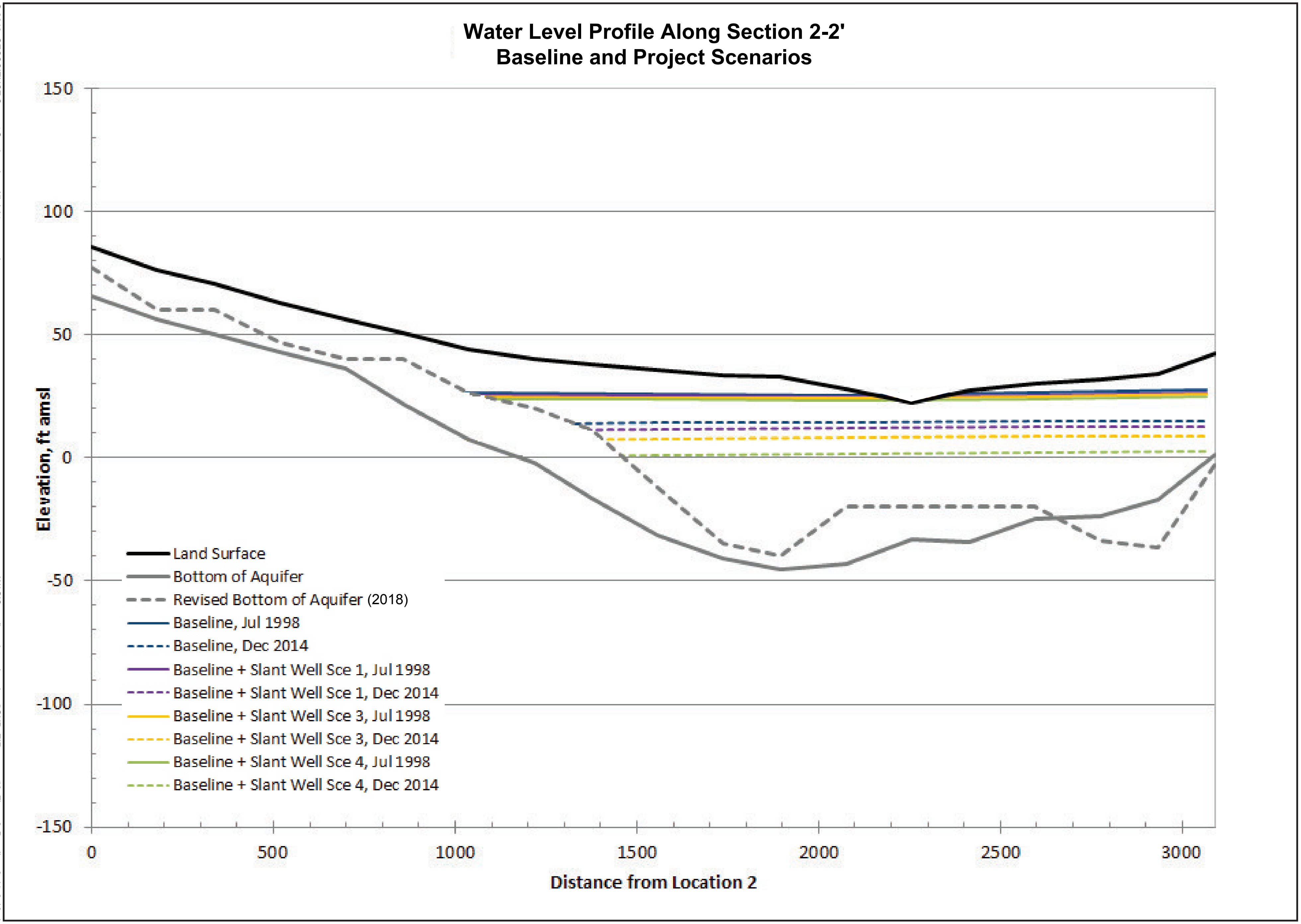
SOUTH COAST WATER DISTRICT

FIGURE 13

GEOSCIENCE



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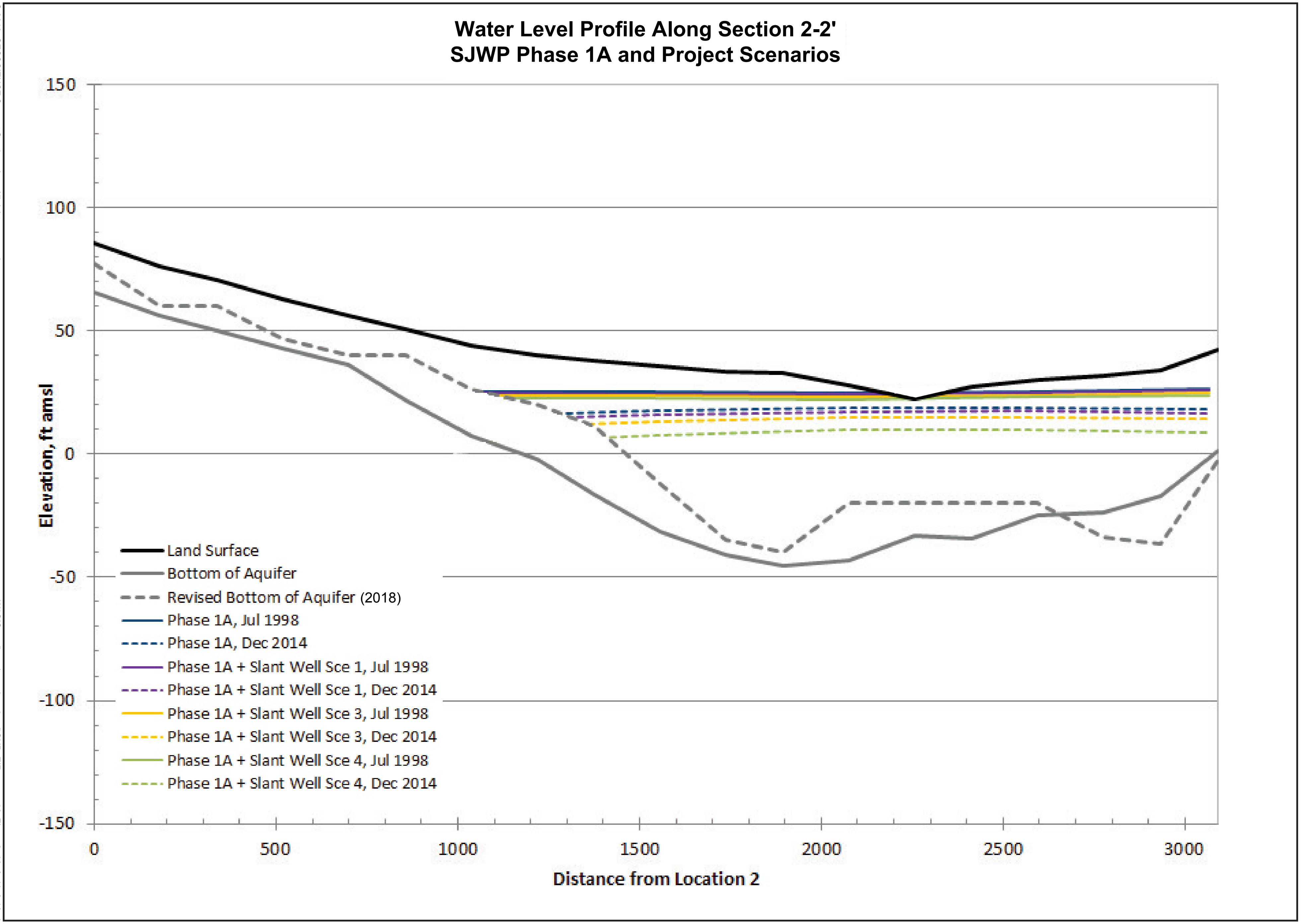
SOUTH COAST WATER DISTRICT

FIGURE 14

GEOSCIENCE



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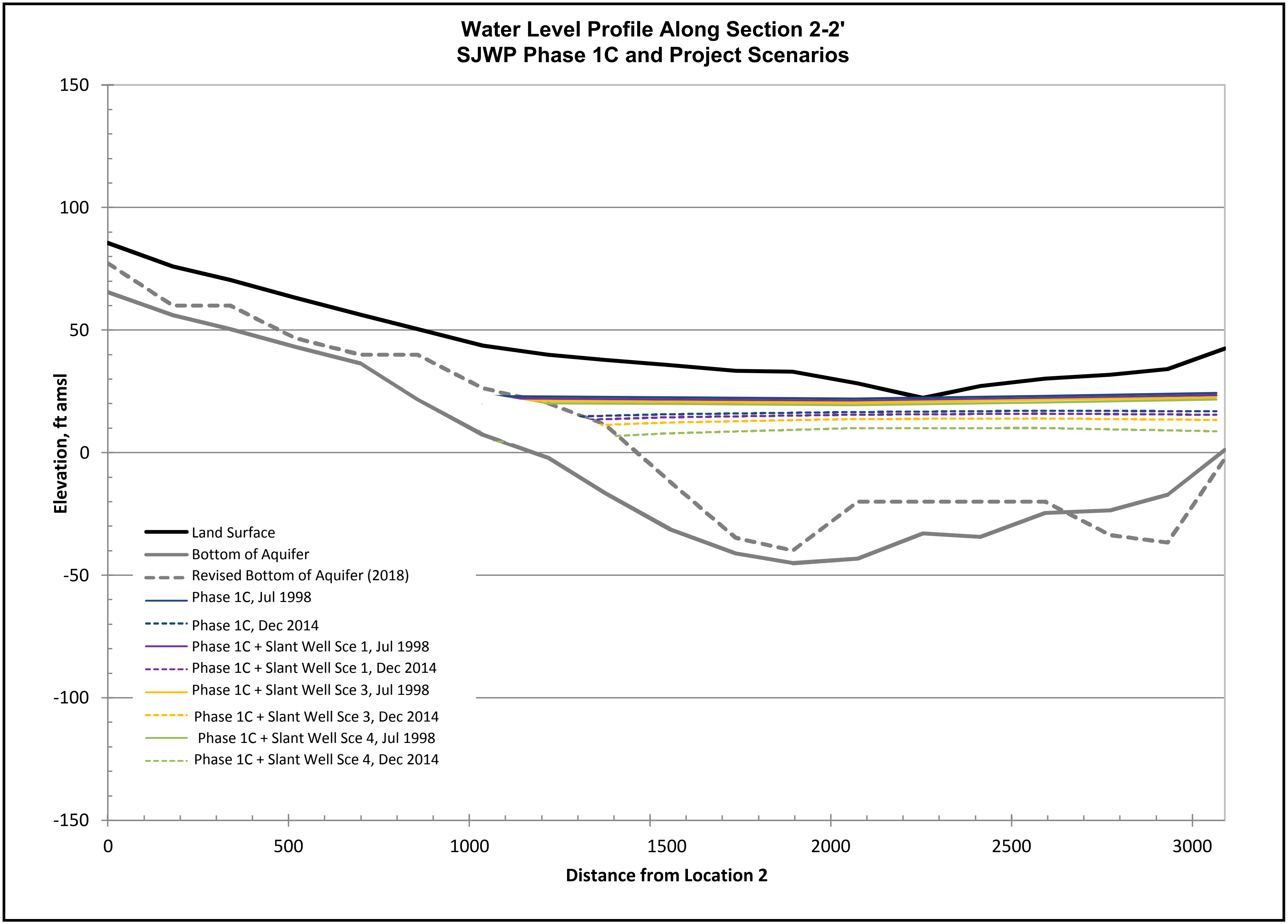
SOUTH COAST WATER DISTRICT

FIGURE 15

GEOSCIENCE



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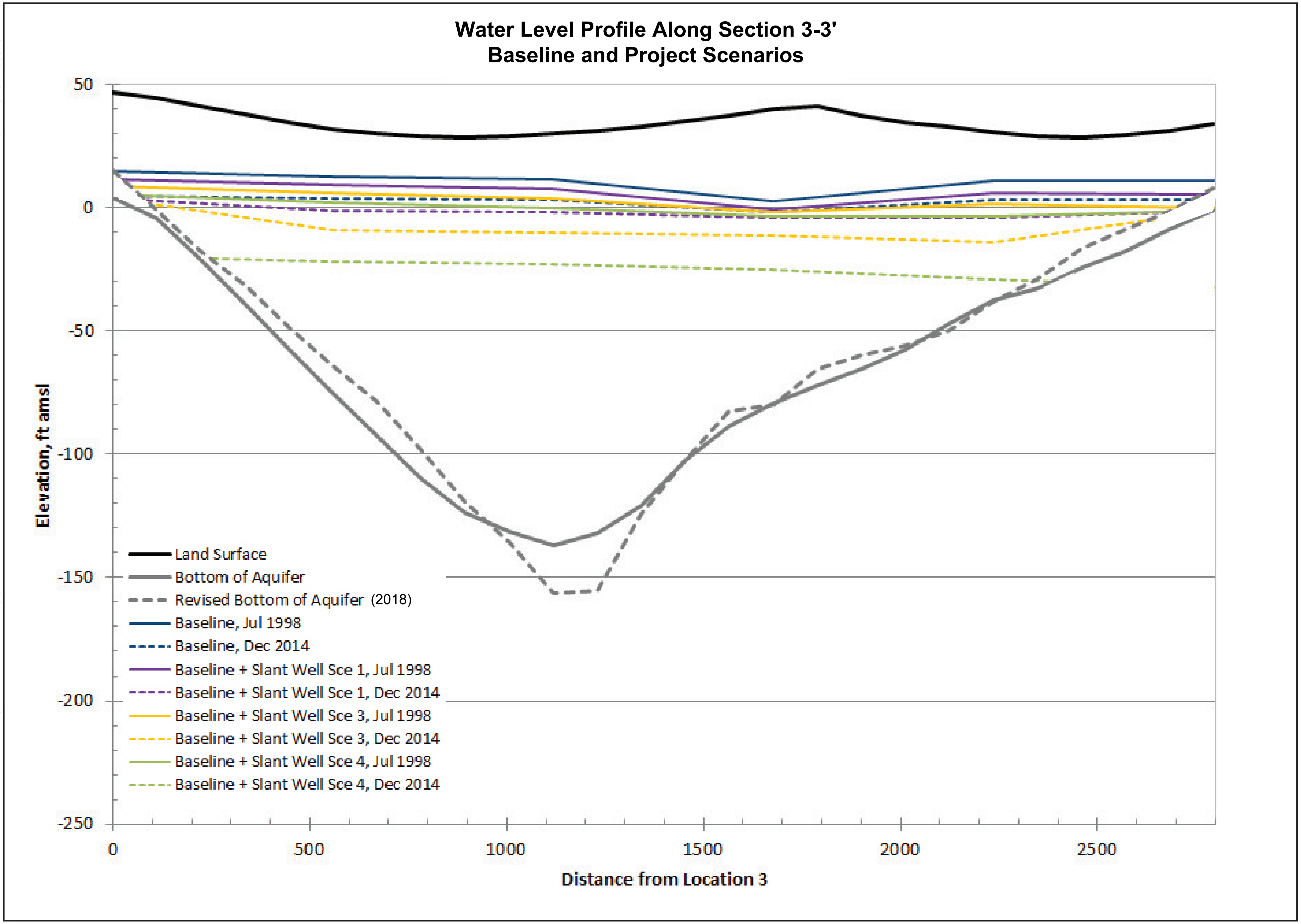
SOUTH COAST WATER DISTRICT

FIGURE 16

GEOSCIENCE



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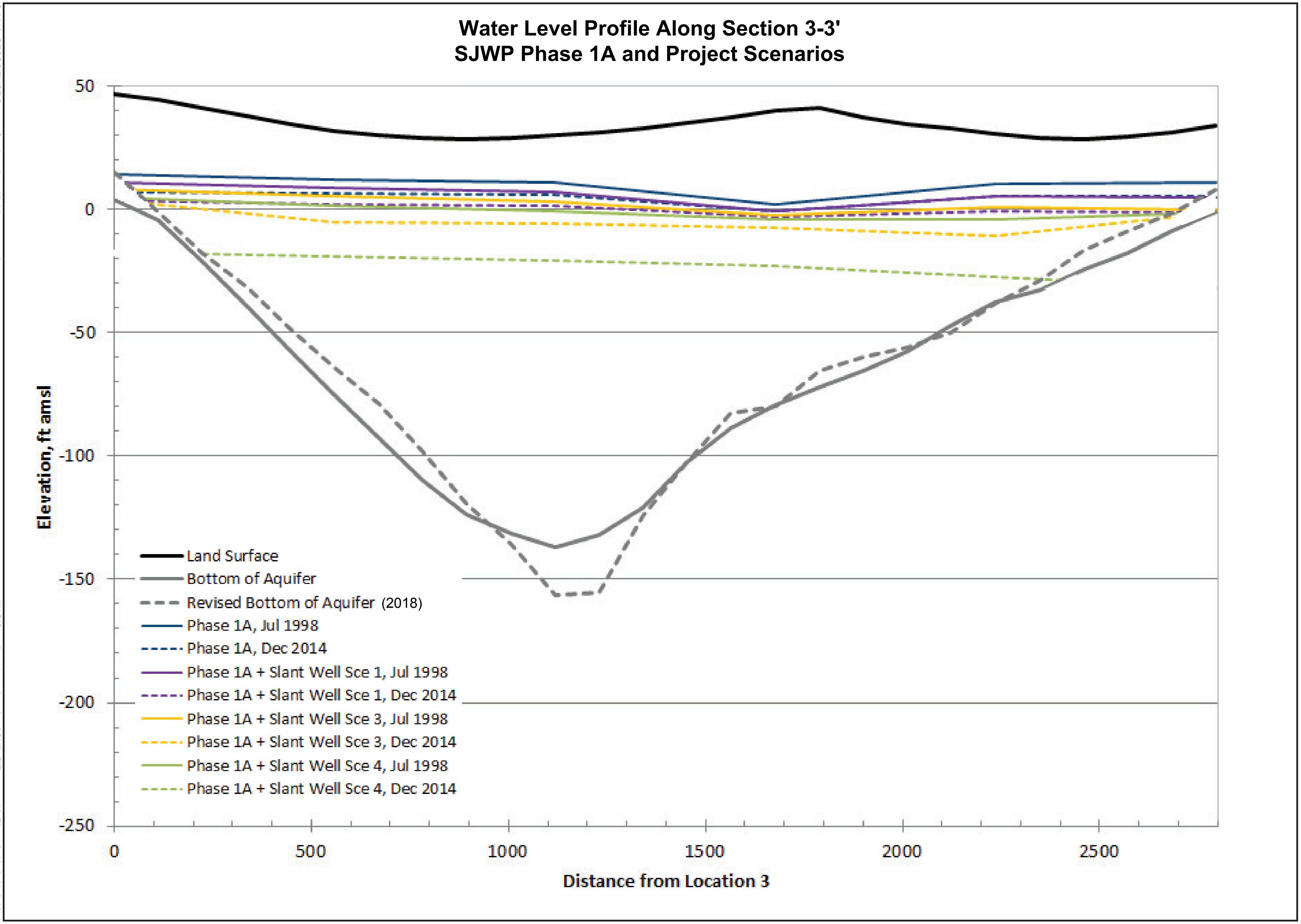
SOUTH COAST WATER DISTRICT

FIGURE 17

GEOSCIENCE



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SOUTH COAST WATER DISTRICT

FIGURE 18

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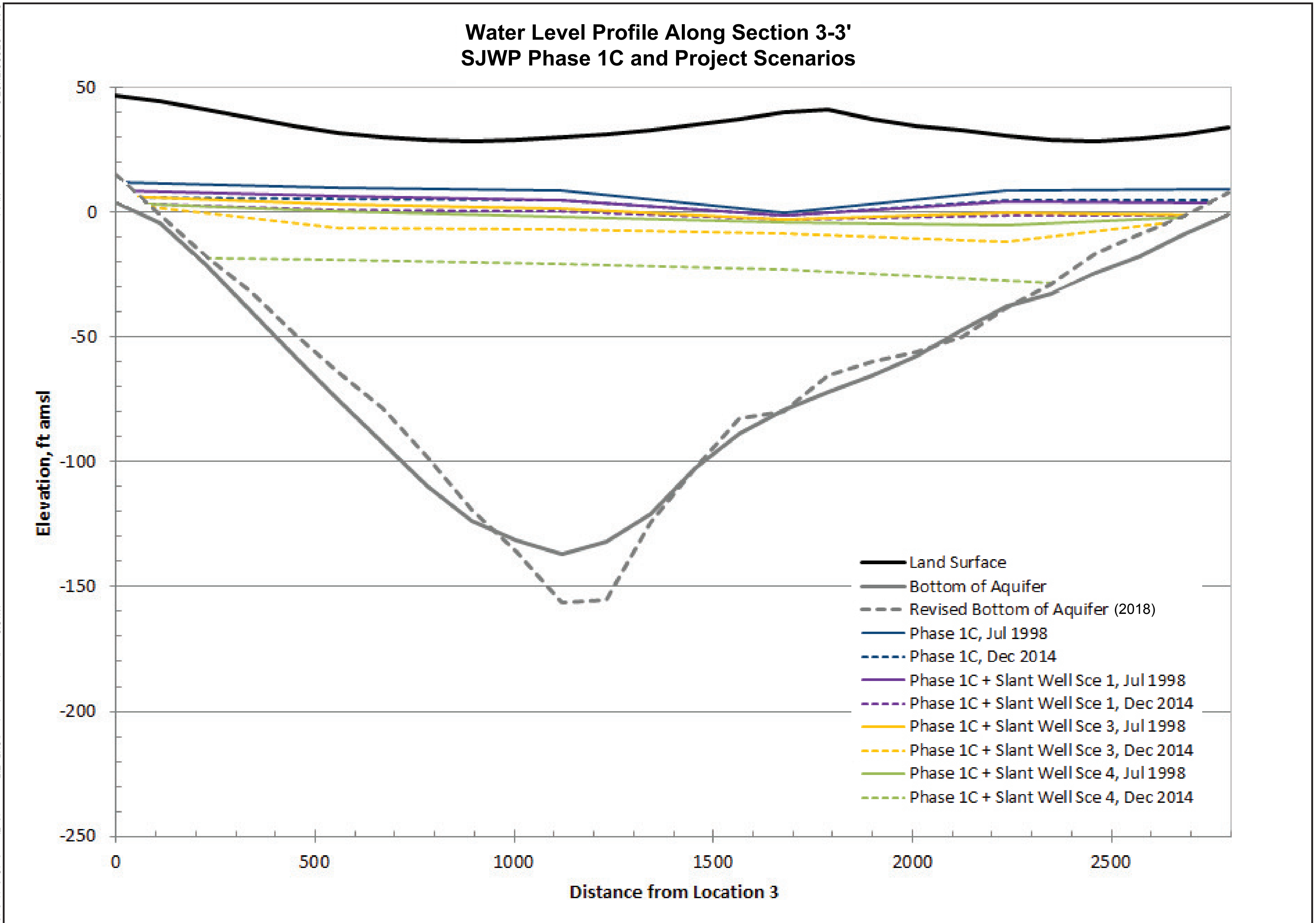

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SOUTH COAST WATER DISTRICT

FIGURE 19

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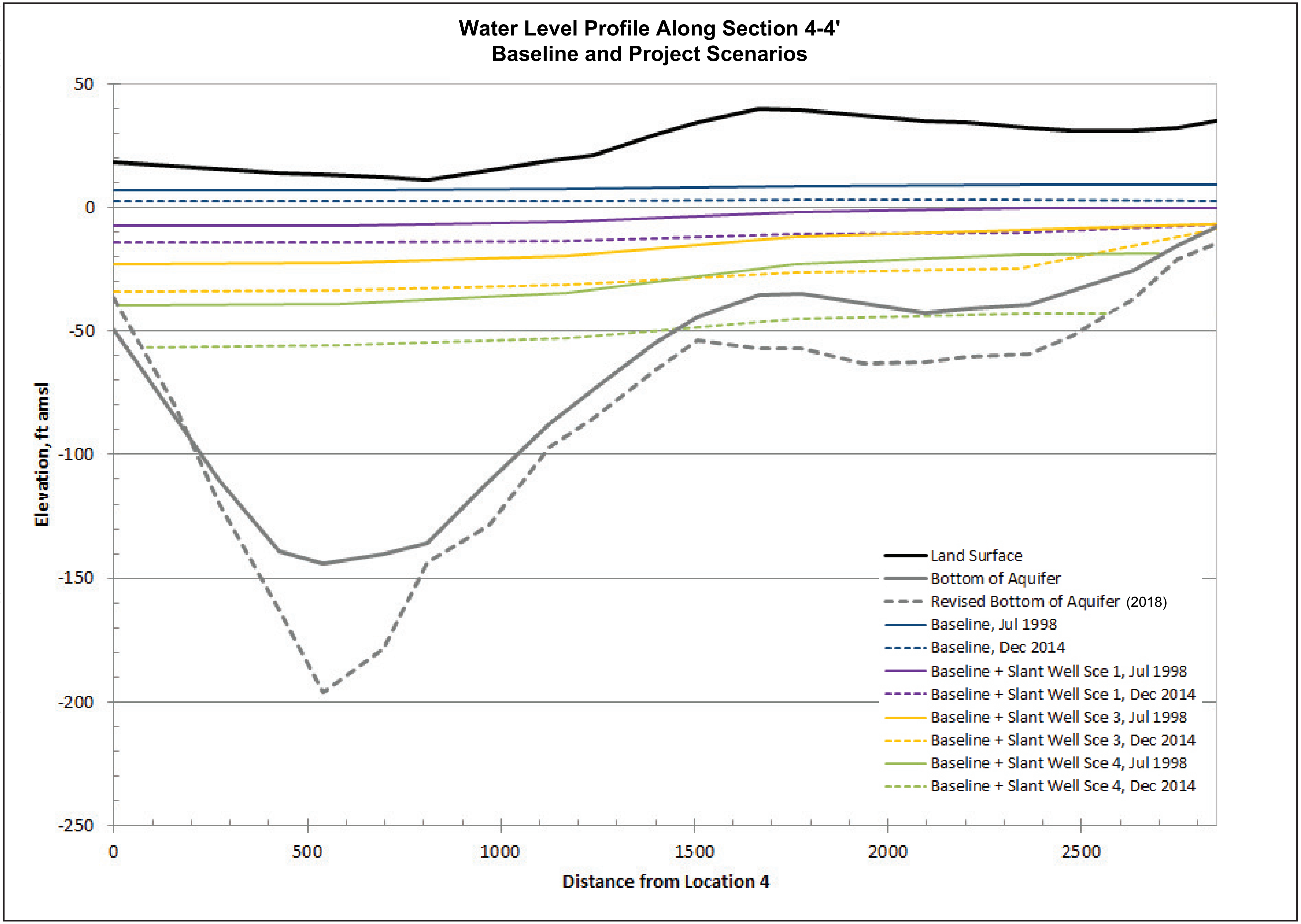

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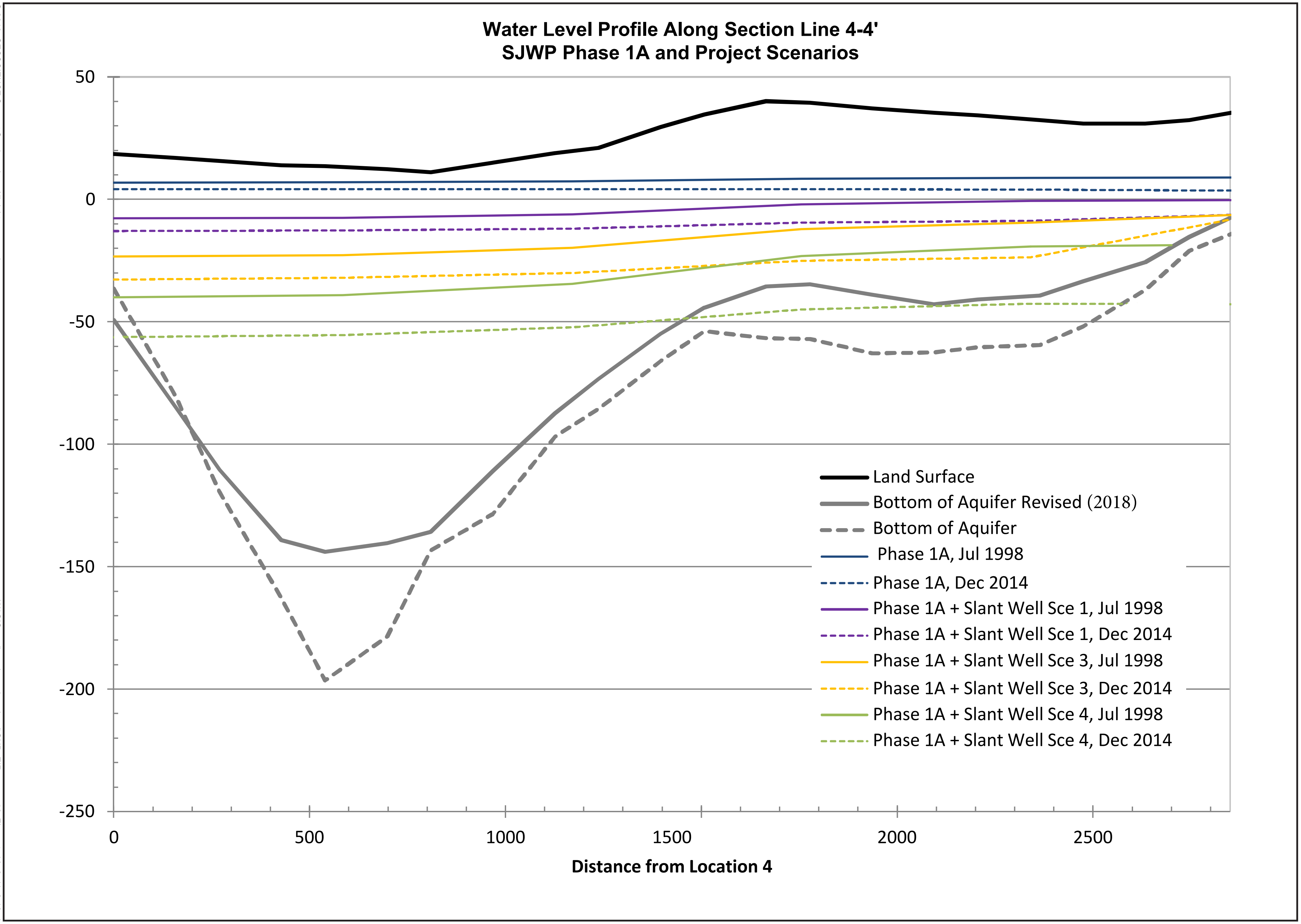
SOUTH COAST WATER DISTRICT

FIGURE 20

GEOSCIENCE



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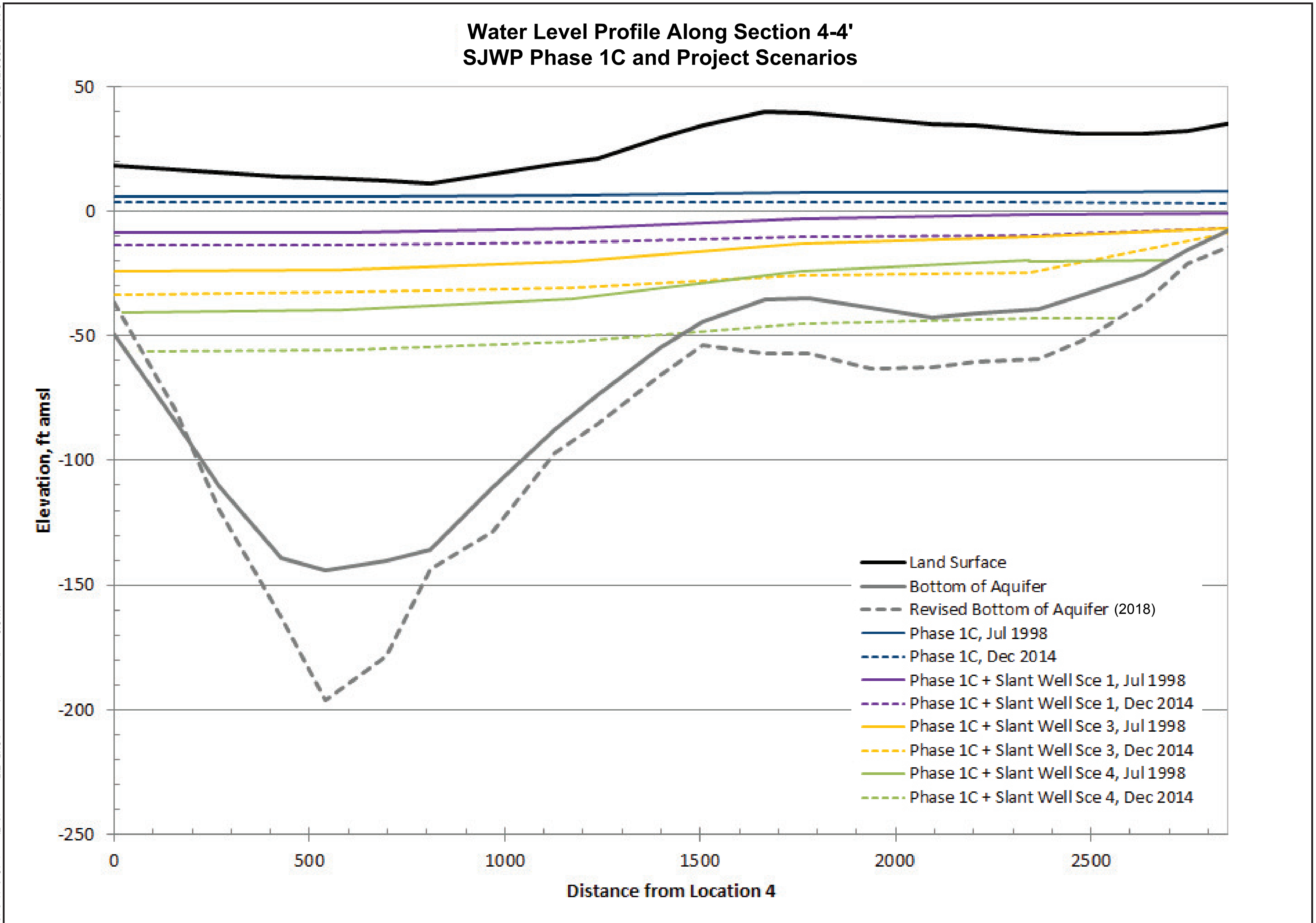
SOUTH COAST WATER DISTRICT

FIGURE 21

GEOSCIENCE



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7-Mar-19

SOUTH COAST WATER DISTRICT

FIGURE 22

GEOSCIENCE

GEOSCIENCE

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620 Arrow Highway, Suite 2000, La Verne, CA 91750 | Mailing: P.O. Box 220, Claremont, CA 91711

APPENDIX 4.2.3.2

SAN JUAN CREEK LAGOON TECHNICAL MEMO

Technical Memorandum

GEOSCIENCE Support Services, Inc.
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To:	Mark Donovan, PE Principal Engineer GHD Engineering 175 Technology Dr. #200 Irvine, CA 92618
From:	Brian Villalobos, PG, CHG, CEG Principal Geohydrologist GEOSCIENCE Support Services, Inc.
Date:	November 19, 2018
Subject:	**EIR Doheny Ocean Desalination Project – Hydrogeologic Analysis Related to Responses to Comments – Task 1: Evaluate Project Impacts on San Juan Creek Surface Water Levels**

1.0 INTRODUCTION

The Doheny Ocean Desalination Project Draft Environmental Impact Report (DEIR) was issued on May 17, 2018. GEOSCIENCE Support Services, Inc. (GEOSCIENCE) reviewed the DEIR comments related to project impacts to groundwater and surface water provided to us by the project team, including those provided by the National Oceanic and Atmospheric Administration National Marine Fisheries Service (NOAA-NMFS). In response to these comments, GEOSCIENCE has conducted additional analysis regarding the influence of slant well pumping on San Juan Creek lagoon levels. This technical memorandum summarizes the results of Task 1: Evaluate Project Impacts on San Juan Creek Surface Water Levels, as outlined in the approved scope of work from our proposal dated August 29, 2018.

2.0 EVALUATION OF PROJECT IMPACTS ON SAN JUAN CREEK SURFACE FLOW

The San Juan Basin (SJB) Regional Groundwater Model was used to determine San Juan Creek discharges to the ocean under "No Project" (i.e., Baseline) and "Project" (i.e., pumping) conditions. Evaluating the surface outflow component under No Project and Project conditions allows for the quantification of the potential reduction in surface flow, in cubic feet per second (cfs), due to Project impacts. This quantification can be used for further biological evaluations of potential impacts, as requested by NOAA-NMFS.

2.1 San Juan Basin Regional Groundwater Model

The SJB Regional Groundwater Model was originally developed in 2013 to evaluate the basin yield and groundwater level response from existing and planned groundwater development, to determine potential impacts on groundwater levels and pumping interference from the installation of sheet piling along the San Juan Creek flood control channel, and to assess impacts associated with the Doheny Ocean Desalination Project (GEOSCIENCE, 2013). Later, the model was updated for the Doheny Desalination Project during work for the Foundational Actions Funding Program – Advancement of Slant Well Technology and Groundwater Flow and Solute Transport Modeling (GEOSCIENCE, 2015) to better understand 1) feedwater quality produced over time from a slant well system, 2) drawdown effects and environmental strategies along coastal reaches, and 3) the behavior of seawater flow and intrusion control in a multi-layered aquifer system. Following onshore and offshore geophysical surveys and the drilling of a borehole in 2017-2018 to better define the geometry of coastal paleochannels, the SJB Regional Model was further refined by incorporating the newer hydrogeological data.

The SJB Regional Model is a three (3) layered MODFLOW[1] model covering the lower and middle SJB area of approximately 47.5 square miles (30,400 acres), including an offshore area to incorporate infiltration from the ocean (see Figure 1). The model consists of a finite-difference grid with 1,012 rows in the north to south direction and 524 columns in the west to east direction, for a total of 530,288 cells per layer, or 1,590,864 cells total. Each model cell of the SJB Regional Model represents an area of 50 ft x 50 ft. The active model area represents unconsolidated and semi-consolidated fluvial deposits interbedded with numerous fine-grained silt and clay deposits. Inactive model areas and the base of the groundwater model represent surrounding and underlying consolidated geologic formations (i.e., bedrock).

The regional model was calibrated for the period from January 2004 to December 2014 for purposes of analyzing the impacts of full-scale pumping. This period was selected due to the importance of recent stresses (dry hydrologic period) on the basin for predicting future performance. The calibration was based on 2,435 groundwater level measurements from 36 target wells and measured streamflow from the San Juan Creek at La Novia gaging station and Trabuco Creek at San Juan Capistrano gaging station.

[1] MODFLOW is a block-centered, finite-difference groundwater flow code developed by the United States Geologic Survey (USGS) (McDonald and Harbaugh, 1988) for the purpose of modeling both saturated and unsaturated groundwater flow.

2.2 Model-Calculated San Juan Creek Lagoon Elevations

The SJB Focused Model was developed and calibrated as part of the Foundational Actions Funding Program (GEOSCIENCE, 2015) to more accurately predict slant well pumped water quality over time, injection water flow/water quality/reactants, and ocean water intrusion. The finer cell size (resolution) used for the focused model was also important to understand seasonal coastal lagoon drawdown effects. Lagoon levels were calculated by the Focused Model, which employs the Lake Package to simulate the surface water-groundwater interaction.

Project impacts were evaluated for the hydrologic period from January 1947 through December 2010 for the following scenarios:

- Baseline (i.e., no Project pumping)
- Scenario 1: Project pumping of 10 million gallons per day (MGD) from three slant wells
- Scenario 2: Project pumping of 10 MGD from seven slant wells at Capistrano Beach
- Scenario 3: Project pumping of 30 MGD from sixteen slant wells (20 MGD from slant well pods at Doheny and 10 MGD from pods at Capistrano Beach)

For the purposes of this evaluation, surface flows in San Juan Creek under baseline conditions (representing no Project pumping conditions) will be compared to Scenario 1 (representing low Project pumping conditions) and Scenario 3 (representing higher Project pumping conditions).

As presented in Table 4-3 of Model Update and Refinement Using Results from Onshore and Offshore Geophysical Surveys and Exploratory Borehole Data Technical Memorandum, dated March 1, 2018, Project pumping impacts on the lagoon water levels were reported as ranging from -0.14 to -0.26 ft under Scenario 1 conditions and -0.16 to -0.85 ft under Scenario 3 conditions. To clarify decreases in lagoon levels for dry and wet hydrologic conditions, the table below summarizes the model-calculated decreases in lagoon levels.

Table 1. Project Impacts on Lagoon Surface Levels

Hydrologic Cycle	Hydrologic Period	Impact on Lagoon Levels, ft	
		Scenario 1	Scenario 3
Dry	1947-1976	-0.14	-0.16
Wet	1978-1983	-0.26	-0.85

It is important to note that the model-calculated decreases in San Juan Creek lagoon levels occur over the entire lagoon area of approximately 13.2 acres. The following section provides the model-calculated reduction in outflow that will occur as a result of Scenario 1 and Scenario 3 pumping over the area influenced by Project pumping under dry and wet hydrologic conditions.

2.3 Model-Calculated San Juan Creek Outflow to the Ocean under No Project and Project Conditions

Surface flow in San Juan Creek was simulated in the SJB Regional Model using the Streamflow Routing Package (SFR Package) from MODLFOW, which accounts for the interaction between surface water and groundwater. The SFR Package assigns recharge to stream cells that are sequentially numbered in the downstream direction. The downward leakage of streamflow, or streambed percolation, is calculated as a function of the hydraulic conductivity of the streambed, the wetted perimeter of the streambed, the length of the stream reach, the underlying groundwater head, stream stage, and streambed thickness. Model input for the routing package includes stream inflow, stream channel geometry, and streambed conductance (Niswonger and Prudic, 2006). Information on streamflow was available from observed measurements at the San Juan Creek at La Novia and Trabuco Creek at San Juan Capistrano gaging stations, which were also used for model calibration. These gaging stations are shown on Figure 1.

In order to evaluate the long-term outflow at the ocean, streamflow in San Juan Creek and surface outflow to the ocean was calculated by the SJB Regional Model for the hydrologic period from January 1947 through December 2010 for the same scenarios presented above. The model-calculated streamflow from San Juan Creek to the ocean is shown on Figures 2 through 4 under Baseline, Scenario 1, and Scenario 3 conditions, respectively.

For baseline conditions, outflow at the ocean ranges from 15.95 cubic feet per second (cfs) under dry hydrologic conditions to 56.04 cfs under wet hydrologic conditions. Discharge under Scenario 1 Project pumping conditions ranges from 15.86 cfs under dry hydrologic conditions to 55.59 cfs under wet hydrologic conditions. Discharge under Scenario 3 ranges from 15.83 cfs under dry hydrologic conditions to 55.41 cfs under wet hydrologic conditions. The corresponding decrease in San Juan Creek streamflow under Project pumping conditions, as compared to baseline, is summarized in the following table.

Table 2. Project Impacts on San Juan Creek Outflow to the Ocean

Hydrologic Cycle	Hydrologic Period	Impact on San Juan Creek Discharge, cfs	
		Scenario 1	Scenario 3
Dry	1947-1976	-0.09	-0.12
Wet	1978-1983	-0.45	-0.63

The decreases in San Juan Creek streamflow from Project pumping correspond to approximately 0.6 to 0.8 percent of the baseline outflow under Scenario 1 conditions and 0.8 to 1.1 percent of the baseline outflow under Scenario 3 conditions.

3.0 EVALUATION OF PROJECT IMPACTS ON SHALLOW AQUIFER GROUNDWATER LEVELS

3.1 Groundwater Level Monitoring

Historical groundwater levels for the shallow aquifer in the vicinity of the San Juan Creek lagoon were collected both during and after the long-term slant well pumping test from transducers placed in District-owned nested monitoring wells MW-1 through MW-4 (12 total monitoring wells). MW-1 and MW-2, which are closest to the lagoon, are shown on Figure 5 (inset below).

Figure 5. Monitoring Well Locations in the Vicinity of the San Juan Creek Lagoon

During the initial field investigations conducted in 2005, boreholes B-2 and B-4 were completed as nested monitoring wells (MW-1 and MW-2, respectively). Each borehole contains three nested 2-inch PVC wells screened in the shallow, middle, or deep aquifer. A basic monitoring well construction diagram for B-2/MW-1 and B-4/MW-2 is shown on the sketch below (inset Figure 6), which illustrates the general configuration of the nested monitoring wells in MW-1 and MW-2.

After the monitoring wells were constructed, they were developed and sampled. The depth to static groundwater level in each nested monitoring well was initially measured with an electronic sounder. Each nested monitoring well was later equipped with a pressure transducer to measure groundwater levels every 15 minutes. Transducer data were downloaded on a weekly basis during the long-term pumping test (June 2010 through April 2012) and on a monthly basis after the long-term pumping test to present. An on-site barometer was used to compensate the transducer data downloaded from each nested monitoring well. The compensated water level data were then converted to elevation (NAVD88) and plotted over time.

The nested well design allows for accurate water level data from each of the nearshore aquifers (shallow, middle, and deep). For example, the shallow screen in each nested monitoring well (MW-1S and MW-2S) provides water level data for the shallow aquifer, which is in direct hydraulic connection with San Juan Creek. Since the shallow aquifer is separated from the middle aquifer by an aquitard that is approximately 10 ft thick, water level measurements in the shallow aquifer are not influenced by the deeper systems. Therefore, water level data from MW-1S and MW-2S can be used to monitor and evaluate Project pumping impacts on the shallow aquifer. In particular, MW-2S was used to evaluate groundwater levels in the shallow aquifer under the lagoon due to its location in relation to both the Test Slant Well and the lagoon.

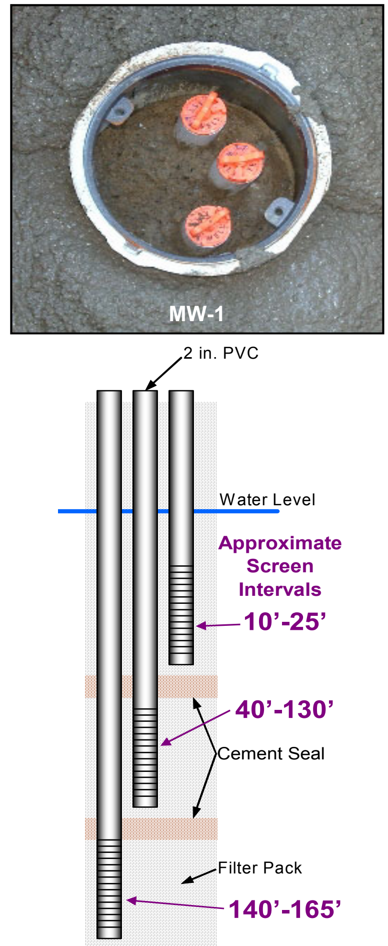

Figure 6. General Monitoring Well Construction

In addition, MW-2 is located at approximately the same location as one of the lagoon bottom profiles surveyed by the Chambers Group (2016) from Spring 2015 to Spring 2016 – providing a reference on the relative position of shallow aquifer water levels with respect to the lagoon bottom (see Figure 5). These profiles, presented originally as Figure 2-14 of the Chambers Group report, are shown as attached Figure 7. The cross-sectional transects indicate most erosion taking place within the southwest corner of the lagoon where the sand berm is typically breached. Some accretion of sediment occurs on the eastern bank of the lagoon (see Chambers Group, 2016). An average bottom elevation of the lagoon was estimated to be approximately 4 ft NAVD88 from the Lagoon S cross-section (shown on Figure 5 as A-A').

3.2 Shallow Aquifer Groundwater Levels during Historical Hydrologic Cycles

Shallow aquifer groundwater levels from MW-2S are shown on Figure 8 from just before the start of the long-term pumping test in June, 2010, through present. The shallow aquifer groundwater levels are also shown in comparison to the average estimated lagoon bottom elevation of 4 ft NAVD88 and monthly precipitation from the Laguna Beach #2 precipitation gage. The water levels represent a combination of two sets of collected data that were joined together to display shallow groundwater levels during the pumping test and post-pumping period. One dataset was collected and processed by GEOSCIENCE during slant pumping test monitoring, and the other was collected and processed by South Coast Water District (SCWD) during post-pumping test monitoring.

As shown, groundwater levels in the shallow aquifer fluctuated above and below the average lagoon bottom elevation both during and after the long-term pumping test. Based on the observed fluctuations and local precipitation, groundwater levels in MW-2S appear to correlate well with dry (below average) or wet (above average) hydrologic periods. At the start of the long-term pumping test, shallow aquifer water levels were hovering at the elevation of the lagoon bottom. Just after the initiation of the test, water levels fall below the lagoon bottom elevation. Water levels then rose above the lagoon bottom elevation in response to the increased precipitation that occurred from January 2011 through July 2011. Subsequently, water level in the shallow aquifer fell below the lagoon bottom elevation during the following dry years (2012 through 2015) and rose again following increased precipitation events in 2015 and 2016. However, despite the lower groundwater levels in the shallow aquifer during dry hydrologic conditions, review of aerial photos and land-based photos taken during monitoring events indicate surface water was still present in the lagoon during these times (Figure 9).

Based on the observed response of water levels to changes in precipitation, it appears that groundwater levels in the shallow aquifer in the vicinity of the San Juan Creek lagoon are heavily influenced by hydrologic conditions.

3.3 Project Impacts during Wet, Dry, and Average Hydrologic Cycles

Figure 10 shows historical annual precipitation from 1928 through 2017, along with the calculated cumulative departure from the mean. The 70-year average precipitation is approximately 11.77 in/year during the period from 1928 through 2017. A downward slope on the cumulative departure curve indicates a less than average or “dry” conditions and upward slope shows periods during “wet” conditions. The long-term cumulative departure from the mean precipitation show that the study period during slant well pumping (June 2010 through April 2012) and post slant well pumping represented an overall a dry hydrologic period.

A snapshot of the recent precipitation data, shown on Figure 11 (below), suggests that the average precipitation over the past 12 years is only 6.84 in/year, which is nearly half of the long-term precipitation average of 11.77 in/year. The precipitation data shown on Figure 11 corresponds to the period for which groundwater level data were recorded, from 2005 to 2017, in the monitoring wells near the San Juan Creek lagoon.

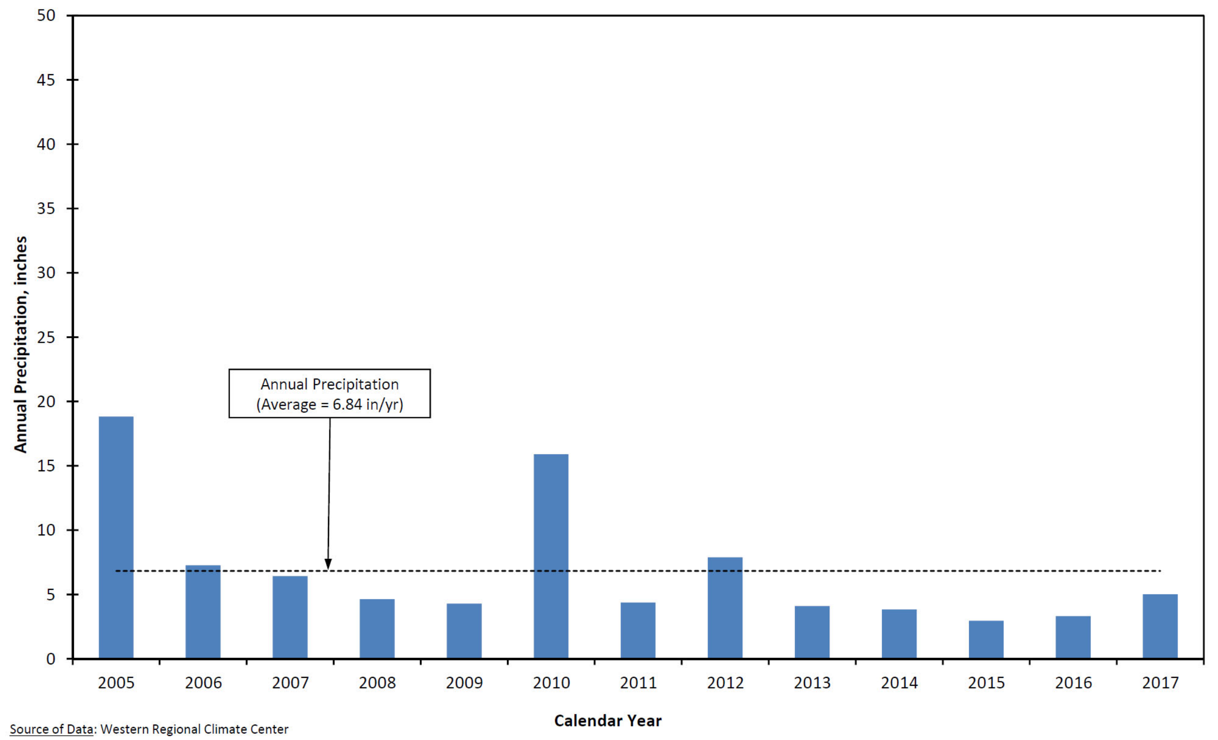

Figure 11. Annual Precipitation at Laguna Beach #2 Precipitation Gage (2005-2017)

Water levels in the shallow aquifer during the long-term pumping test and during the post-pumping period, which occurred during primarily dry hydrologic conditions (2005 to 2017), showed fluctuations that rose and fell above/below the average bottom elevation of the San Juan Creek lagoon. These fluctuations appear to be driven largely by local hydrologic cycles and precipitation patterns. However, even when shallow aquifer water levels fell below the lagoon bottom elevation in dry conditions, aerial imagery and field observations showed that water remained in the lagoon. During these conditions, the water percolated from the lagoon is in "free fall" conditions; that is, as water percolates into the subsurface it must percolate some distance before it reaches the water table. Therefore, the percolation rate is driven by the streambed hydraulic conductivity and not by depth to the groundwater elevation. As such, the degree of water level fluctuation expected under Project pumping conditions for the full scale wellfield will not significantly affect surface outflow and lagoon levels.

4.0 SUMMARY AND CONCLUSIONS

- Both the creek outflow and the shallow aquifer near the lagoon are highly affected by hydrologic conditions (i.e., precipitation patterns).

- During periods of low precipitation (dry hydrologic conditions), water levels in the shallow aquifer generally fall below the average estimated lagoon bottom elevation – both during pumping conditions and in the absence of slant well pumping .

- Even during dry conditions when groundwater levels in the shallow aquifer fall below the lagoon bottom during No Project (no pumping) and Project (pumping) conditions, water is still present in the lagoon.

- When groundwater levels in the shallow aquifer fall below the lagoon/river bottom, surface water level in the lagoon is controlled by the hydraulic conductivity of the underlying sediments and is independent of groundwater levels.

- During periods of high precipitation (wet hydrologic conditions) groundwater levels in the shallow aquifer generally rise above the lagoon bottom.

- Additional seepage from the lagoon and streambed upgradient of the lagoon occurs under Project pumping conditions. However, decreases in San Juan Creek streamflow from Project pumping correspond to approximately 0.6 to 0.8 percent of the baseline outflow under Scenario 1 conditions and 0.8 to 1.1 percent of the baseline outflow under Scenario 3 conditions.

5.0 REFERENCES

Chambers Group (Chambers Group, Inc.), 2016. Lower San Juan Creek and Seasonal Coastal Lagoon Habitat Assessment – Orange County, California. Prepared for South Coast Water District and Municipal Water District of Orange County, dated July.

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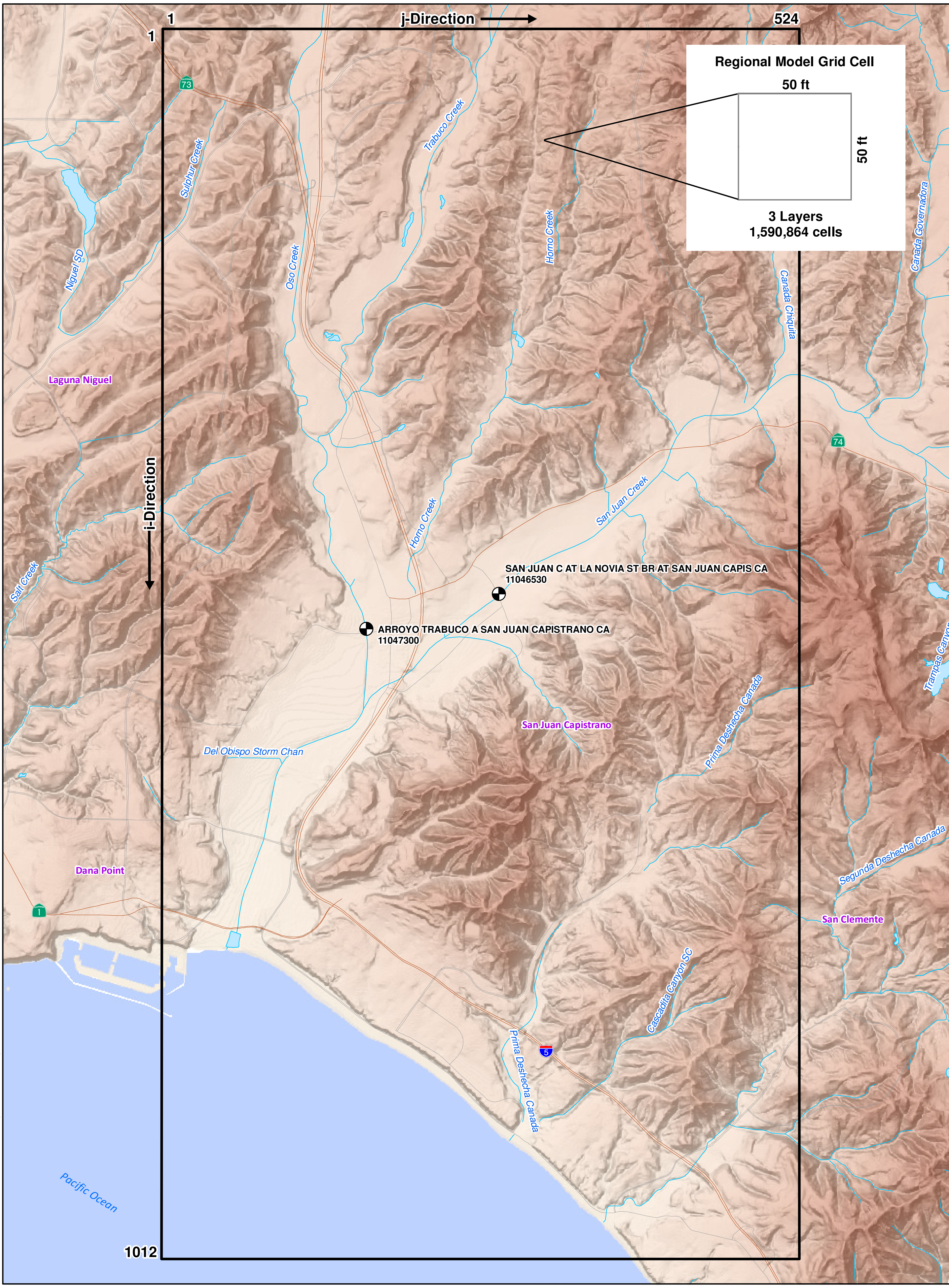
Niswonger, R.G. and Prudic, D.E., 2006. Documentation of the Streamflow-Routing (SFR2) Package to Include Unsaturated Flow Beneath Streams – A Modification to SFR1. Chapter 13 of Section A, Ground Water, of Book 6, Modeling Techniques. U.S. Geological Survey Techniques and Methods 6-A13, Version 1.10, April 2006.



San Juan Basin Regional Groundwater Model Boundary

Gaging Station Name and Designation

SAN JUAN BASIN REGIONAL GROUNDWATER MODEL

19-Nov-18

Prepared by: DB. Map Projection: State Plane 1983, Zone VI.

NORTH

0 2,000 4,000 8,000 Feet

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FIGURE 1

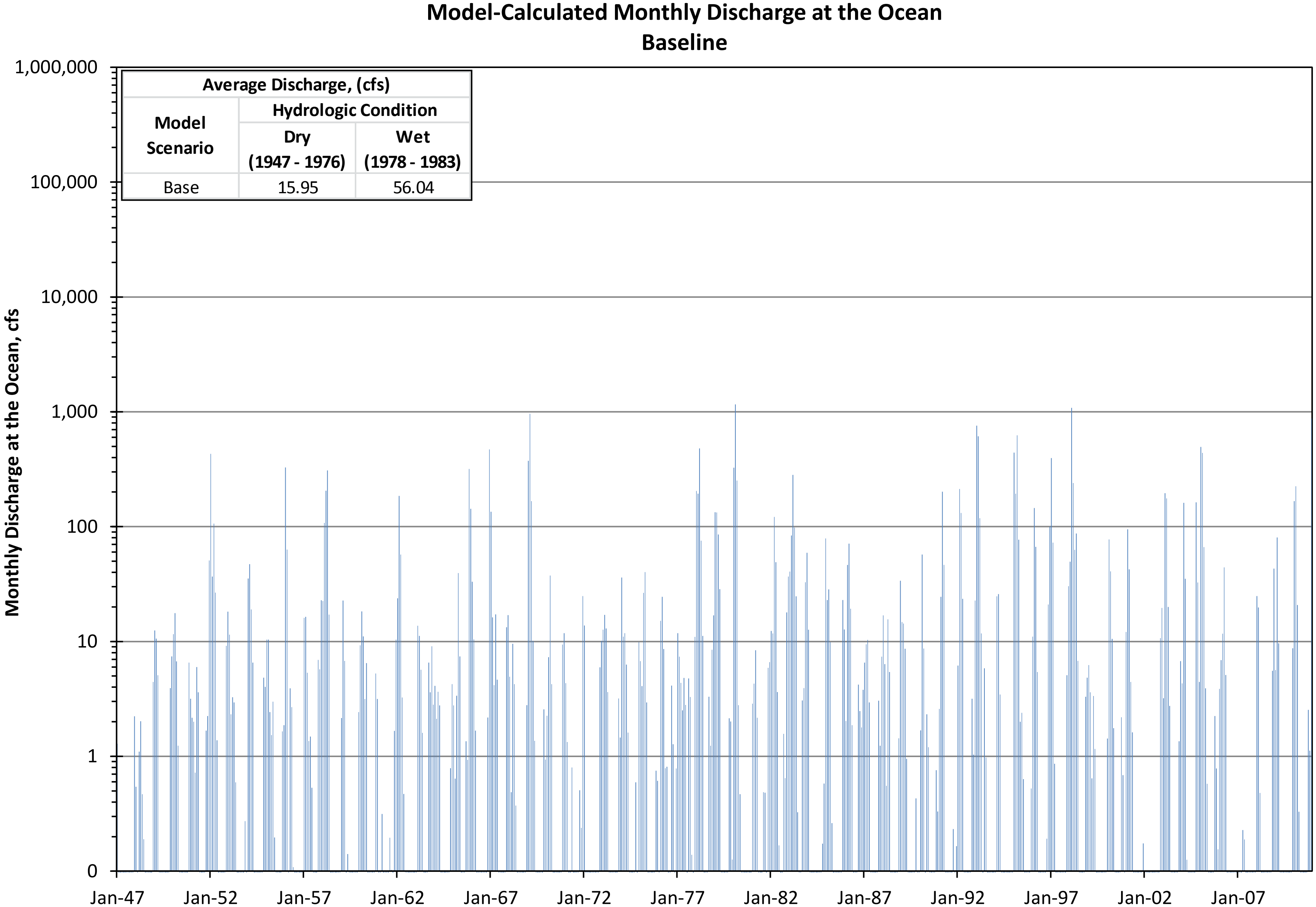

Model-Calculated Monthly Discharge at the Ocean
Baseline
Average Discharge, (cfs)
Model Scenario
Hydrologic Condition
Dry (1947 - 1976)
Wet (1978 - 1983)
Base
15.95
56.04
Monthly Discharge at the Ocean, cfs
1,000,000
100,000
10,000
1,000
100
10
1
0
Jan-47
Jan-52
Jan-57
Jan-62
Jan-67
Jan-72
Jan-77
Jan-82
Jan-87
Jan-92
Jan-97
Jan-02
Jan-07

Figure 2

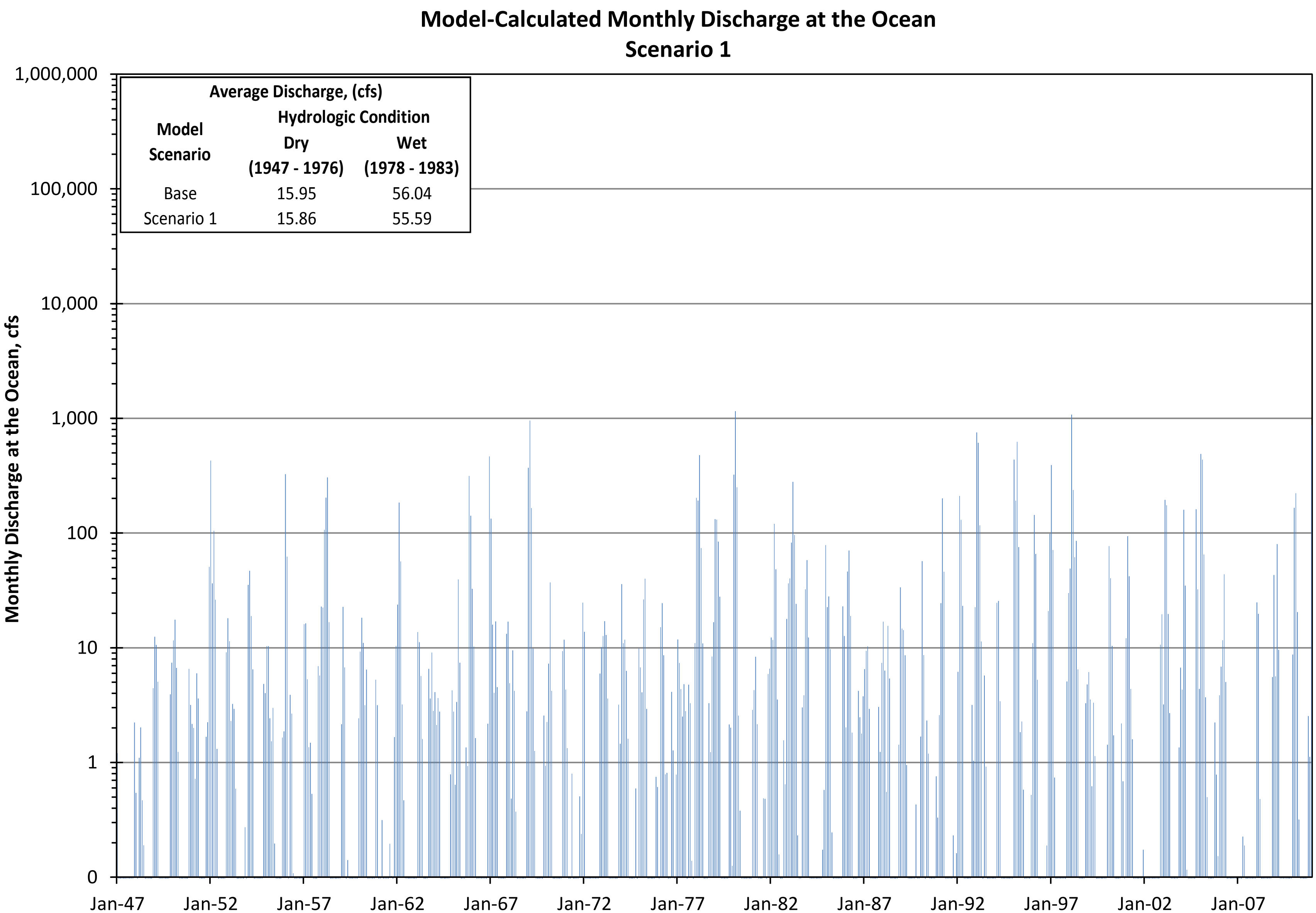

Model-Calculated Monthly Discharge at the Ocean
Scenario 1
Average Discharge, (cfs)
Model Scenario
Hydrologic Condition
Dry (1947 - 1976)
Wet (1978 - 1983)
Base 15.95 56.04
Scenario 1 15.86 55.59
Monthly Discharge at the Ocean, cfs
1,000,000
100,000
10,000
1,000
100
10
1
0
Jan-47
Jan-52
Jan-57
Jan-62
Jan-67
Jan-72
Jan-77
Jan-82
Jan-87
Jan-92
Jan-97
Jan-02
Jan-07

Figure 3

Model-Calculated Monthly Discharge at the Ocean
Scenario 3

Average Discharge, (cfs)		
Model Scenario	Hydrologic Condition	
	Dry (1947 - 1976)	Wet (1978 - 1983)
Base	15.95	56.04
Scenario 3	15.83	55.41

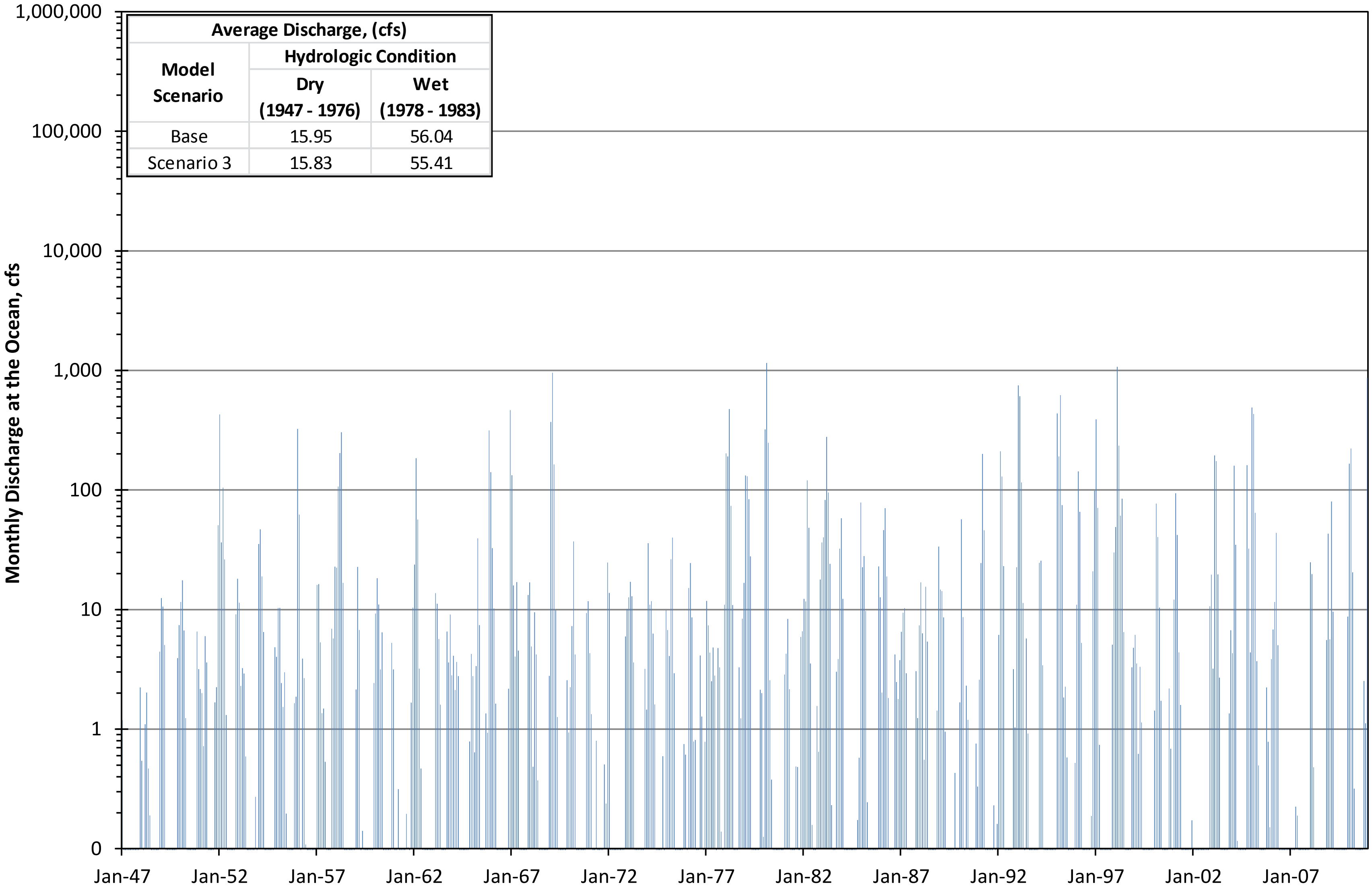
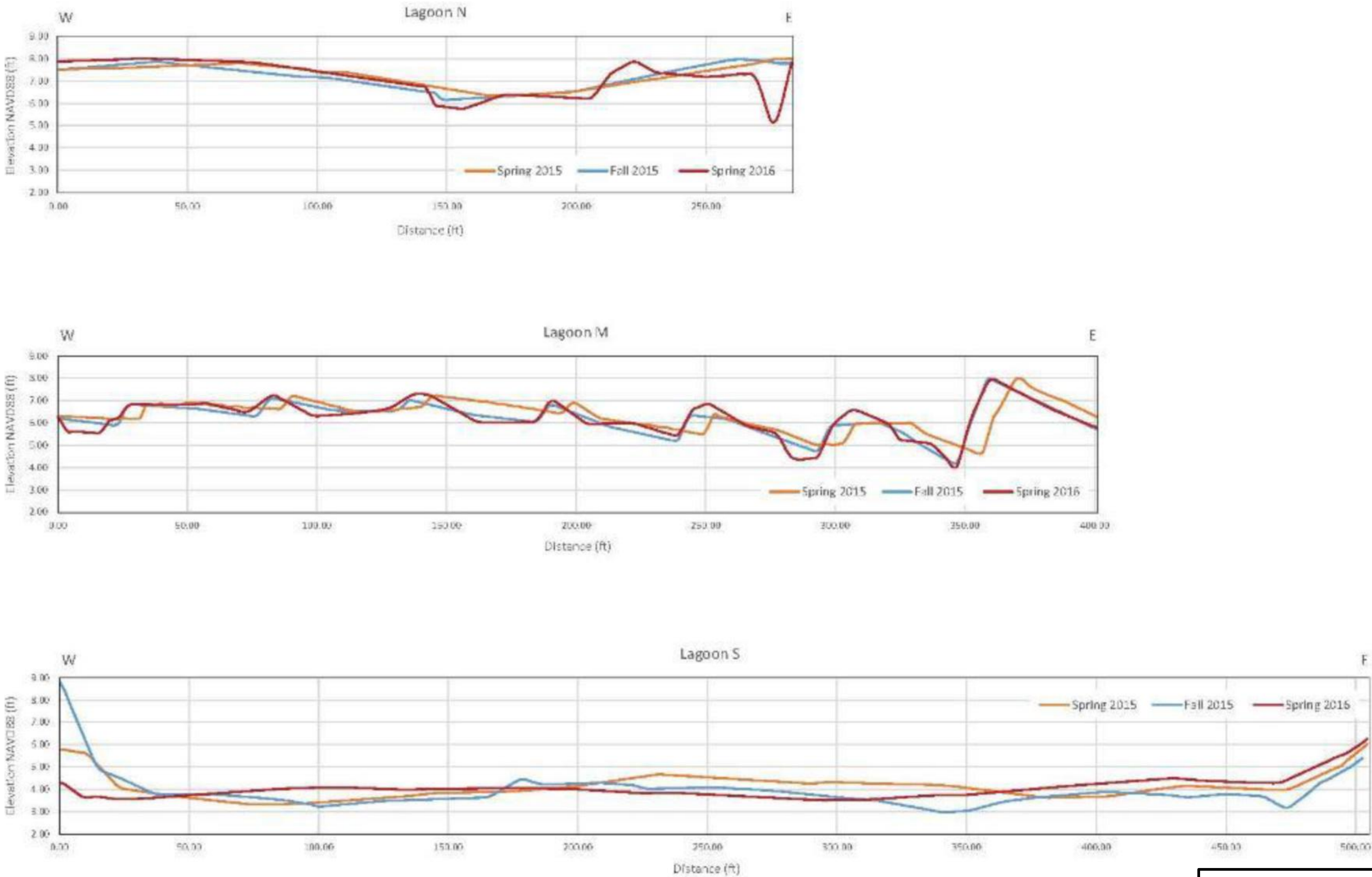


Figure 4



SOURCE: FIGURE 2-14 OF CHAMBERS GROUP, 2016

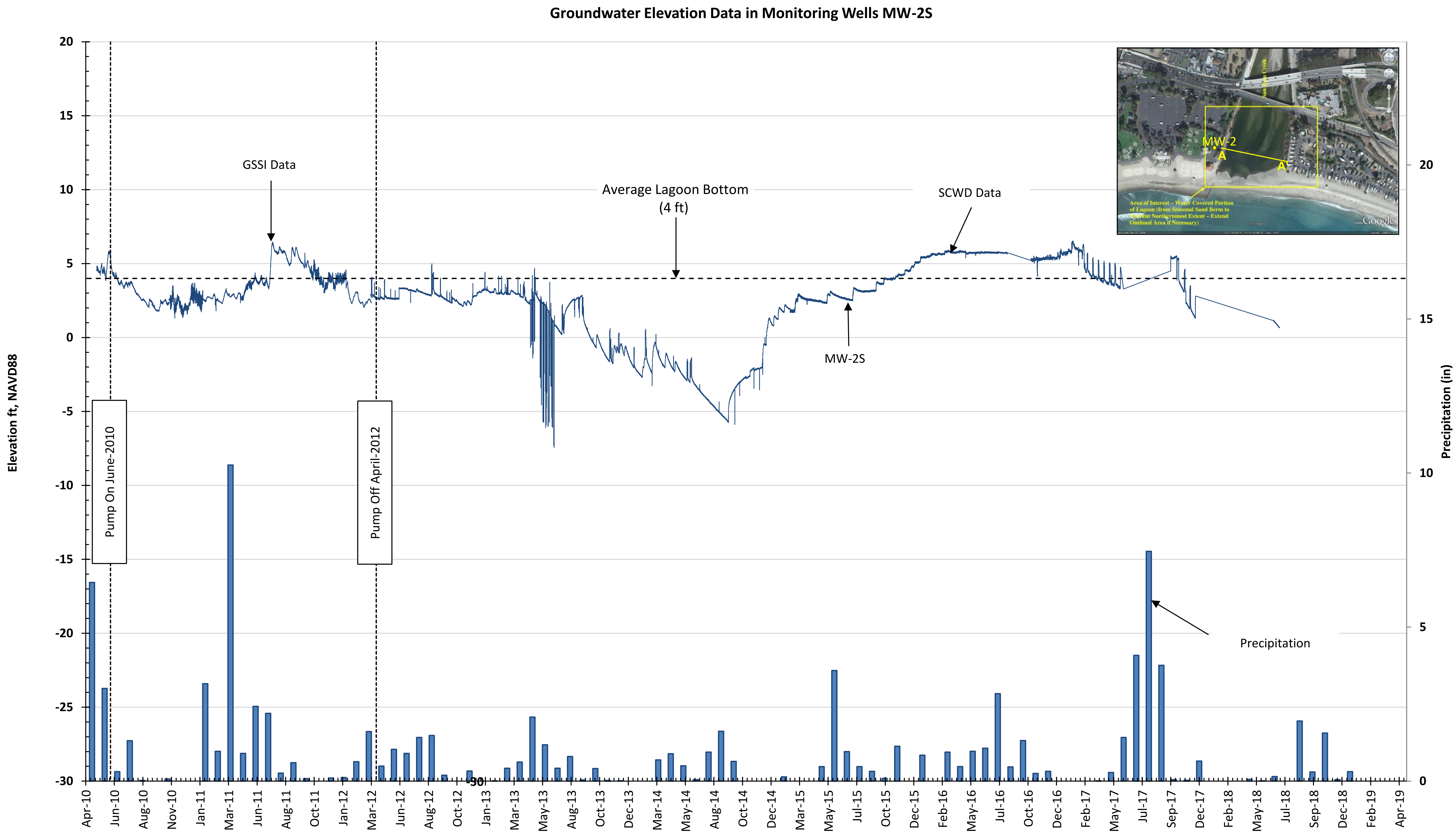
LAGOON BOTTOM PROFILES

SOUTH COAST WATER DISTRICT/GHD

FIGURE 7

EIR DOHENY OCEAN DESALINATION PROJECT – HYDROGEOLOGIC ANALYSIS RELATED TO RESPONSES TO COMMENTS
TASK 1: EVALUATE PROJECT IMPACTS ON SAN JUAN CREEK SURFACE WATER LEVELS

GEOSCIENCE


Groundwater Elevation Data in Monitoring Wells MW-2S
Elevation ft, NAVD88
Precipitation (in)
GSSI Data
Average Lagoon Bottom (4 ft)
SCWD Data
MW-2S
Precipitation
Pump On June-2010
Pump Off April-2012
MW-2
A
A'
20
15
10
5
0
-5
-10
-15
-20
-25
-30
Apr-10
Jun-10
Aug-10
Nov-10
Jan-11
Mar-11
Jun-11
Aug-11
Oct-11
Jan-12
Mar-12
Jun-12
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Oct-12
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Oct-14
Dec-14
Mar-15
May-15
Jul-15
Oct-15
Dec-15
Feb-16
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Jul-16
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Feb-17
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Feb-19
Apr-19

Figure 8

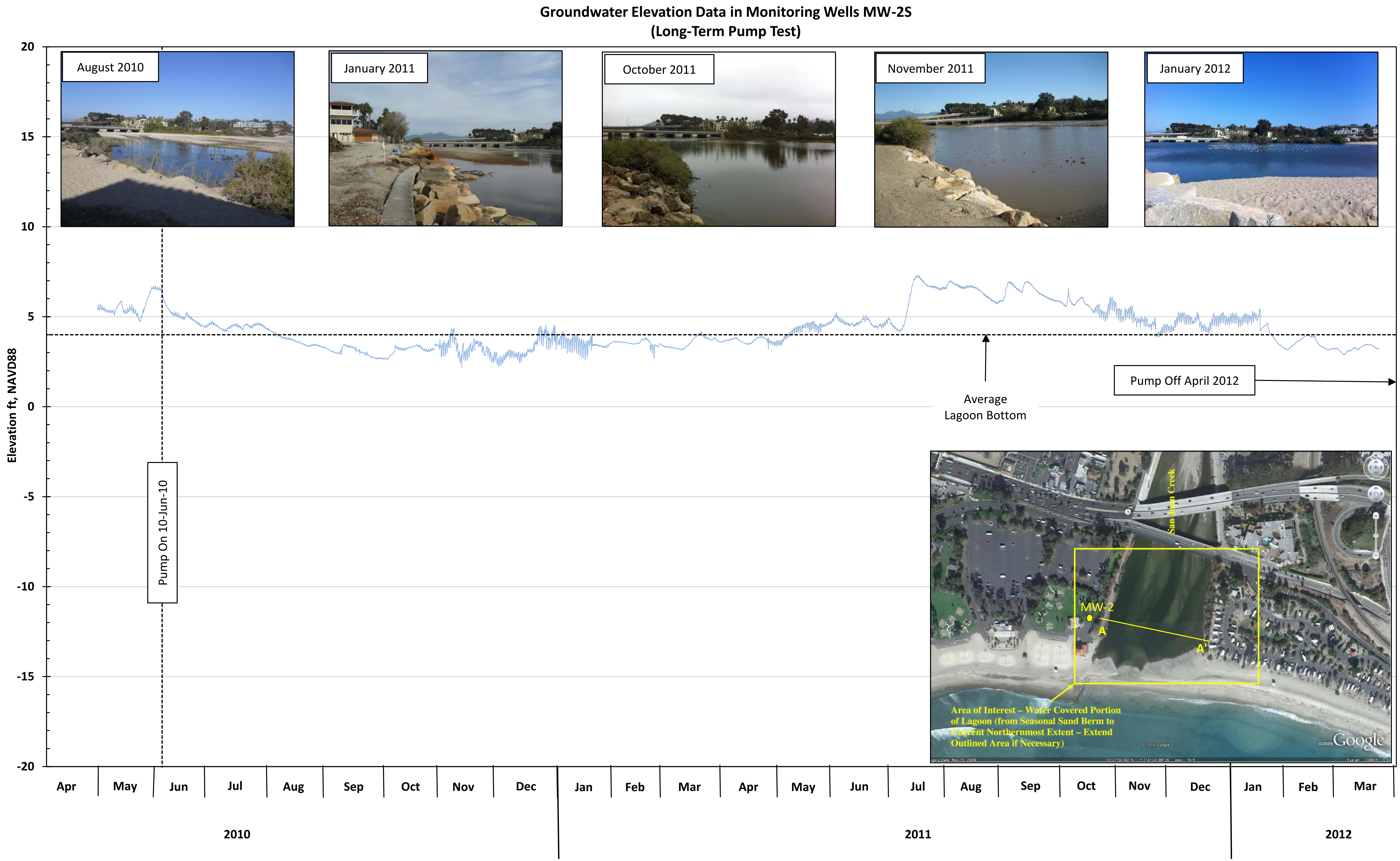
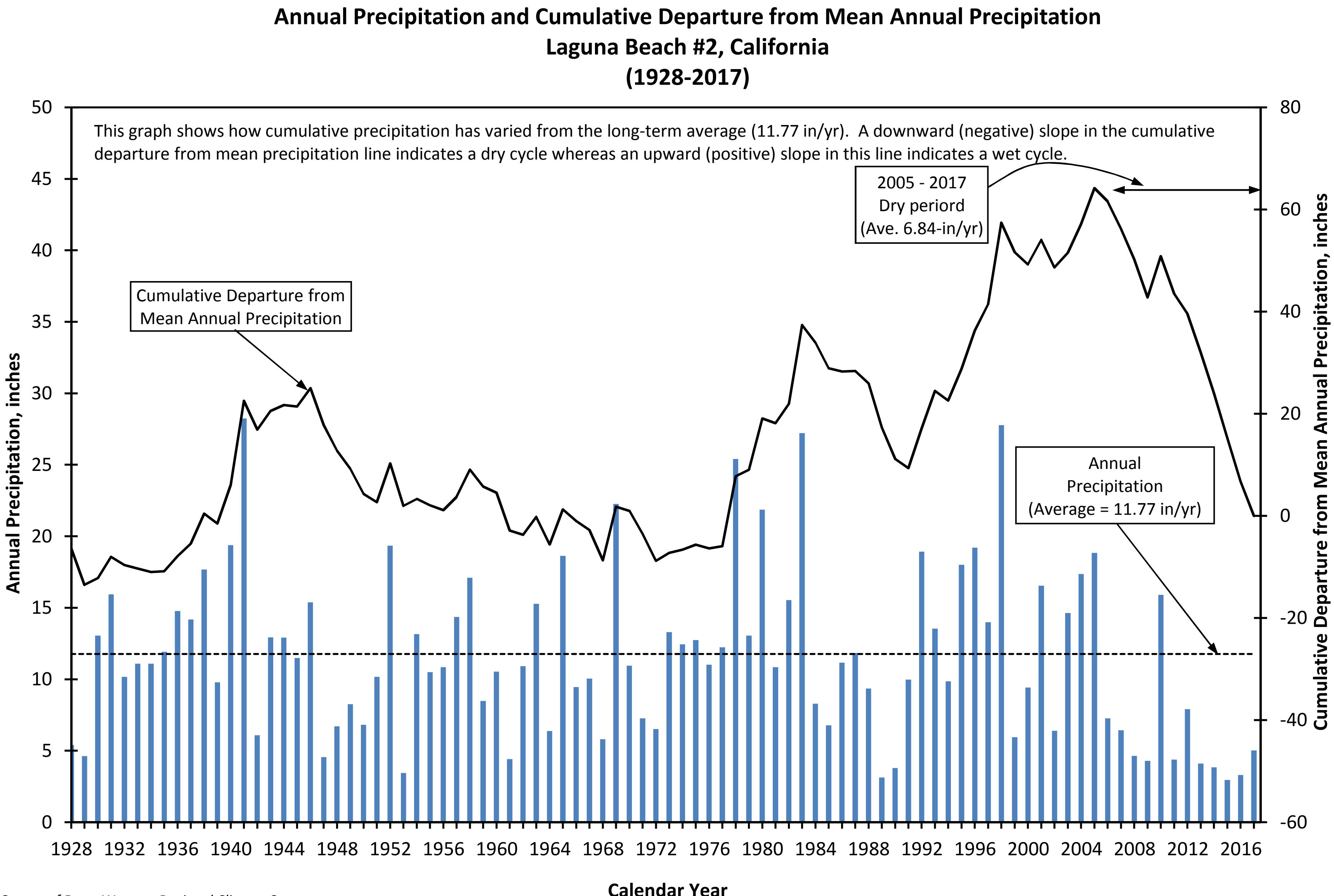

Groundwater Elevation Data in Monitoring Wells MW-2S
(Long-Term Pump Test)
August 2010
January 2011
October 2011
November 2011
January 2012
Elevation ft, NAVD88
20
15
10
5
0
-5
-10
-15
-20
Pump On 10-Jun-10
Average
Lagoon Bottom
Pump Off April 2012
San Juan Creek
MW-2
A
A'
Area of Interest – Water Covered Portion of Lagoon (from Seasonal Sand Berm to Current Northernmost Extent – Extend Outlined Area if Necessary)
Google
Apr
May
Jun
Jul
Aug
Sep
Oct
Nov
Dec
Jan
Feb
Mar
Apr
May
Jun
Jul
Aug
Sep
Oct
Nov
Dec
Jan
Feb
Mar
2010
2011
2012

Figure 9



Source of Data: Western Regional Climate Center

Figure 10

APPENDIX 4.2.4

LOCAL HAZARD AND DRAINAGE CALCULATIONS FOR FINAL EIR

South Coast Water District

Doheny Desalination Project
Local Hazard Conditions and Drainage Study
January 2019

GHD Inc. | 320 Goddard Way Suite 200 Irvine CA 92618

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1. Introduction

1.1 Introduction

Imported water from northern California (State Water Project) and the Colorado River Aqueduct make up most of the water supply to south Orange County. With the continuous growth in California and the susceptibility of imported water to drought or other natural disaster, the availability and the reliability of water from imported sources is increasingly becoming a concern for local water providers. 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 north of Highway 1, could supply up to 15 million gallons of local drinking water a day by desalinating seawater from the 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 analyses 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, 2018.
- 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 storage. 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 to San Juan Creek through the project site. Owned by Orange County Flood Control District, the L01S02 system is a double concrete box culvert that runs parallel to the landside of the San Juan Creek levee before outfalling 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 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 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 land use 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, 2018. 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, and the sea level rise projections in Appendix G, Table G-11 (California Coastal Commission 2018).

The study was completed by Michael Baker International and presented in the September 2018 report titled Coastal Hazards Analysis for the Doheny Desalination Project by Scott A. Jenkins (Appendix B). Additional 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.

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 (2018) 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 water level province tabulation from NOAA tide gage stations with extended periods of record (California Coastal Commission 2018). The Doheny Desalination Project falls within the La Jolla tide gage water level province. Sea level rise projections are provided in Table G-11 in Appendix G of the California Coastal Commission 2018. 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 (CCC, 2018)	2070	2.0	3.6
Critical Infrastructure Planning Horizon (CCC, 2018)	2100	3.6	7.1

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. Projected sea level rise for 2070 and 2100 were available from Table G-11 in Appendix G of California Coastal Commission 2018. 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)
EHW (Extreme High Water)	7.47	9.47	11.07	11.07	14.57
MHHW (Mean Higher High Water)	5.13	7.13	8.73	8.73	12.23
MHW (Mean High Water)	4.41	6.41	8.01	8.01	11.51
MSL (Mean Sea Level)	2.54	4.54	6.14	6.14	9.64

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 (2018) 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 (2018) 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 2018, based on California Coastal Commission 2018. 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 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 Figure 2.2 based on topography generated from USACE 2014 LiDAR data. The figure shows total water levels for the accreted beach conditions for each event since these were higher water levels than the eroded beach condition. 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. Also note that the mapping shows flooding for all areas below the given flood elevation even though there may not be a direct flow path to all locations. It can be seen that the extremal total water level for the low and 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. It also may flood along an existing South Drainage Swale due to backwater from the creek to the low grade area along the swale. 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 (Barnard et al., 2018). Flooding extents at the study site for the 0.5 m, 2 m and 5 m sea level rise scenarios for a 100-year storm event are presented in Figure 2.3. The flooding extents were much less than those based on the estimates of extremal total water levels provided by Michael Baker International (2018) 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 2100 sea level projection scenarios are illustrated in Figure 2.4. Flood limits were very similar to the 100-year wave storm event for the 2100 low and high range sea level rise predictions (Figure 2.2). Flood levels were approximately 0.4 ft higher for the low range and high range sea level rise limits for a 100-year event. It can be seen that the tsunami for the 2100 low and high range sea level rise scenarios may reach a very small portion at the seaward tip of the property where there is no proposed infrastructure. Flooding also impacts the area around the existing South Drainage Swale due to backwater from the creek to the low grade area along the swale.

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 2.5. While the mapping resolution is low, the flooding extent appears to be similar to that provided in Figure 2.4.

The assessment shows that the projected sea level rise scenarios considered in this study does not pose significant flood risk to the project site. The back water ponding shown along the South Drainage Swale can be mitigated by site design to regrade the low ground area along the swale.

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, five 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 four tidal elevation scenarios are the Years 2070 and 2100 projected low and high sea level rise elevations. All five 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)	2.87	5.13
2070 Low Sea Level Rise Projection under MHHW	4.87	7.13
2070 High Sea Level Rise Projection under MHHW	6.47	8.73
2100 Low Sea Level Rise Projection under MHHW	6.47	8.73
2100 High Sea Level Rise Projection under MHHW	9.97	12.23

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 2009c).

3.2.4 Analysis Results

The HEC-RAS model was run using the 25-year and 100-year Expected Value hydrographs under Year 2070 scenarios. The creek hydraulic analysis based on these 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.

The HEC RAS model also simulated the Year 2100 High MHHW Sea Level Rise projection for the 100-year storm event, to determine if it had any effect on the water surface elevation in San Juan creek upstream of the Highway 1 Bridge. Using the high sea level rise projection, the water surface elevation in the creek did change upstream of the bridge, however only during the receding limb of the streamflow hydrograph beginning at hour 30 of the simulation (note: the peak discharge in San Juan Creek occurs at hour 17.25). At this time, the flow in the stream is approximately 300 cfs, which is just a fraction of the flow that occurs at the peak of the storm (approximately 45,000 cfs for the peak of the 100-year storm). The change in water surface elevation only persists for about 500 feet upstream of the bridge, after that there is no effect that the boundary condition has on the water surface elevation at any point in time.

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 five 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
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

Watershed	Time of Concentration (min)	Lag Time (min)	Percent Impervious	Curve Number (AMC II)	Curve Number (AMC III)
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
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
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

Watershed	Model Node Applied	Receiving Infrastructure
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 3.9 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

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)				
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
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 Sepulveda 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, backflow up the pipe and inundate 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

In the alternative evaluation, it is found that Alternative 1 has a number of benefits and advantages over the other alternatives considered in the evaluation. 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.

As a refinement to the project recommendation, Alternative 1 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 modified Alternative 1, identified as Alternative 1a, is identical to Alternative 1, except the southern grading limit is shifted north to align with the South Coast Water District property boundary. The modification of the southern grading limit is to ensure the proposed grading does not encroach into the existing Caltrans right of way. 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.

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 compare to the existing condition.

Same as Alternative 1, Alternative 1a does not depend on improvements at San Juan Creek nor the existing storm drain facilities. In addition, as shown in Table 3.12, the proposed project does not increase the peak flow at the existing storm drain facilities under the 100-year design storm.

Table 3.12 Peak Flow Summary at Storm Drain Facilities Creek Outfall

Storm Drain Facility	100-Year Storm	
	Pre-Project Peak Flow	Post-Project Peak Flow
54” Pipe	101 cfs	57 cfs
LO1SO2	1154 cfs	1150 cfs

Note that the reduced peak flow at the 54” pipe under post-project condition is due to the proposed detention basin attenuating the peak flow.

It is recommended that Alternative 1a as the preferred flood improvement alternative for the project site.

4. 500-Year Storm Sensitivity Analysis

4.1 Introduction

This section documents a sensitivity analysis to evaluate the flood inundation under a 500-year design storm event. The sensitivity analysis included a hydrology analysis for the watershed upstream of the project site contributing to the storm drain facility LO1SO2, a hydraulic analysis at San Juan Creek to estimate the water surface elevation of the creek channel and overtopping, and a watershed analysis to map the floodplain inundation.

4.2 Hydrology Analysis

The hydrology analysis is based on the same method as outlined in Section 3.3 for the design storms analysis. The only exception is the total rainfall depth for a 500-year design storm is not available in Orange County Hydrology Manual, so 500-year design storm data from NOAA Atlas 14 is used for the analysis. Table 4.1 lists the estimated peak flow from a 500-year storm in the project watersheds.

Table 4.1 500-Year Storm Peak Flow Summary for Project Watersheds

Watershed	Peak Flow (cfs)
	500-year Expected Value
1A	41.6
1B	20.8
2	40.4
3	31.7
4	25.9
5	19.8
6	57.0
7	73.8
8	5.2
9	179.8
10	74.9
11	172.2
12	59.7
13	79.1
14	131.87

Watershed	Peak Flow (cfs)
	500-year Expected Value
15	2048.1
16	354.5
17	194.8
1A Developed	62.4
1B Developed	31.2

The 500-year design flow for San Juan Creek is based on FEMA FIS for Orange County, California and Incorporated Areas (FEMA 2009c). Table 7 in the FIS listed the San Juan Creek 500-year design flow of 80,000 cfs, at the City of San Juan Capistrano corporate limits.

4.3 Hydraulic Analysis

The hydraulic analysis included San Juan Creek hydraulic analysis in HEC-RAS model to estimate the creek water surface elevation, and watershed hydraulic analysis in PCSWMM model to estimate the floodplain inundation.

In the San Juan Creek hydraulic analysis, the 80,000 cfs 500-year design flow is input to the upstream extent of the model at Stonehill Drive (River Station 5917 in the model).

During the existing conditions Q500 flood scenario San Juan Creek overtops the river left bank onto the project site. Overtopping begins at the upstream and middle portions of the project site where flood waters from the creek combine with excess flood volumes from the open portion of LO1SO2. From the northern portions of the site, flood waters then move south paralleling the creek, following the site’s natural grades. Flood waters eventually reach their highest concentration in the southwest portion of the project site, near where the existing grades are lowest adjacent to the flood wall and the Highway 1 roadway prism. At this location, even though the water surface elevation in the creek is higher than the floodwall, flood water from the site re-enters the creek or flows under the bridge given that floodplain water surface elevation at the project site is generally higher than the lower portions of the creek during the peak flooding hours.

Table 4.2 below shows the time series that were developed for each storm drain facility outfall, and the corresponding cross section from the HEC-RAS model that was used to determine the values.

Table 4.2 500-Year Storm Outfall Boundary Conditions at San Juan Creek

HEC-RAS Model STA	1855	1502	4555	3955	3355
Outfall	OF1	OF2	OF3	OF4	OF5
Hour	500-year Storm Outfall Water Surface Elevations (ft, NAVD 88)				
0	14.55	13.76	25.04	23.09	20.67
1	14.52	13.73	25.03	23.05	20.64
3	16.04	15.11	26.66	24.88	22.39
6	17.39	16.3	28.01	26.42	23.87
9	18.14	17.13	28.76	27.27	24.68
12	19.28	18.25	29.9	28.53	25.91
15	26.44	27.87	35.94	34.07	30.27
17.25	27.01	28.72	41.54	38.17	31.37
18	27.06	28.76	41.36	38.01	31.27
21	25.7	26.84	33.37	32.38	29.24
24	18.13	17.13	28.74	27.26	24.67
27	14.08	13.32	24.44	22.5	20.11
30	10.92	10.02	21.11	18.17	16.15
33	10.7	9.7	20.71	17.78	15.7
36	10.28	9.14	20.1	17.13	15
39	10.1	8.92	19.81	16.85	14.67
42	9.99	8.86	19.73	16.78	14.59
45	9.95	8.83	19.69	16.73	14.55
48	9.92	8.81	19.65	16.69	14.52

The water surface elevations in San Juan Creek and storm drain facility outfalls sets the downstream boundary condition for the watershed model in PCSWMM. The watershed model analyzed the routing of stormwater flow via surface runoff and storm drain facilities, and estimated inundation extent, depth, and duration in the floodplain. Figure 4.1 mapped the floodplain extent and depth at the project site and the adjacent watershed under the pre-project condition.

4.4 500-Year Storm Facility Protection

The hydraulic analysis showed the project site and the surrounding area has significant floodwater inundation under a 500-year storm event. Figure 4.1 shows that some area has over 3’ of flood depth under the pre-project condition. The maximum flood depth at the project site is at elevation 28.3’.

Under the post-project condition, the recommended alternative will raise the project site to 28.2’, to protect the project site from a 100-year flood plus 1’ of freeboard. As the post-project ground elevation at the project site is 28.2’, the maximum 500-year flood elevation of 28.3’ would likely results in minimal flooding at the project site.

As a sensitivity analysis, an improvement scenario is tested to provide the project site with 500-year flood protection with 1’ of freeboard. Under this test condition, the recommended alternative would need to raise the project site to 29.3’, with approximately 90,000 CY of fill material. Figure 4.2 shows the resultant floodplain with this test scenario. Figure 4.3 shows that with the improvements, the flood inundation depth at the vicinity of the project site will be reduced, since the fill at the project site created a barrier to block off overtopping flow from San Juan Creek to the floodplain in the watershed.

This analysis shows that under the recommended improvement to raise the site to elevation 28.2’, the project site will likely have minimal inundation. Also considering the fact that the site improvement to provide 500-year flood protection with 1’ freeboard will result in approximately 23,000 CY of additional fill, this study recommends the project to raise the project site to 28.2’ to provide 100-year flood protection with 1’ freeboard, as per the Alternative 1a outlined in Section 3.6. In addition, the project will identify and include flood proofing design at the critical facilities to minimize potential flood impacts and the resultant damage and system downtime.

5. References

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Figures

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Grid: NAD 1983 StatePlane California III FIPS 0403 Feet

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GHD

South Coast Water District
Doheny Desalination Project - EIR

Job Number	11125157
Revision	A
Date	08 May 2017

Vicinity and Project Location

Figure 1.1

718 Third Street Eureka CA 95501 USA T 707 443 8326 F 707 444 8330 E eureka@ghd.com W www.ghd.com

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Data source: ESRI StreetMap/TeleAtlas; GHD GIS; Created by:jclark2

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Map Projection: Transverse Mercator
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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

6705 Millcreek Drive Unit 1 Mississauga ON L5N 5M4 Canada T 1 416 213 7121 F 1 905 890 8499 E mississauga@ghd.com W www.ghd.com

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

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

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LEGEND

Site Location
Extent of Assessment

Flooded Areas for Extremal Total Water Level

2100 Low Range Sea Level Rise (18.4 ft*)
2100 High Range Sea Level Rise (21.9 ft*)

*Source: Michael Baker International, 2018. Coastal Hazards Analysis for the Doheny Desalination Project. February 2018

GHD

Doheny Desalination Project
Coastal Hazard Assessment

Future Flooding 2100

Job Number 11140040
Revision A
Date 30 Nov 2018

Figure 2.2

6705 Millcreek Drive Unit 1 Mississauga ON L5N 5M4 Canada T 1 416 213 7121 F 1 905 890 8499 E mississauga@ghd.com W www.ghd.com

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, 2018, based on U.S. Army Corps of Engineers (USACE) digital elevation model, 2016. Created by:aboone

Paper Size ANSI A

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

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LEGEND

- Site Location
- CosMos Flood Limits - 0.5 m Sea Level Rise
- CosMos Flood Limits - 2.0 m Sea Level Rise
- CosMos Flood Limits - 5 m Sea Level Rise

Flood Data: Barnard, P.L., Erikson, L.H., Foxgrover, A.C., Limber, P.W., O'Neill, A.C., and Vitousek, S., 2018, Coastal Storm Modeling System (CoSMoS) for Southern California, v3.0, Phase 2 (ver. 1g, May 2018): U.S. Geological Survey data release, https://doi.org/10.5066/F7T151Q4.

Doheny Desalination Project
Coastal Hazard Assessment

Job Number	11140040
Revision	A
Date	30 Nov 2018

CosMos 3.0 Sea Level Rise

Figure 2.3

6705 Millcreek Drive Unit 1 Mississauga ON L5N 5M4 Canada T 1 416 213 7121 F 1 905 890 8499 E mississauga@ghd.com W www.ghd.com

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, 2018. Created by:aboone

Paper Size ANSI A

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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 (18.82 ft*)

2100 High Range Sea Level Rise (22.31 ft*)

*Source: Michael Baker International, 2018. Coastal Hazards Analysis for the Doheny Desalination Project. February 2018

Doheny Desalination Project
Coastal Hazard Assessment

Job Number 11140040
Revision A
Date 30 Nov 2018

Tsunami Flooding 2100

Figure 2.4

6705 Millcreek Drive Unit 1 Mississauga ON L5N 5M4 Canada T 1 416 213 7121 F 1 905 890 8499 E mississauga@ghd.com W www.ghd.com

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, 2018, based on U.S. Army Corps of Engineers (USACE) digital elevation model, 2016. Created by:aboone

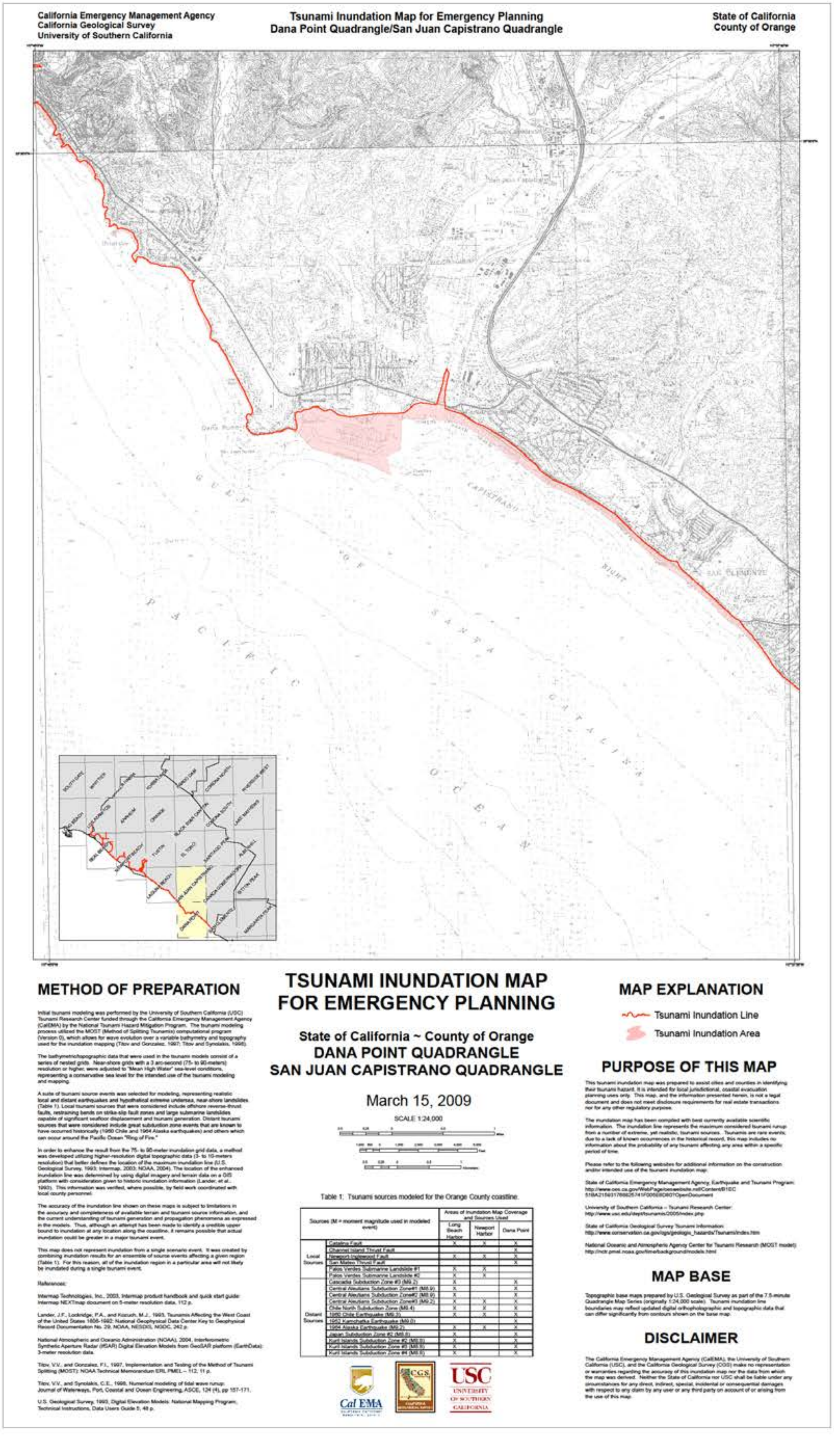


Figure 2.5 – Tsunami Inundation Map for Emergency Planning

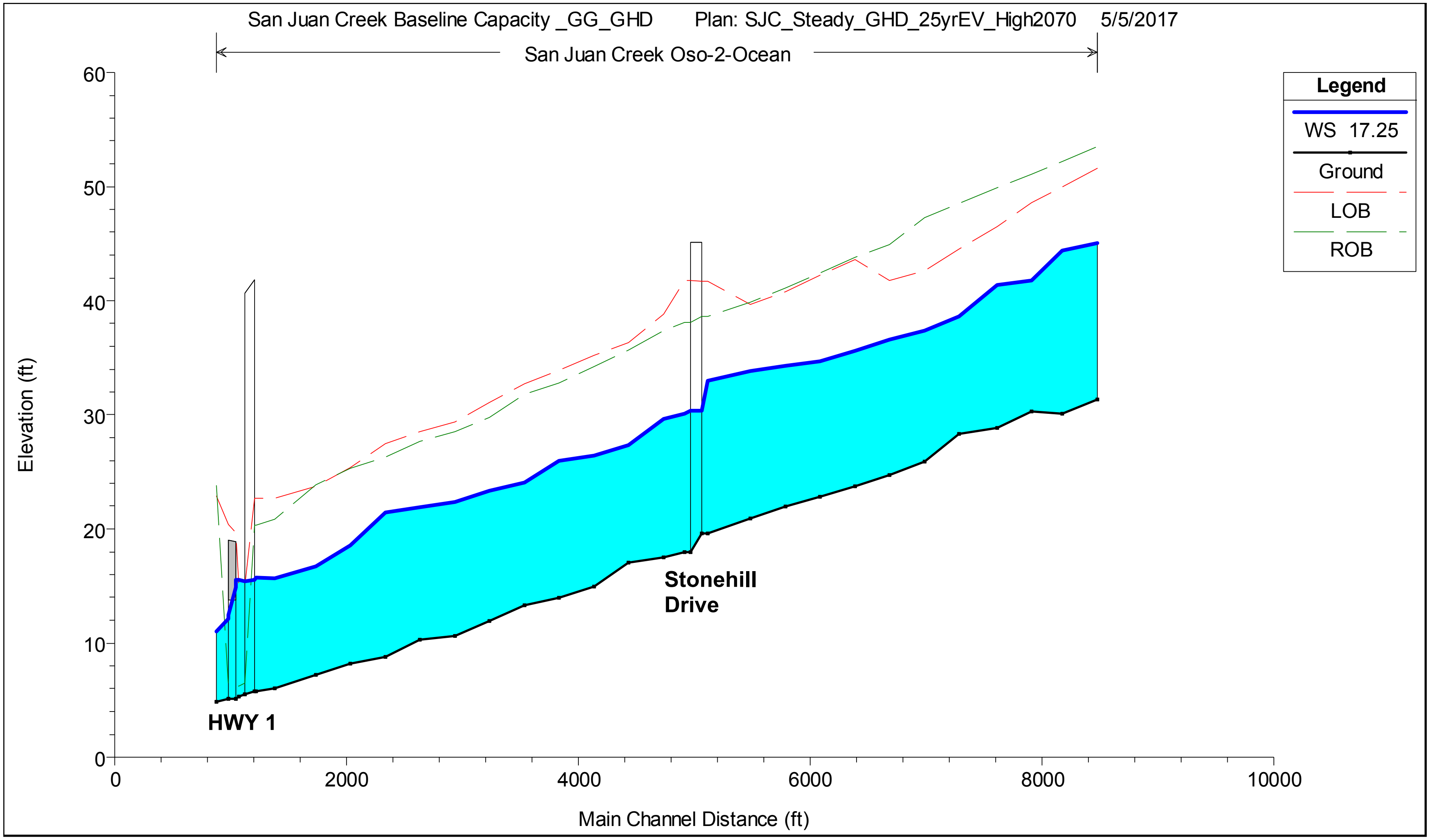


Figure 3.1 – San Juan Creek maximum water surface profile, 25-year event.

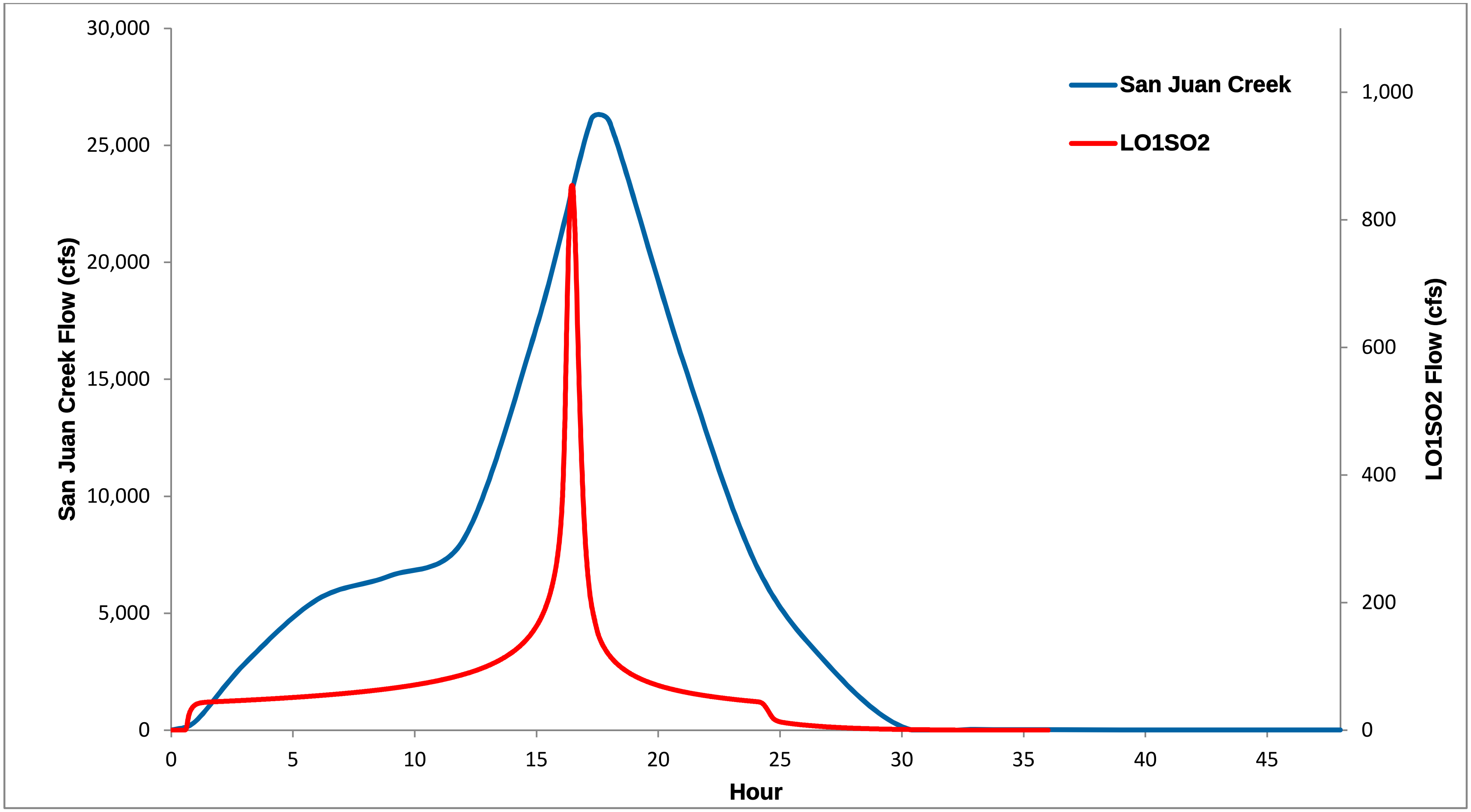


Figure 3.2 – San Juan Creek and LO1SO2 hydrograph comparison, 25-year event.

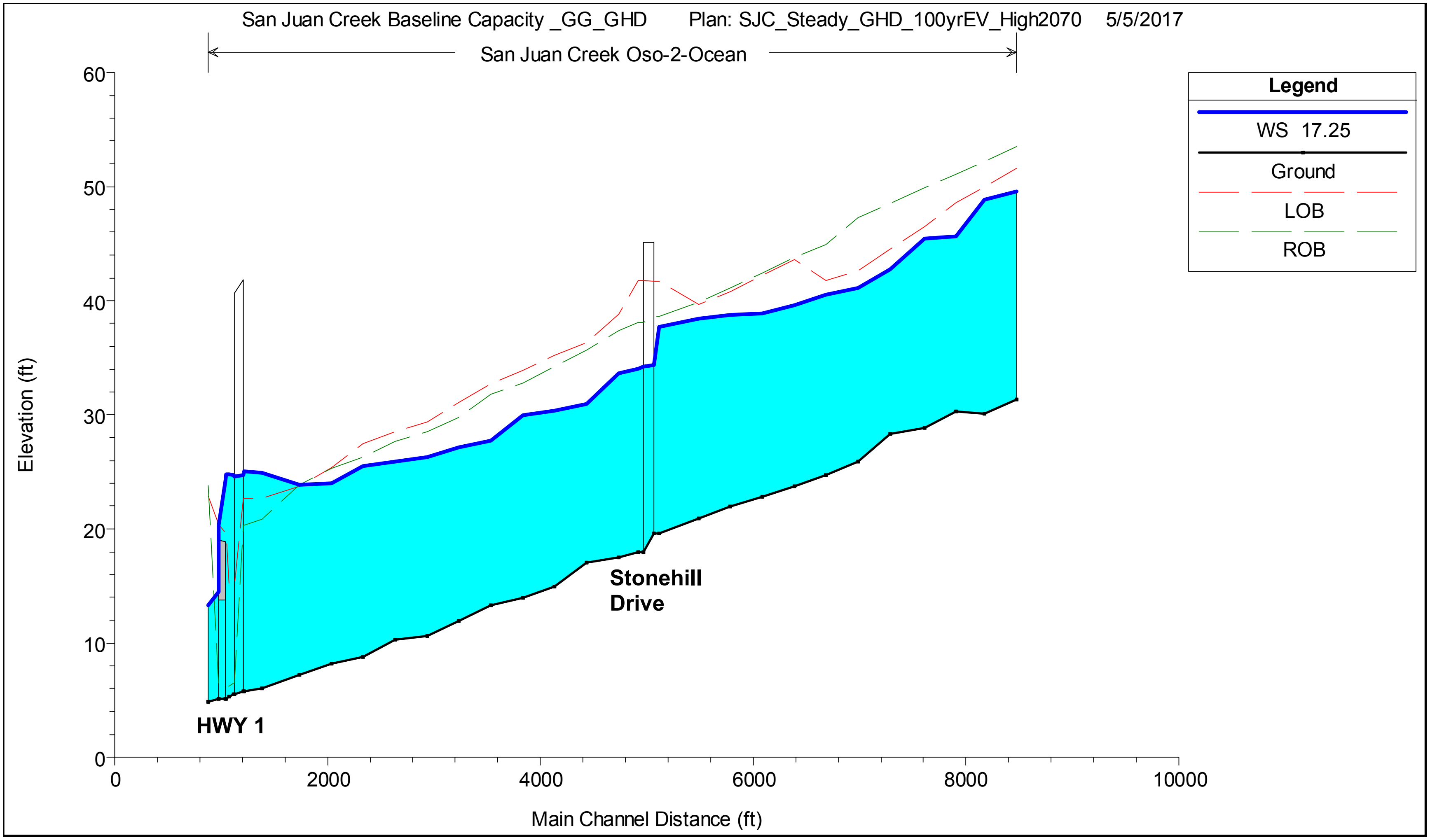


Figure 3.3 – San Juan Creek maximum water surface profile, 100-year event.

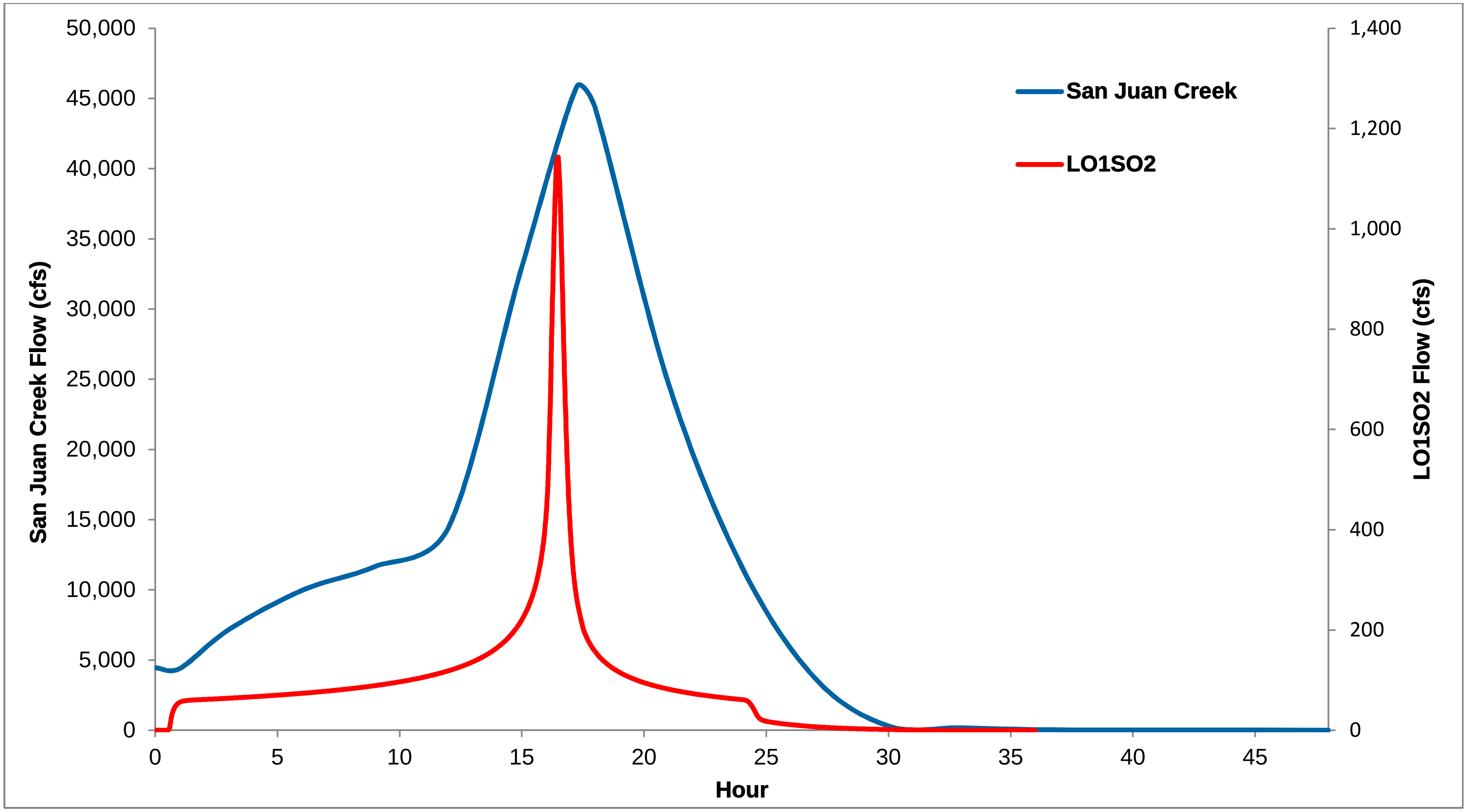
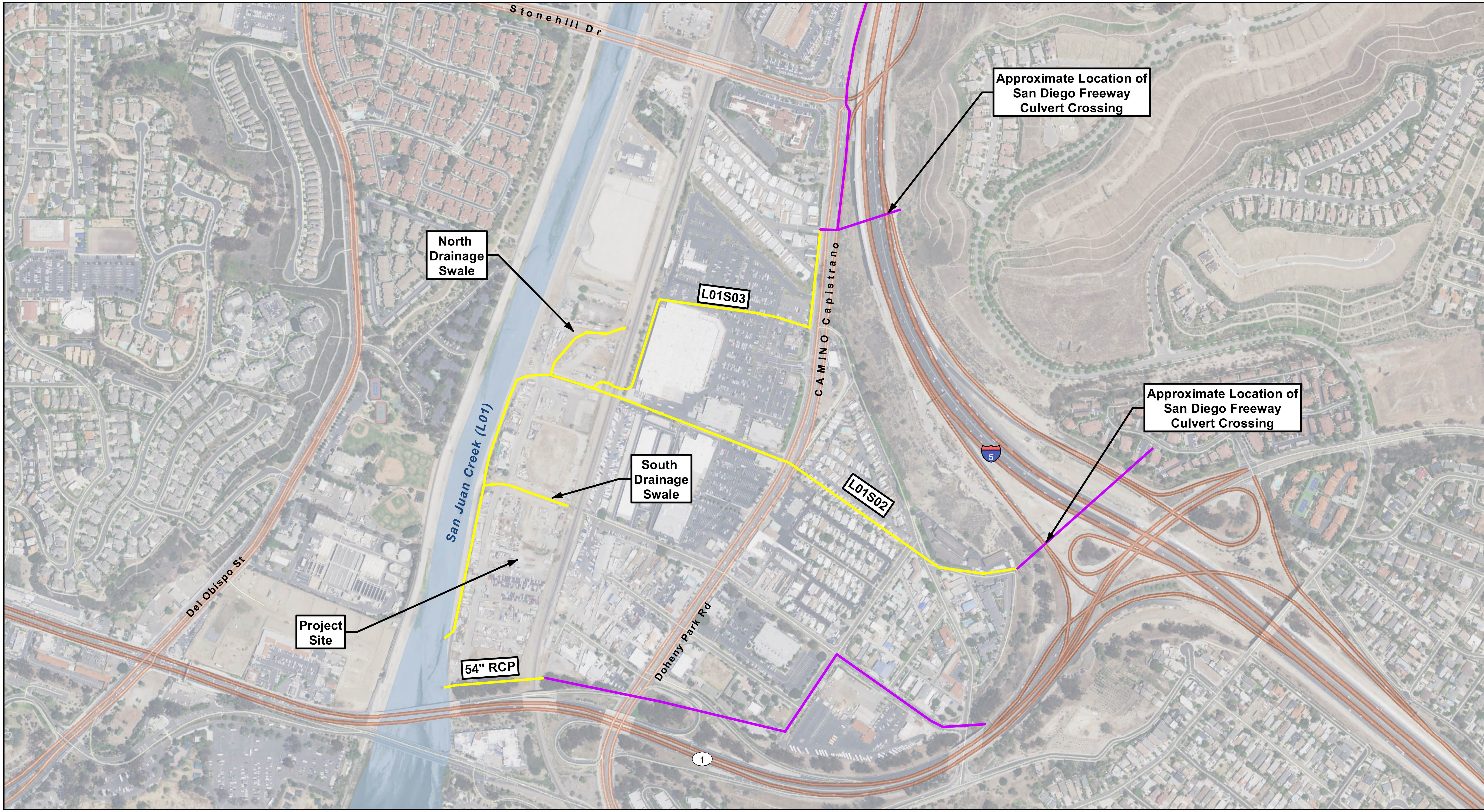


Figure 3.4 – San Juan Creek and LO1SO2 hydrograph comparison, 100-year event.



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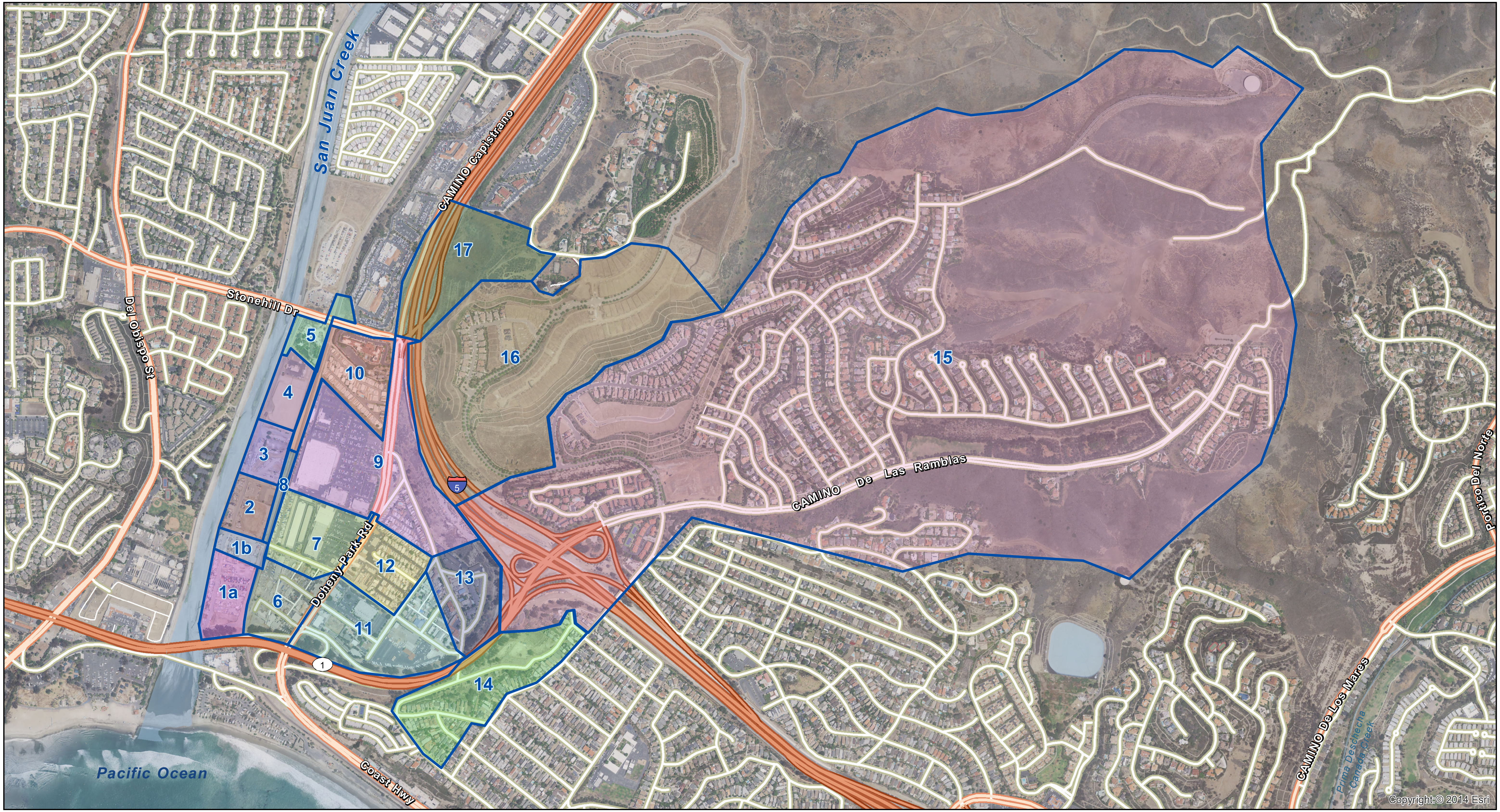
Data source: Aerial: NAIP, 2016; ESRI Street Map; GHD: watershed delineation. Created by:jclark2

718 Third Street Eureka CA 95501 USA T 707 443 8326 F 707 444 8330 E eureka@ghd.com W www.ghd.com



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LEGEND

Doheny Watersheds

15 Watershed ID

South Coast Water District
Doheny Desalination Project Drainage Analysis

Doheny Watersheds Overview Map

Job Number	11125157
Revision	A
Date	08 May 2017

Figure 3.6

718 Third Street Eureka CA 95501 USA T 707 443 8326 F 707 444 8330 E eureka@ghd.com W www.ghd.com

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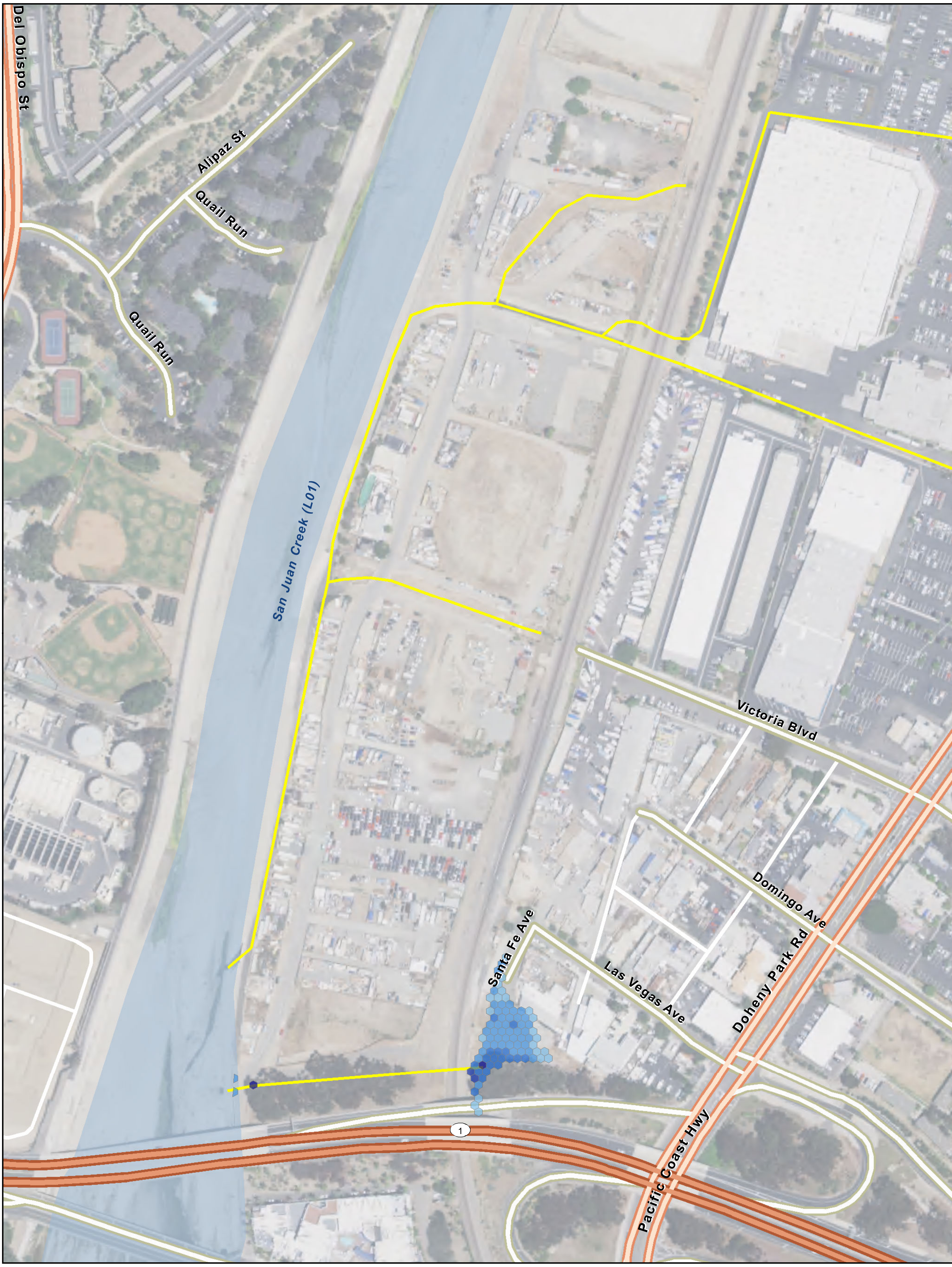

Storm Water Conduits (in model)

Maximum Flooding Depth

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

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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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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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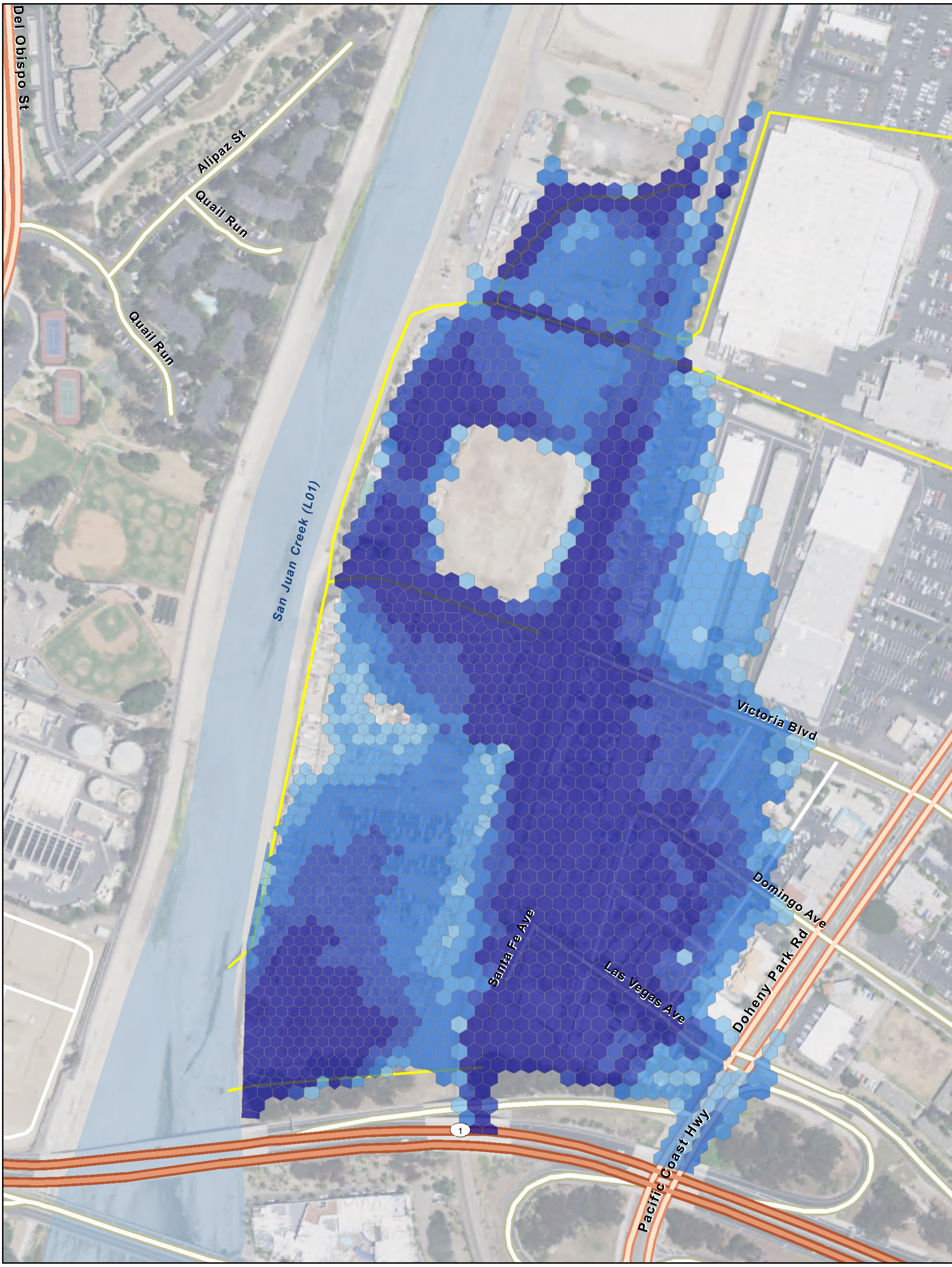
718 Third Street Eureka CA 95501 USA **T** 707 443 8326 **F** 707 444 8330 **E** eureka@ghd.com **W** www.ghd.com

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
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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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GHD

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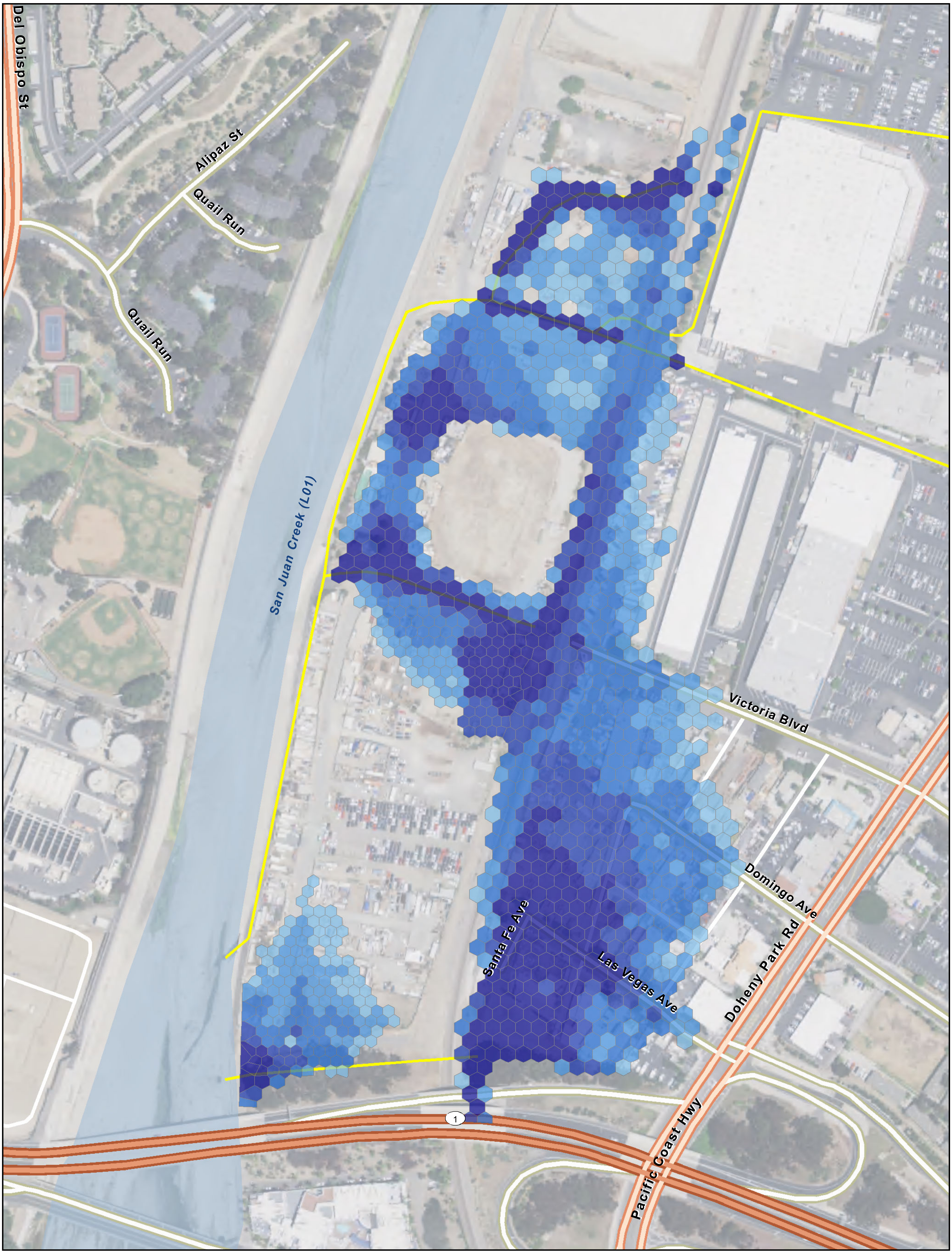
Data source: Aerial: NAIP, 2016; ESRI Street Map; GHD: elevation model. Created by:jclark2



Storm Water Conduits (in model)

Maximum Flooding Depth

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

Paper Size ANSI B

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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

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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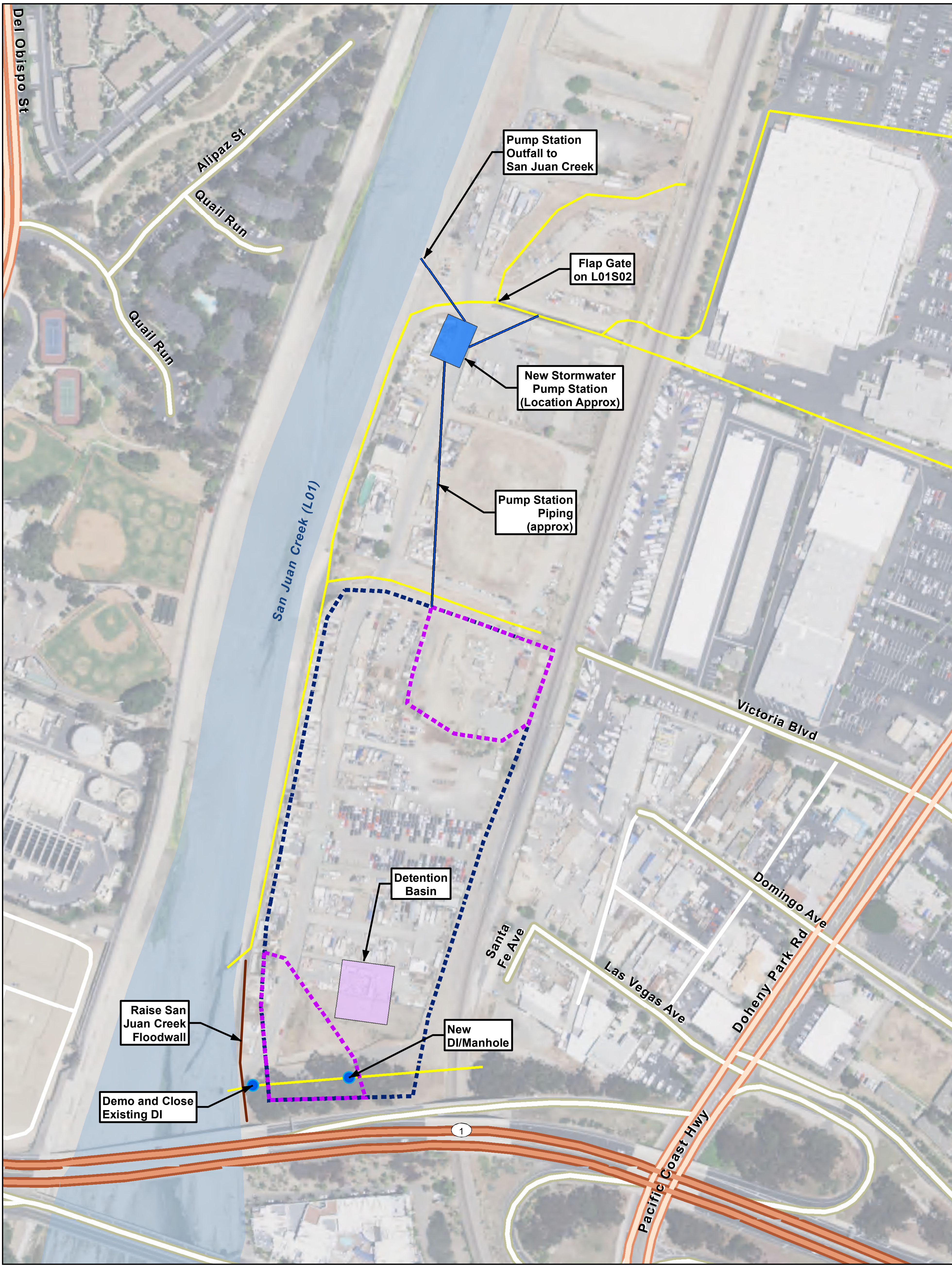
718 Third Street Eureka CA 95501 USA **T** 707 443 8326 **F** 707 444 8330 **E** eureka@ghd.com **W** www.ghd.com

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



Storm Water Conduits (in model)
Proposed Drainage Inlet
Proposed Floodwall
Proposed Pump Station Piping
Proposed Pump Station
Proposed Detention Basin
Approx. Limits of Grading, Alternative 4
Approx. Limits of Grading, Alternatives 1, 2, & 3

Paper Size ANSI B

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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

Job Number 11125157
Revision A
Date 08 May 2017

Proposed Improvement Options Figure 3.10

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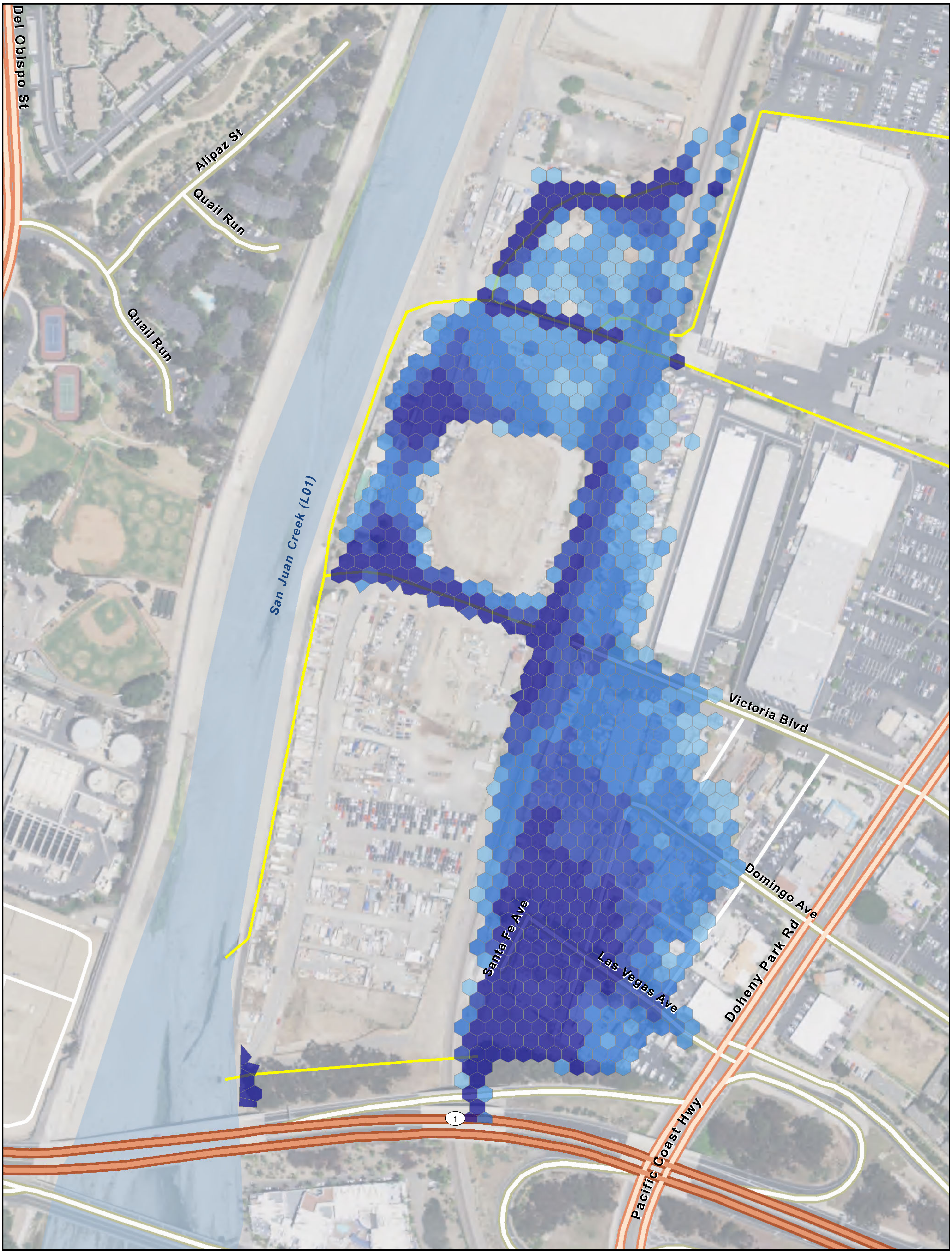
718 Third Street Eureka CA 95501 USA T 707 443 8326 F 707 444 8330 E eureka@ghd.com W www.ghd.com

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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GHD

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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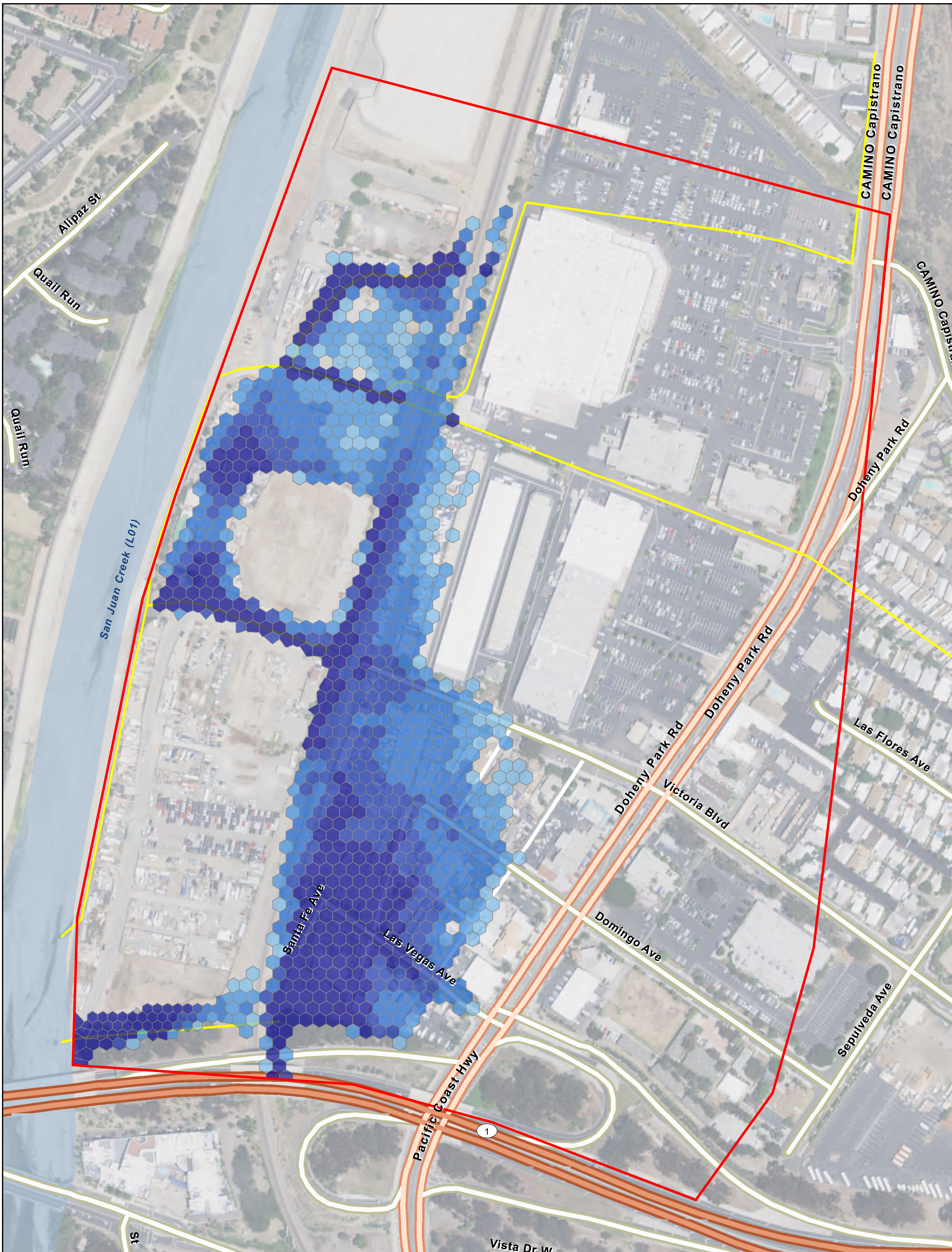

Storm Water Conduits (in model) | Two-Dimensional Model Boundary

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 1a

Job Number 11125157
Revision A
Date 12 Apr 2018

Figure 3.11a

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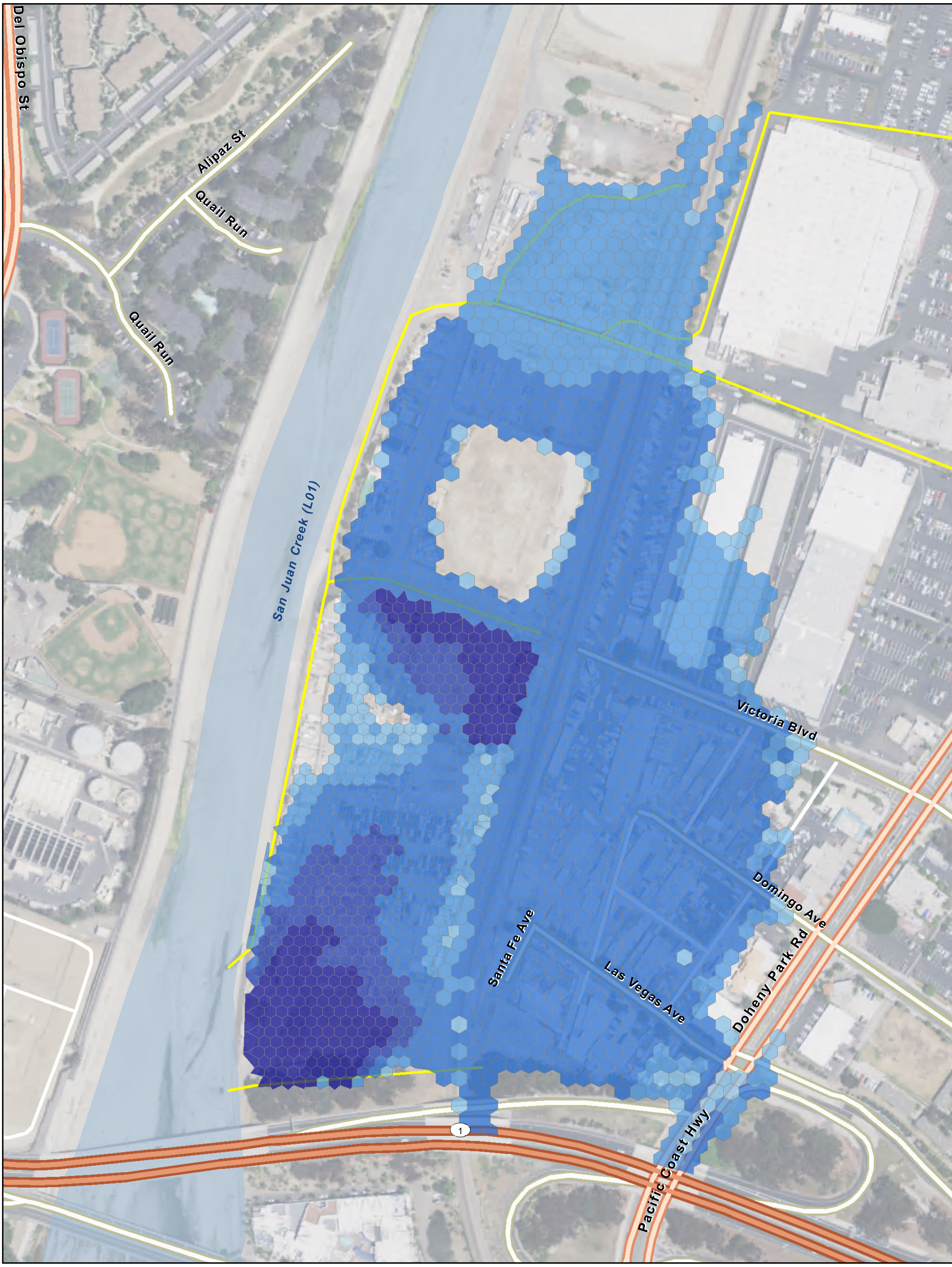
718 Third Street Eureka CA 95501 USA T 707 443 8326 F 707 444 8330 E eureka@ghd.com W www.ghd.com

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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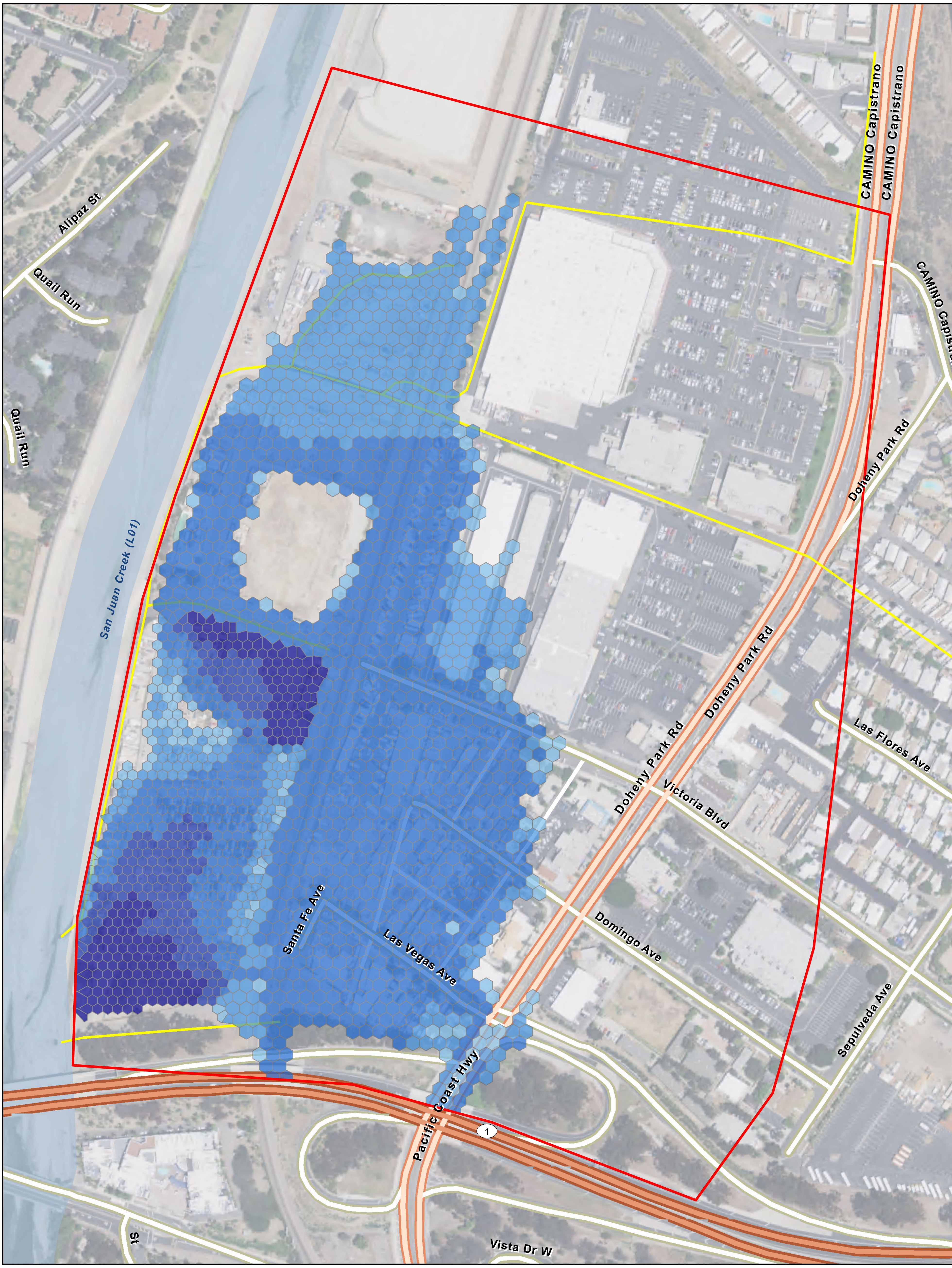
718 Third Street Eureka CA 95501 USA **T** 707 443 8326 **F** 707 444 8330 **E** eureka@ghd.com **W** www.ghd.com

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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GHD

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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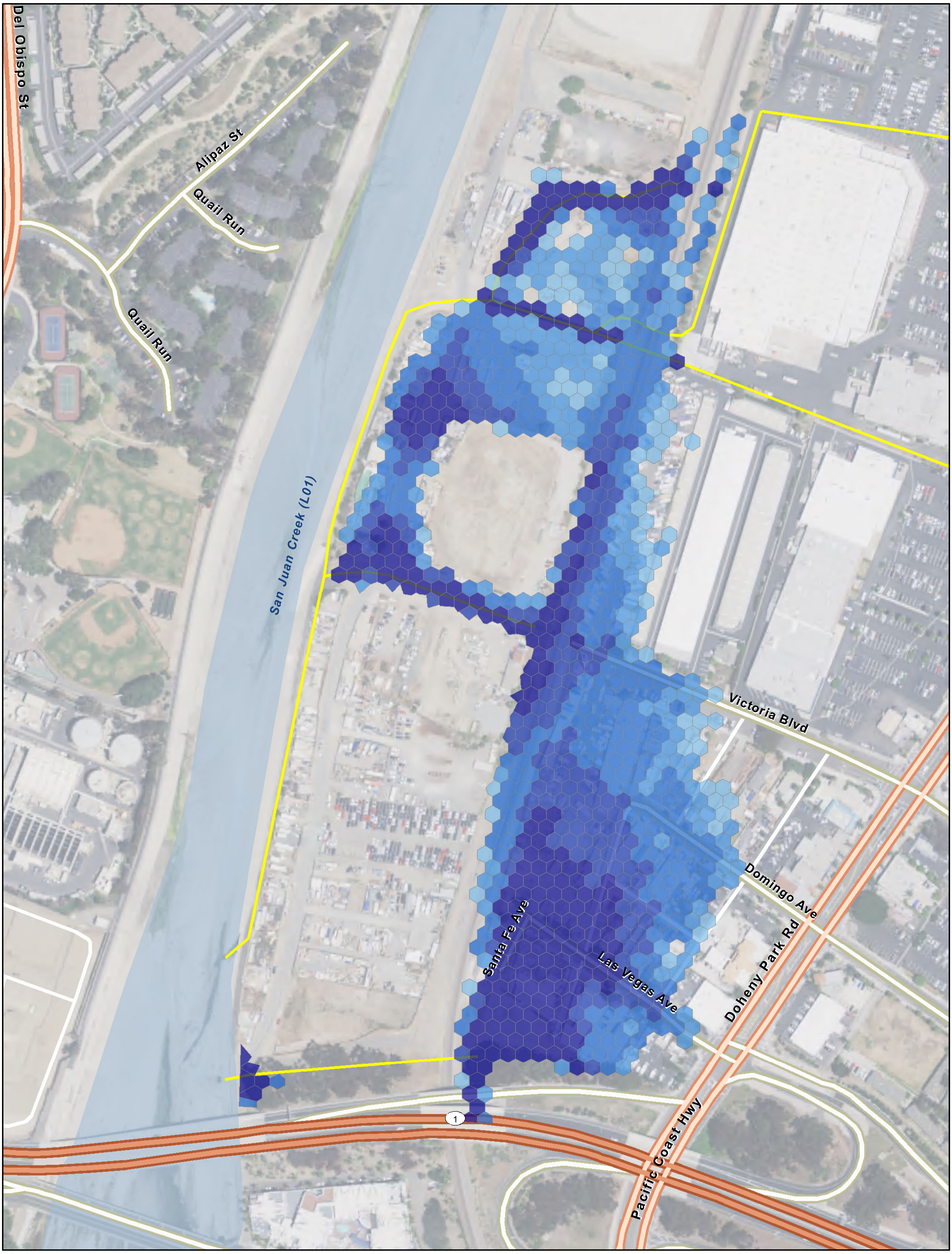

Storm Water Conduits (in model)

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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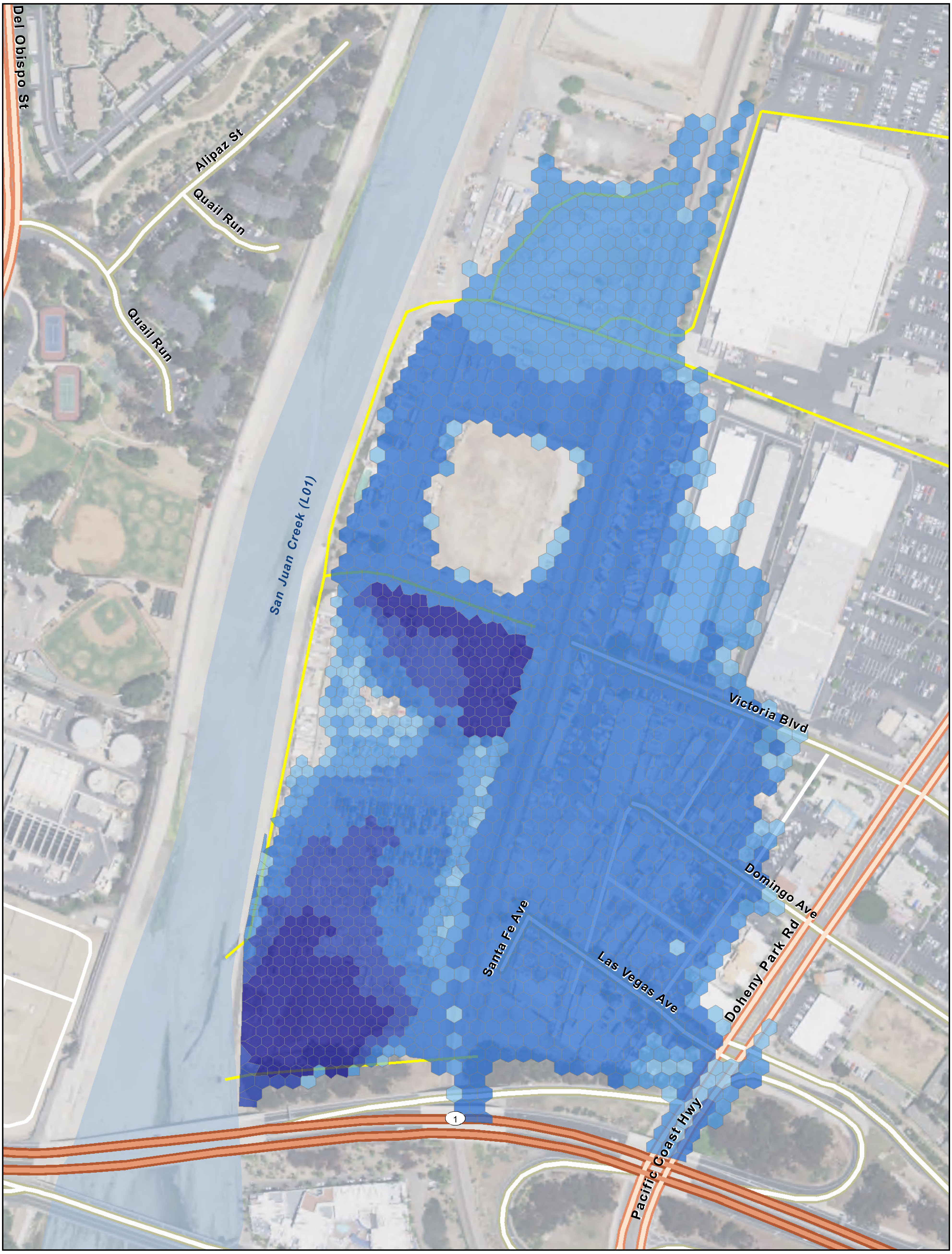

Storm Water Conduits (in model)

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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GHD

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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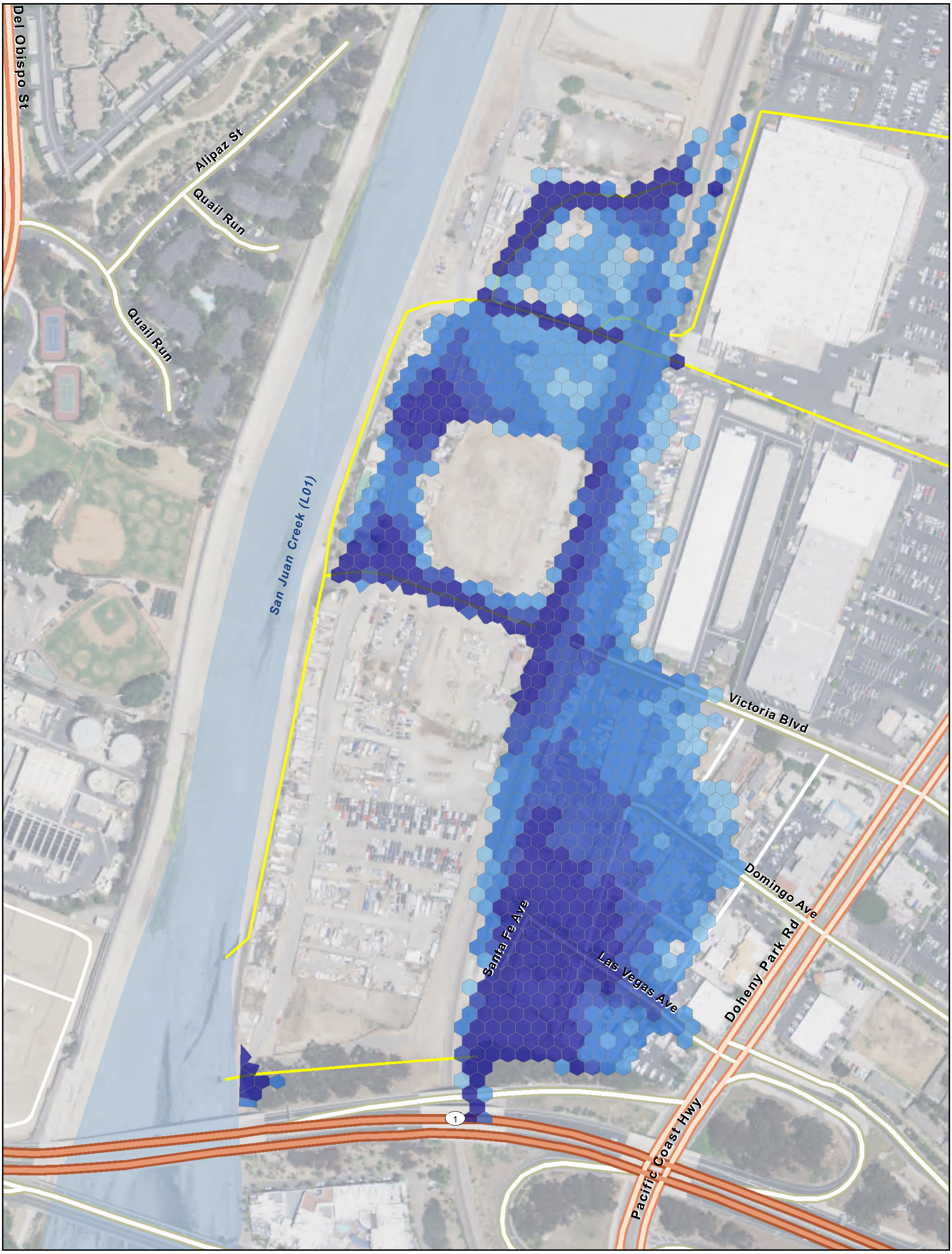
718 Third Street Eureka CA 95501 USA **T** 707 443 8326 **F** 707 444 8330 **E** eureka@ghd.com **W** www.ghd.com

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 3

Job Number	11125157
Revision	A
Date	09 May 2017

Figure 3.15

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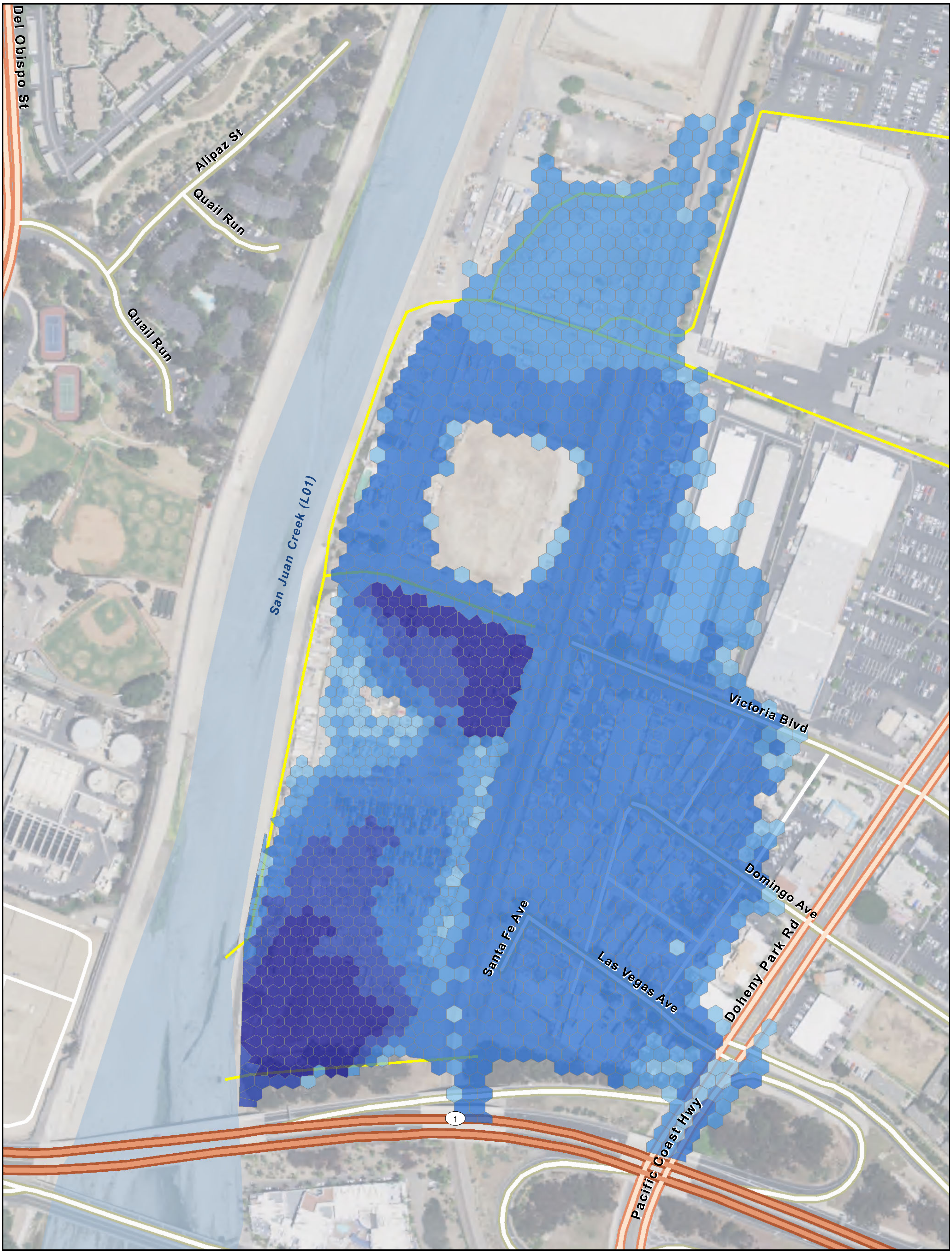

Storm Water Conduits (in model)

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

N

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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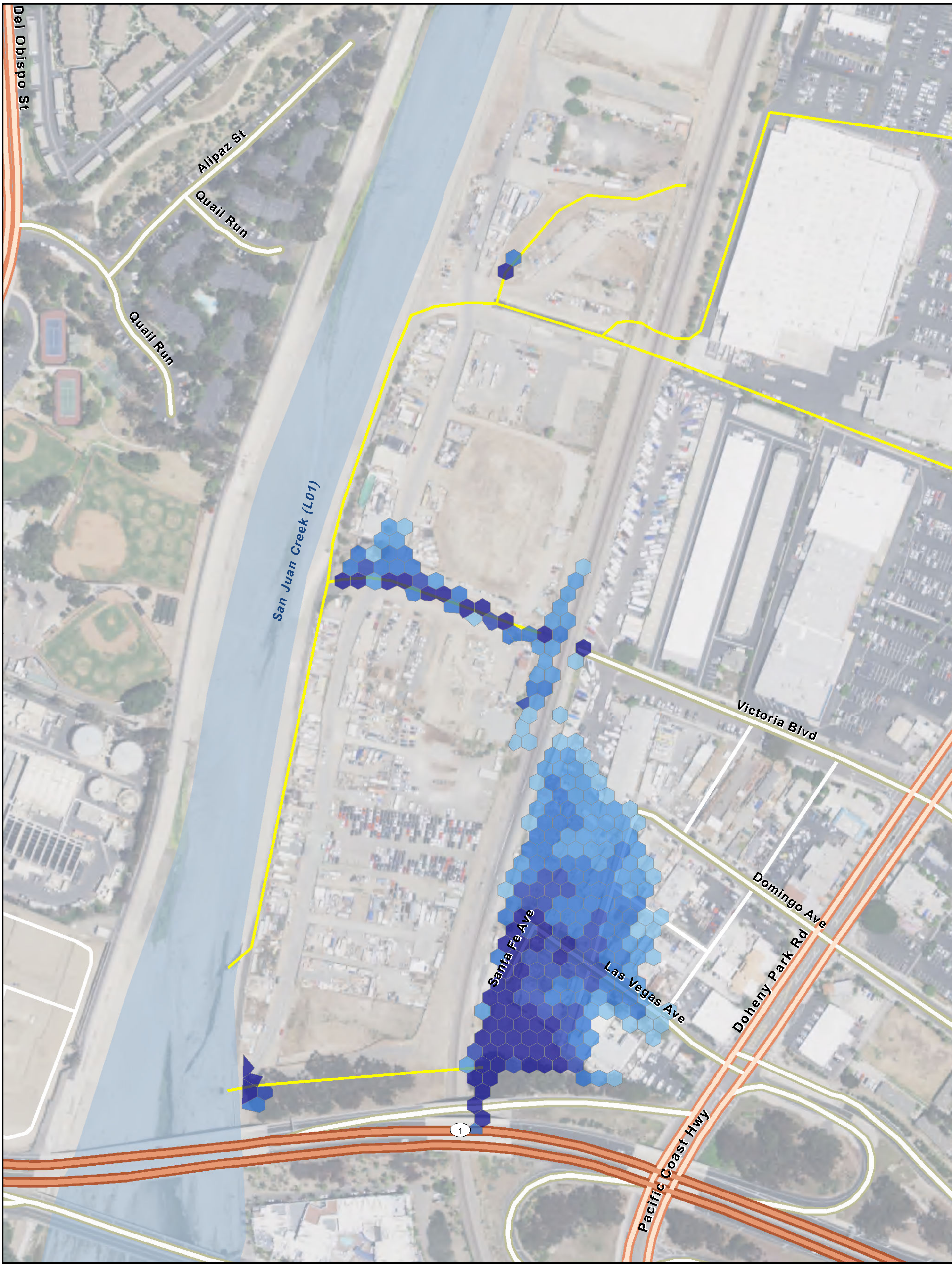

Storm Water Conduits (in model)

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

N

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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718 Third Street Eureka CA 95501 USA T 707 443 8326 F 707 444 8330 E eureka@ghd.com W www.ghd.com

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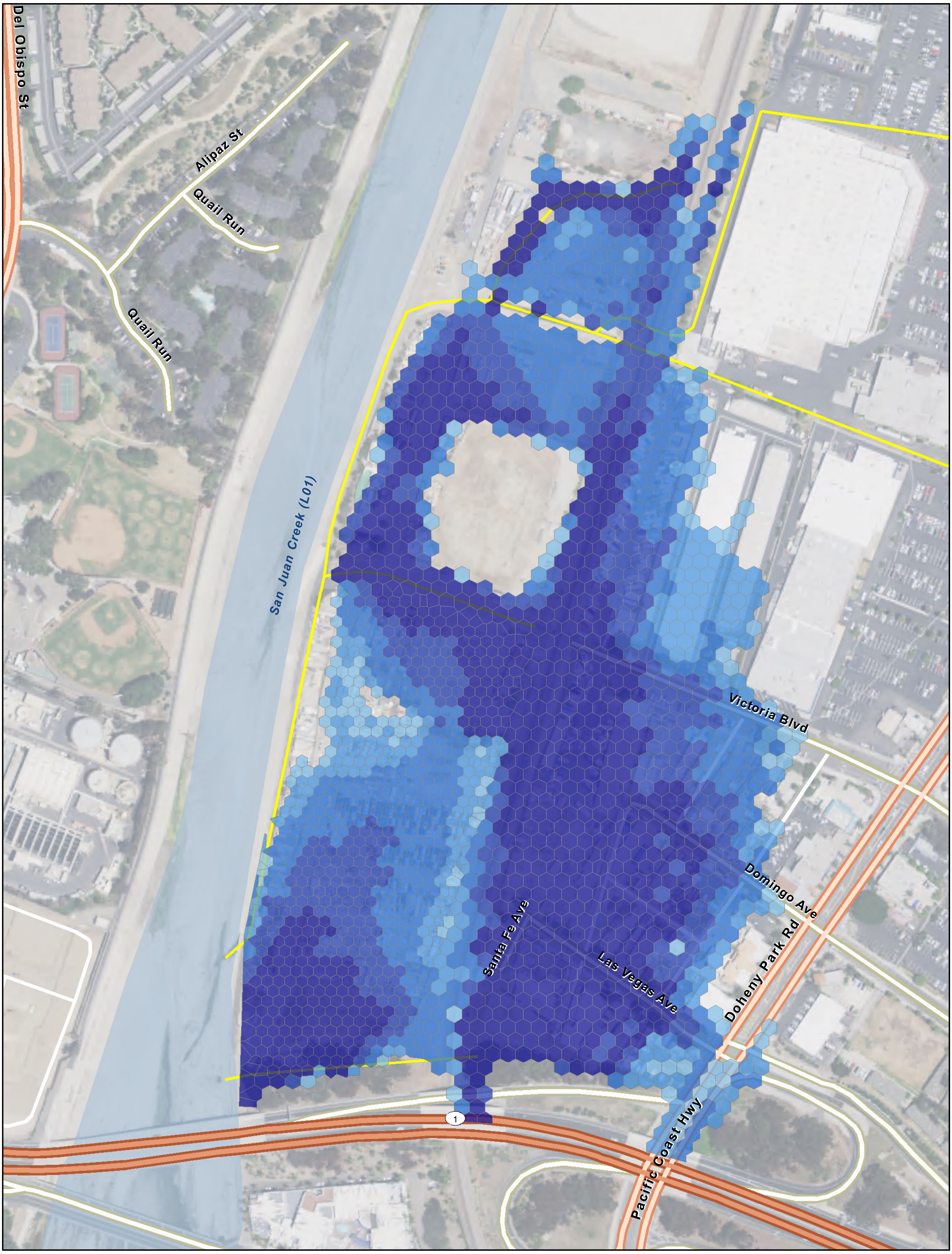

Storm Water Conduits (in model)

Reduction in Maximum Flood Depth

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

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

N

GHD

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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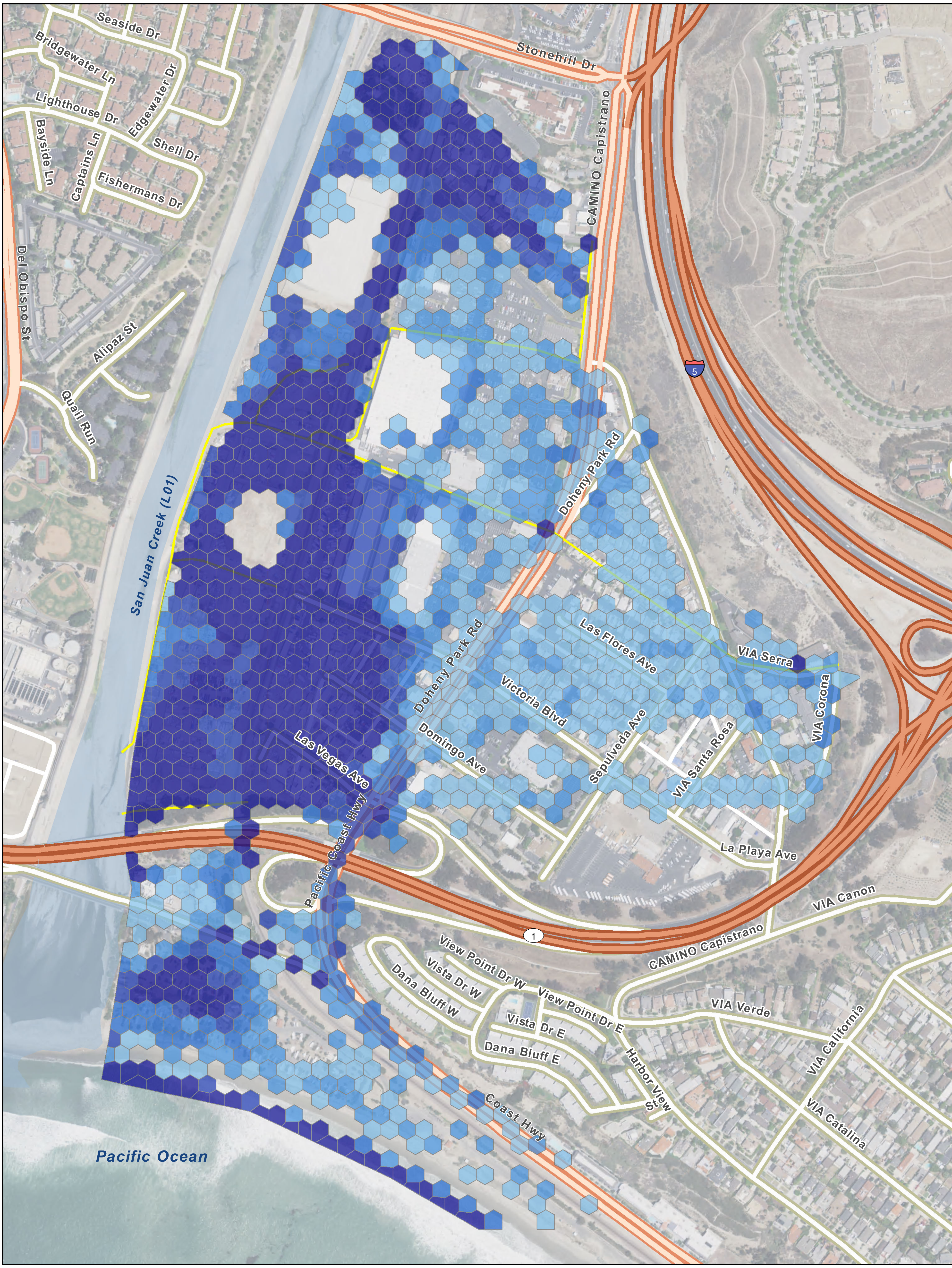

Storm Water Conduits (in model)

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, 500-Year Event, Existing Condition

Job Number 11125157
Revision A
Date 16 Jan 2019

Figure 4.1

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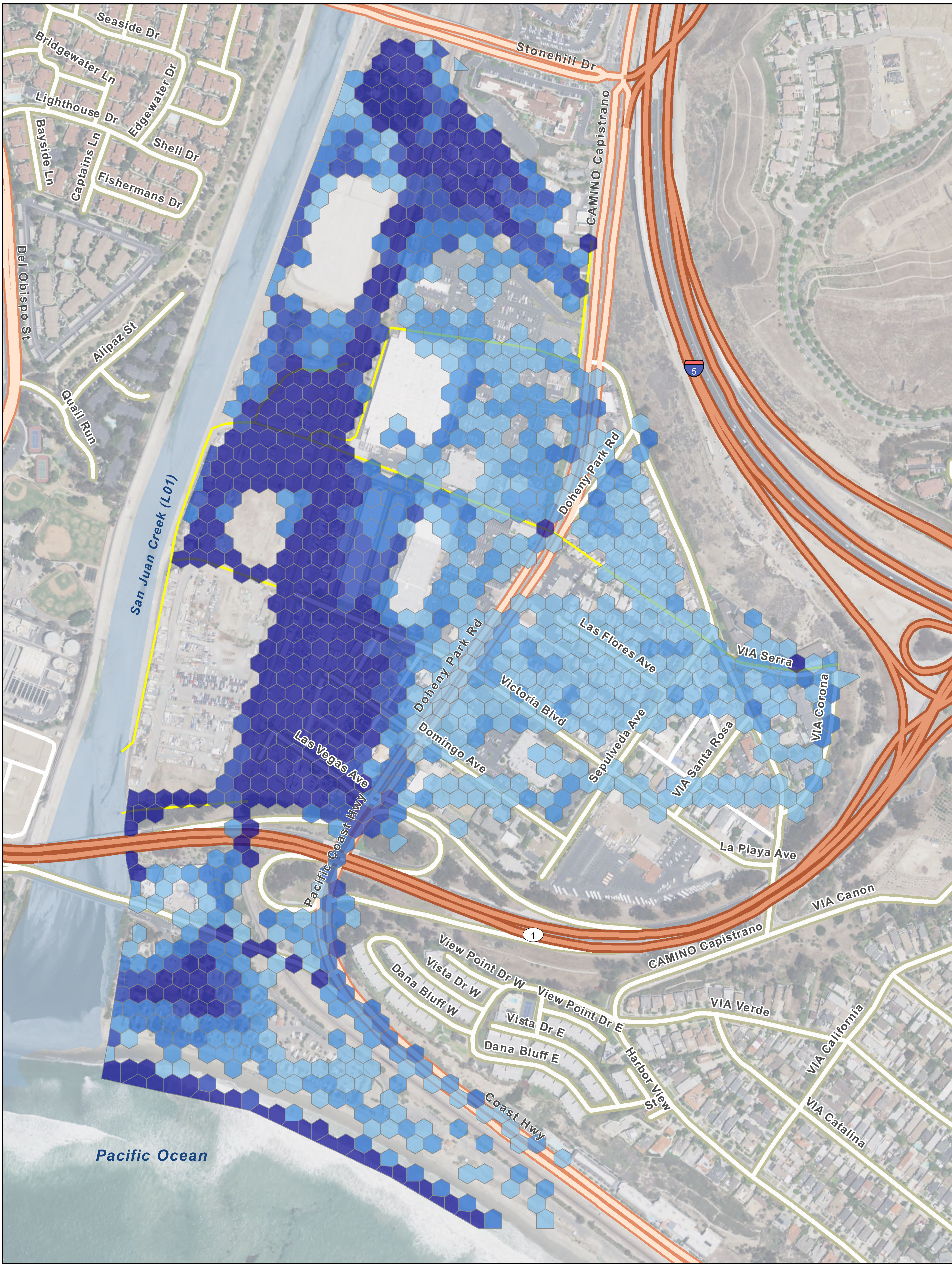

Storm Water Conduits (in model)

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

GHD

South Coast Water District
Doheny Desalination Project Drainage Analysis

Flood Inundation Map, 500-Year Event, Future Condition

Job Number 11125157
Revision A
Date 16 Jan 2019

Figure 4.2

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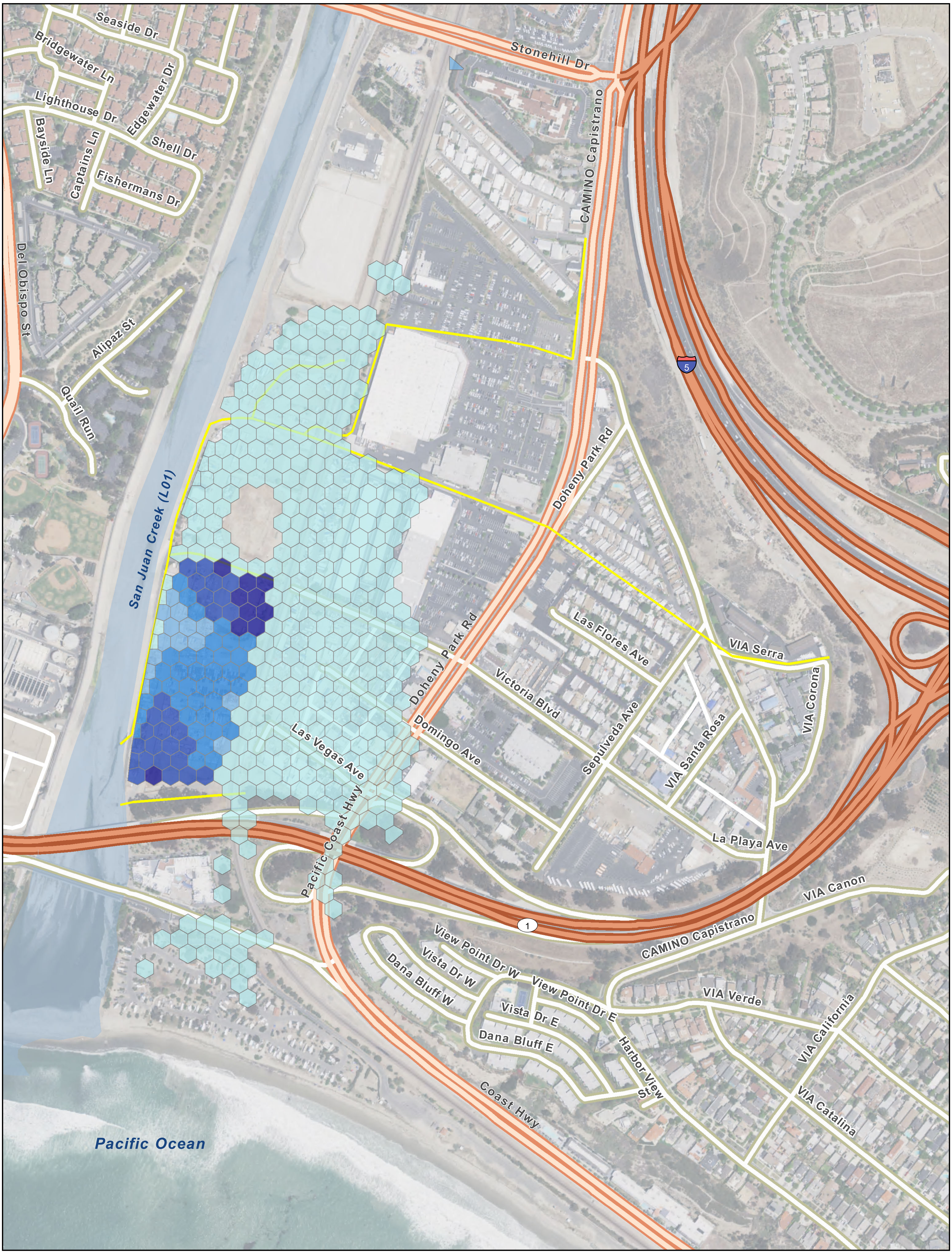
718 Third Street Eureka CA 95501 USA T 707 443 8326 F 707 444 8330 E eureka@ghd.com W www.ghd.com

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



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

GHD

South Coast Water District

Doheny Desalination Project Drainage Analysis

Change in Flood Inundation, 500-Year Event, Existing vs Future Condition

Job Number 11125157
Revision A
Date 16 Jan 2019

Figure 4.3

\\ghdnet\ghd\US\Irvine\Projects\111\11125157- SCWD TO #8B\08-GIS\Maps\Deliverables\380_11125157_500_RevA.mxd

718 Third Street Eureka CA 95501 USA T 707 443 8326 F 707 444 8330 E eureka@ghd.com W www.ghd.com

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

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.

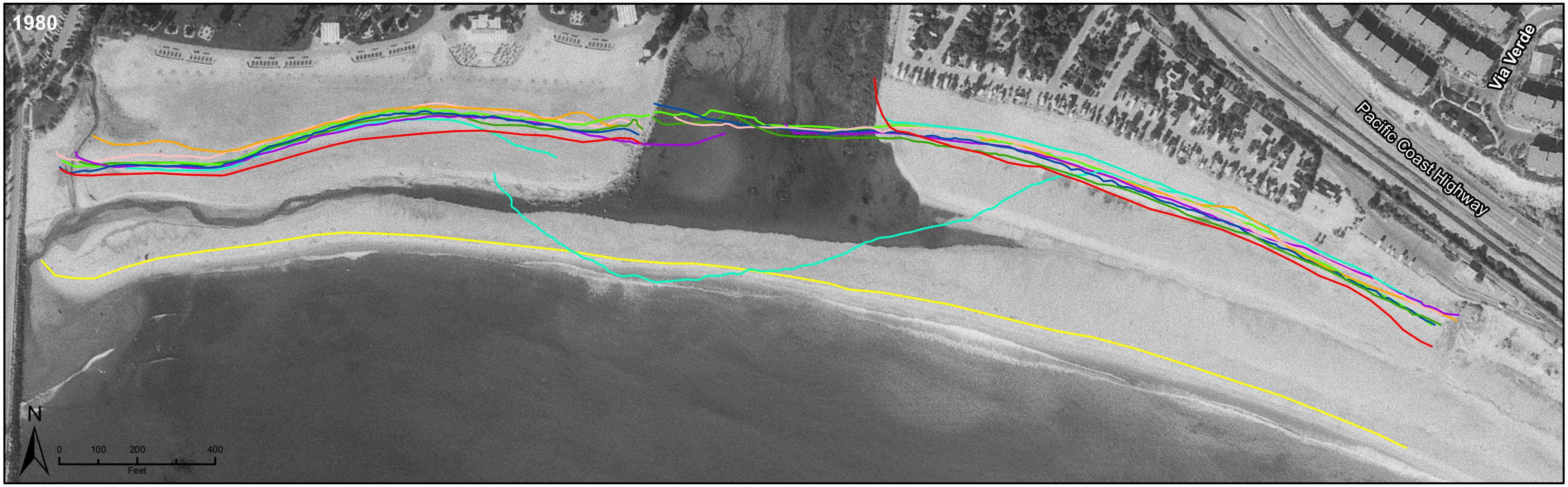
An historical aerial photographic assessment was conducted to confirm the observations that the beach width has been relatively stable by using older aerial photographs to determine if there has been any systematic shoreline change over time. The shoreline was traced in aerial photographs for 1975, 1980, 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 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

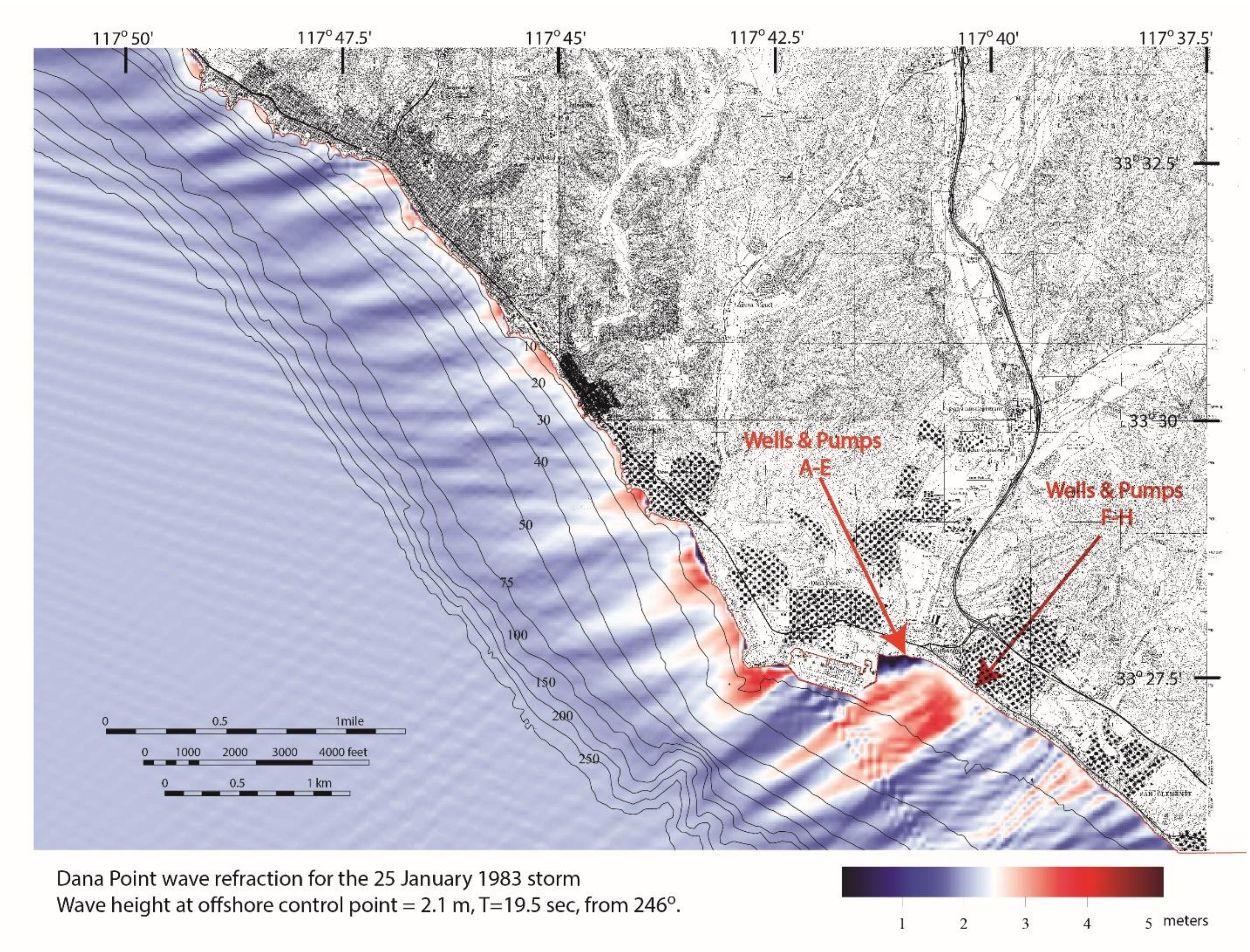
ATTACHMENT A
DATE: MARCH 2017
PROJECT: 11140040
DRAWN BY: J.D., R.G.

Appendix B:

Coastal Hazards Analysis

Coastal Hazards Analysis for the Doheny Desalination Project for the Final EIR

By Scott A. Jenkins, Ph.D.



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°.

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

TABLE OF CONTENTS

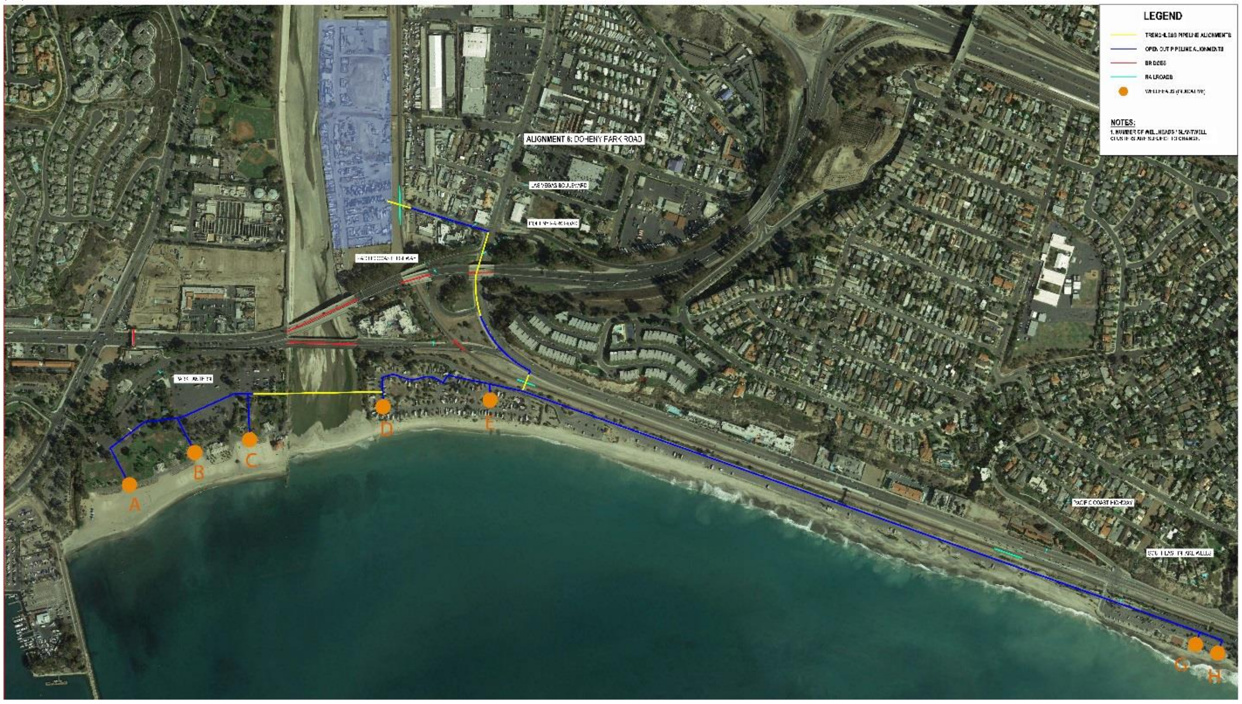
EXECUTIVE SUMMARY:

This 2019 study, prepared in response to comments for the Final EIR, provides further analysis to amplify the Coastal Hazards Analysis prepared in 2017 for the Draft EIR of the Doheny Desalination Project (DDP). That earlier work is being amplified herein in response to a revision of the *California Coastal Commission Sea Level Rise Policy Guidance* document that was originally released in August 2015, (CCC, 2015), but has been updated in July 2018 with new sea level rise projections. In addition, there have been minor adjustments in the locations of a number of the well heads and pump stations being proposed for the Doheny Desalination Project. The following study accounts for these intervening changes in policy guidance and minor modifications to the project description.

The primary analysis tool used in this study is the *Coastal Evolution Model* (CEM) developed at the Scripps Institution of Oceanography was used to evaluate Appendix-B requirements of the *California Coastal Commission Sea Level Rise Policy Guidance* document (CCC, 2015) for a sea level rise/coastal hazards analysis of the 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 grain 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 Oceanic and Atmospheric Administration (NOAA); and stream flow and sediment flux for the San Juan Creek from the United States Geological Survey (USGS) and the Federal Emergency Management Agency (FEMA). Sea level rise projections used in this study were based on the *best fit equation* from Appendix-B of the *California Coastal Commission 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 or tsunami concurrent with extreme ocean water levels and sea level rise are placed at two sites, namely Doheny State Beach and Capistrano Beach Park (cf. Figure ES-1a & b). At the Doheny Beach site, five potential locations are being evaluated for vaulted well heads with submersible pumps, including: ***Well Head A***, elevation 17 ft. NAVD, at 33°27'44.38"N, 117°41'16.32"W; ***Well Head B***, elevation 17 ft. NAVD, at 33°27'45.07"N, 117° 41'10.30"W; ***Well Head C***, elevation 17 ft. NAVD at 33°27'45.12"N, 117°41'6.62"W; ***Well Head D***, at elevation 18 ft. NAVD at 33°27'44.48"N, 117°40'55.30"W; and ***Well Head E***, at elevation 18 ft. NAVD at 33°27'42.45"N, 117°40'47.33"W; (see Figure ES-1a). Two additional vaulted well heads with submersible pumps are being evaluated at the Capistrano Beach site (Figure ES-1b), which includes: ***Well Head G***, at elevation 18 ft. NAVD at 33°27'14.94"N, 117° 39'59.91"W; and ***Well Head H***, at elevation 19 ft. NAVD at 33°27'13.17"N, 117°39'57.15"W.

This study is based on sea level rise projections appearing in Appendix-G, Table G-11, of the recently updated *California Coastal Commission Sea Level Rise Policy Guidance* document (CCC, 2018). This document provides no specific guidance on the redline frequency for flooding or inundation. In the absence of such guidance we have adopted FEMA standards for flooding

(a)



(b)

Figure ES-1: Critical shore-front infrastructure locations for the Doheny Desalination Project: a) Doheny State Beach site; and b) Capistrano Beach Park site 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 14 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 at present and at year 2100 sea levels, is summarized in Table ES-1a for structures at the Doheny Beach site and Table ES-1 b for the Capistrano Beach site. Inspection of Table ES-1a & b reveals that all the beach front well sites 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 for event return periods between 1 yr. and 100 yr. However, once we admit to 2100 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 low-range 2100 sea level projections, the three well sites on the north side of San Juan Creek (Well Heads A-C) and one of the wells at the Capistrano Beach site (Well Head G) will experience minor overtopping, even for a 1 year event if the beaches have been accreted by additional sands from water shed floods or still retain a built-out summer equilibrium beach profile, with overtopping rates of about Q'(1yr) = 0.038 cfs per lineal ft. of shoreline. However, if a 100-yr total water level event occurs during the low-range projection of 2100 sea levels, then all of the well sites will be overtopped to varying degrees if the beaches remain in an accreted condition with elevated berms and steep beach slopes. Under these beach conditions, overtopping rates will range from a high of Q'(100yr) = 0.094 cfs per lineal ft. of shoreline at Well Heads A- C, to a low of Q'(100yr) = 0.014 cfs/ft at Well Head H. Interestingly enough, none of the well heads would experience overtopping during a 100 year event when occurring during the low range 2100 sea levels if the beach were eroded, which would be the most likely condition during a 100-year event. 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-1a indicates the 100 year total water level events at the Doheny Beach site reach TWL(100) = 21.9 ft. NAVD for the steeply sloping accreted beach conditions and TWL(100) = 20.2 ft. NAVD for the eroded beach conditions. At the Capistrano Beach site, shoaling wave heights are higher and total water levels for a 100 year event superimposed on the high range projections for 2100 sea levels produce total water levels reaching TWL(100) = 22.7 ft. NAVD for the steeply sloping accreted beach conditions and TWL(100) = 21.1 ft. NAVD for the eroded beach conditions. Consequently, all beach front well head vaults for the Doheny Desalination Project will be overtopped when extreme waves happen concurrently with extreme ocean water levels that are superimposed on the high range of 2100 sea levels. The lowest lying well heads (Well Heads A-C) would experience the highest overtopping rates, ranging from Q'(100yr) = 0.216 cfs/ft. to 0.331 cfs/ft. depending on the eroded or accreted condition of Doheny State Beach. According Table VI-5-6 in the Coastal Engineering Manual (USACE, 2006) overtopping rates of this order of magnitude are very dangerous for pedestrian and vehicle traffic, and may cause structural damage to adjacent buildings, but the well heads and pumps for the Doheny Desalination project will be protected by steel vault enclosures. The smallest overtopping rates during the 100-year event at the high range

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

	Well Head-A Elevation = 17 ft. NAVD	Well Head-B Elevation = 17 ft. NAVD	Well Head-C Elevation = 17 ft. NAVD	Well Head-D Elevation = 18 ft. NAVD	Well Head-E Elevation = 18 ft. NAVD
*$TWL(1)$ Present Sea Level (eroded/accreted)	8.7/10.5 ft. NAVD status = dry	8.7/10.5ft. NAVD status = dry	8.7/10.5ft. NAVD status = dry	8.7/10.5ft. NAVD status = dry	8.7/10.5ft. NAVD status = dry
*$Q'(1)$ 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.
*$TWL(1)$ 2100 Sea Level Low Range Projection (eroded/accreted)	12.3/14.1 ft. NAVD status = dry	12.3/14.1 ft. NAVD status = dry	12.3/14.1 ft. NAVD status = dry	12.3/14.1 ft. NAVD status = dry	12.3/14.1 ft. NAVD status = dry
*$Q'(1)$ 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.
*$TWL(1)$ 2100 Sea Level High Range Projection (eroded/accreted)	15.8/17.6 ft. NAVD status = flooded accreted beach	15.8/17.6 ft. NAVD status = flooded accreted beach	15.8/17.6 ft. NAVD status = flooded accreted beach	15.8/17.6 ft. NAVD status = dry	15.8/17.6 ft. NAVD status = dry
*$Q'(1)$ 2100 Sea Level High Range Projection (eroded/accreted)	0.0/0.038 cfs/ft.	0.0/0.038 cfs/ft.	0.0/0.038 cfs/ft.	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.
**$TWL(100)$ 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.
**$TWL(100)$ 2100 Sea Level Low Range Projection (eroded/accreted)	16.7/18.4 ft. NAVD status = flooded accreted beach	16.7/18.4 ft. NAVD status = flooded accreted beach	16.7/18.4 ft. NAVD status = flooded accreted beach	16.7/18.4 ft. NAVD status = flooded accreted beach	16.7/18.4 ft. NAVD status = flooded accreted beach
**$Q'(100)$ 2100 Sea Level Low Range Projection (eroded/accreted)	0.0/0.094 cfs/ft.	0.0/0.094 cfs/ft.	0.0/0.094 cfs/ft.	0.0/0.027 cfs/ft.	0.0/0.027 cfs/ft.
**$TWL(100)$ @ 2100 Sea Level High Range Projection (eroded/accreted)	20.2/21.9 ft. NAVD status = flooded	20.2/21.9 ft. NAVD status = flooded	20.2/21.9 ft. NAVD status = flooded	20.2/21.9 ft. NAVD status = flooded	20.2/21.9 ft. NAVD status = flooded
**$Q'(100)$ 2100 Sea Level High Range Projection (eroded/accreted)	0.216/0.331 cfs/ft.	0.216/0.331 cfs/ft.	0.216/0.331 cfs/ft.	0.149/0.263 cfs/ft.	0.149/0.263 cfs/ft.

*Evaluated for the 1-yr return period; ** Evaluated for the 100-yr return period

Table ES-1b: Capistrano Beach Extremal Total Water Level (TWL) and Overtopping Rates (Q')

	Well Head-G Elevation = 18 ft. NAVD	Well Head-H Elevation = 19 ft. NAVD
*$TWL(1)$ Present Sea Level (eroded/accreted)	9.7/11.5 ft. NAVD status = dry	9.7/11.5 ft. NAVD status = dry
*$Q'(1)$ Present Sea Level (eroded/accreted)	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.
*$TWL(1)$ 2100 Sea Level Low Range Projection (eroded/accreted)	13.3/15.1 ft. NAVD status = dry	13.3/15.1 ft. NAVD status = dry
*$Q'(1)$ 2100 Sea Level Low Range Projection (eroded/accreted)	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.
*$TWL(1)$ 2100 Sea Level High Range Projection (eroded/accreted)	16.8/18.6 ft. NAVD status = flooded accreted beach	16.8/18.6 ft. NAVD status = dry
*$Q'(1)$ 2100 Sea Level High Range Projection (eroded/accreted)	0.0/0.038 cfs/ft.	0.0/0.00 cfs/ft.
**$TWL(100)$ Present Sea Level (eroded/accreted)	14.0/15.6 ft. NAVD status = dry	14.0/15.6 ft. NAVD status = dry
**$Q'(100)$ Present Sea Level (eroded/accreted)	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.
**$TWL(100)$ 2100 Sea Level Low Range Projection (eroded/accreted)	17.6/19.2 ft. NAVD status = flooded accreted beach	17.6/19.2 ft. NAVD status = flooded accreted beach
**$Q'(100)$ 2100 Sea Level Low Range Projection (eroded/accreted)	0.0/0.081 cfs/ft.	0.0/0.014 cfs/ft.
**$TWL(100)$ @ 2100 Sea Level High Range Projection (eroded/accreted)	21.1/22.7 ft. NAVD status = flooded	21.1/22.7 ft. NAVD status = flooded
**$Q'(100)$ 2100 Sea Level High Range Projection (eroded/accreted)	0.209/0.318 cfs/ft.	0.142/0.250 cfs/ft.

*Evaluated for the 1-yr return period; ** Evaluated for the 100-yr return period

2100 sea level projections will occur at the highest located well head (Well Head H) at the Capistrano Beach site where overtopping rates will range from Q'(100yr) = 0.142 cfs/ft. to 0.250 cfs/ft. While these overtopping rates are still dangerous to pedestrian and vehicle traffic, they are easily managed by the steel vault enclosures of the well heads and pumps being placed at Capistrano Beach.

Tsunami induced erosion, runup, and inundation were analyzed for the Doheny and Capistrano Beaches 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 analysis was based on numerical refraction/diffraction codes for a shoaling solitary wave. 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 and Capistrano Beaches. 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 Table ES-2a for the Doheny Beach site, and Table ES-2b for the Capistrano Beach site. These tables indicate that all of the shore facilities of the Doheny Desalination Project are above tsunami inundation levels at present sea level. However, all of the well heads at both Doheny and Capistrano Beaches would suffer some degree of tsunami overtopping if concurrent with 2100 sea levels, and the overtopping rates could be quite severe, especially for the high 2100 sea level rise projections. At the low range of 2100 sea level projections, total water levels would reach *TWL* = 18.82 ft. NAVD at Doheny Beach and *TWL* = 18.83 ft. NAVD at Capistrano Beach. Well Heads A-C at Doheny Beach would experience the highest overtopping surges of Q' = 1.142 cfs/ft while Well Head G at Capistrano Beach would remain high and dry. However, if the tsunami occurred atop the high range sea level rise projections for year 2100, then total water levels would reach *TWL* = 22.31 ft. NAVD at Doheny Beach and *TWL* = 22.4 ft. NAVD at Capistrano Beach, sufficient to overtop all the well sites of the Doheny Desalination project. In this case the tsunami surge could produce very high, although short-lived, overtopping rates reaching a maximum of Q' = 5.691 cfs/ft at Well Heads A-C on Doheny Beach and a minimum of Q' = 2.916 cfs/ft at Well Head H on Capistrano Beach. Undoubtedly, the steel vault enclosures of the well heads can be designed to withstand these high surge rates, but particular attention should be given to the foundations of the vaults to assure those foundations have adequate depth to prevent undercutting by scour. 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 inundated by a shoaling tsunami solitary wave.

Table ES-2a: Tsunami Total Water Level (*TWL*) and Overtopping Rates (Q') Analysis at the Doheny Beach Site

	Well Head-A Elevation = 17 ft. NAVD	Well Head-B Elevation = 17 ft. NAVD	Well Head-C Elevation = 17 ft. NAVD	Well Head-D Elevation = 18 ft. NAVD	Well Head-E 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 cfs/ft.	0.0 cfs/ft.	0.0 cfs/ft.	0.0 cfs/ft.	0.0 cfs/ft.
TWL 2100 Sea Level Low Range Projection	18.82 ft. NAVD status = flooded	18.82 ft. NAVD status = flooded	18.82 ft. NAVD status = flooded	18.82 ft. NAVD status = flooded	18.82 ft. NAVD status = flooded
Q' 2100 Sea Level Low Range Projection	1.142 cfs/ft.	1.142 cfs/ft.	1.142 cfs/ft.	0.345 cfs/ft.	0.345 cfs/ft.
TWL @ 2100 Sea Level High Range Projection	22.31 ft. NAVD status = flooded	22.31 ft. NAVD status = flooded	22.31 ft. NAVD status = flooded	22.31 ft. NAVD status = flooded	22.31 ft. NAVD status = flooded
Q' 2100 Sea Level High Range Projection	5.691 cfs/ft.	5.691 cfs/ft.	5.691 cfs/ft.	4.162 cfs/ft.	4.162 cfs/ft.

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

Table ES-2b: Tsunami Total Water Level (*TWL*) and Overtopping Rates (Q') Analysis at the Capistrano Beach Site

	Well Head-G Elevation = 18 ft. NAVD eroded/accreted	Well Head-H Elevation = 19 ft. NAVD eroded/accreted
**TWL* Present Sea Level (eroded)	15.3 ft. NAVD status = dry	15.3 ft. NAVD status = dry
*Q' Present Sea Level (eroded)	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.
**TWL* 2100 Sea Level Low Range Projection (eroded)	18.83 ft. NAVD status = flooded	18.83 ft. NAVD status = dry
*Q' 2100 Sea Level Low Range Projection (eroded)	0.352 cfs/ft.	0.0/0.0 cfs/ft.
**TWL* 2100 Sea Level High Range Projection (eroded)	22.4 ft. NAVD status = flooded	22.4 ft. NAVD status = flooded
*Q' 2100 Sea Level High Range Projection (eroded)	4.293 cfs/ft.	2.916 cfs/ft.

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

Coastal Hazards Analysis for the Doheny Desalination Project for the Final EIR

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-channel 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-channel 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 sea levels; 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.

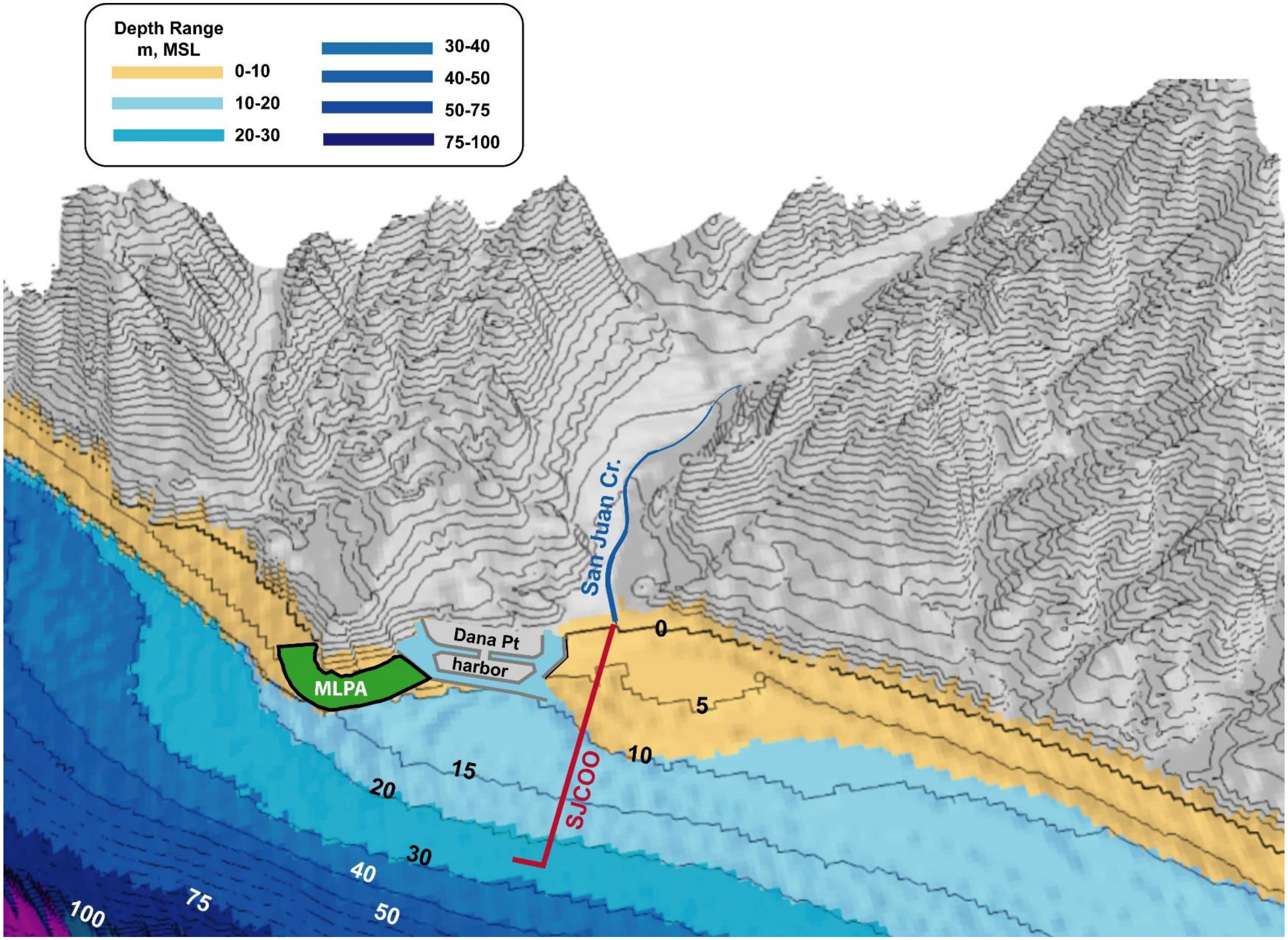


Figure 1.1: Project site map in GIS. Bathymetry contours in meters MSL. Data from GEODAS 3 arc-second database

2) Regulatory Requirements:

The *California Coastal Commision Sea Level Rise Guidance Policy Guidance* document (CCC, 2015) and CCC (2018) 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 interpolation[100], 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. 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 present analysis will use 2100 as the ultimate planning horizon for a critical infrastructure project.

Originally, 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). In our previous study, we selected 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), which appear as the cyan colored curve in Figure 3.1 for the low-range projection; and the magenta colored curve for the high range projection. For the 2100 planning horizon, sea level rise was originally projected to range from 1.37 ft to 5.50 ft. However, in the updated sea level rise policy guidance document, (CCC, 2018), equations (1) and (2) were abandoned in favor of a water level province tabulation centered around NOAA tide gage stations having long periods of record. The Doheny Desalination Projects lies in the La Jolla tide gage water level province, for which sea level rise projections are listed in Table G-11 in Appendix-G of CCC (2018). These new projections are plotted in Figure 3.1 as the blue curve for the low range projections and the red curve for the high range projections. Clearly the clarified sea level rise curves in Figure 3.1 project significantly higher future sea levels, particularly for the low range estimates. For the 2100 planning horizon sea level rise is projected range from 3.6 ft to 7.1 ft. The low range projection represents a 17% probability that sea level rise exceeds these values; while the high range projection represents a 0.5% probability that sea level rise exceeds these values. These values will be used in the calculations of extreme total water levels (TWL's) in the following sections.

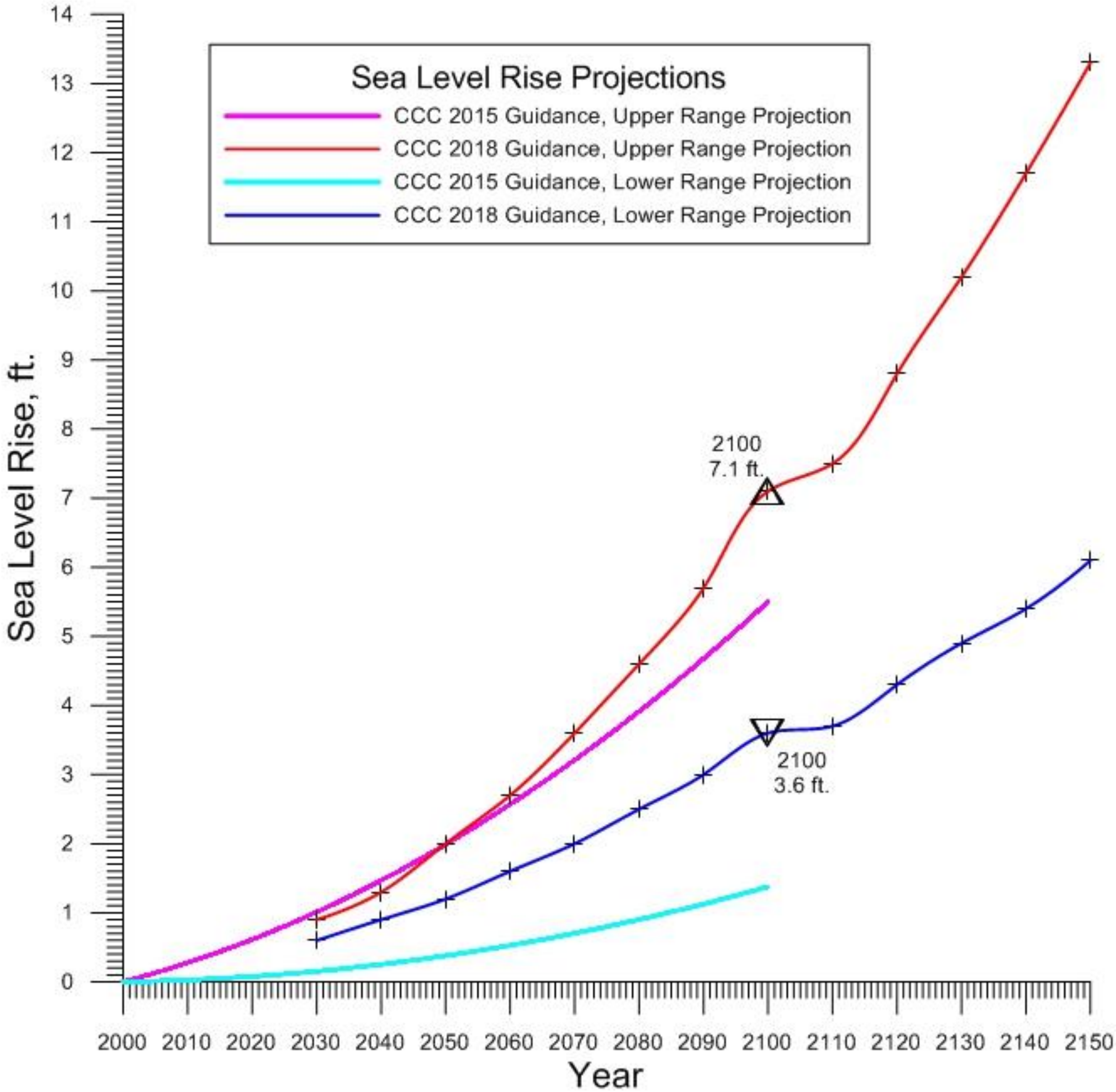


Figure 3.1: Range of sea level rise projections from the California Coastal Commission sea level rise guidance document, (CCC, 2018, Appendix-G). The 2100 planning horizon is indicated by symbols on the upper and lower range curves. Blue curve represents a 17% probability that sea level rise exceeds these values; red curve represents a 0.5% probability that sea level rise exceeds these values

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 CCC, (2015) and CCC (2018). 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 was not used as the basis for a water level province in Appendix-G of CCC (2018). 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:

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

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

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

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

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 :

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

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.

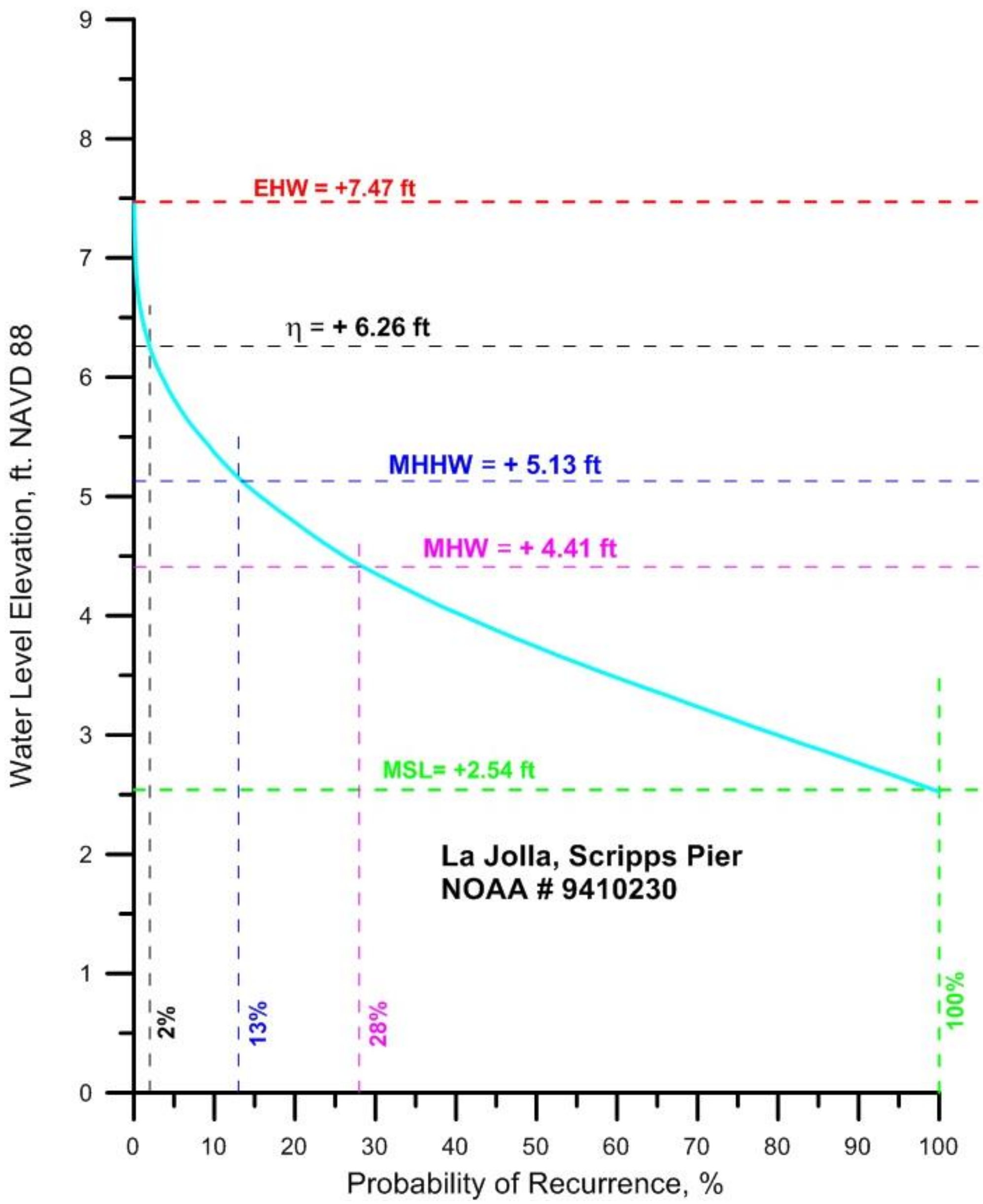


Figure 4.1: Hydroperiod function of still-water level elevations at present sea level, based on ocean water level measurements at the Scripps Pier tide gage station, NOAA #9410230, for the period of record 1924-2016. Tidal datums based on the 1983-2001 tidal epoch (latest datum analysis period).

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

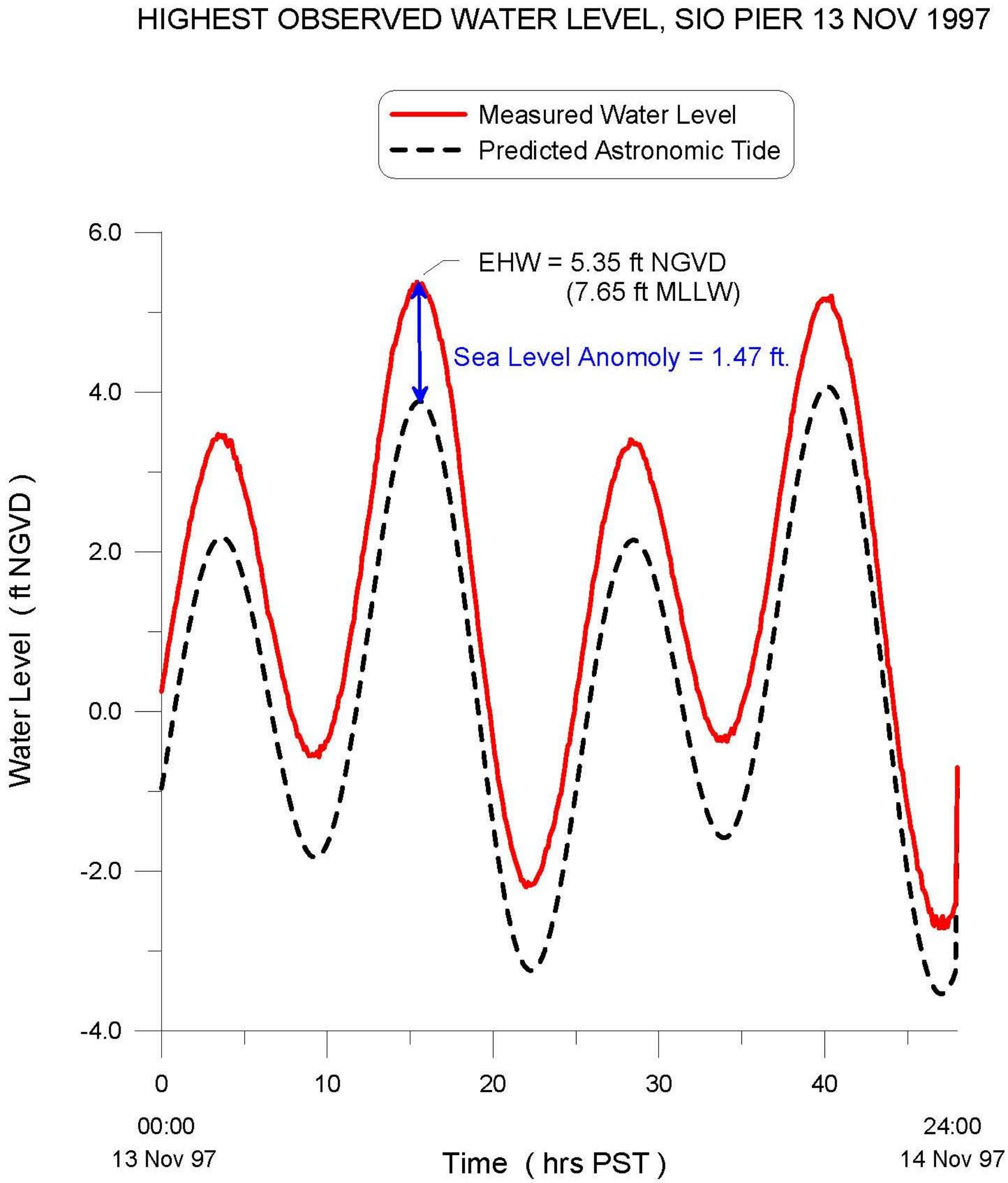


Figure 4.2: Comparison of measured ocean water level (red) at the Scripps Pier tide gage vs. predicted water level based on tidal constituents (black dashed) for the extreme high water event of 13 November 1997 (from Jenkins and Wasyl, 2005).

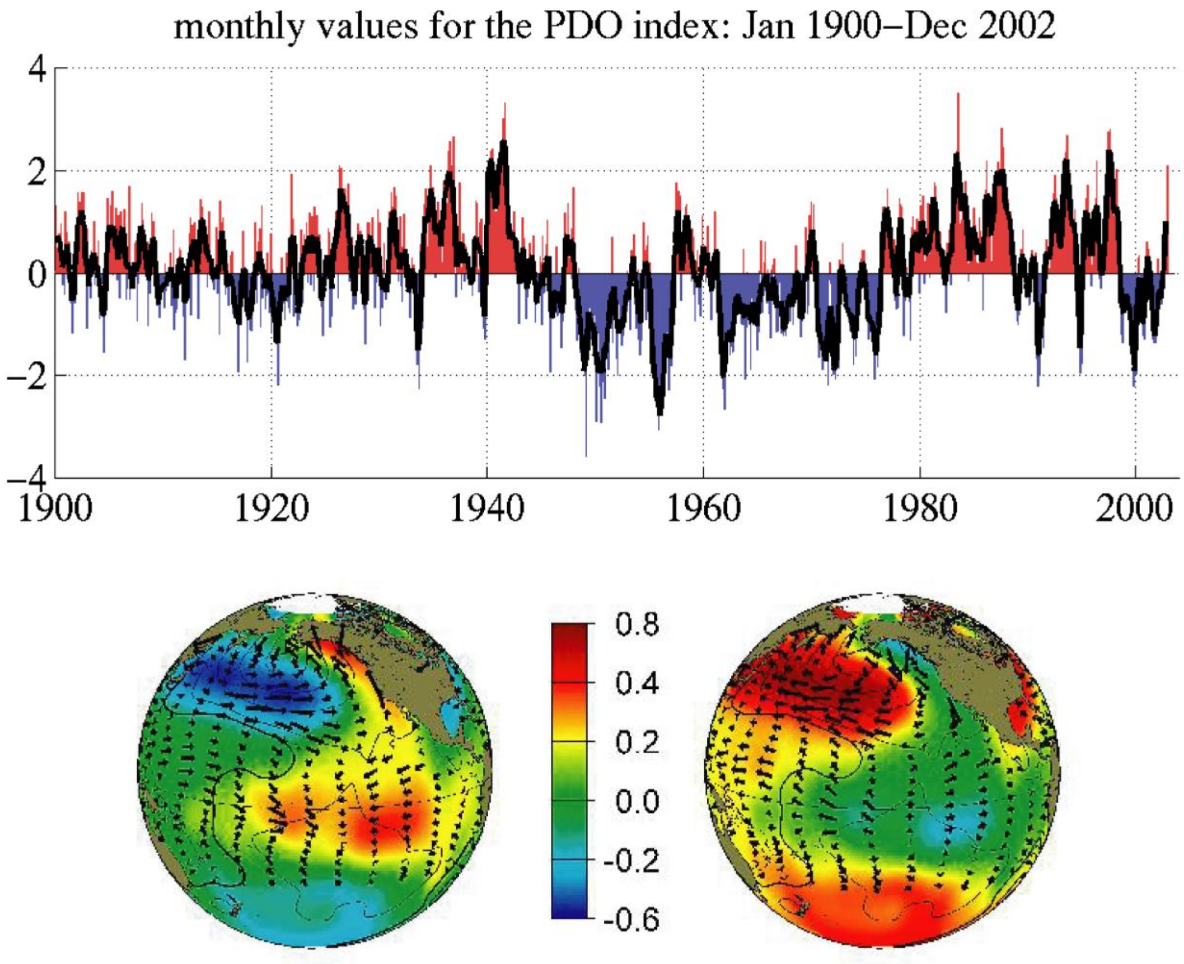


Figure 4.3. Typical wintertime Sea Surface Temperature (colors), Sea Level Pressure (contours) and surface wind stress (arrows) anomaly patterns during warm and cool phases of PDO.

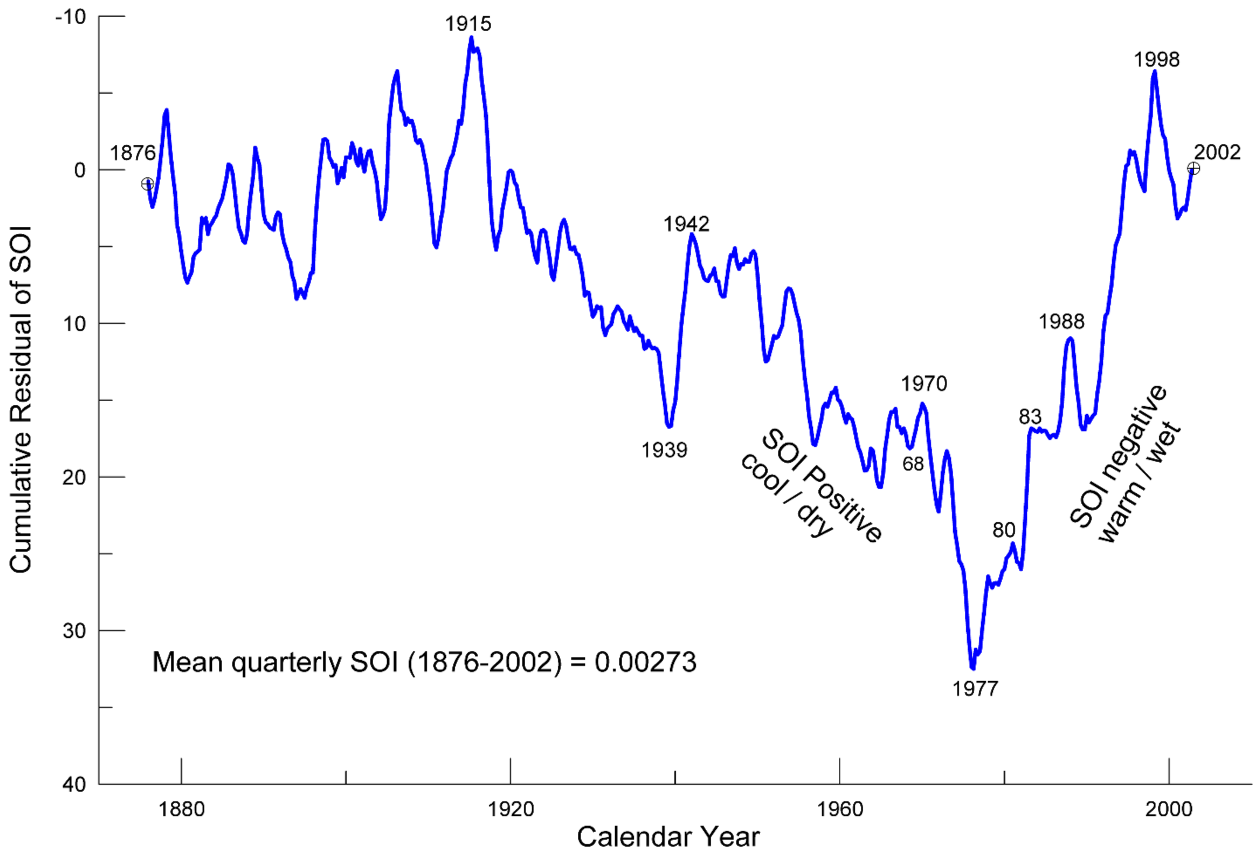


Figure 4.4. Cumulative residual of quarterly values of Southern Oscillation Index (SOI) [data from Australian Commonwealth Bureau of Meteorology].

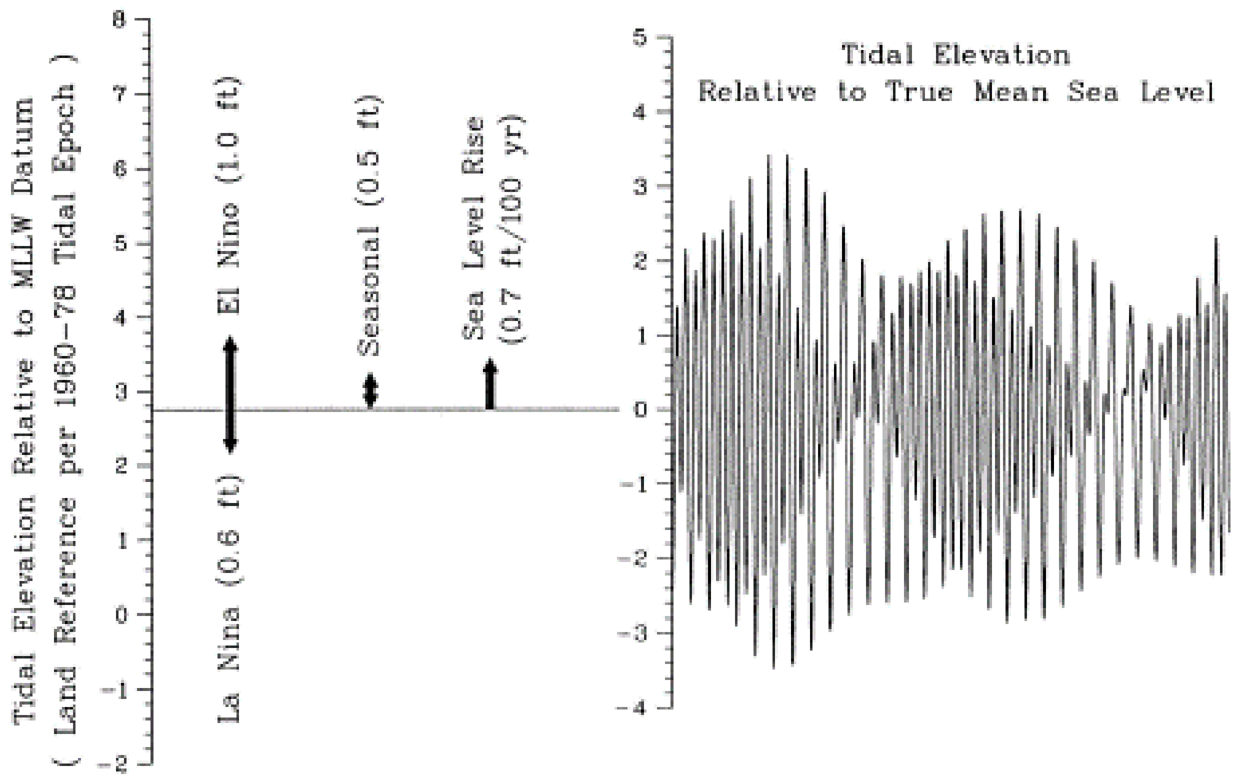


Figure 4.5 Schematic decomposition ocean water levels according to astronomic tides, and seasonal and ENSO/PDO cycle effects on steric sea levels.

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 2100 sea level in Figure 4.6.

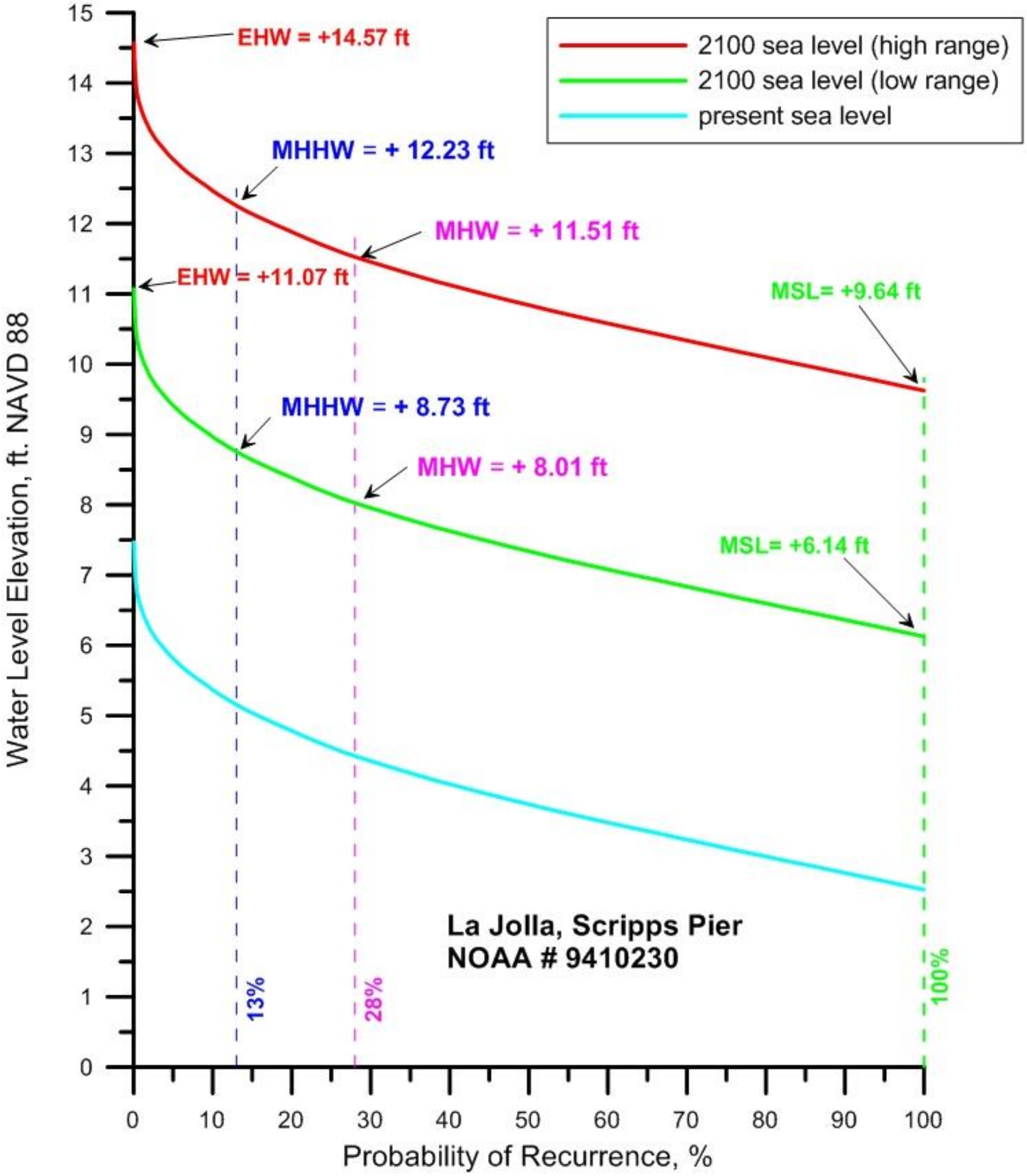


Figure 4.6: Hydroperiod function of still-water level elevations at 2100 sea level, based on ocean water level measurements at the Scripps Pier tide gage station, NOAA #9410230, for the period of record 1924-2016. Tidal datums based on the 1983-2001 tidal epoch (latest datum analysis period). Sea level rise component from Appendix-G, Table G-11 in CCC (2018)

At the year 2100 planning horizon for desalination projects, low range projections in Figure 4.6. indicate that mean sea level increases to MSL = +6.14 ft NAVD while extreme high water increases to EHW = +11.07 ft. NAVD, while mean higher high water increases to MHHW = + 8.73 ft. NAVD. At the high range 2100 projections, mean sea level increases to MSL = +9.64 ft. NAVD; extreme high water increases to an astonishing EHW = +14.57 ft. NAVD, and mean higher high water increases to MHHW = + 12.23 ft. NAVD. The still water elevations inferred at future sea levels from the linear superposition assumption are summarized in Table 4.2 below. It is interesting to note that under the updated policy guidance (CCC,2018) water levels for the high range sea level rise projections for 2070 are the same as water levels for the low range sea level rise projections for year 2100.

Table 4.2: Still Water elevations at present and future sea levels. Based on NOAA #941-0230 tide gage records and sea level rise from Appendix-G, Table G-11 in CCC (2018)

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 Low Range Projection (ft. NAVD 88)	*2100 Sea Level High Range Projection (ft. NAVD 88)
Mean Sea Level (MSL)	2.54	4.54	6.14	6.14	9.64
Mean High Water (MHW)	4.41	6.41	8.01	8.01	11.51
Mean Higher-HighWater (MHHW)	5.13	7.13	8.73	8.73	12.23
Extreme High Water (EHW)	7.47	9.47	11.07	11.07	14.57

*Planning horizon for the Doheny Desalination Project.

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

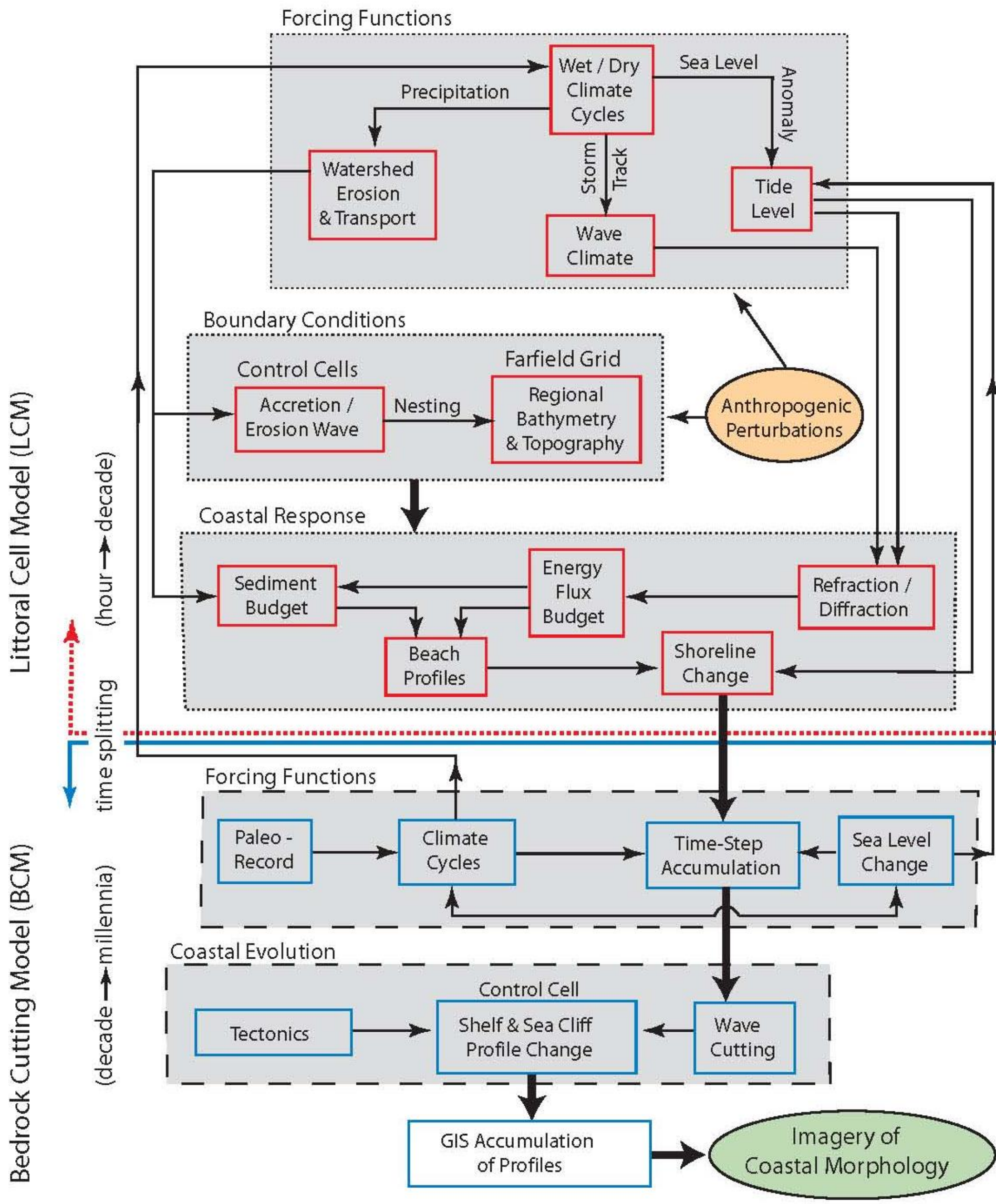


Figure 5.1: Architecture of the Coastal Evolution Model consisting of the Littoral Cell Model (above) and the Bedrock Cutting Model (below). Modules (shaded) are formed of coupled primitive process models. (Jenkins and Wasyl, 2005).

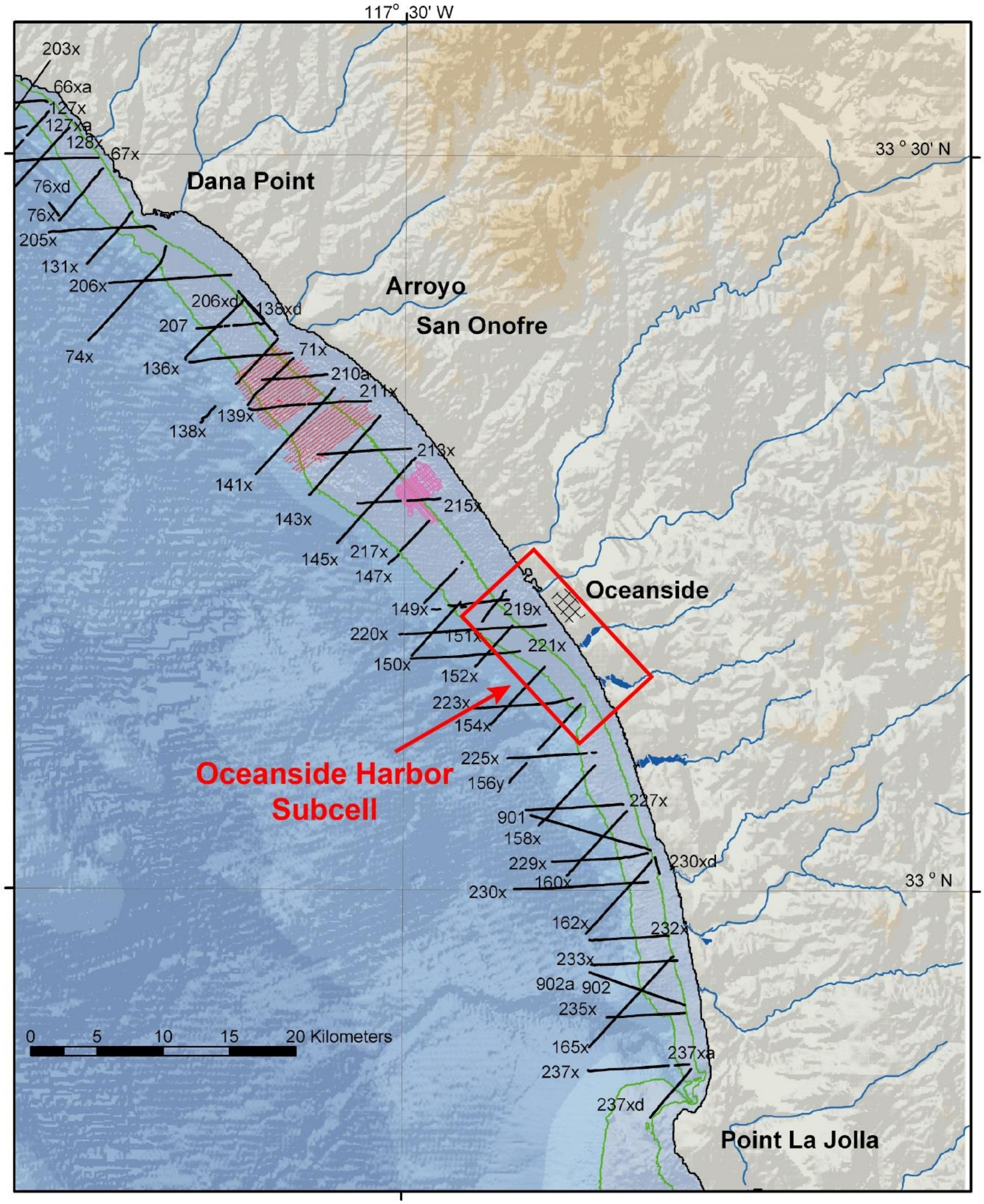


Figure 5.2: Oceanside Littoral Cell and Oceanside Harbor Sub-Cell. Composite bathymetry from NOS data base and equilibrium profiles after Jenkins and Inman (2006) for wave conditions of wet weather scenario. Depth contours shown in meters mean sea level. USGS cross-shelf survey tracks shown as numbered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

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

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:

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

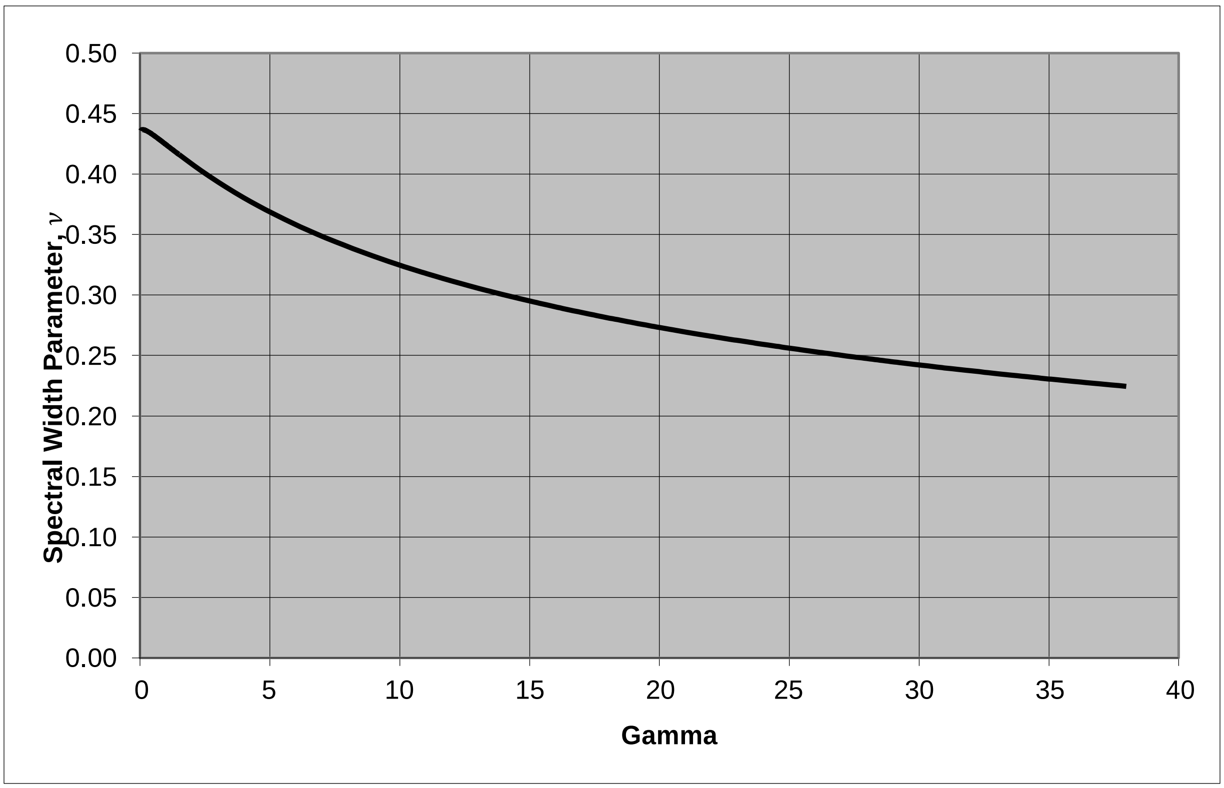


Figure 5.3 *Gamma* values are limited from 1 to 38, based on the range of wave data used (Section 4.4) to relate the spectral narrowness, ν, to the peak parameter, *Gamma*, as shown in Figure 3.3.

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 swash level, 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.

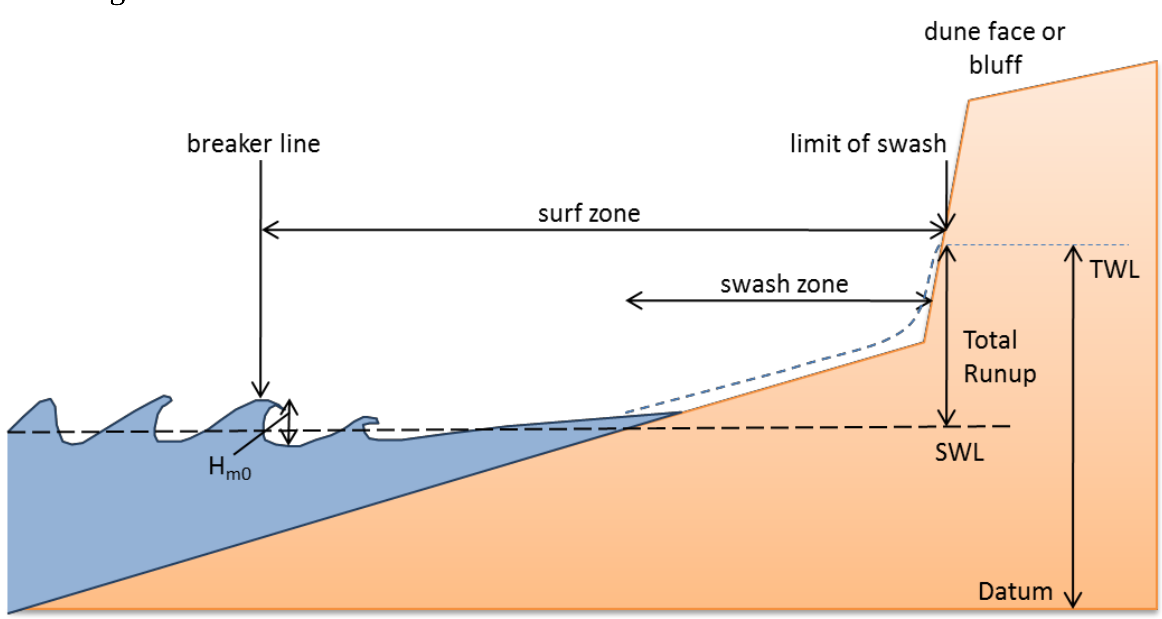


Figure 5.4: Conceptual Model Showing the Components of Wave Runup Associated with Incident Waves.

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 $\left(H_b\right)_{rms}$, and the depth at the toe of the barrier relative to SWL, h. The $\left(H_b\right)_{rms}$ is determined using the deepwater equivalent significant wave height, H'_0, and the peak wave period, T_P , according to:

$$\left(H_b\right)_{rms} = 0.714\left(\frac{\kappa}{g}\right)^{1/5}\left(\frac{{H'_0}^2 C_0}{2}\right)^{2/5} \tag{13}$$

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:

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

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

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

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,

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

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

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

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.

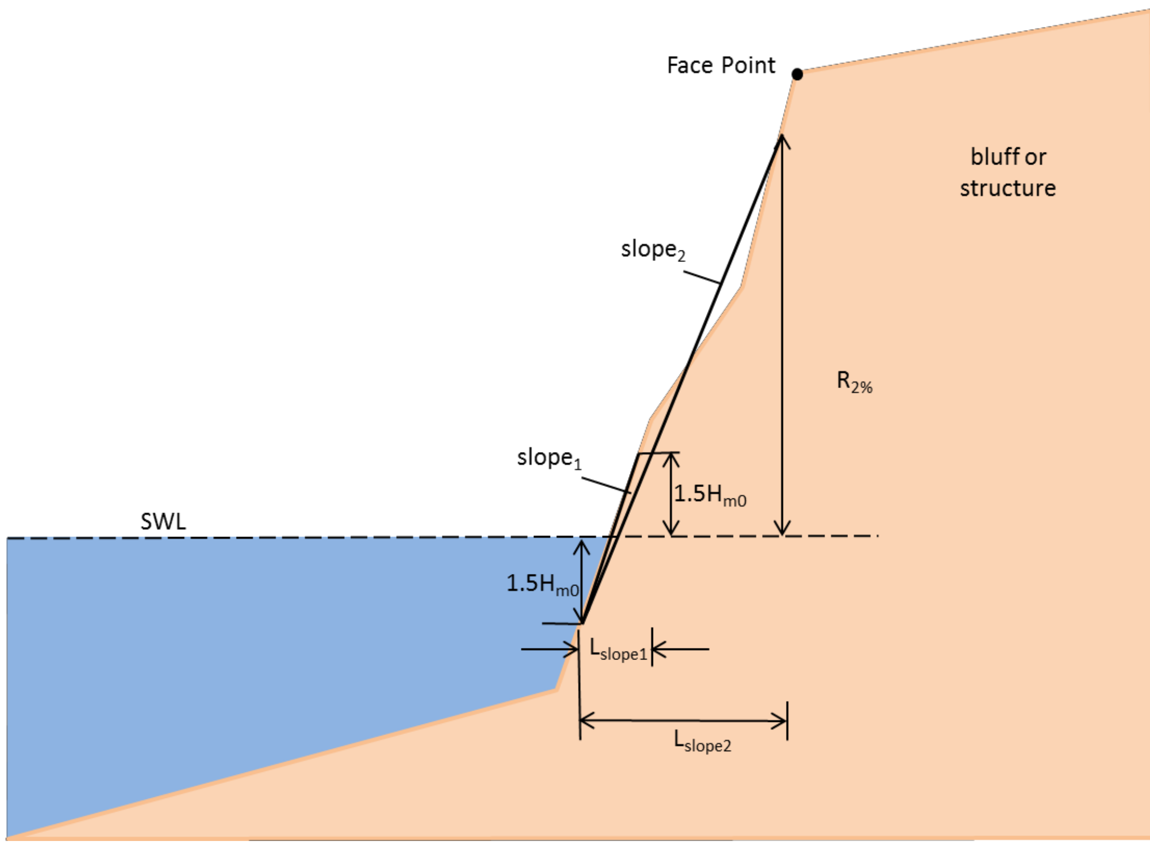


Figure 5.5: Determination of an Average Slope of Hard Back-Shore Formations (Bluff or Barriers) Based on an Iterative Approach, (Corrected from van der Meer, 2002)

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:

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

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 $\xi_{0m} = m_{TAW} / (H_{m0} / L_m)^{0.5}$ 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-5-8 in the Coastal Engineering Manual (USACE, 2006) which presents an overtopping formula for impermeable and permeable barriers and structures according to:

$$Q' = a\, g H_s T_{om} \exp\left(-\frac{bR_c}{H_s \gamma_r} \sqrt{\frac{s_{om}}{2\pi}} \right) \tag{17}$$

Where H_s, is the significant wave height at the structure, R_c is the freeboard, γ_r is the influence factor for surface roughness, T_{om} is the wave period associated with the spectral peak in deep water, s_{om} is the wave steepness associated with the spectral peak in deep water, and a and b are empirical constants based on beach slope and berm width as determined from measured beach profiles plotted in Section 6.4. 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).

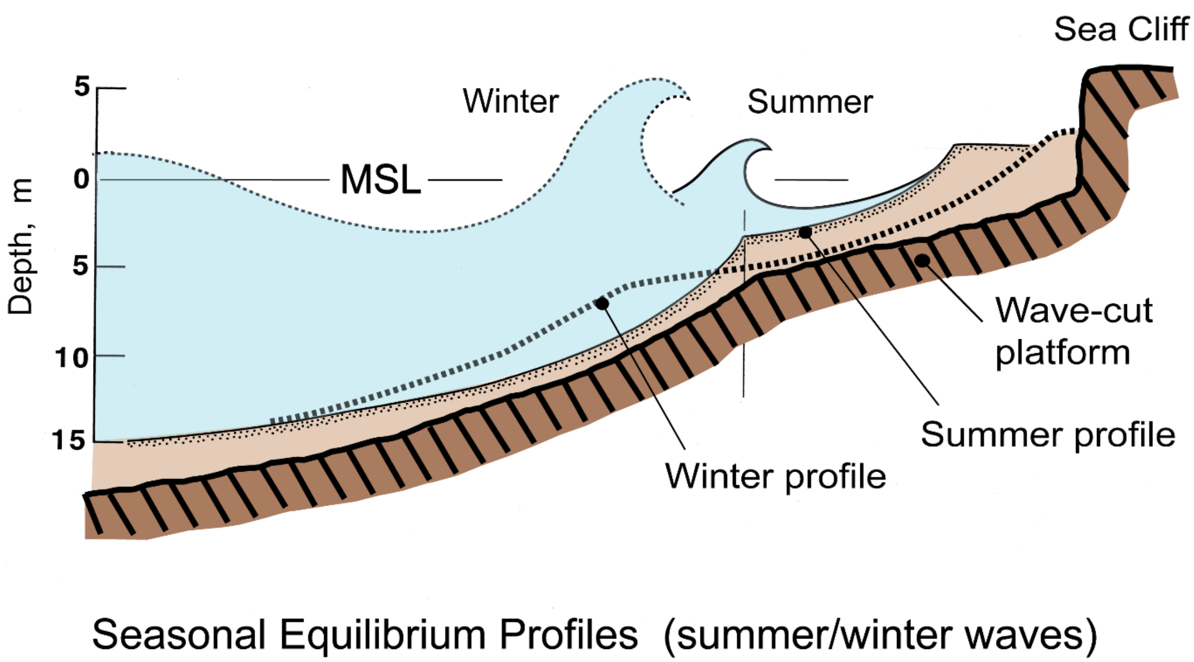


Figure 5.6: Schematic of summer and winter equilibrium beach profiles, from Inman, et al (1993).

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:

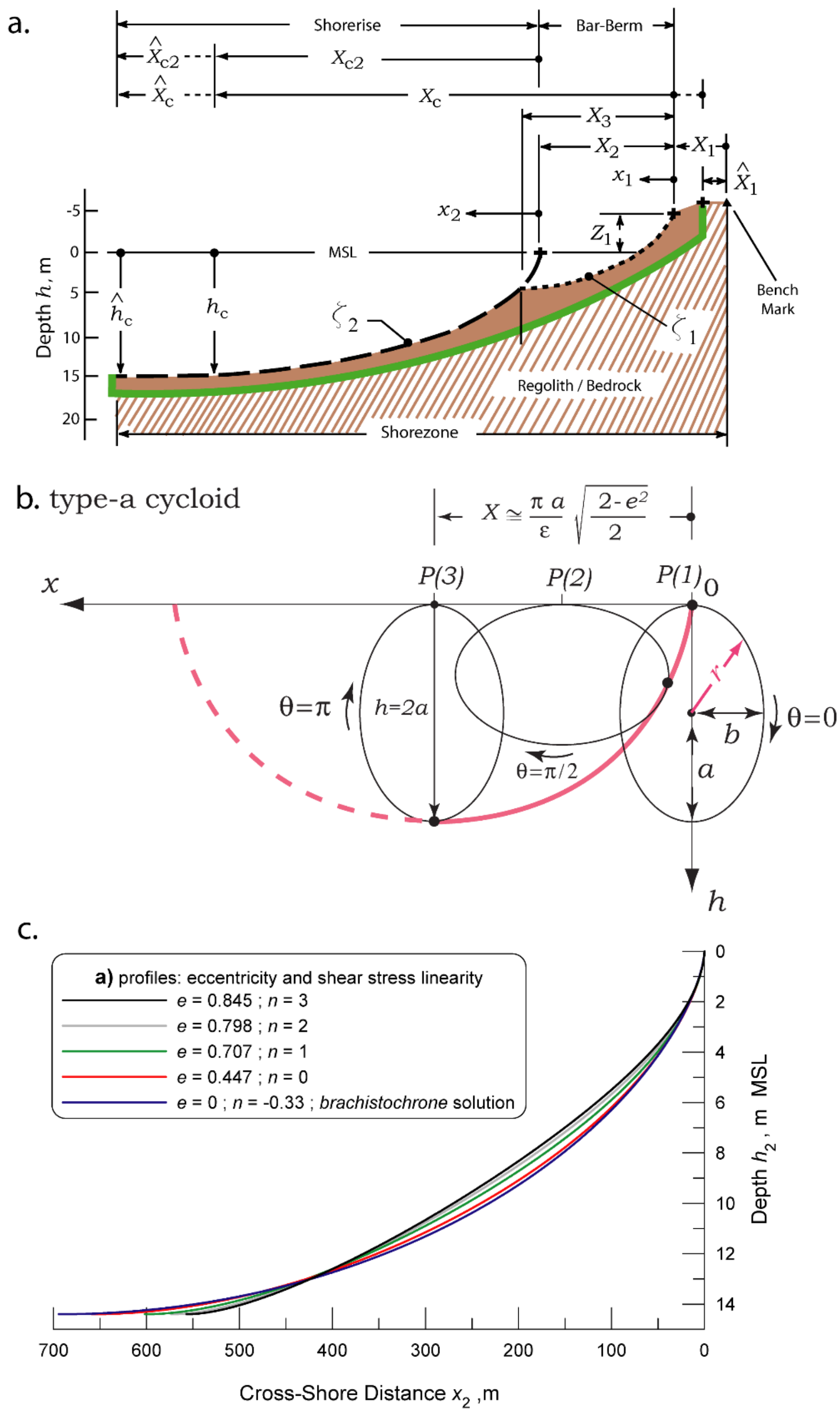


Figure 5.7. Equilibrium beach profile a) nomenclature, b) elliptic cycloid, c) Type-a cycloid solution.

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

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:

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

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:

$$h_{\mathbf{c}} = \frac{K_{\mathbf{e}} H_{\infty}}{\sinh k h_{\mathbf{c}}} \left(\frac{D_{\mathrm{o}}}{D_{\mathbf{2}}} \right)^{\psi} \tag{24}$$

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 $K_{\mathrm{e}} \sim 2.0, \psi \sim 0.33$ and $D_{\mathrm{o}} \sim 100 \mu\mathrm{m}$, 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.

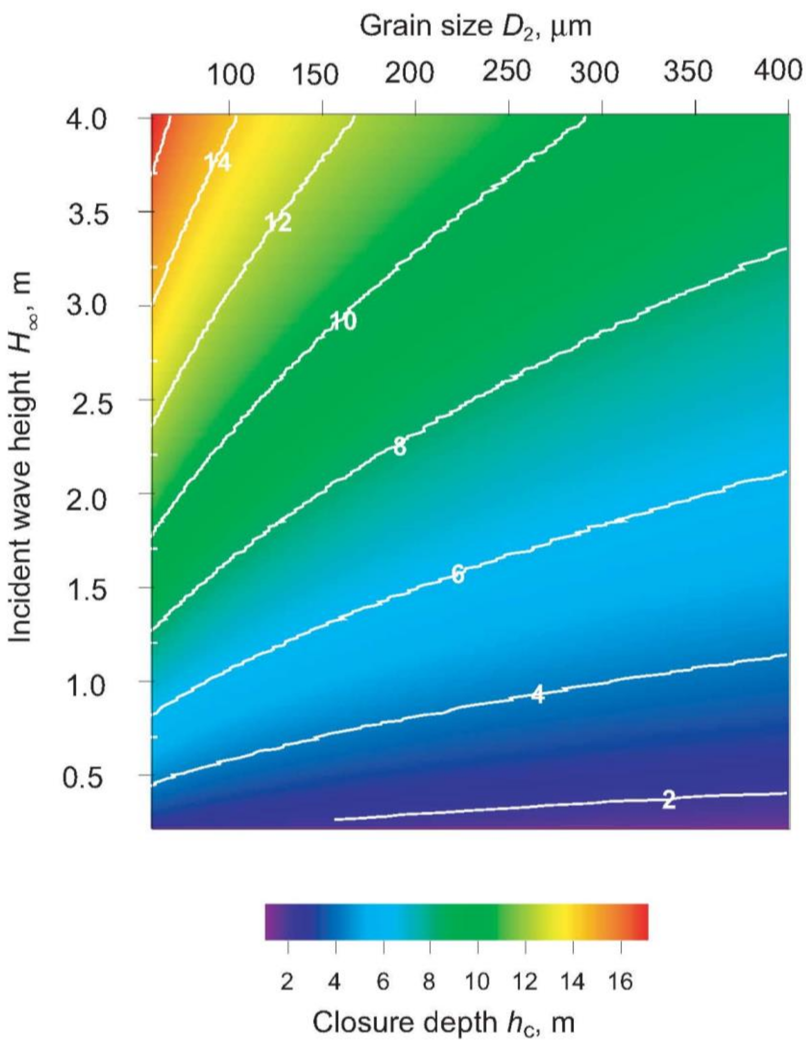


Figure 5.8: Closure depth contoured versus incident wave height and sediment grain size for waves of 15 second period, with $K_{\mathrm{e}} \sim 2.0, \psi \sim 0.33$ and $D_{\mathrm{o}} \sim 100 \mu\mathrm{m}$. D_2 is the shorerise median grain size; and D_{o} is a reference grain size.

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: ***Well Head A***, elevation 17 ft.NAVD, at 33°27'44.38"N, 117°41'16.32"W; ***Well Head B***, elevation 17 ft. NAVD, at 33°27'45.07"N, 117°41'10.30"W; ***Well Head C***, elevation 17 ft. NAVD at 33°27'45.12"N, 117°41'6.62"W; ***Well Head D***, at elevation 18 ft. NAVD at 33°27'44.48"N, 117°40'55.30"W; and ***Well Head E***, at elevation 18 ft. NAVD at 33°27'42.45"N, 117°40'47.33"W; (see Figure 6.3a). Two additional vaulted well heads with submersible pumps will be placed at the Capistrano Beach site (Figure 6.3b), which includes: ***Well Head G***, at elevation 18 ft. NAVD at 33°27'14.94"N, 117°39'59.91"W; and ***Well Head H***, at elevation 19 ft. NAVD at 33°27'13.17"N, 117°39'57.15"W.

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.

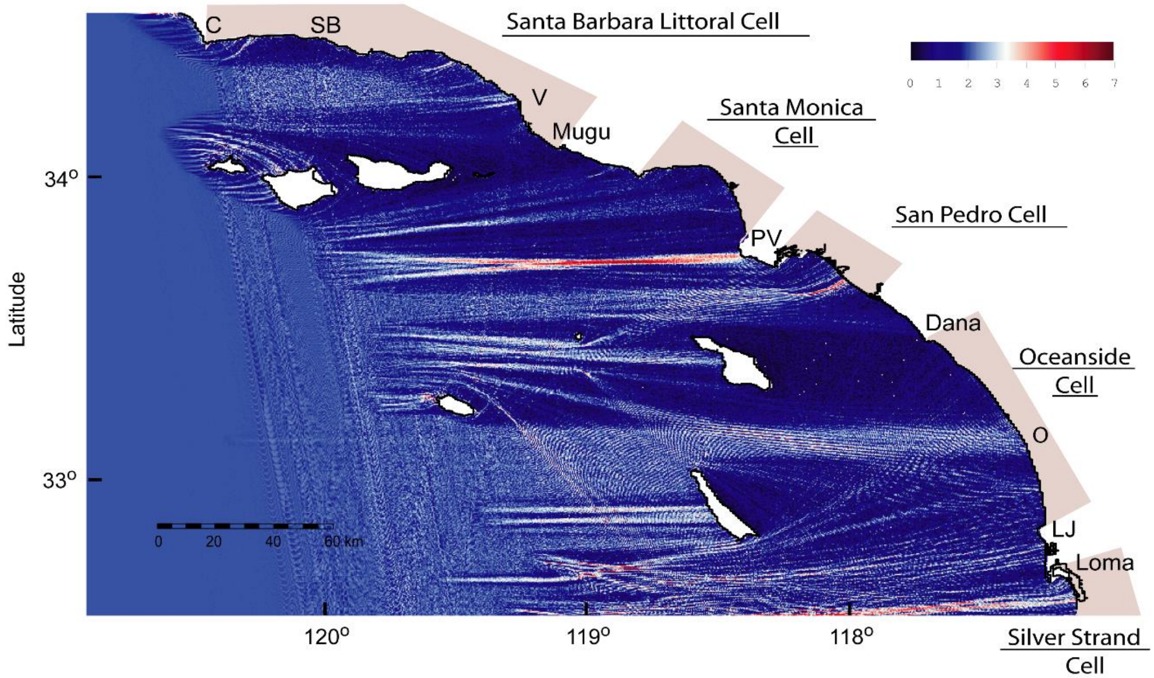
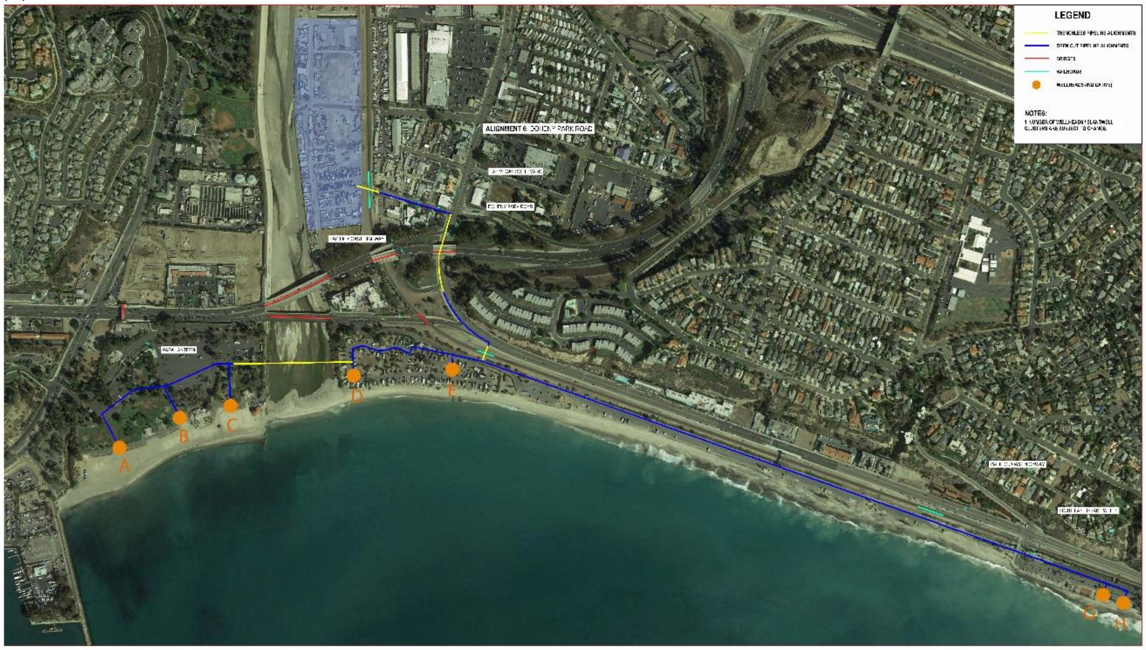


Figure 6.1: Far-field refraction/diffraction grid to simulate shoaling waves entering the Southern California Bight and Oceanside Littoral Cell. Results based on the 5 largest storms of the 1998 El Nino winter (from Jenkins and Wasyl, 2008b).

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

(a)



(b)

Figure 6.3: Critical shore-front infrastructure locations for the Doheny Desalination Project: a) Doheny Beach site; and b) Capistrano Beach site

Wave forcing to the Coastal Evolution Model (CEM) 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

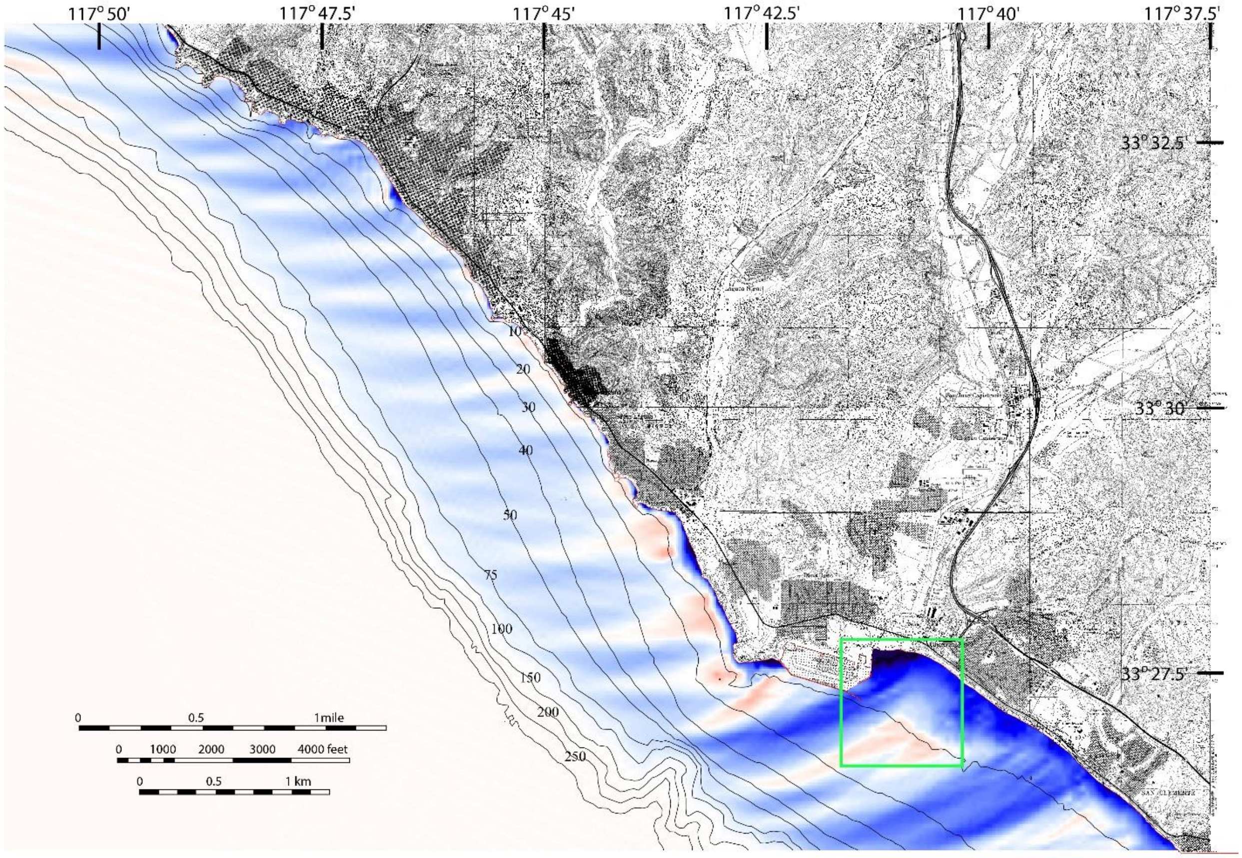


Figure 4. Maximum event wave refraction for period of record, 1980-2000 (18 January 1988). Wave height at offshore control point = 4.8 m, T=14 sec, from 270°.

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.

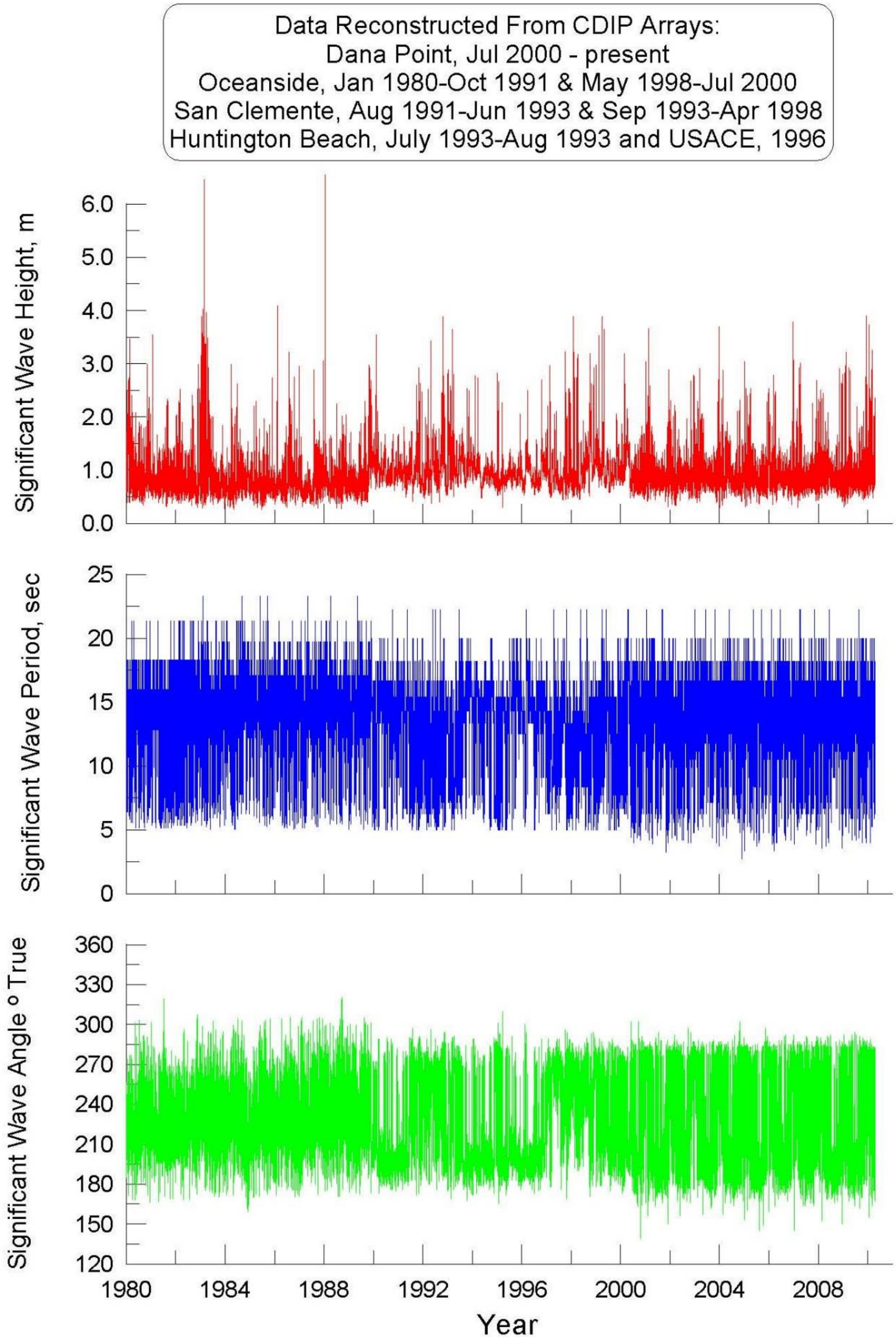


Figure 6.5: Archival wave forcing data 1980-2010 reconstructed for the SJCOO and Doheny Desalination Project modeling, from backward/forward refraction of regional CDIP wave monitoring data.

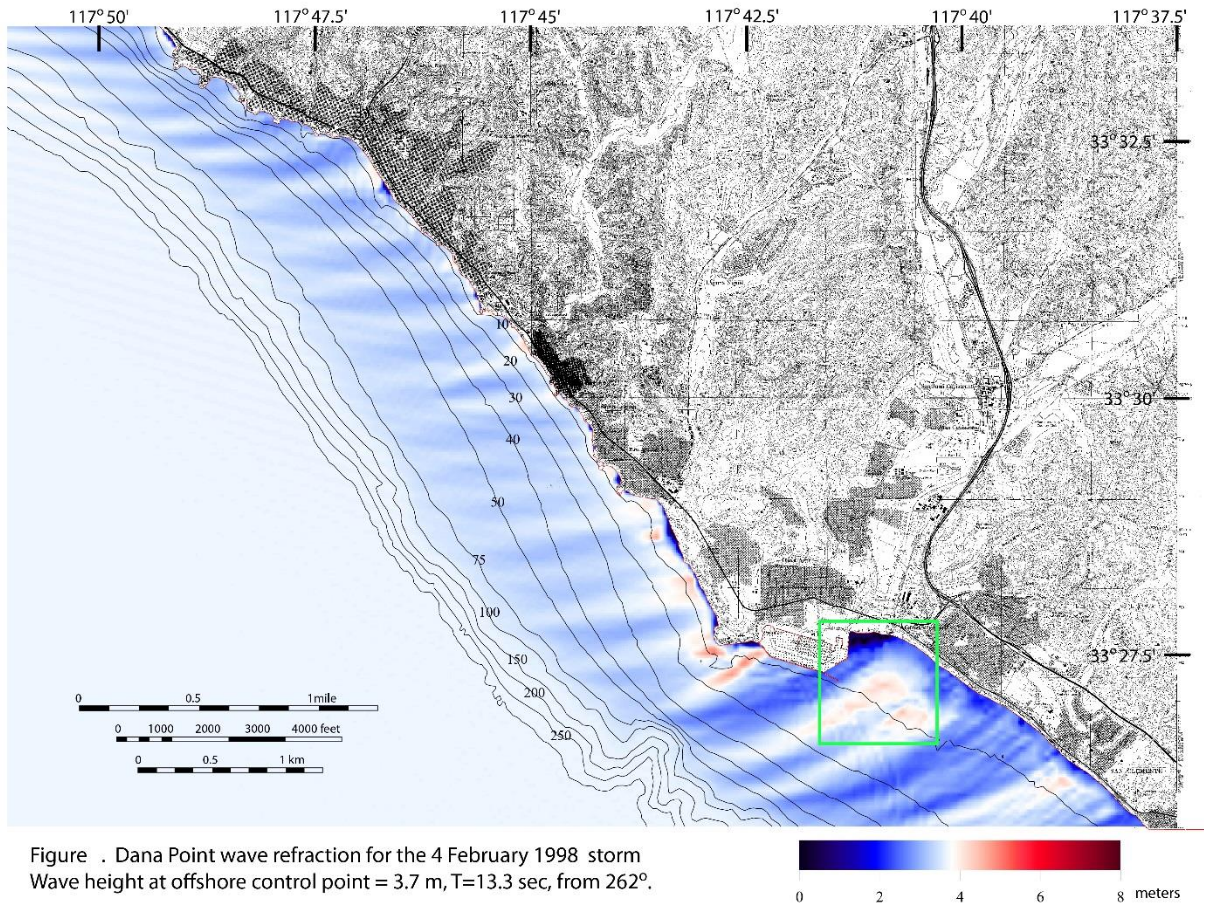


Figure 6.6: Wave refraction/diffraction field around the SJCOO site and the Doheny Desalination Project site derived from wave burst measurements from the Acoustic Doppler Current Profiler (ADCP) records taken under the MBC Applied Environmental Sciences (MBC) NPDES monitoring studies. Nearfield domain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

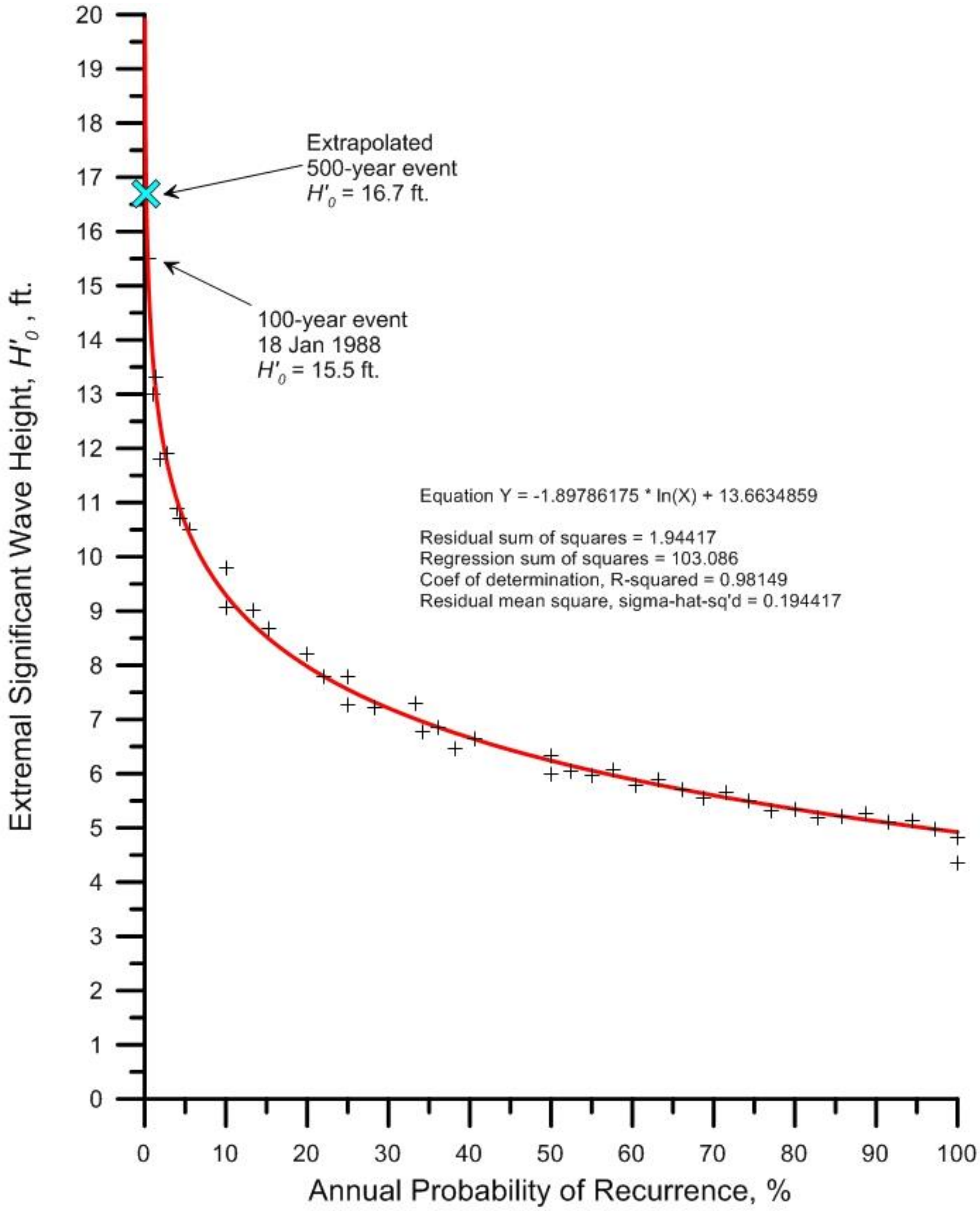


Figure 6.7 : Probability of recurrence of design wave heights based on Weibull extremal analysis of significant wave heights at Doheny & Capistrano Beaches. Analysisbased on Weibull Type III distribution applied to 12 m local water depth with K = 1.0. Recurrence Probability $P(H)$ = 100%/T, where T = 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, (proxy slope for an *eroded beach*). Figure 6.11 indicates that the average nearshore slope in summer steepens to m_{DIM} = 0.10, (proxy slope for an *accreted beach*). 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):

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

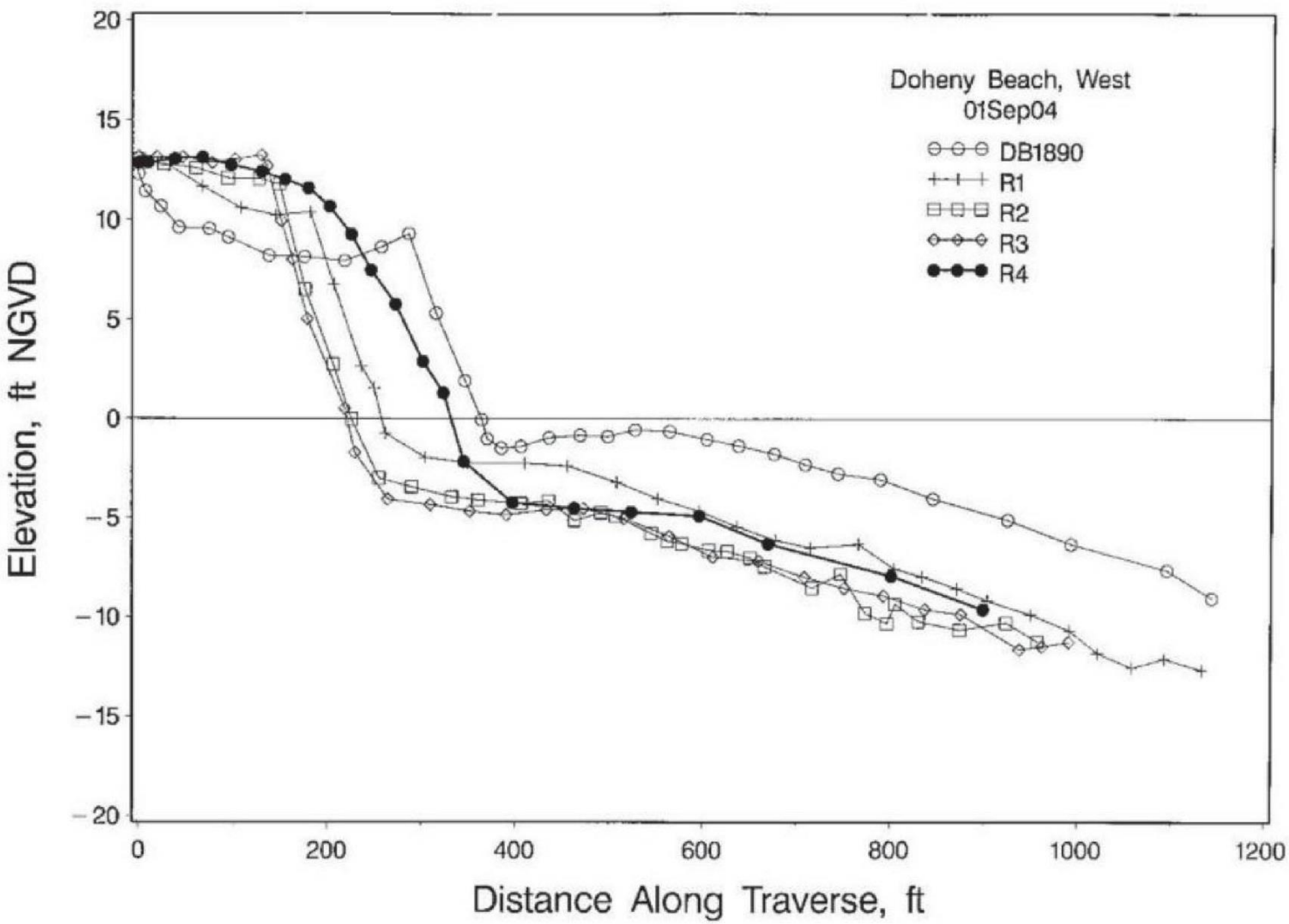


Figure 6.8: Beach profile surveys of Doheny Beach range lines adjacent to Well Heads A & B. Data from Coastal Environments, (2014).

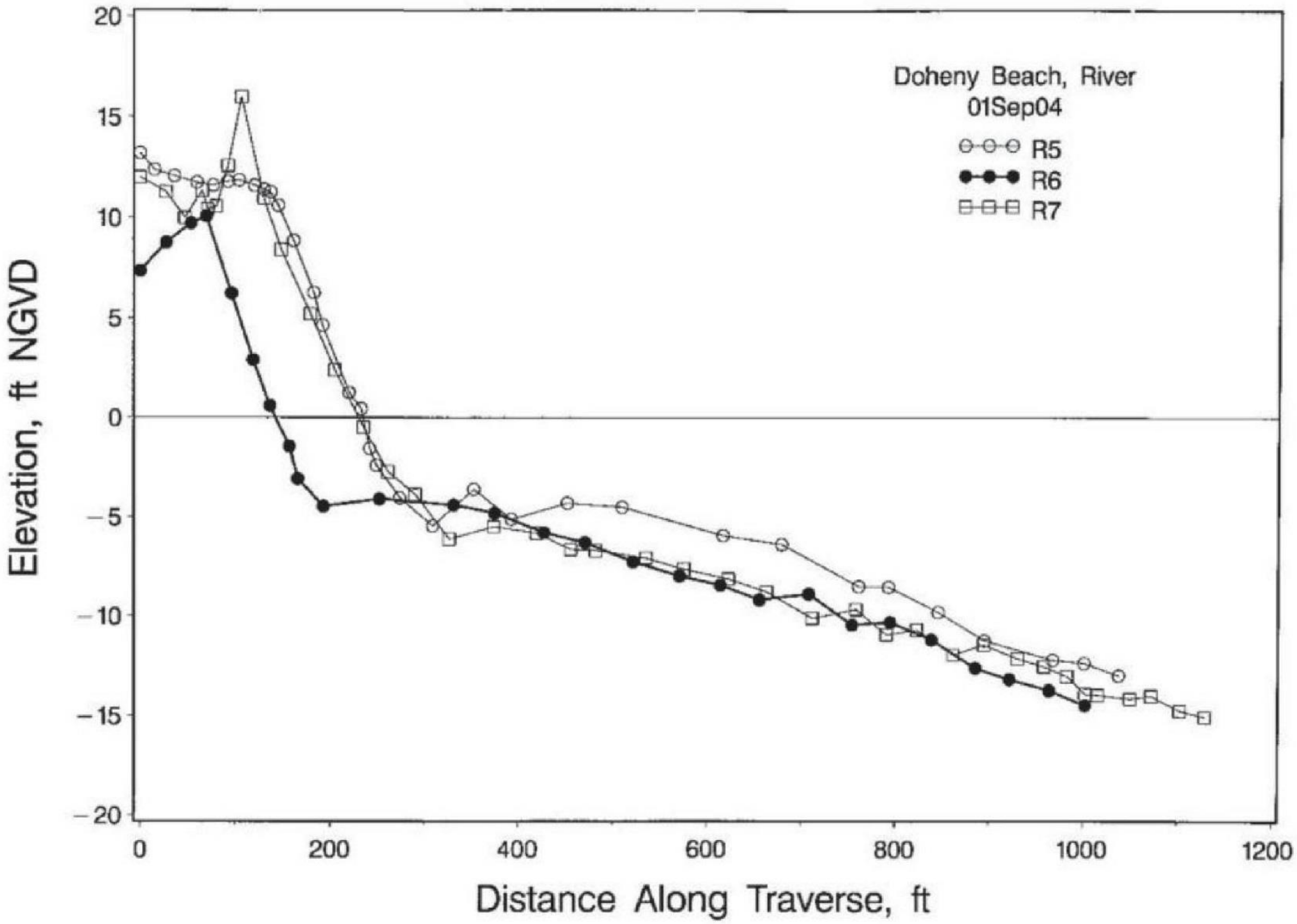


Figure 6.9: Beach profile surveys of Doheny Beach range lines adjacent to Well Heads C & D. Data from Coastal Environments, (2014).

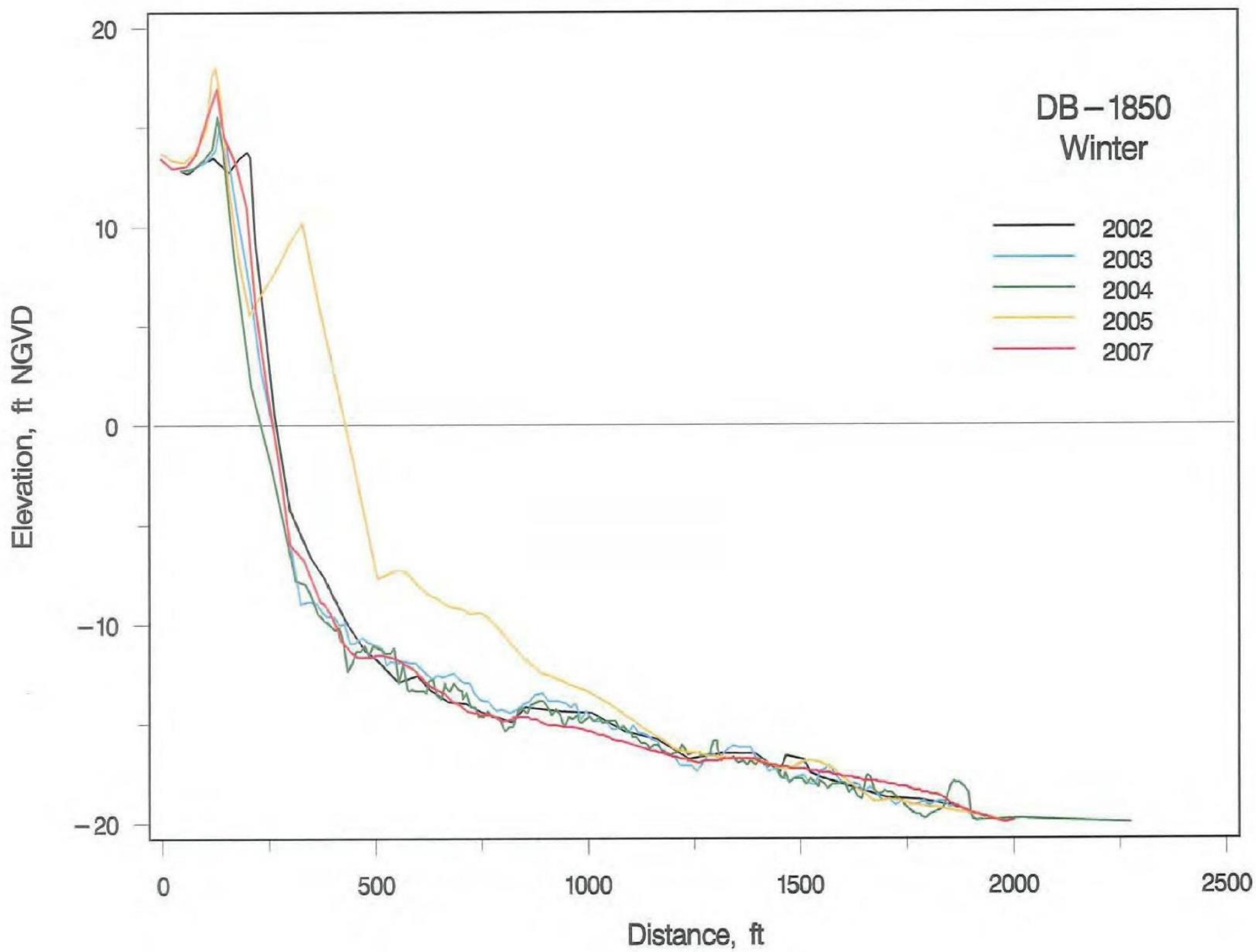


Figure 6.10: Winter beach profile surveys of Doheny Beach range lines adjacent to Well Heads D & E. Surveys due to the US Army Corps of Engineers. Data provided by Coastal Frontiers, (2014).

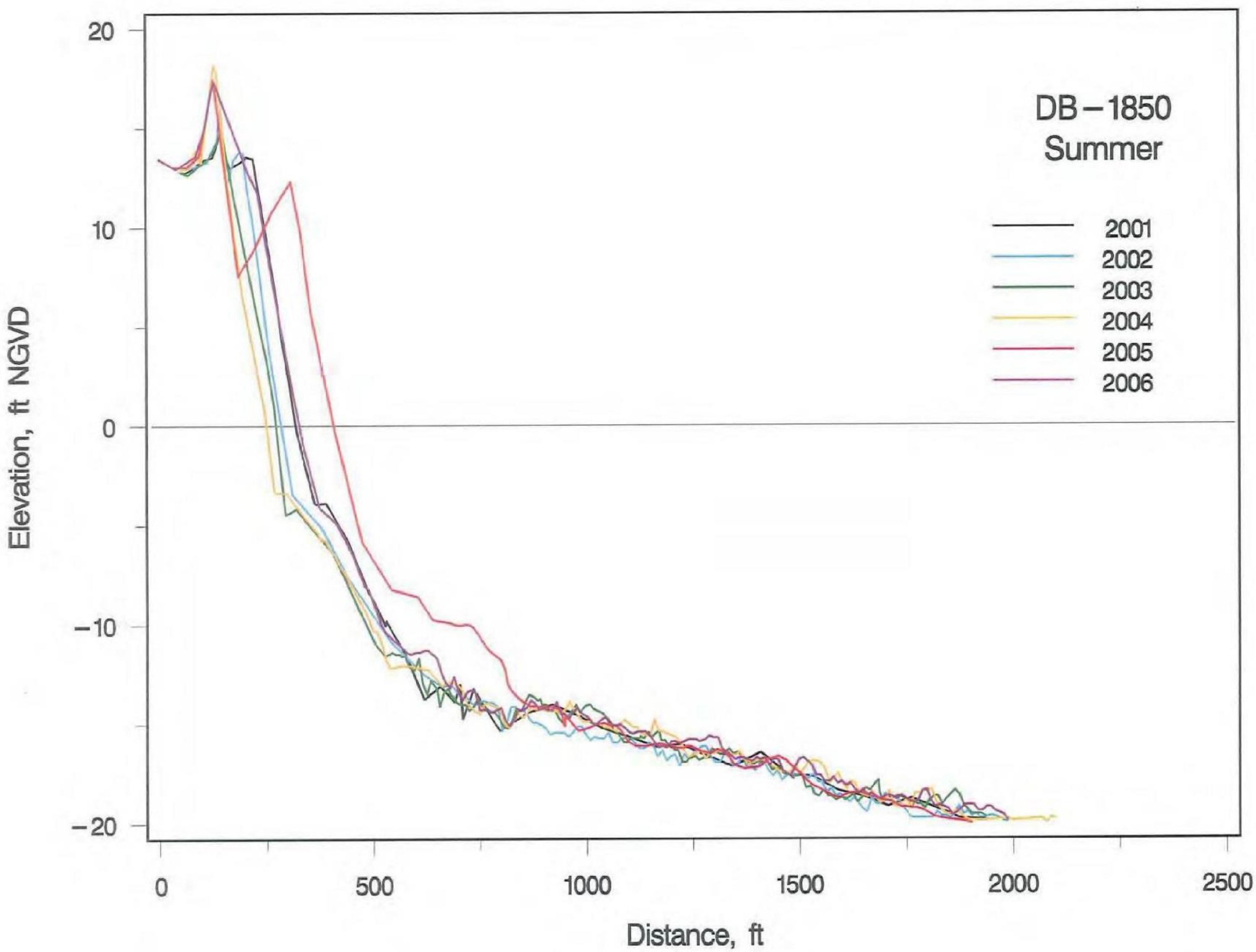


Figure 6.11: Summer beach profile surveys of Doheny Beach range lines adjacent to Well Heads D & E. Surveys due to the US Army Corps of Engineers. Data provided by Coastal Frontiers, (2014).

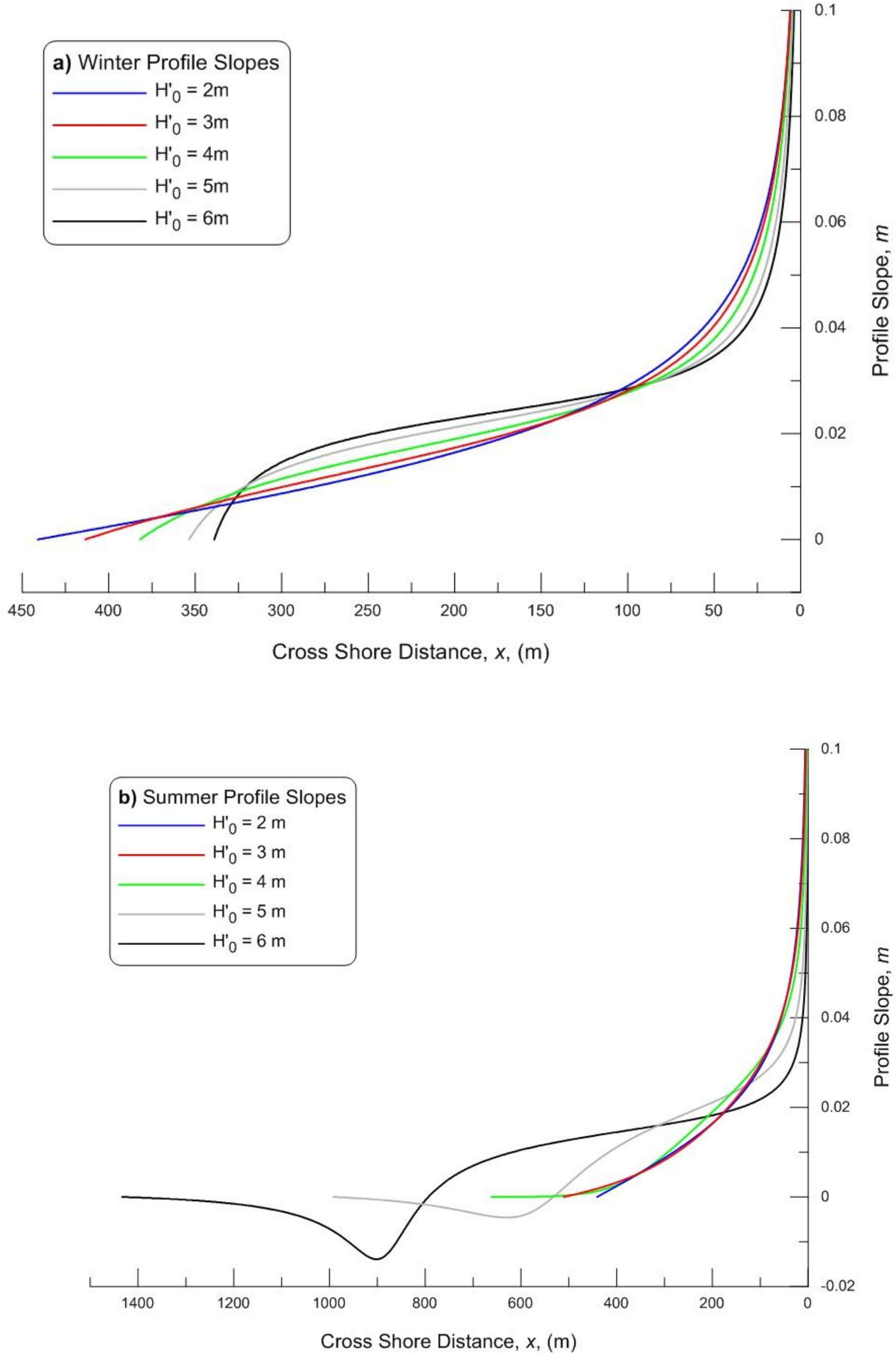


Figure 6.12. Family of elliptic cycloid slope solutions in the bar berm: a) type-a cycloids; b) type-b cycloids. Cycloids scaled for : H'_0 = 2 - 6 m; T = 15 sec; m_{DIM} = 0.06 (winter); m_{DIM} = 0.1 (summer); γ = 0.8; Γ = 0.76; Λ = 0.81

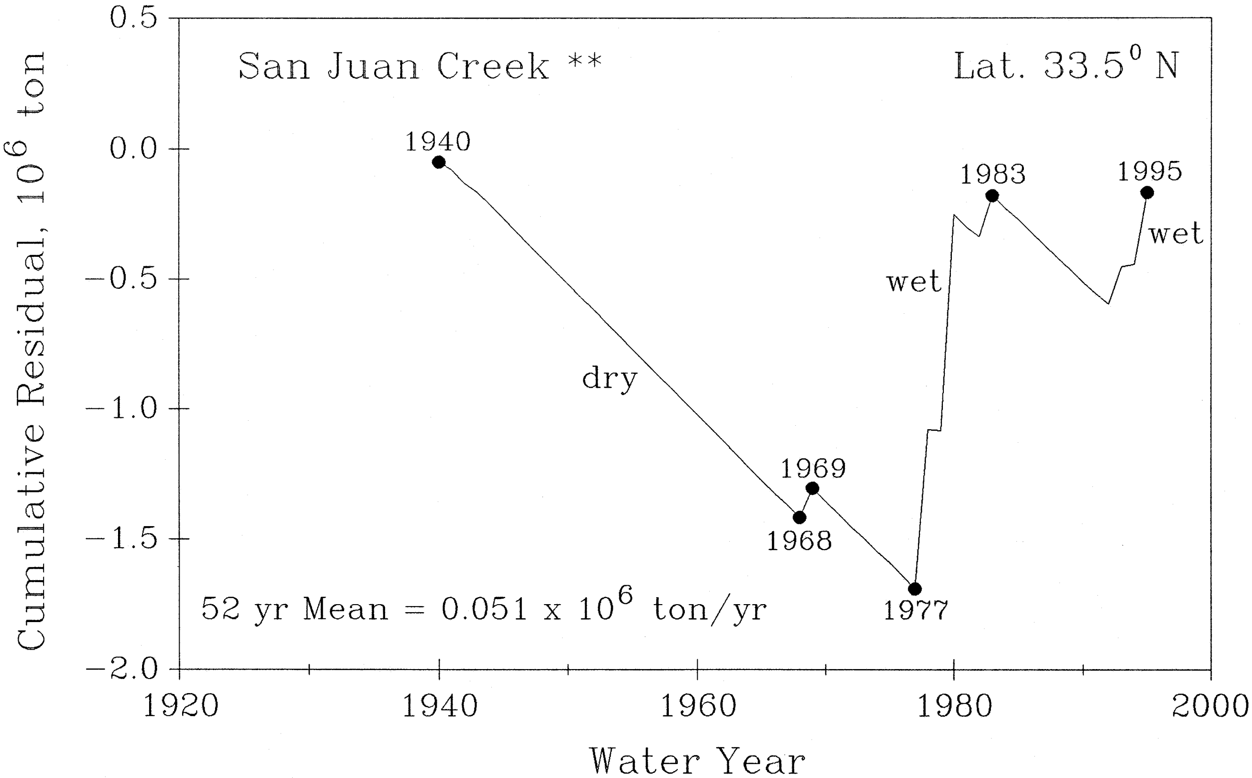


Figure 6.13. Cumulative residual time series of sediment flux for the San Juan Creek calculated using a 56-year mean (1940-1995), from Inman and Jenkins (1999).

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

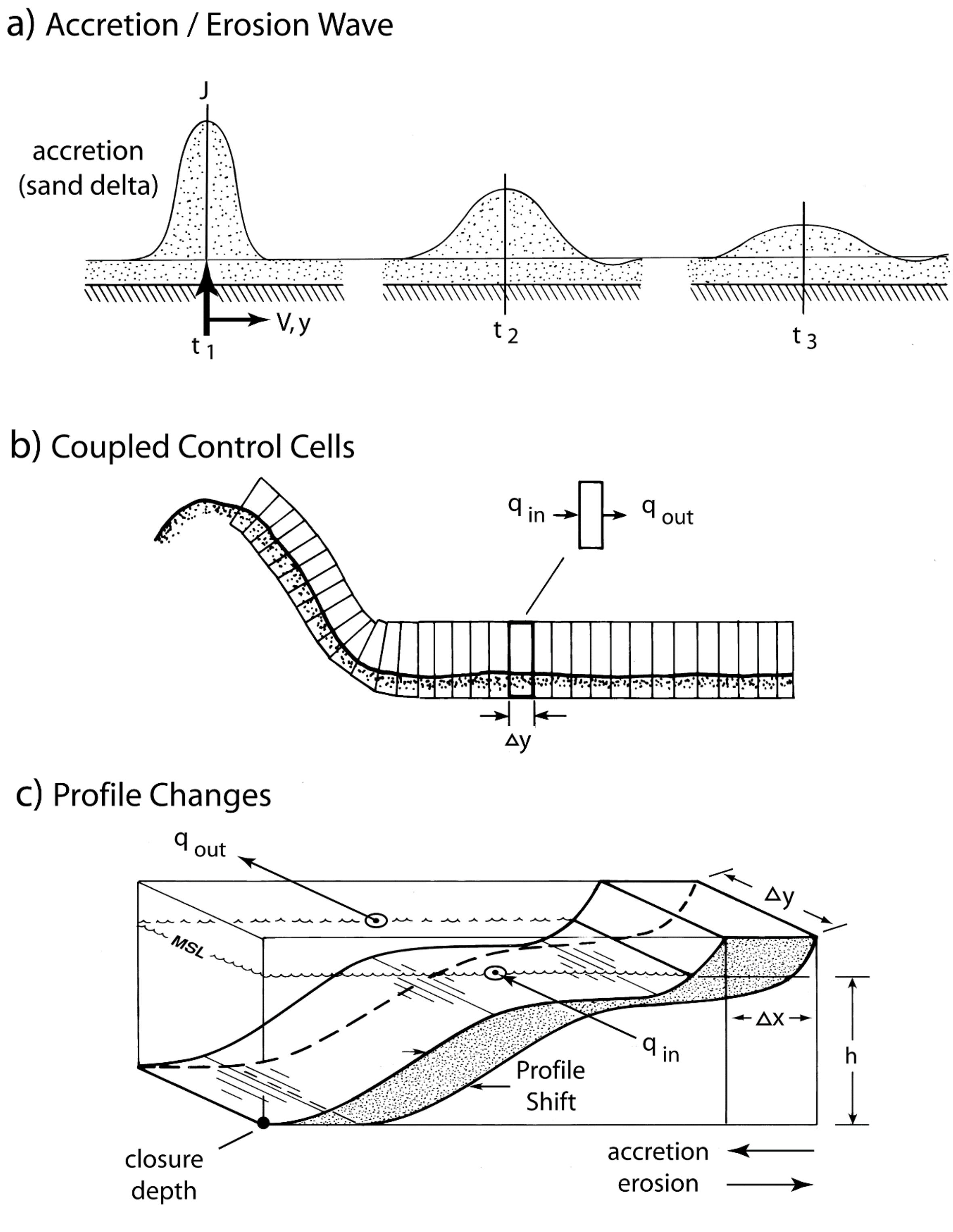


Figure 6.14: Computational approach for modeling changes in beach width and shoreline positon after Jenkins, et. al., (2007).

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 Doheny and Capistrano State Beaches. In 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 7.2 actually gives maximum total wave run-up of 11.88 ft. for the accreted beach conditions at Doheny Beach and 9.98 ft. for the eroded beach conditions. At Capistrano Beach, shoaling wave heights are greater and maximum total wave run-up is 12.73 ft. for the accreted beach conditions and 10.83 ft. for the eroded beach conditions

The annualized probability of recurrence of total water level is plotted in Figures 7.4 and 7.5 at Doheny and Capistrano Beaches, respectively, under the stationary hypothesis for still

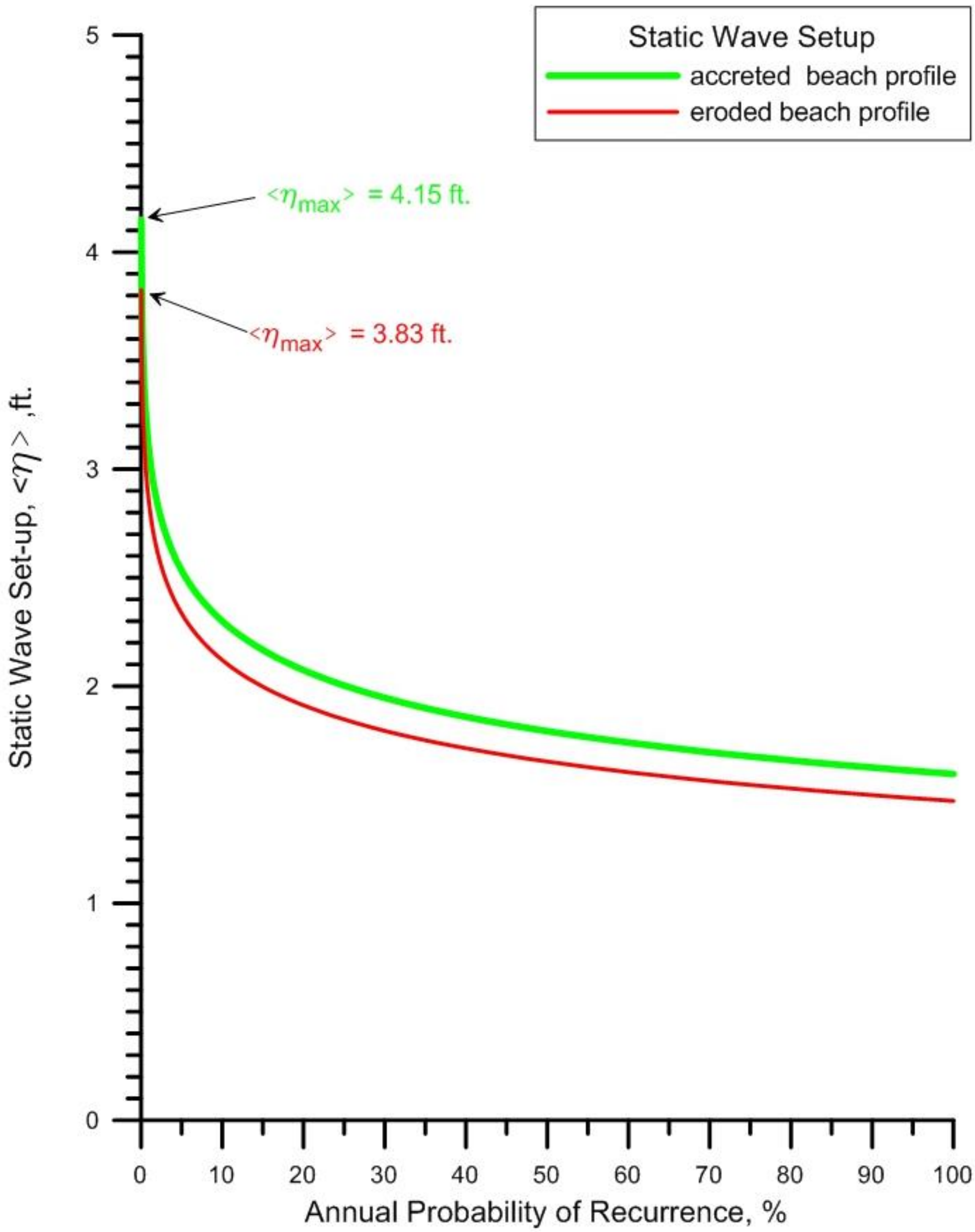


Figure 7.1: Probability of recurrence of static wave setup based on on extremal design wave heights from Weibull Type III distribution and beach profiles from Figures 6.8 - 6.11.

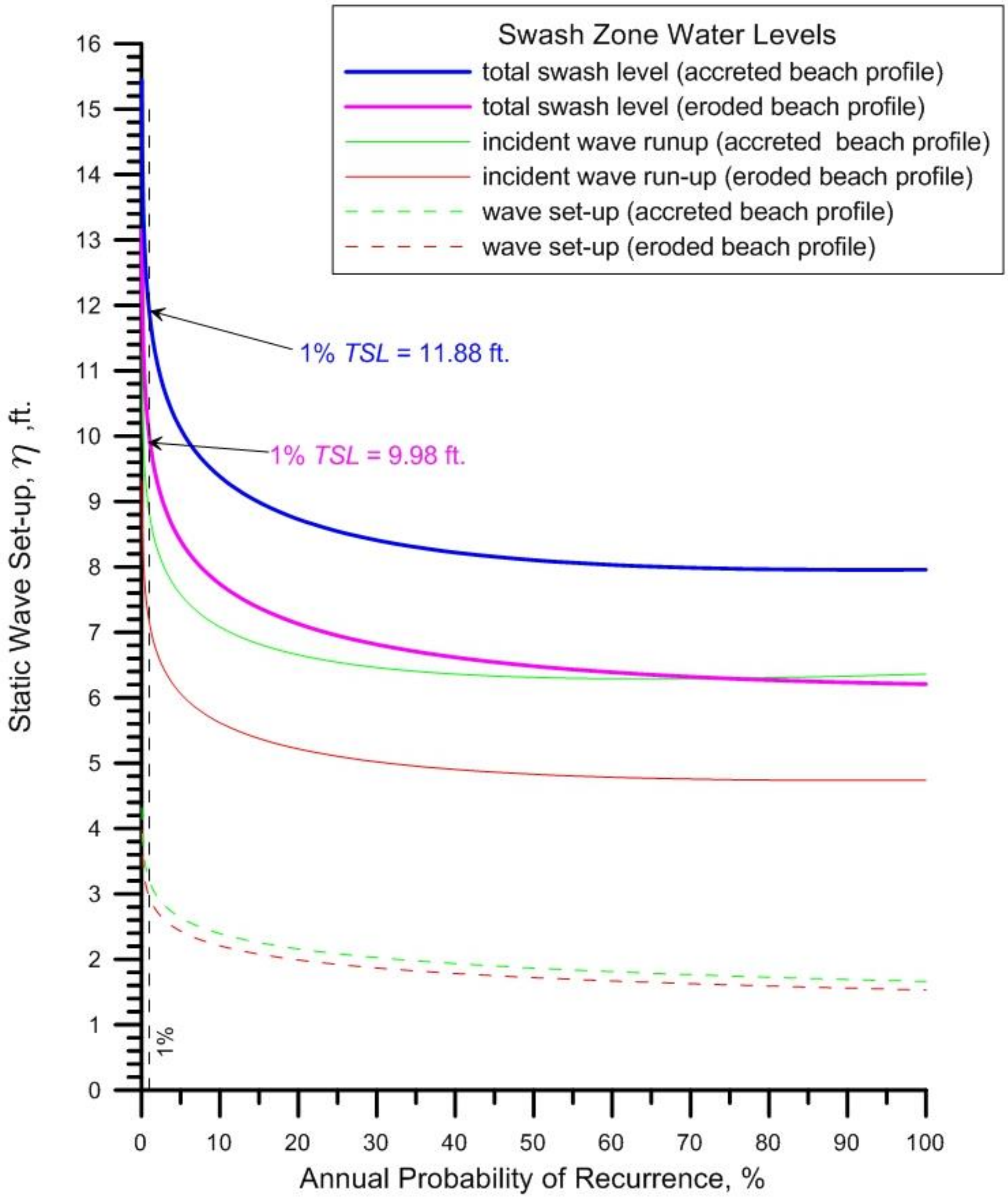


Figure 7.2 : Probability of recurrence of total swash level (*TSL*) at Doheny State Beach based on extremal design wave heights from Weibull Type III distribution

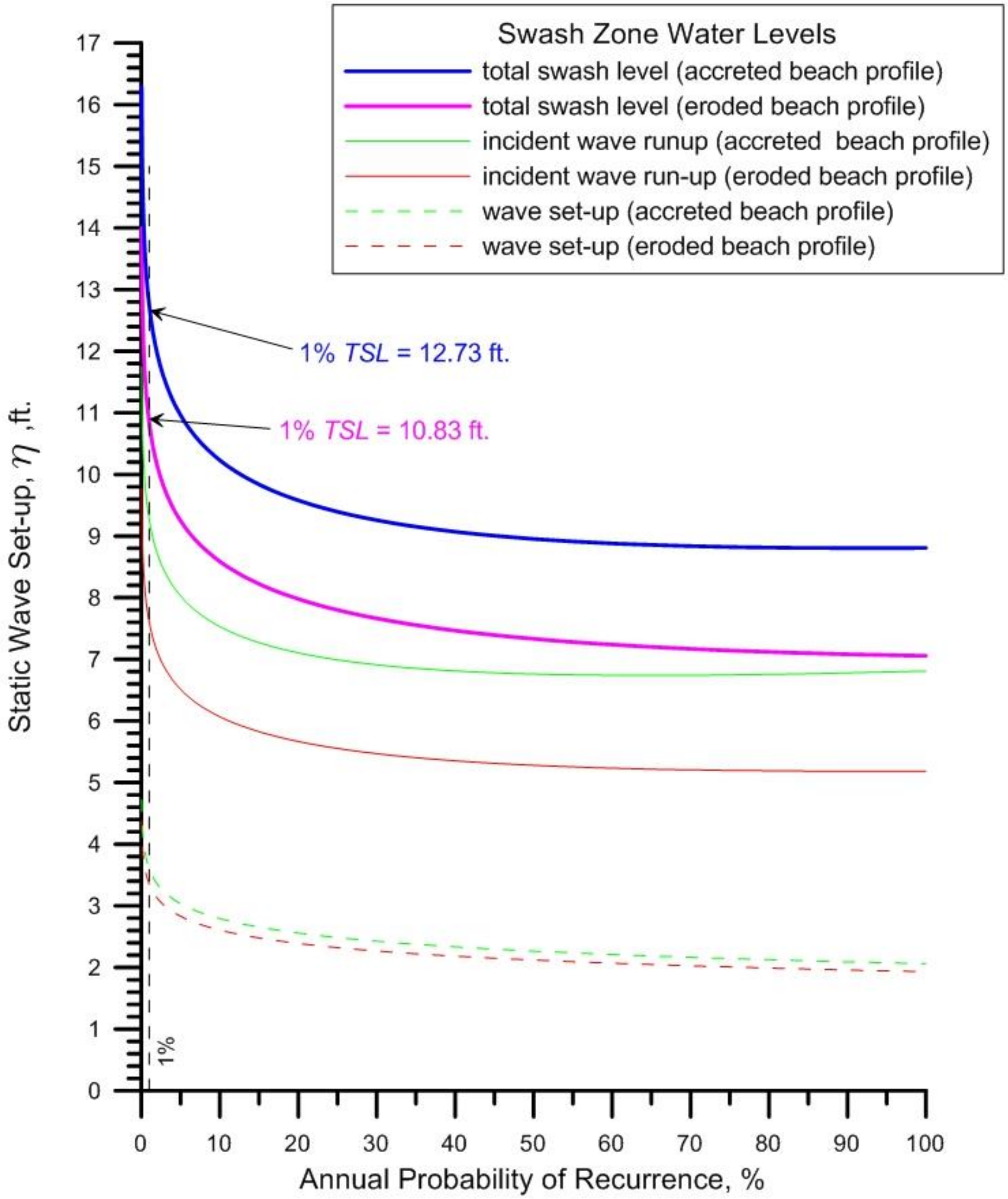


Figure 7.3 : Probability of recurrence of total swash level (*TSL*) at Capistrano State Beach based on extremal design wave heights from Weibull Type III distribution

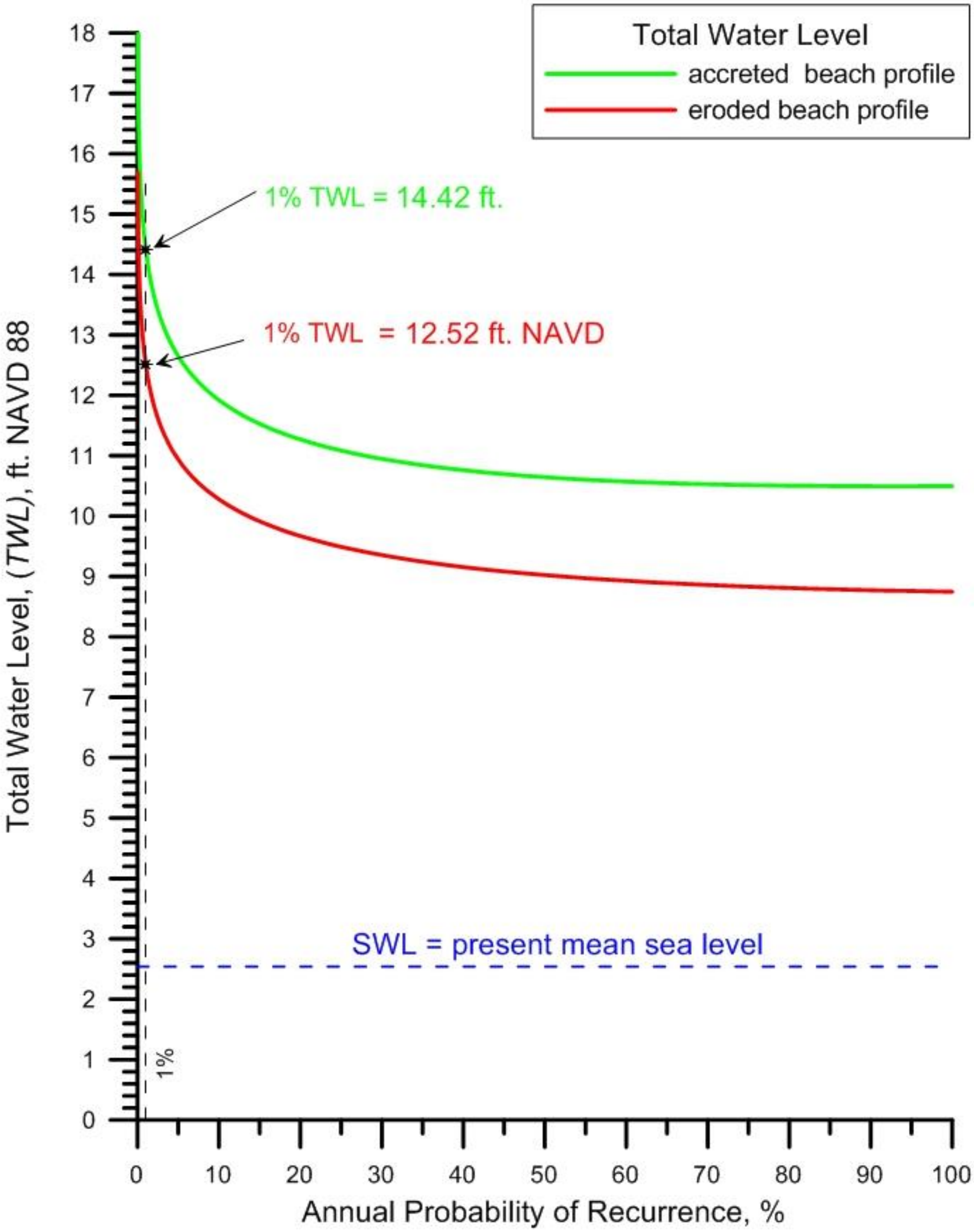


Figure 7.4: Annualized probability of recurrence of total water level at Doheny State Beach based on present sea level and extremal design wave heights from Weibull Type III distribution. SWL = MSL

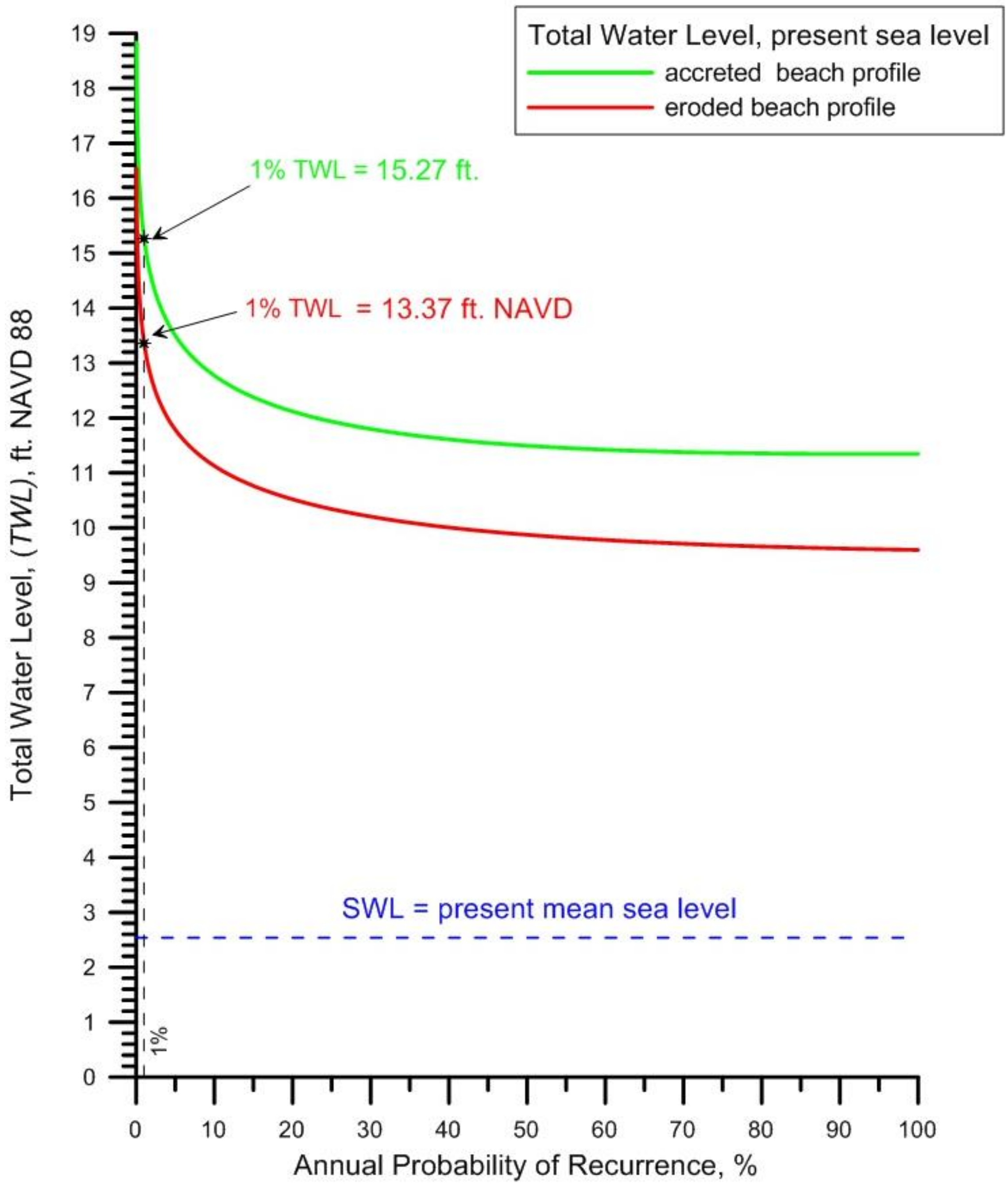


Figure 7.5 : Probability of recurrence of total water level at Capistrano State Beach for still water level at persent mean sea level based on extremal design wave heights from Weibull Type III distribution; SWL = MSL

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 at Doheny Beach is *TWL* = 17.98 ft. NAVD for the accreted beach conditions and *TWL* = 15.69 ft. NAVD for the eroded beach conditions. At Capistrano Beach (Figure 7.5), the maximum total water level is *TWL* = 18.83 ft. NAVD for the accreted beach conditions and *TWL* = 16.54 ft. NAVD for the eroded beach conditions. (Total water levels are higher at Capistrano Beach because shoaling waves during the 100-year event are higher at that location, cf. Figure 6.4). The total water level achieved under accreted beach conditions at present sea level exceeds the elevations of well heads A, B, C and G, which are located at Z_i == 17 ft. NAVD and Z_i == 17 ft. NAVD, respectively; 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, Figures 7.4 & 7.5 have 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 at Doheny Beach. At Capistrano Beach. The 1% probability (100-yr event) yields *TWL*(1%) = 15.27 ft. NAVD for the accreted beach conditions and *TWL*(1%) = 13.37 ft. NAVD for the eroded beach conditions.. Consequently we conclude that all the beach front facilities for the Doheny Desalination Project (Figure 6.3a & b) 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 7.6 and 7.7 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 at Doheny Beach, (Figure 7.6), the 1% total water level events reach *TWL*(1%) = 18.02 ft. NAVD for the accreted beach conditions and *TWL*(1%) = 16.12 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 if the beach is in an eroded winter condition. However, in the unlikely event that the 100 year storm occurs while the beach is still in a summer equilibrium condition (accreted beach), then Well Heads A-C will be overtopped by about 1 ft of excess runup, while Well Heads D and E would be partially wetted. At Capistrano Beach, the 1% total water level events at the low range projection for 2100 sea level, (Figure 7.7), reach *TWL*(1%) = 18.87 ft. NAVD for the accreted beach conditions and *TWL*(1%) = 16.97 ft. NAVD for the eroded beach conditions. While both well heads at Capistrano Beach would be safe from overtopping if Capistrano Beach were in an eroded winter state, Well Head G would be overtopped by about 0.87 ft. of runup if the beach remained in an accreted summer condition.

For the high-range 2100 sea level projections, (Figures 7.8 and 7.9) the 1% total water level events will overtop all of the well sites. At Doheny Beach, (Figure 7.8), the 1% total water level events reach *TWL*(1%) = 21.52 ft. NAVD for the accreted beach conditions and *TWL*(1%) = 19.62 ft. NAVD for the eroded beach conditions, exceeding the elevations of all well sites regardless of beach erosion or accretion. Similarly, at Capistrano Beach (Figure 7.9), the 1% total water level events reach *TWL*(1%) = 22.37 ft. NAVD for the accreted beach conditions and *TWL*(1%) = 20.47 ft. NAVD for the eroded beach conditions, again exceeding the elevations of all well sites regardless of beach erosion or accretion.

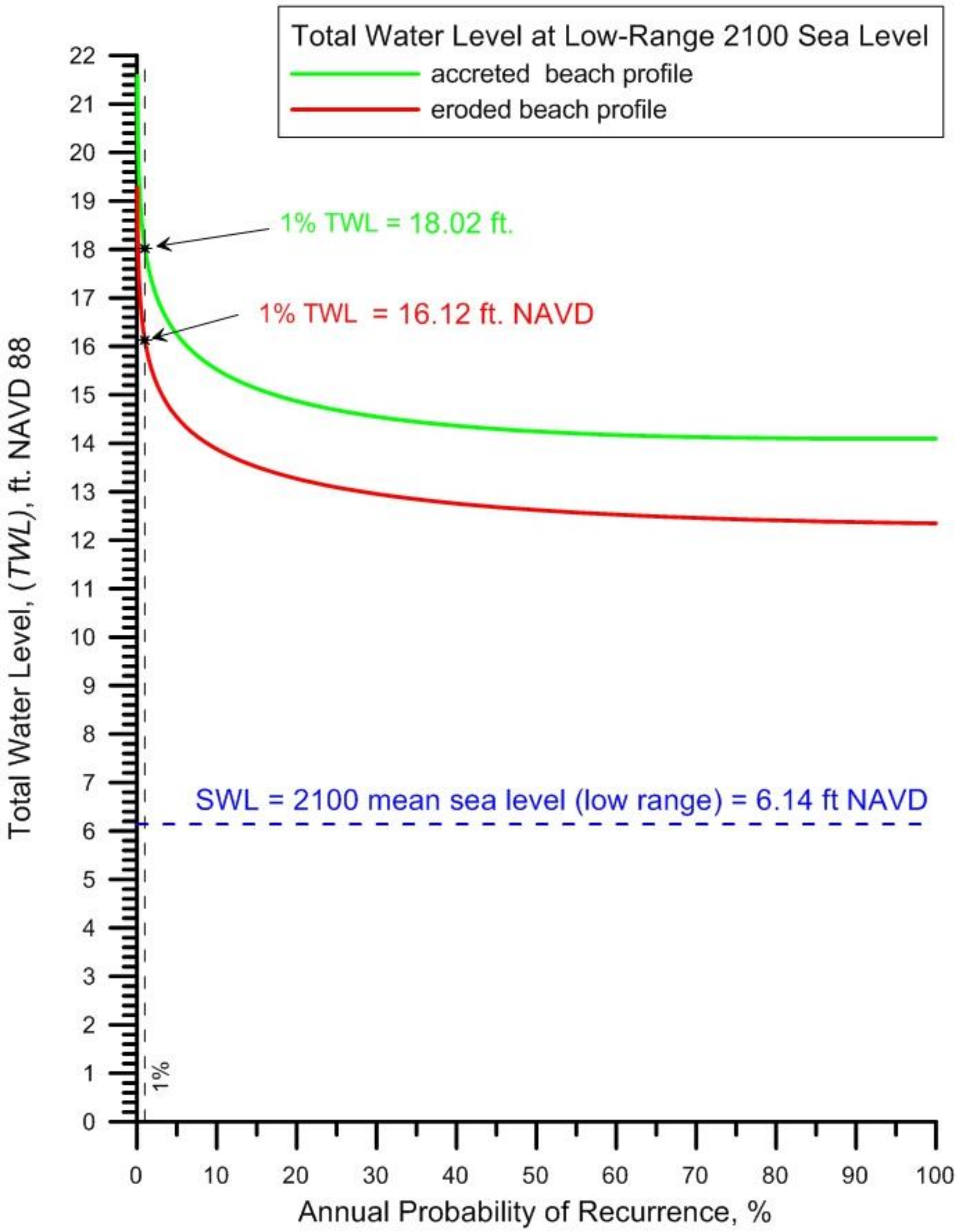


Figure 7.6: Annualized probability of recurrence of total water level at Doheny State Beach for still water level at 2100 (low range) mean sea level, based on extremal design wave heights from Weibull Type III distribution. Sea level based on CCC (2018), Appendix-G, Table G-11

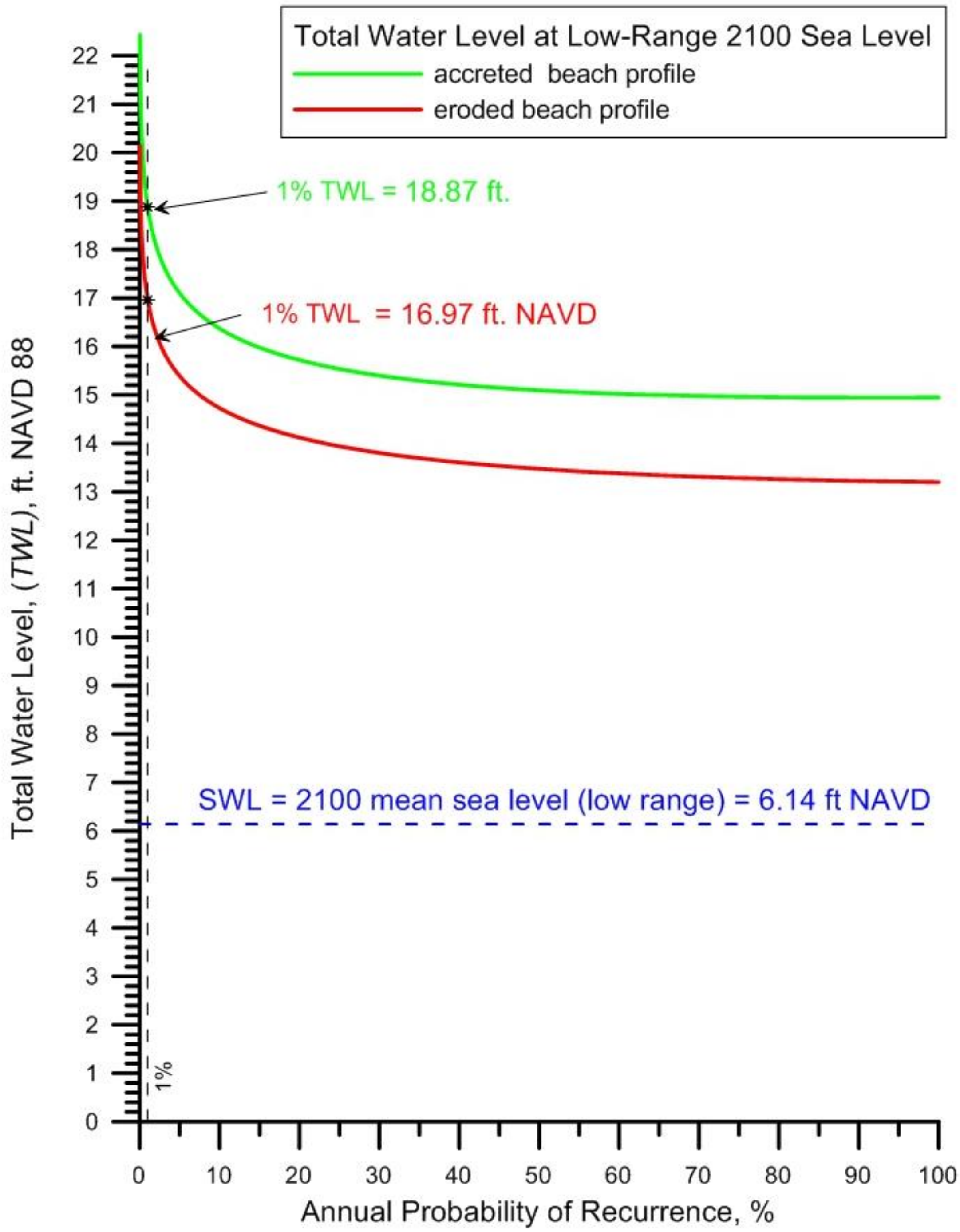


Figure 7.7: Annualized probability of recurrence of total water level at Capistrano State Beach for still water level at 2100 (low range) mean sea level, based on extremal design wave heights from Weibull Type III distribution. Sea level based on CCC (2018), Appendix-G, Table G-11

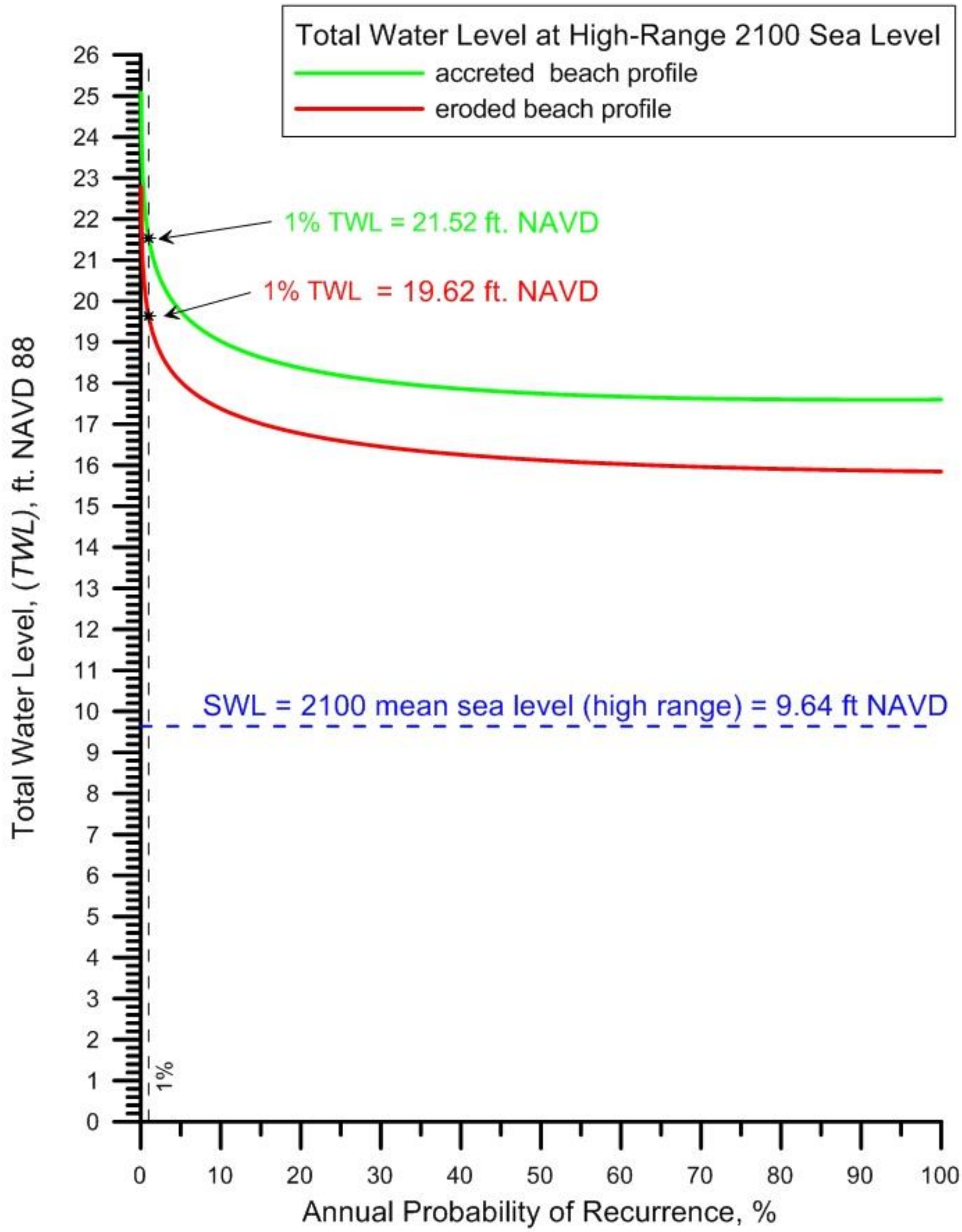


Figure 7.8: Annualized probability of recurrence of total water level at Doheny State Beach for still water level at 2100 (high range) mean sea level, based on extremal design wave heights from Weibull Type III distribution. Sea level based on CCC (2018), Appendix-G, Table G-11

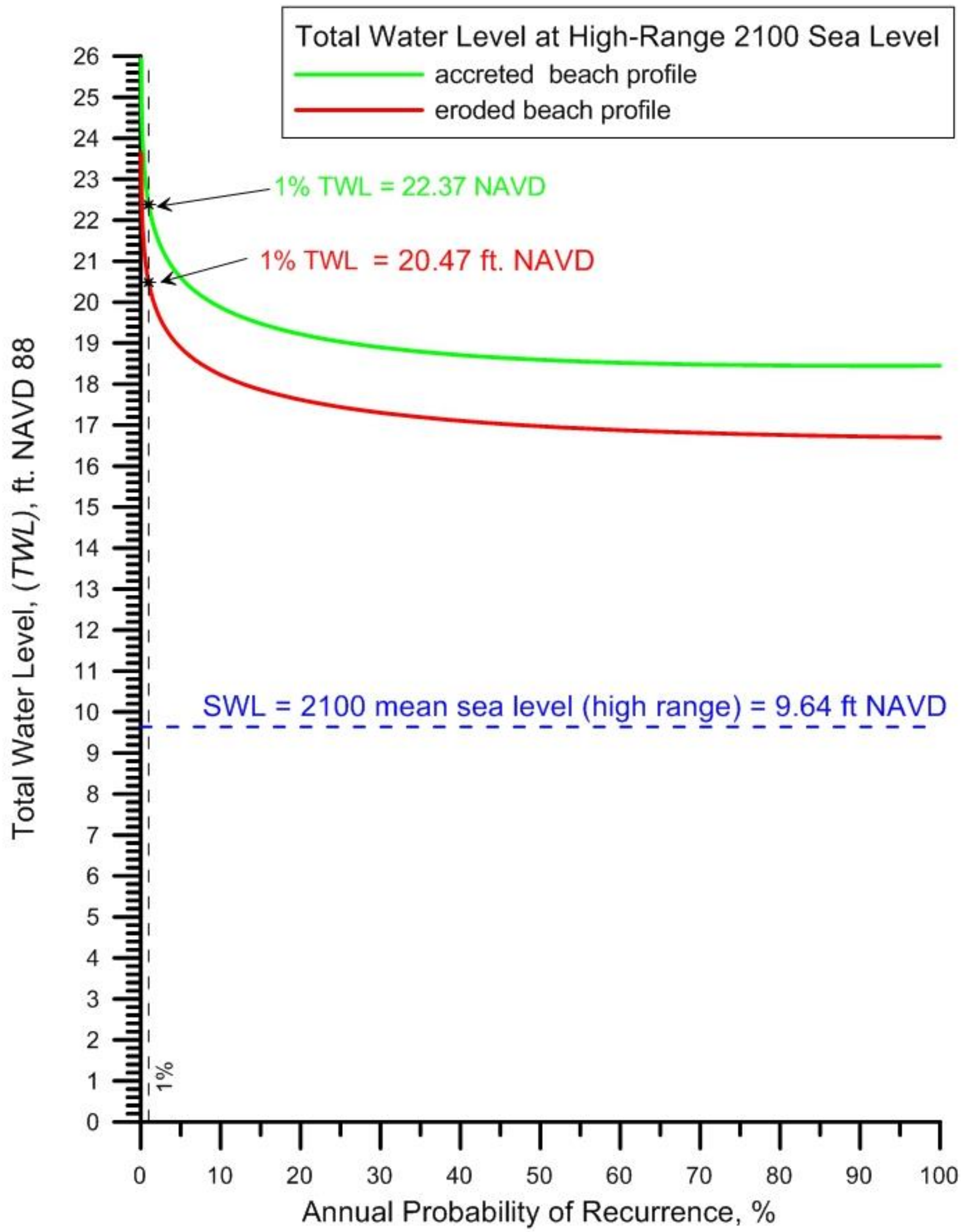


Figure 7.9: Annualized probability of recurrence of total water level at Capistrano State Beach for still water level at 2100 (high range) mean sea level, based on extremal design wave heights from Weibull Type III distribution. Sea level based on CCC (2018), Appendix-G, Table G-11

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 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 7.10 & 7.11 for Doheny and Capistrano Beaches, respectively, while those for 2100 sea levels are found in Figures 7.12 – 7.15. Comparing these results with the total water level results in Figures 7.4-7.9 (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 at present sea level at Doheny Beach in Figure 7.10 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 present 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 Figures 7.10 & 7.11 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. However, once we admit to 2100 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 low-range 2100 sea level projections, (Figures 7.12 &7.13) the three well sites at Doheny Beach on the north side of San Juan Creek (Well Heads A-C) and one of the wells at the Capistrano Beach site (Well Head G) will experience minor overtopping, even for a 1 year event if the beaches have been accreted by additional sands from water shed floods or still retain a built-out summer equilibrium beach profile, with overtopping rates of about Q'(1yr) = 0.038 cfs per lineal ft. of shoreline, per equation (17). If a 100-yr total water level event occurs during the low-range projection of 2100 sea levels, then Figures 7.12 & 7.13 indicate that all of the well sites will be overtopped to varying degrees if the beaches remain in an accreted condition (i.e.,with elevated berms and steep beach slopes). Under these beach conditions, overtopping rates will range from a high of Q'(100yr) = 0.094 cfs per lineal ft. of shoreline at Well Heads A- C on Doheny Beach, to a low of Q'(100yr) = 0.014 cfs/ft at Well Head H on Capistrano Beach, while overtopping rates at Well Heads D & E would be Q'(100yr) = 0.027 cfs/ft at Doheny Beach and Q'(100yr) = 0.081 cfs/ft at Well Head G on Capistrano Beach. Interestingly enough, none of the well heads would experience overtopping during a 100 year event when occurring during the low range 2100 sea levels if the beach were eroded, which would be the most likely condition during a 100-

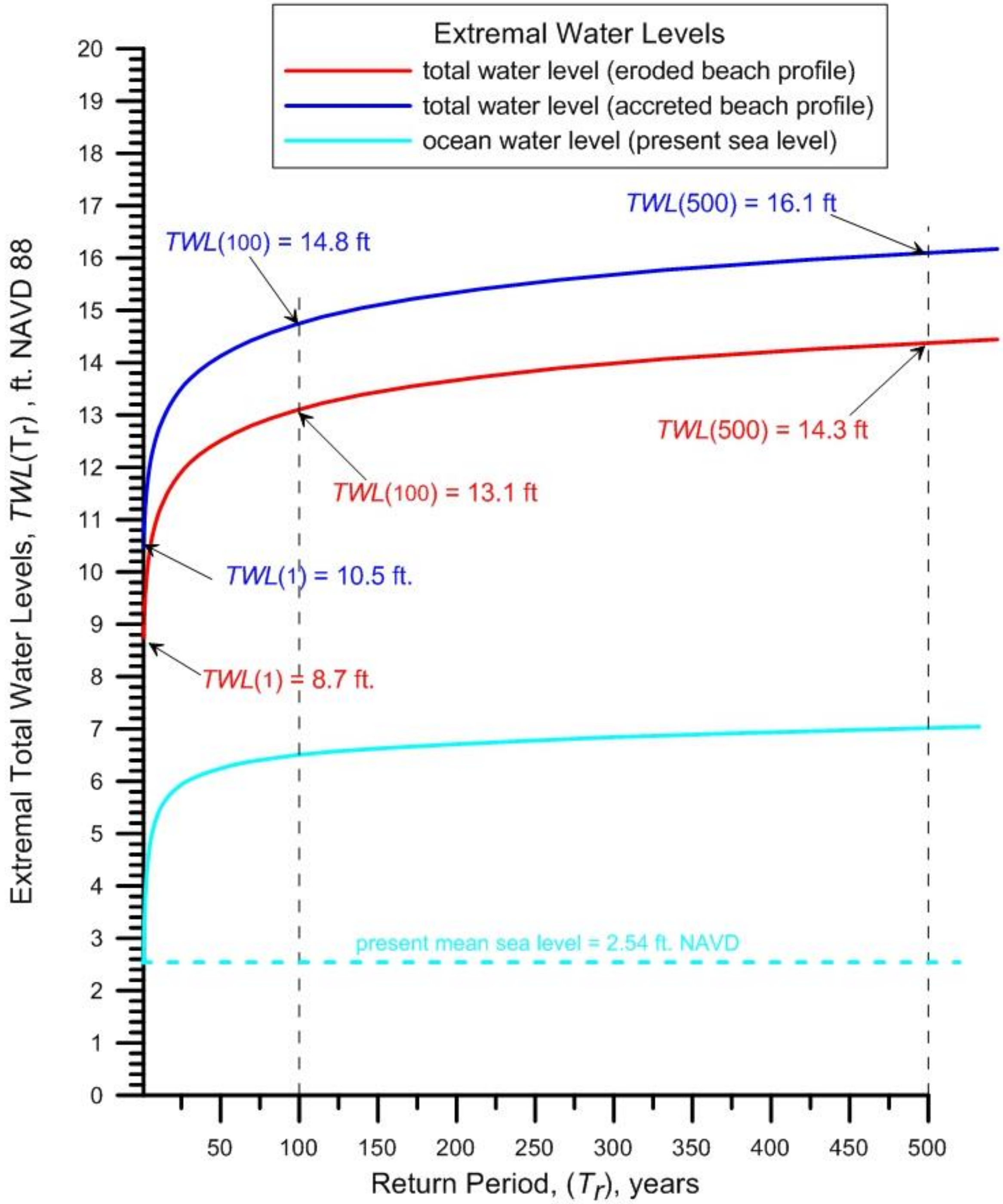


Figure 7.10: Return period of extremal total water level (TWL) at Doheny State Beach based on extremal design wave heights from Weibull Type III distribution and still water levels set at extremal ocean water levels for present sea level, per NOAA tide gage #941-0230

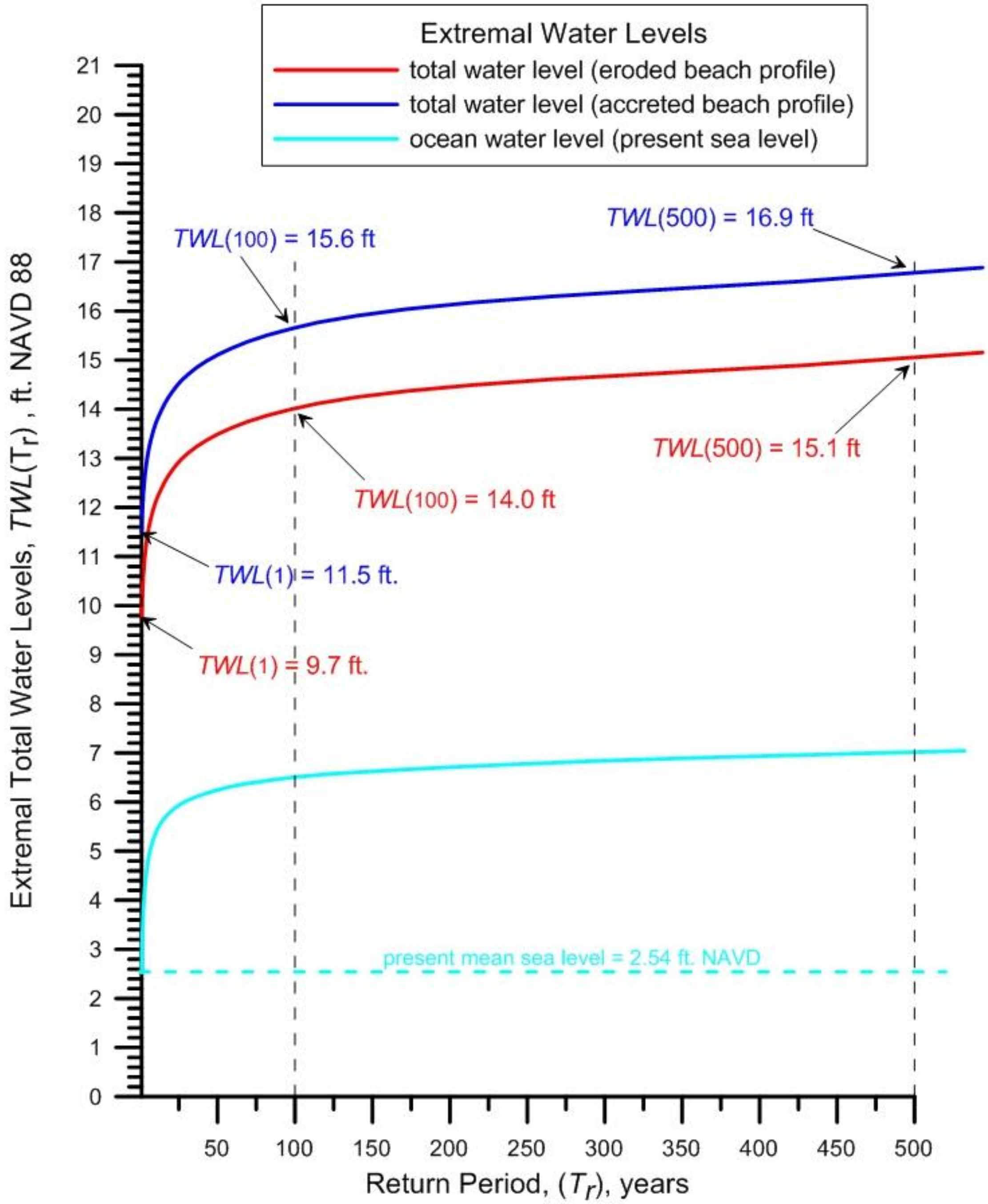


Figure 7.11: Return period of extremal total water level (TWL) at Capistrano State Beach based on extremal design wave heights from Weibull Type III distribution and still water levels set at extremal ocean water levels for present sea level, per NOAA tide gage #941-0230.

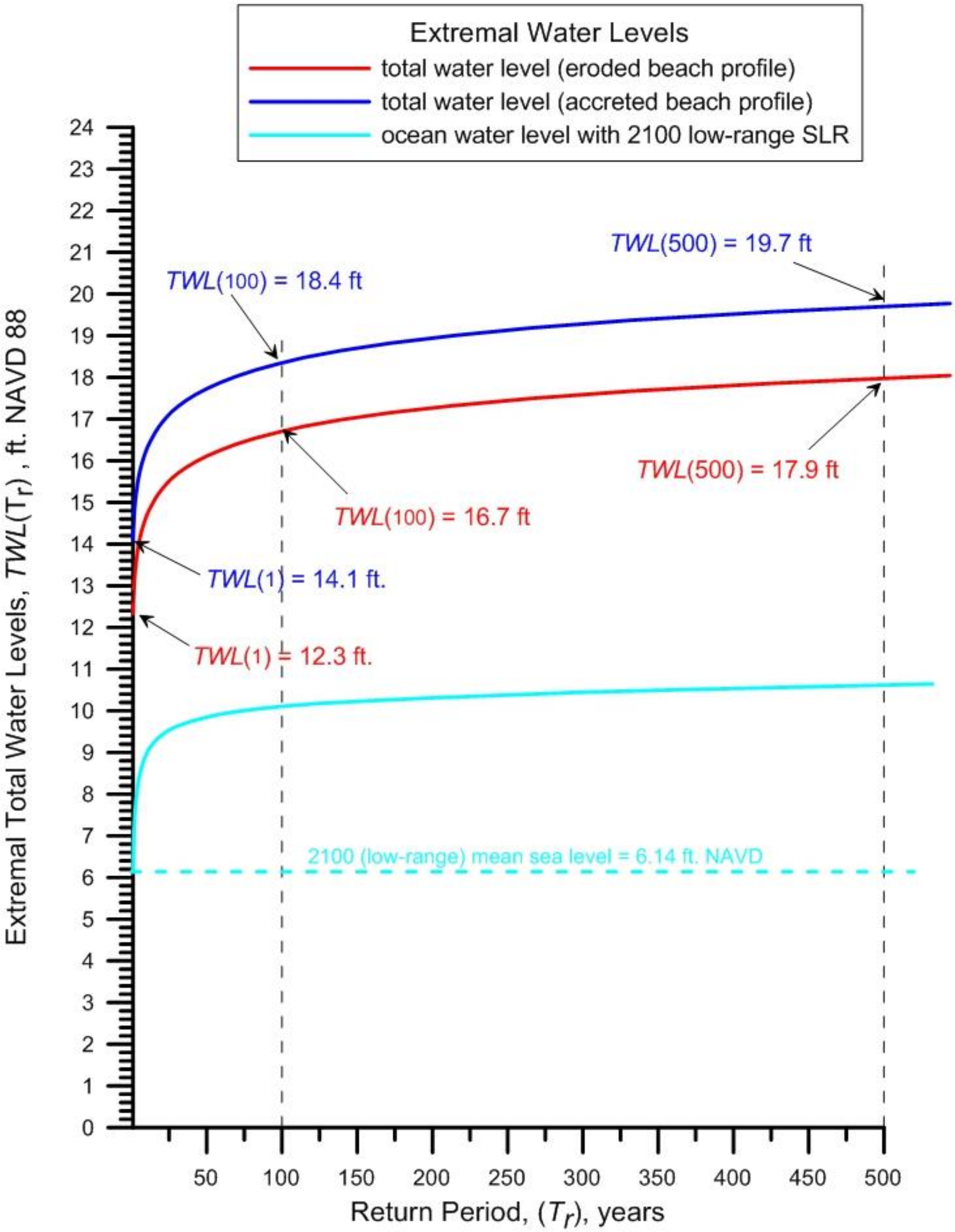


Figure 7.12: Return period of extremal total water level (TWL) at Doheny State Beach based on extremal design wave heights from Weibull Type III distribution and still water levels set at extremal ocean water levels for the low-range 2100 sea level rise, per CCC (2018), Appendix-G, Table G-11

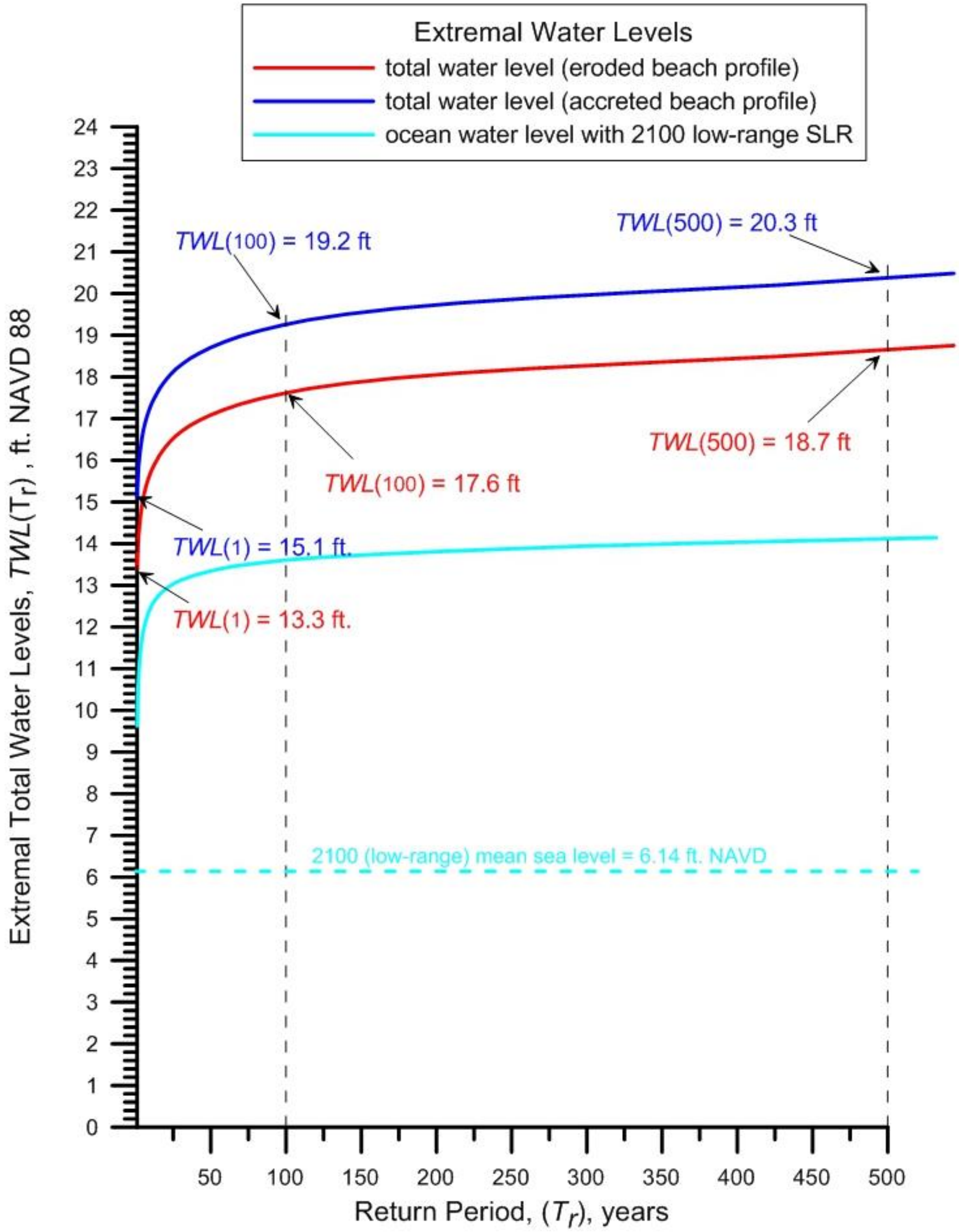


Figure 7.13: Return period of extremal total water level (TWL) at Capistrano State Beach based on extremal design wave heights from Weibull Type III distribution and still water levels set at extremal ocean water levels for the low-range 2100 sea level rise, per CCC (2018), Appendix-G, Table G-11

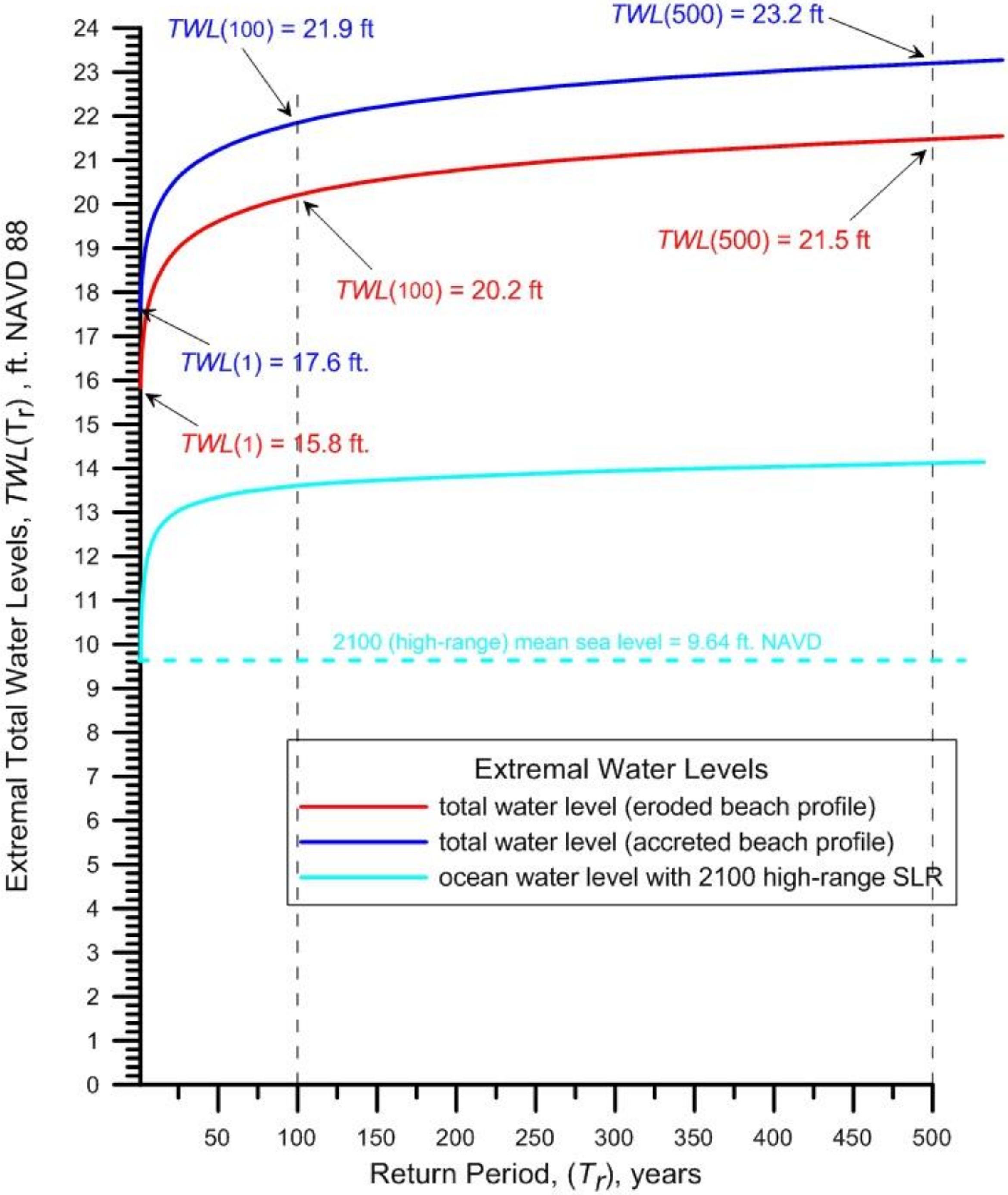


Figure 7.14: Return period of extremal total water level (TWL) at Doheny State Beach based on extremal design wave heights from Weibull Type III distribution and still water levels set at extremal ocean water levels for the high-range 2100 sea level rise, per CCC (2018), Appendix-G, Table G-11

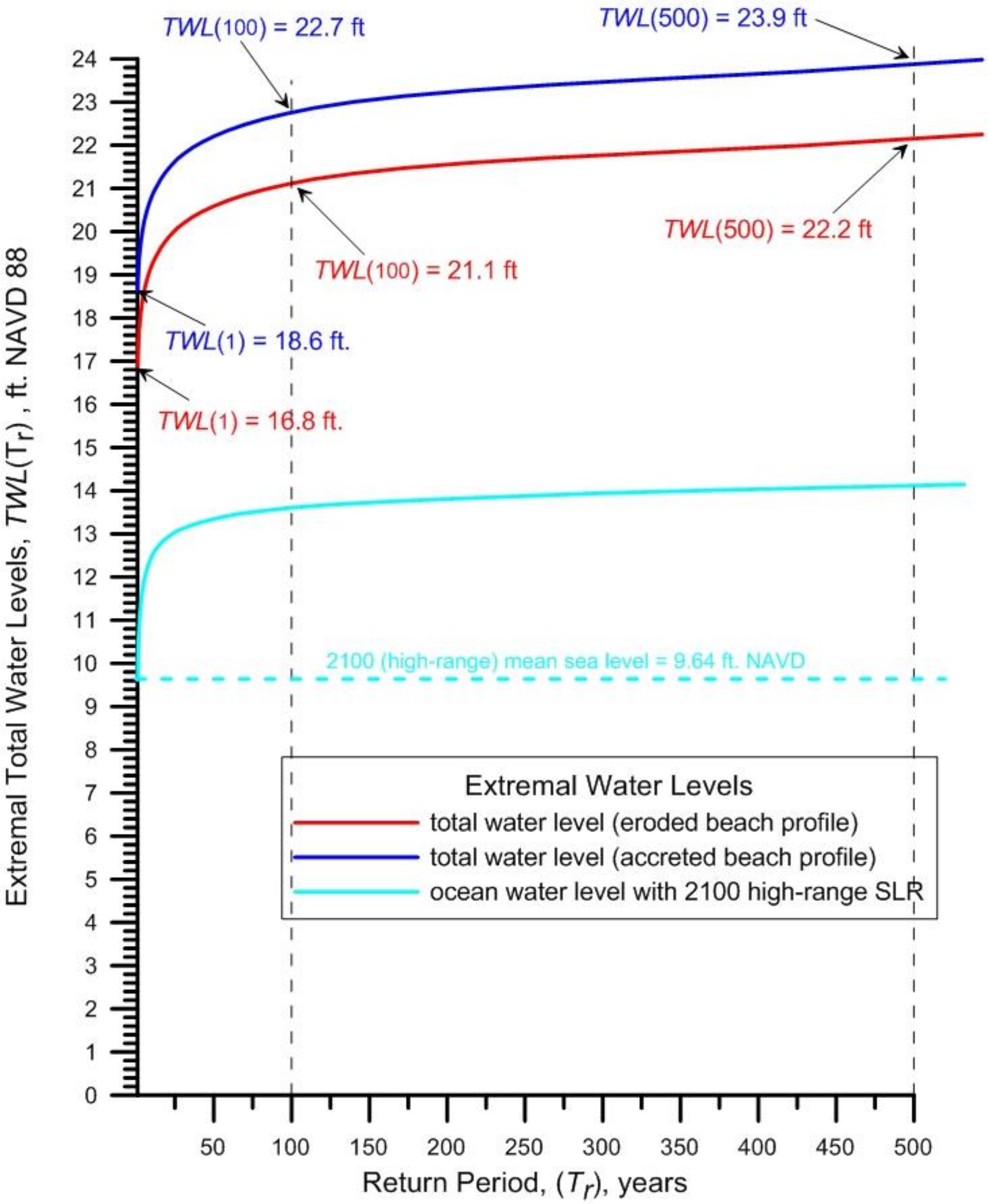


Figure 7.15: Return period of extremal total water level (TWL) at Capistrano State Beach based on extremal design wave heights from Weibull Type III distribution and still water levels set at extremal ocean water levels for the high-range 2100 sea level rise, per CCC (2018), Appendix-G, Table G-11

year event. 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 at Doheny Beach (Figure 7.14), the 100 year total water level events reach *TWL*(100) = 21.9 ft. NAVD for the accreted beach conditions and *TWL*(100) = 20.2 ft. NAVD for the eroded beach conditions; while at Capistrano Beach, (Figure 7.15), *TWL*(100) = 22.7 ft. NAVD for the accreted beach conditions and *TWL*(100) = 21.1 ft. NAVD for the eroded beach conditions. Consequently all beach front facilities for the Doheny Desalination Project would be vulnerable to flooding by the 100-year event if it were occur during 2100 high range sea level projections. The lowest lying well heads (Well Heads A-C at Doheny Beach) would experience the highest overtopping rates, ranging from Q'(100yr) = 0.216 cfs/ft. to 0.331 cfs/ft. depending on the eroded or accreted condition of Doheny State Beach. According Table VI-5-6 in the Coastal Engineering Manual (USACE, 2006) overtopping rates of this order of magnitude are very dangerous for pedestrian and vehicle traffic, and may cause structural damage to adjacent buildings; but the well heads and pumps for the Doheny Desalination project will be protected by steel vault enclosures. The smallest overtopping rates during the 100-year event at the high range 2100 sea level projections will occur at the highest located well head (Well Head H) at the Capistrano Beach site where overtopping rates will range from Q' (100yr) = 0.142 cfs/ft. to 0.250 cfs/ft., with overtopping rates at Well Heads D & E on Doheny Beach ranging from Q'(100yr) = 0.149 cfs/ft to 0.263cfs/ft and Q'(100yr) = 0.209 cfs/ft to 0.318 cfs/ft at Well Head G on Capistrano Beach. While these overtopping rates are still dangerous to pedestrian and vehicle traffic, they are easily mitigated by the steel vault enclosures of the well heads and pumps being placed at Capistrano Beach. The results for total water levels and overtopping rates based on extremal still water levels analysis methods are summarized in Table 7.1 for the Doheny Beach well sites, and in Table 7.2 for the Capistrano Beach well sites.

Table 7.1:Doheny Beach Extremal Total Water Level (*TWL) and Overtopping Rates (Q')

	Well Head-A Elevation = 17 ft. NAVD	Well Head-B Elevation = 17 ft. NAVD	Well Head-C Elevation = 17 ft. NAVD	Well Head-D Elevation = 18 ft. NAVD	Well Head-E Elevation = 18 ft. NAVD
*$TWL(1)$ Present Sea Level (eroded/accreted)	8.7/10.5 ft. NAVD status = dry	8.7/10.5ft. NAVD status = dry	8.7/10.5ft. NAVD status = dry	8.7/10.5ft. NAVD status = dry	8.7/10.5ft. NAVD status = dry
*$Q'(1)$ 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.
*$TWL(1)$ 2100 Sea Level Low Range Projection (eroded/accreted)	12.3/14.1 ft. NAVD status = dry	12.3/14.1 ft. NAVD status = dry	12.3/14.1 ft. NAVD status = dry	12.3/14.1 ft. NAVD status = dry	12.3/14.1 ft. NAVD status = dry
*$Q'(1)$ 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.
*$TWL(1)$ 2100 Sea Level High Range Projection (eroded/accreted)	15.8/17.6 ft. NAVD status = flooded accreted beach	15.8/17.6 ft. NAVD status = flooded accreted beach	15.8/17.6 ft. NAVD status = flooded accreted beach	15.8/17.6 ft. NAVD status = dry	15.8/17.6 ft. NAVD status = dry
*$Q'(1)$ 2100 Sea Level High Range Projection (eroded/accreted)	0.0/0.038 cfs/ft.	0.0/0.038 cfs/ft.	0.0/0.038 cfs/ft.	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.
**$TWL(100)$ 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.
**$TWL(100)$ 2100 Sea Level Low Range Projection (eroded/accreted)	16.7/18.4 ft. NAVD status = flooded accreted beach	16.7/18.4 ft. NAVD status = flooded accreted beach	16.7/18.4 ft. NAVD status = flooded accreted beach	16.7/18.4 ft. NAVD status = flooded accreted beach	16.7/18.4 ft. NAVD status = flooded accreted beach
**$Q'(100)$ 2100 Sea Level Low Range Projection (eroded/accreted)	0.0/0.094 cfs/ft.	0.0/0.094 cfs/ft.	0.0/0.094 cfs/ft.	0.0/0.027 cfs/ft.	0.0/0.027 cfs/ft.
**$TWL(100)$ @ 2100 Sea Level High Range Projection (eroded/accreted)	20.2/21.9 ft. NAVD status = flooded	20.2/21.9 ft. NAVD status = flooded	20.2/21.9 ft. NAVD status = flooded	20.2/21.9 ft. NAVD status = flooded	20.2/21.9 ft. NAVD status = flooded
**$Q'(100)$ 2100 Sea Level High Range Projection (eroded/accreted)	0.216/0.331 cfs/ft.	0.216/0.331 cfs/ft.	0.216/0.331 cfs/ft.	0.149/0.263 cfs/ft.	0.149/0.263 cfs/ft.

*Evaluated for the 1-yr return period; ** Evaluated for the 100-yr return period

Table 7.2: Capistrano Beach Extremal Total Water Level (*TWL*) and Overtopping Rates (Q')

	Well Head-G Elevation = 18 ft. NAVD	Well Head-H Elevation = 19 ft. NAVD
*$TWL(1)$ Present Sea Level (eroded/accreted)	9.7/11.5 ft. NAVD status = dry	9.7/11.5 ft. NAVD status = dry
*$Q'(1)$ Present Sea Level (eroded/accreted)	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.
*$TWL(1)$ 2100 Sea Level Low Range Projection (eroded/accreted)	13.3/15.1 ft. NAVD status = dry	13.3/15.1 ft. NAVD status = dry
*$Q'(1)$ 2100 Sea Level Low Range Projection (eroded/accreted)	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.
*$TWL(1)$ 2100 Sea Level High Range Projection (eroded/accreted)	16.8/18.6 ft. NAVD status = flooded accreted beach	16.8/18.6 ft. NAVD status = dry
*$Q'(1)$ 2100 Sea Level High Range Projection (eroded/accreted)	0.0/0.038 cfs/ft.	0.0/0.00 cfs/ft.
**$TWL(100)$ Present Sea Level (eroded/accreted)	14.0/15.6 ft. NAVD status = dry	14.0/15.6 ft. NAVD status = dry
**$Q'(100)$ Present Sea Level (eroded/accreted)	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.
**$TWL(100)$ 2100 Sea Level Low Range Projection (eroded/accreted)	17.6/19.2 ft. NAVD status = flooded accreted beach	17.6/19.2 ft. NAVD status = flooded accreted beach
**$Q'(100)$ 2100 Sea Level Low Range Projection (eroded/accreted)	0.0/0.081 cfs/ft.	0.0/0.014 cfs/ft.
**$TWL(100)$ @ 2100 Sea Level High Range Projection (eroded/accreted)	21.1/22.7 ft. NAVD status = flooded	21.1/22.7 ft. NAVD status = flooded
**$Q'(100)$ 2100 Sea Level High Range Projection (eroded/accreted)	0.209/0.318 cfs/ft.	0.142/0.250 cfs/ft.

*Evaluated for the 1-yr return period
** Evaluated for the 100-yr return period

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 the *California Coastal Commision Sea Level Rise Guidance Policy Guidance* document (CCC, 2018), 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 & 8.2 also indicate that all of the shore facilities of the Doheny Desalination Project are above tsunami inundation levels at present sea level. However, all of the well heads at both Doheny and Capistrano Beaches would suffer some degree of tsunami overtopping if concurrent with 2100 sea levels, and the overtopping rates could be quite severe, especially for the high 210 sea level rise projections. At the low range of 2100 sea level projections, total water levels would reach *TWL* = 18.82 ft. NAVD at Doheny Beach and *TWL* = 18.83 ft. NAVD at Capistrano Beach. Well Heads A-C at Doheny Beach would experience the highest overtopping surges of Q' = 1.142 cfs/ft while Well Head G at Capistrano Beach would remain high and dry. However, if the tsunami occurred atop the high range sea level rise projections for year 2100, then total water levels would reach *TWL* = 22.31 ft. NAVD at Doheny Beach and *TWL* = 22.4 ft. NAVD at Capistrano Beach, sufficient to overtop all the well sites of the Doheny Desalination Project. In this case the tsunami surge could produce very high, although short-lived, overtopping rates reaching a maximum of Q' = 5.691 cfs/ft at Well Heads A-C on Doheny Beach and a minimum of Q' = 2.916 cfs/ft at Well Head H on Capistrano Beach. Undoubtedly, the steel vault enclosures of the well heads can be designed to withstand these high surge rates, but particular attention should be given to the foundations of the vaults to assure the foundations have adequate depth to prevent undercutting by scour. 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 inundated by a shoaling tsunami solitary wave.

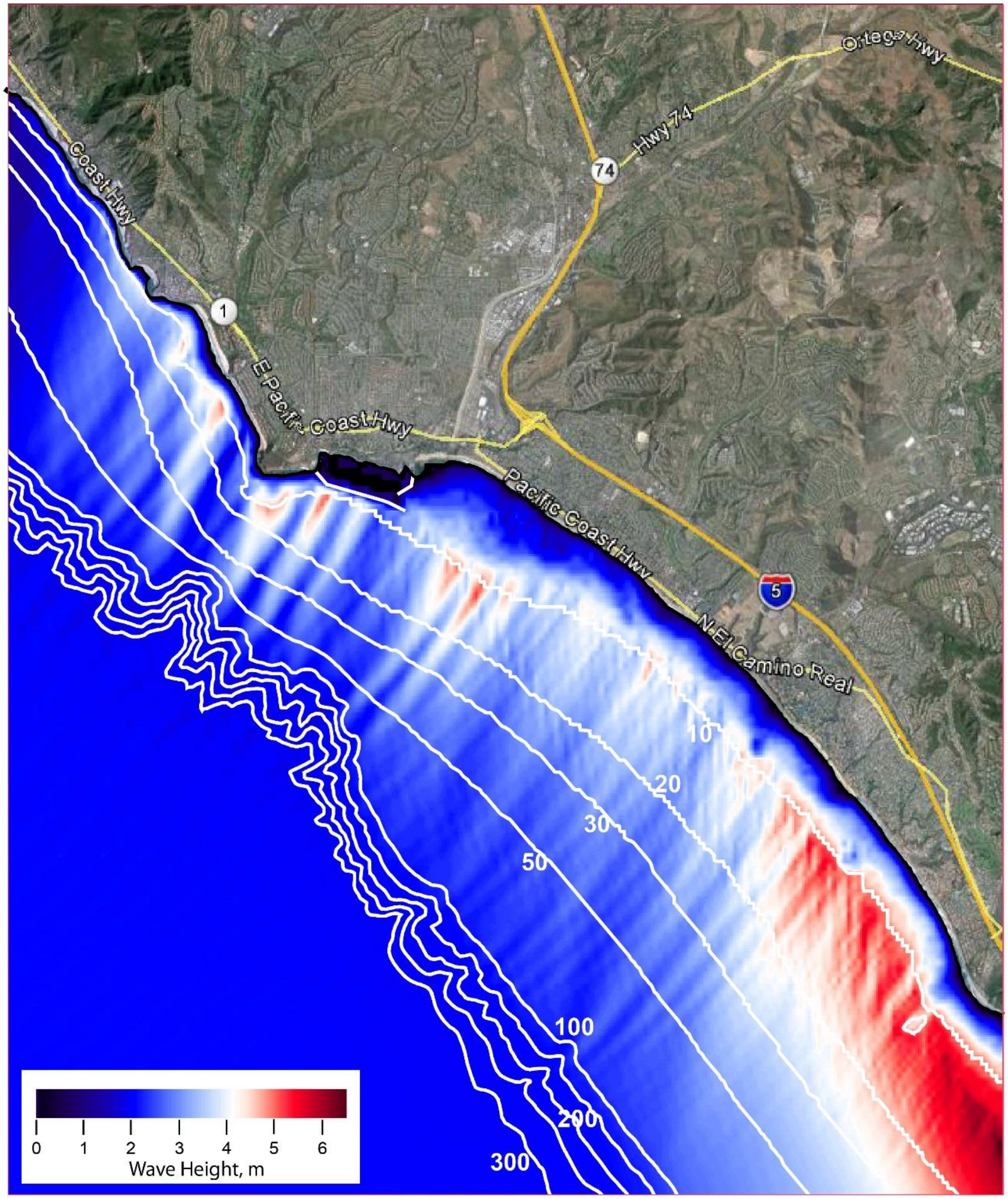


Figure 8.1: High resolution refraction/diffraction computation for a 2m high solitary tsunami wave approaching Doheny Beach from 165 degrees true.

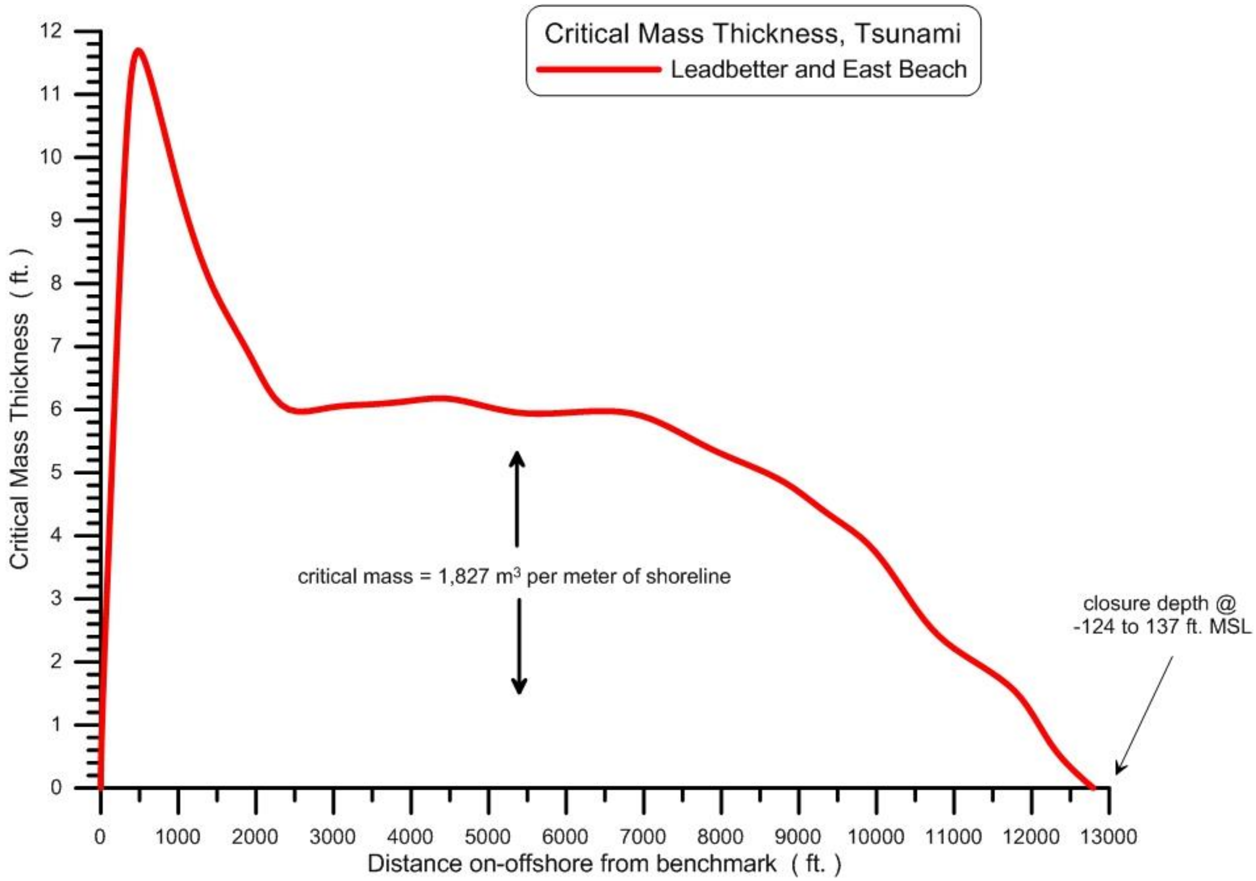


Figure 8.2: Thickness of critical mass envelope at historic survey ranges Doheny Beach, calculated by the calibrated CEM sediment budget based a 2m high solitary tsunami wave approaching Doheny Beach from 165 degrees true. Closure depth = -124 to -137 ft. MSL; critical mass volume = 1,827 m^3 per meter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

Table 8.1: Tsunami Total Water Level (*TWL*) and Overtopping Rates (Q') Analysis at the Doheny Beach Site

	Well Head-A Elevation = 17 ft. NAVD	Well Head-B Elevation = 17 ft. NAVD	Well Head-C Elevation = 17 ft. NAVD	Well Head-D Elevation = 18 ft. NAVD	Well Head-E 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 cfs/ft.	0.0 cfs/ft.	0.0 cfs/ft.	0.0 cfs/ft.	0.0 cfs/ft.
TWL 2100 Sea Level Low Range Projection	18.82 ft. NAVD status = flooded	18.82 ft. NAVD status = flooded	18.82 ft. NAVD status = flooded	18.82 ft. NAVD status = flooded	18.82 ft. NAVD status = flooded
Q' 2100 Sea Level Low Range Projection	1.142 cfs/ft.	1.142 cfs/ft.	1.142 cfs/ft.	0.345 cfs/ft.	0.345 cfs/ft.
TWL @ 2100 Sea Level High Range Projection	22.31 ft. NAVD status = flooded	22.31 ft. NAVD status = flooded	22.31 ft. NAVD status = flooded	22.31 ft. NAVD status = flooded	22.31 ft. NAVD status = flooded
Q' 2100 Sea Level High Range Projection	5.691 cfs/ft.	5.691 cfs/ft.	5.691 cfs/ft.	4.162 cfs/ft.	4.162 cfs/ft.

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

Table ES-2b: Tsunami Total Water Level (*TWL*) and Overtopping Rates (Q') Analysis at the Capistrano Beach Site

	Well Head-G Elevation = 18 ft. NAVD eroded/accreted	Well Head-H Elevation = 19 ft. NAVD eroded/accreted
*TWL Present Sea Level (eroded)	15.3 ft. NAVD status = dry	15.3 ft. NAVD status = dry
*Q' Present Sea Level (eroded)	0.0/0.0 cfs/ft.	0.0/0.0 cfs/ft.
*TWL 2100 Sea Level Low Range Projection (eroded)	18.83 ft. NAVD status = flooded	18.83 ft. NAVD status = dry
*Q' 2100 Sea Level Low Range Projection (eroded)	0.352 cfs/ft.	0.0/0.0 cfs/ft.
*TWL 2100 Sea Level High Range Projection (eroded)	22.4 ft. NAVD status = flooded	22.4 ft. NAVD status = flooded
*Q' 2100 Sea Level High Range Projection (eroded)	4.293 cfs/ft.	2.916 cfs/ft.

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

9) Summary and Conclusions:

This 2019 study, prepared in response to comments for the Final EIR, provides further analysis to amplify the Coastal Hazards Analysis prepared in 2017 for the Draft EIR of the Doheny Desalination Project. That earlier work is being amplified herein in response to a revision of the *California Coastal Commission Sea Level Rise Policy Guidance* document that was originally released in August 2015, (CCC, 2015), but has been updated in July 2018 with new sea level rise projections. In addition, there have been minor adjustments in the locations of a number of the well heads and pump stations being proposed for the Doheny Desalination Project. The following study accounts for these intervening changes in policy guidance and minor modifications to the project description.

The primary analysis tool used in this study is the *Coastal Evolution Model* (CEM) developed at the Scripps Institution of Oceanography was used to evaluate Appendix-B requirements of the *California Coastal Commission 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 grain 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 Oceanic and Atmospheric 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 Commission 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 or tsunami concurrent with extreme ocean water levels and sea level rise are placed at two sites, namely Doheny State Beach and Capistrano State Beach. At the Doheny Beach site, five potential locations are being evaluated for vaulted well heads with submersible pumps, including : ***Well Head A***, elevation 17 ft. NAVD, at 33°27'44.38"N, 117°41'16.32"W; ***Well Head B***, elevation 17 ft. NAVD, at 33°27'45.07"N, 117°41'10.30"W; ***Well Head C***, elevation 17 ft. NAVD at 33°27'45.12"N, 117°41'6.62"W; ***Well Head D***, at elevation 18 ft. NAVD at 33°27'44.48"N, 117°40'55.30"W; and ***Well Head E***, at elevation 18 ft. NAVD at 33°27'42.45"N, 117°40'47.33"W. Two additional vaulted well heads with submersible pumps are being evaluated at the Capistrano Beach site, which includes: ***Well Head G***, at elevation 18 ft. NAVD at 33°27'14.94"N, 117°39'59.91"W; and ***Well Head H***, at elevation 19 ft. NAVD at 33°27'13.17"N, 117°39'57.15"W.

This study is based on sea level rise projections appearing in Appendix-G, Table G-11, of the recently updated *California Coastal Commission Sea Level Rise Policy Guidance* document (CCC, 2018). This document 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 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 14 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 at present and at year 2100 sea levels, is summarized in Table 7.1 for structures at the Doheny Beach site and Table 7.2 for the Capistrano Beach site. Inspection of Table 7.1 & 7.2 reveals that all the beach front well sites for the Doheny Desalination Project are safe from flooding or inundation at present sea levels by extreme event waves concurrent with extreme ocean water levels for event return periods between 1 yr. and 100 yr. However, once we admit to 2100 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 low-range 2100 sea level projections, the three well sites on the north side of San Juan Creek (Well Heads A-C) and one of the wells at the Capistrano Beach site (Well Head G) will experience minor overtopping, even for a 1 year event if the beaches have been accreted by additional sands from water shed floods or still retain a built-out summer equilibrium beach profile, with overtopping rates of about Q'(1yr) = 0.038 cfs per lineal ft. of shoreline. However, if a 100-yr total water level event occurs during the low-range projection of 2100 sea levels, then all of the well sites will be overtopped to varying degrees if the beaches remain in an accreted condition with elevated berms and steep beach slopes. Under these beach conditions, overtopping rates will range from a high of Q'(100yr) = 0.094 cfs per lineal ft. of shoreline at Well Heads A- C, to a low of Q'(100yr) = 0.014 cfs/ft at Well Head H. Interestingly enough, none of the well heads would experience overtopping during a 100 year event when occurring during the low range 2100 sea levels if the beach were eroded, which would be the most likely condition during a 100-year event. 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 at the Doheny Beach site reach TWL(100) = 21.9 ft. NAVD for the steeply sloping accreted beach conditions and TWL(100) = 20.2 ft. NAVD for the eroded beach conditions. At the Capistrano Beach site, shoaling wave heights are higher and total water levels for a 100 year event superimposed on the high range projections for 2100 sea levels produce total water levels reaching TWL(100) = 22.7 ft. NAVD for the steeply sloping accreted beach conditions and TWL(100) = 21.1 ft. NAVD for the eroded beach conditions. Consequently, all beach front well heads for the Doheny Desalination Project will be overtopped and flooded when extreme waves happen concurrently with extreme ocean water levels that are superimposed on the high range of 2100 sea levels. The lowest lying well heads (Well Heads A-C) would experience the highest overtopping rates, ranging from Q'(100yr) = 0.216 cfs/ft. to 0.331 cfs/ft. depending on the eroded or accreted condition of Doheny State Beach. According Table VI-5-6 in the Coastal Engineering Manual (USACE, 2006) overtopping rates of this order of magnitude are very dangerous for pedestrian and vehicle traffic, and may cause structural damage to adjacent buildings, but the well heads and pumps for the Doheny Desalination project will be protected by steel vault enclosures. The smallest overtopping rates during the 100-year event at the high range2100 sea level projections will occur at the highest located well head (Well Head H) at the Capistrano Beach site where overtopping rates will range from Q'(100yr) = 0.142 cfs/ft. to 0.250 cfs/ft. While these overtopping rates are still dangerous to pedestrian and vehicle traffic, they are easily mitigated by the steel vault enclosures of the well heads and pumps being placed at Capistrano Beach.

Tsunami induced erosion, runup, and inundation were analyzed for the Doheny and Capistrano State Beaches 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 analysis was based on numerical refraction/diffraction codes for a shoaling solitary wave. 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 and Capistrano Beaches. 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 Table 8.1 for the Doheny Beach site, and Table 8.2 for the Capistrano Beach site. These tables indicate that all of the shore facilities of the Doheny Desalination Project are above tsunami inundation levels at present sea level. However, all of the well heads at both Doheny and Capistrano Beaches would suffer some degree of tsunami overtopping if concurrent with 2100 sea levels, and the overtopping rates could be quite severe, especially for the high 210 sea level rise projections. At the low range of 2100 sea level projections, total water levels would reach TWL = 18.82 ft. NAVD at Doheny Beach and TWL = 18.83 ft. NAVD at Capistrano Beach. Well Heads A-C at Doheny Beach would experience the highest overtopping surges of Q' = 1.142 cfs/ft while Well Head G at Capistrano Beach would remain high and dry. However, if the tsunami occurred atop the high range sea level rise projections for year 2100, then total water levels would reach TWL = 22.31 ft. NAVD at Doheny Beach and TWL = 22.4 ft. NAVD at Capistrano Beach, sufficient to overtop all the well sites of the Doheny Desalination project. In this case the tsunami surge could produce very high, although short-lived, overtopping rates reaching a maximum of Q' = 5.691 cfs/ft at Well Heads A-C on Doheny Beach and a minimum of Q' = 2.916 cfs/ft at Well Head H on Capistrano Beach. Undoubtedly, the steel vault enclosures of the well heads can be designed to withstand these high surge rates, but particular attention should be given to the foundations of the vaults to assure those foundations have adequate depth to prevent undercutting by scour. 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 inundated by a shoaling tsunami solitary wave.

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

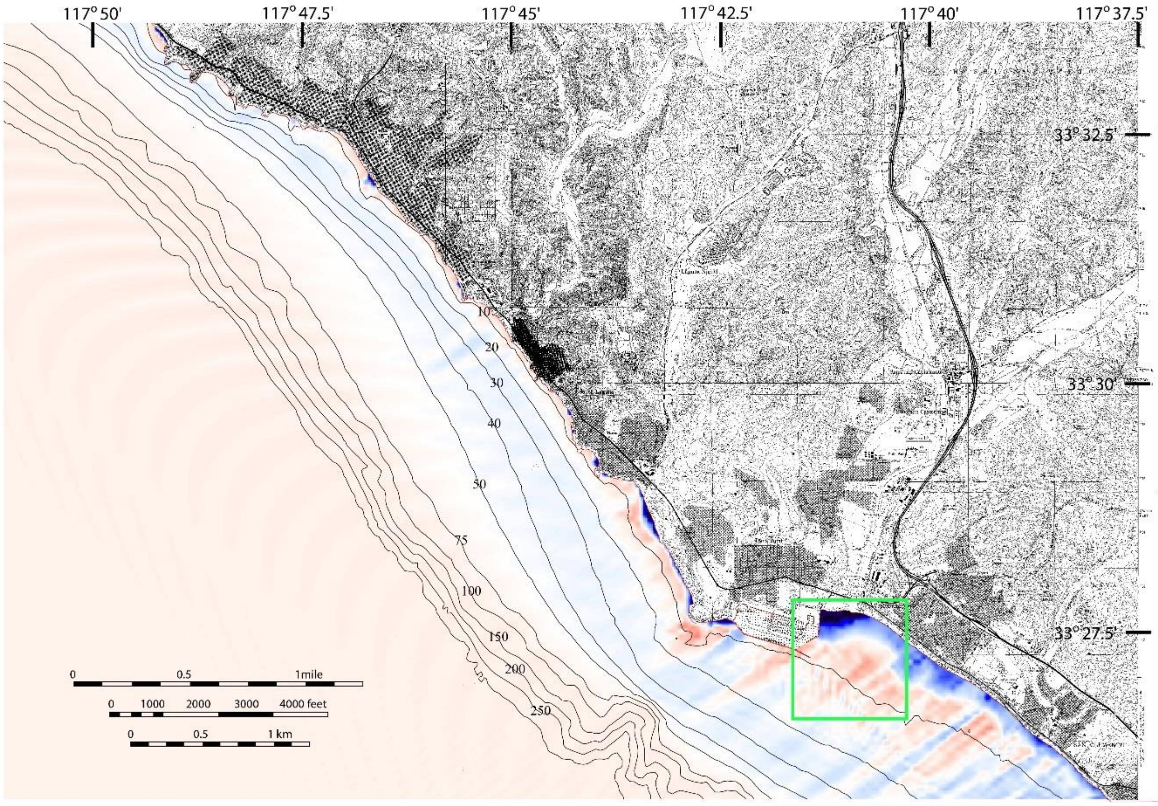


Figure . Dana Point wave refraction for the 16 February 1980 storm
Wave height at offshore control point = 3.5 m, T=10.5 sec, from 220.5°.

0 1 2 3 4 5 6 7 meters

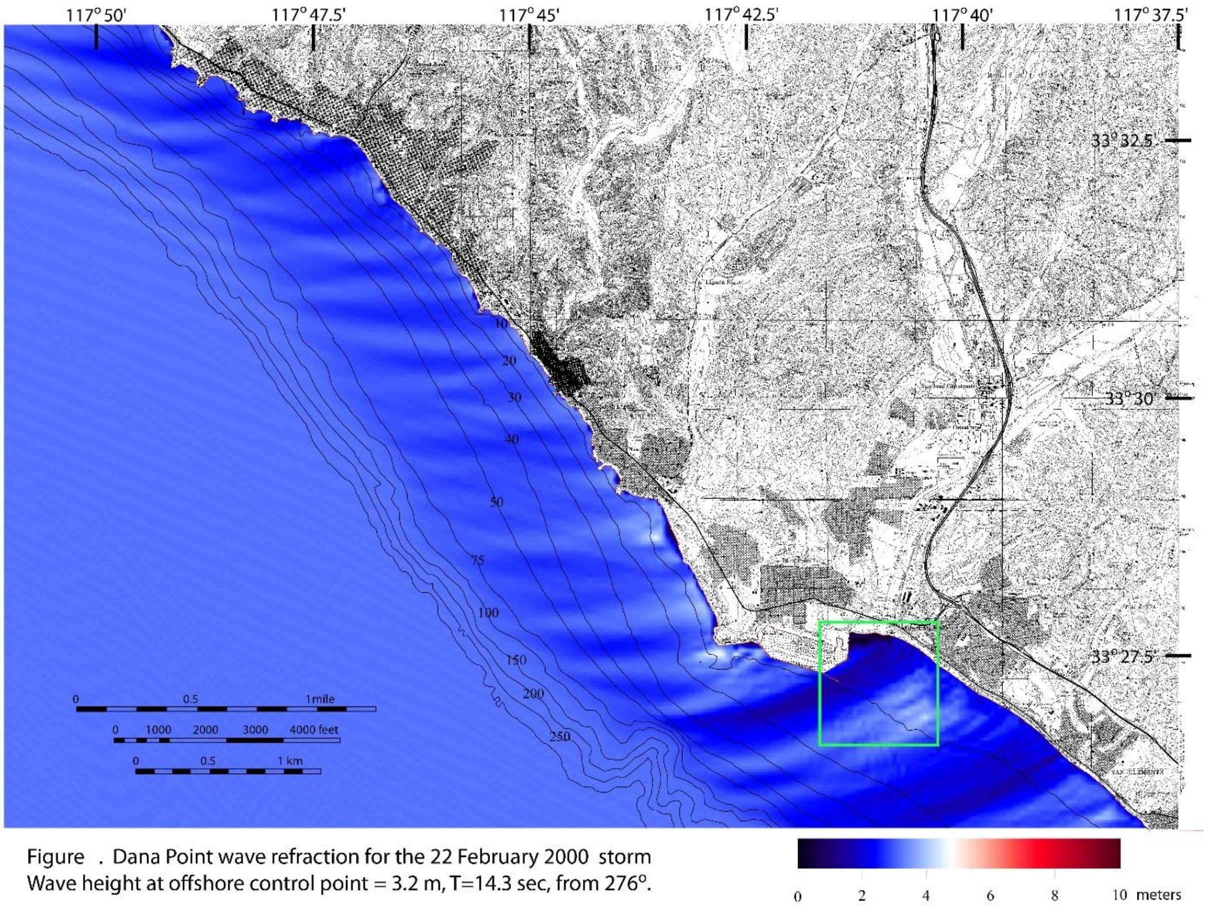


Figure . Dana Point wave refraction for the 22 February 2000 storm
Wave height at offshore control point = 3.2 m, T=14.3 sec, from 276°.

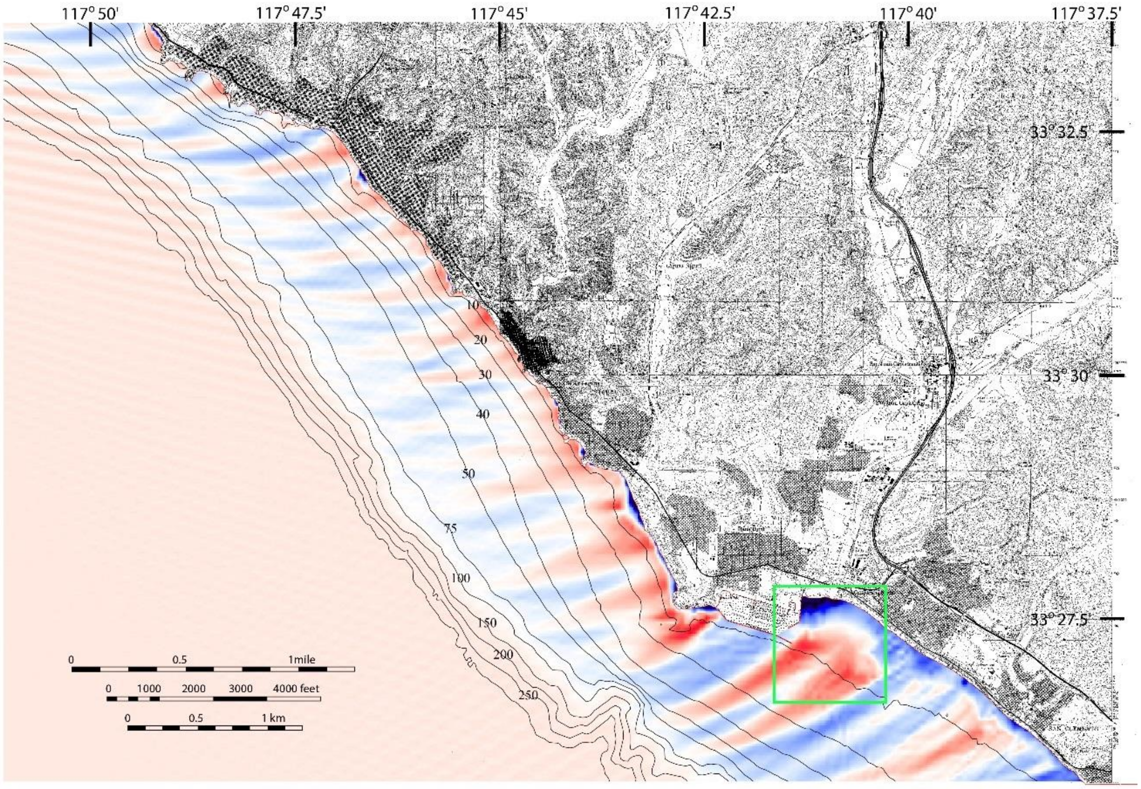


Figure . Dana Point wave refraction for the 28 January 1981 storm
Wave height at offshore control point = 3.5 m, T=15.5 sec, from 251.5°.

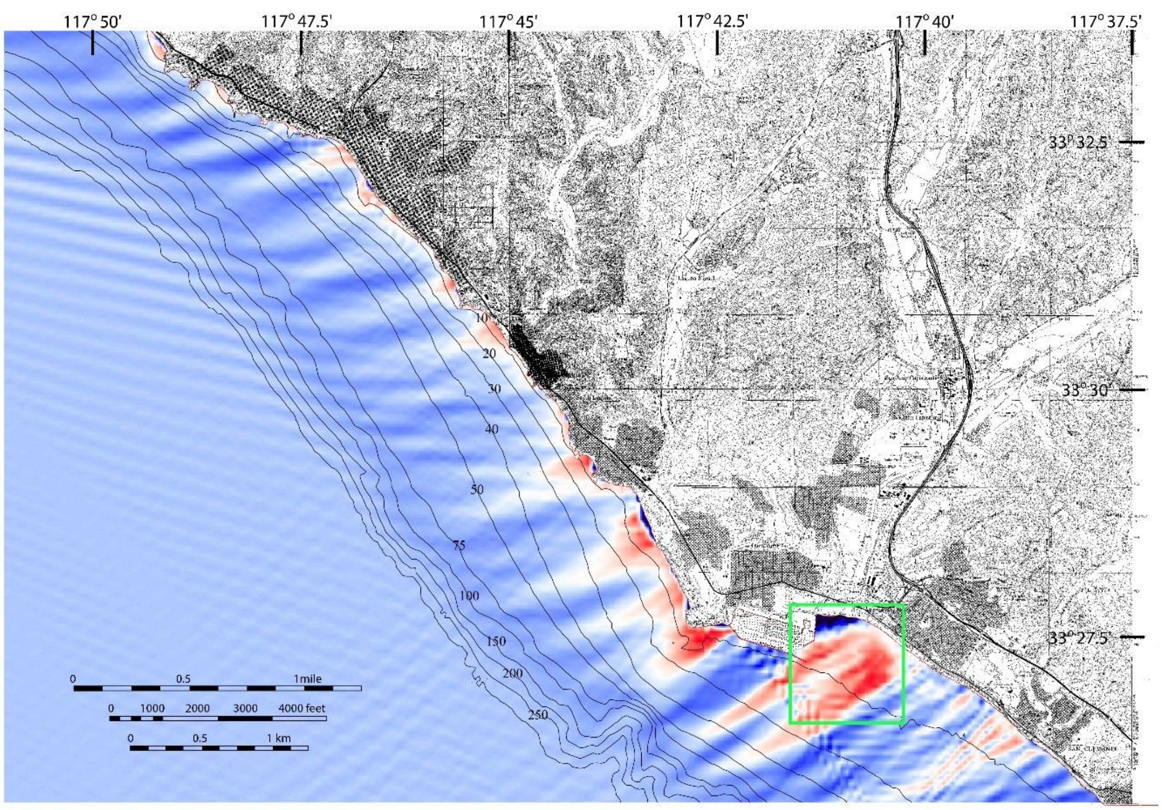
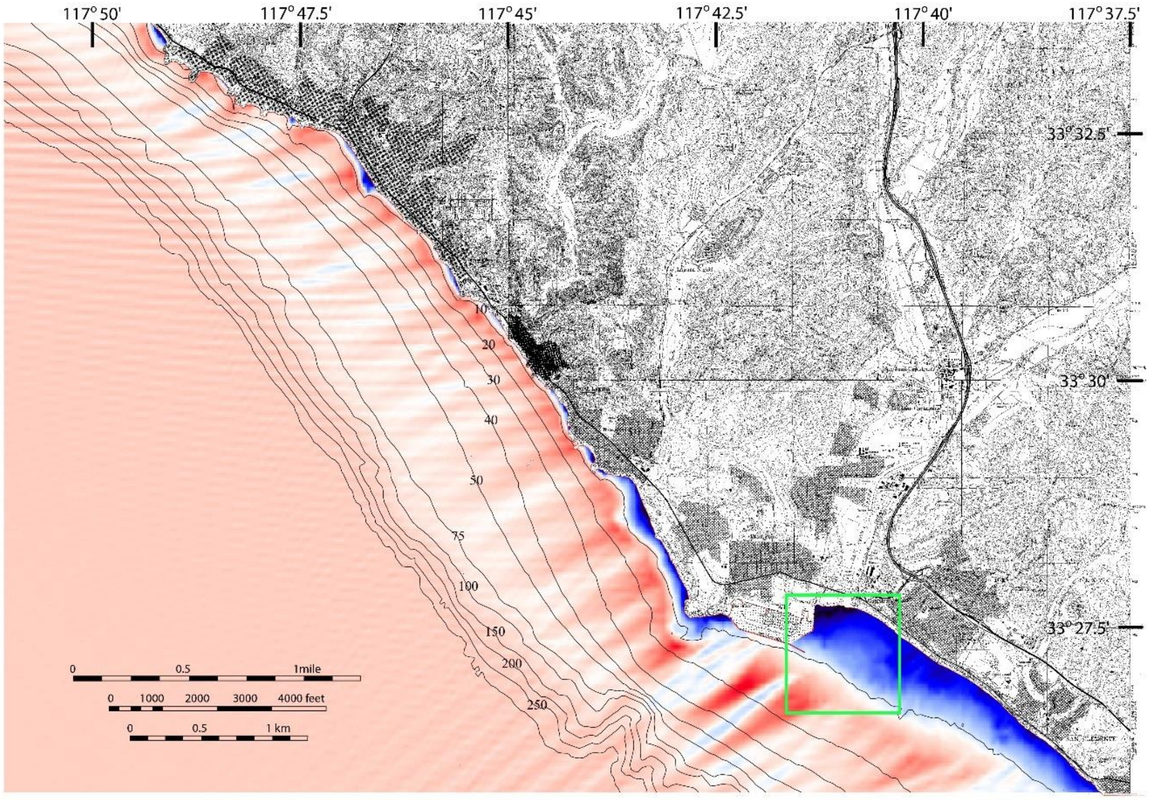


Figure . Dana Point wave refraction for the 25 January 1983 storm
Wave height at offshore control point = 2.1 m, T=19.5 sec, from 246°.



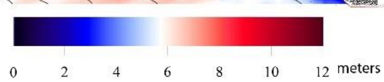


Figure . Dana Point wave refraction for the 1 March 1983 storm
Wave height at offshore control point = 6.5 m, T=15.5 sec, from 223.5°.

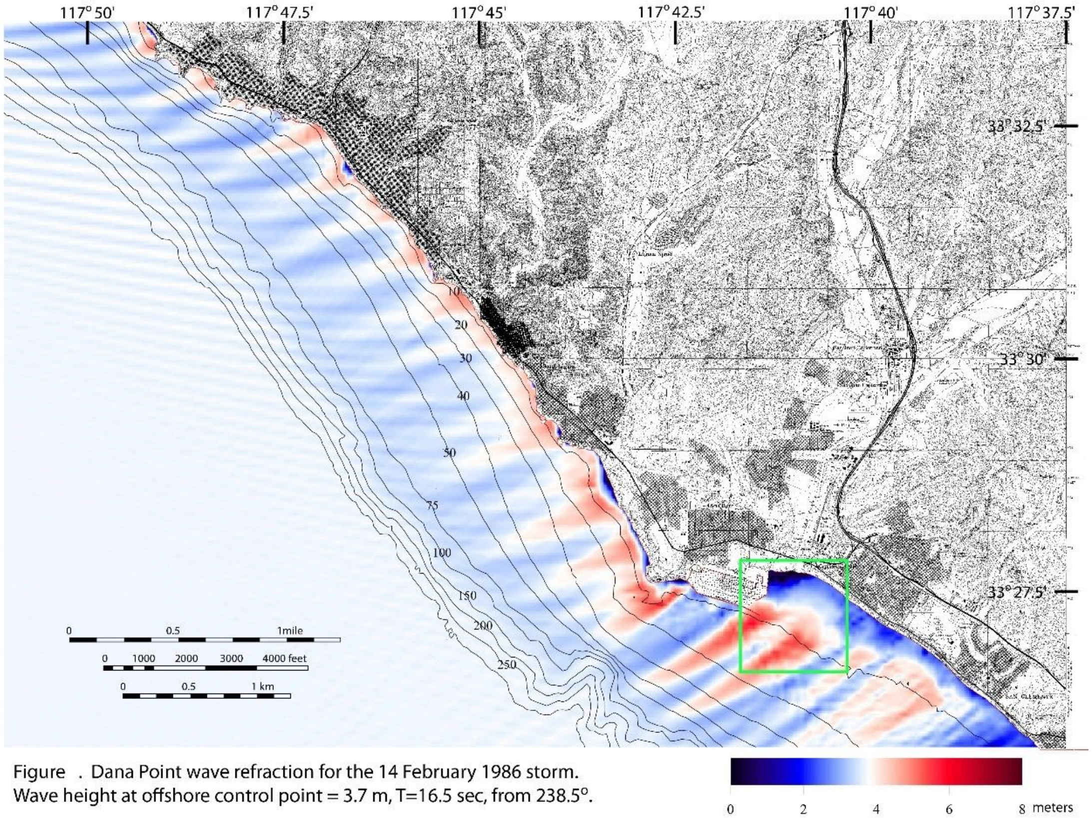


Figure . Dana Point wave refraction for the 14 February 1986 storm.
Wave height at offshore control point = 3.7 m, T=16.5 sec, from 238.5°.

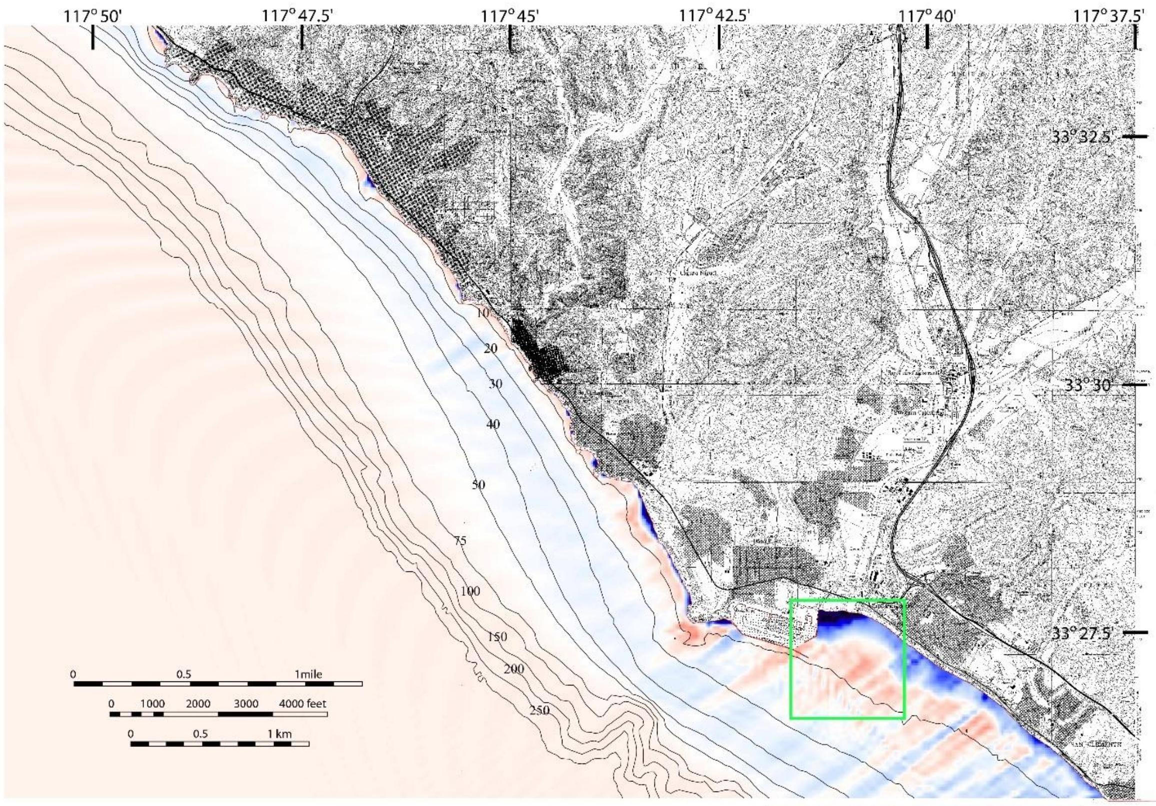


Figure . Dana Point wave refraction for the 16 February 1980 storm
Wave height at offshore control point = 3.5 m, T=10.5 sec, from 220.5°.

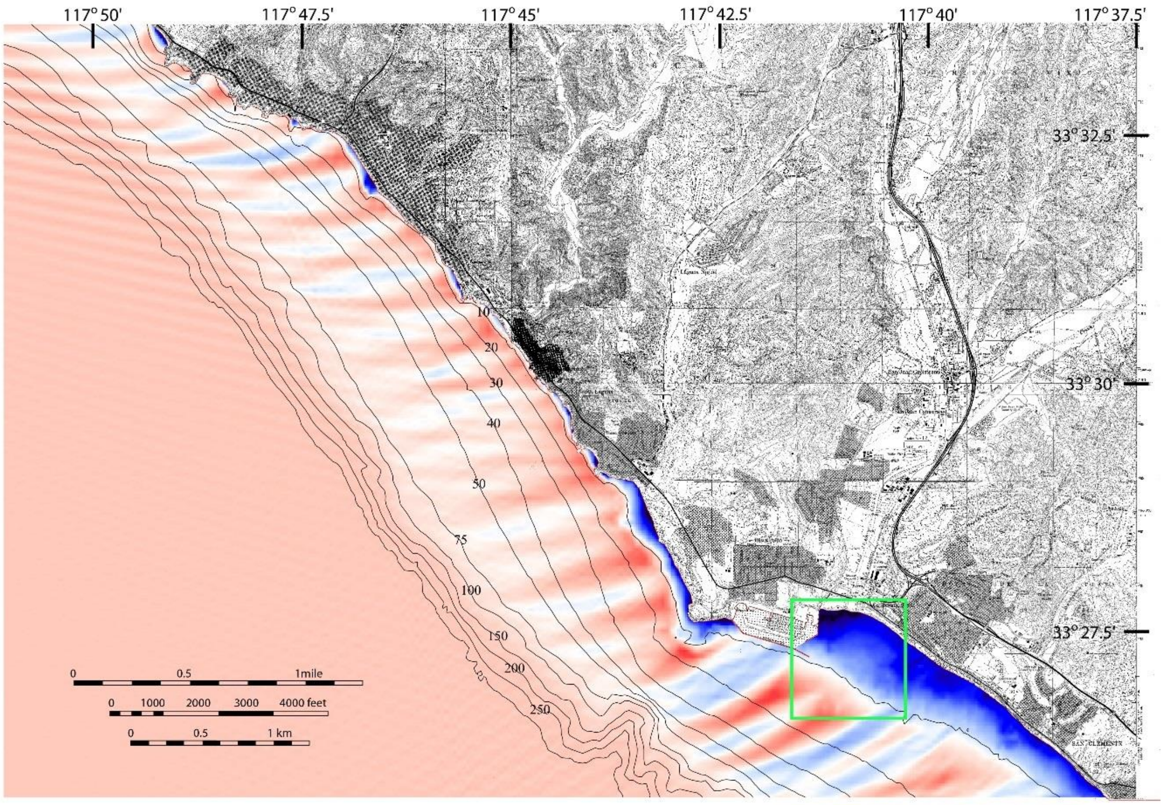


Figure . Dana Point wave refraction for the 17 January 1988 extreme event storm. Wave height at offshore control point = 6.6 m, T=15.5 sec, from 251.5°.

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

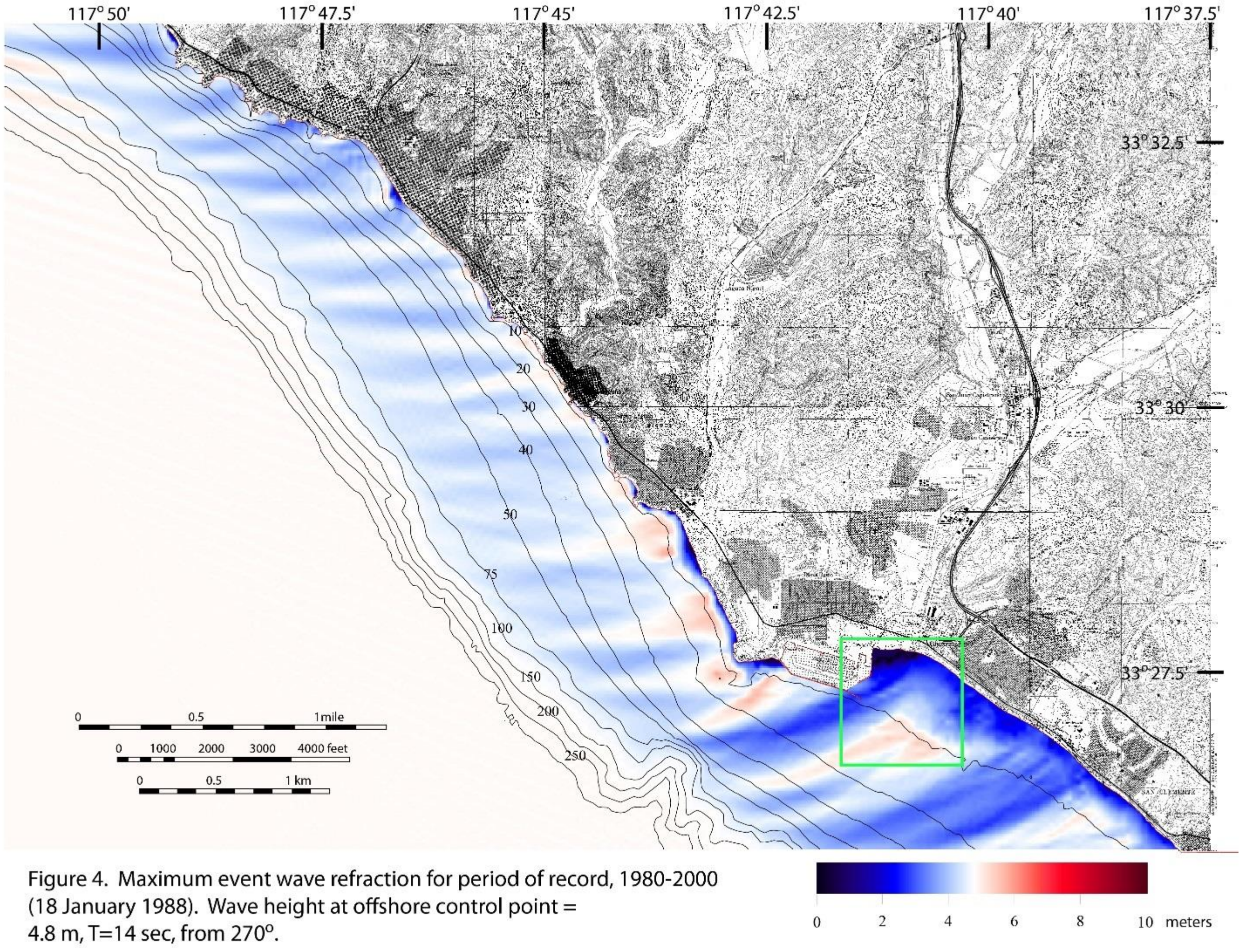


Figure 4. Maximum event wave refraction for period of record, 1980-2000 (18 January 1988). Wave height at offshore control point = 4.8 m, T=14 sec, from 270°.

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

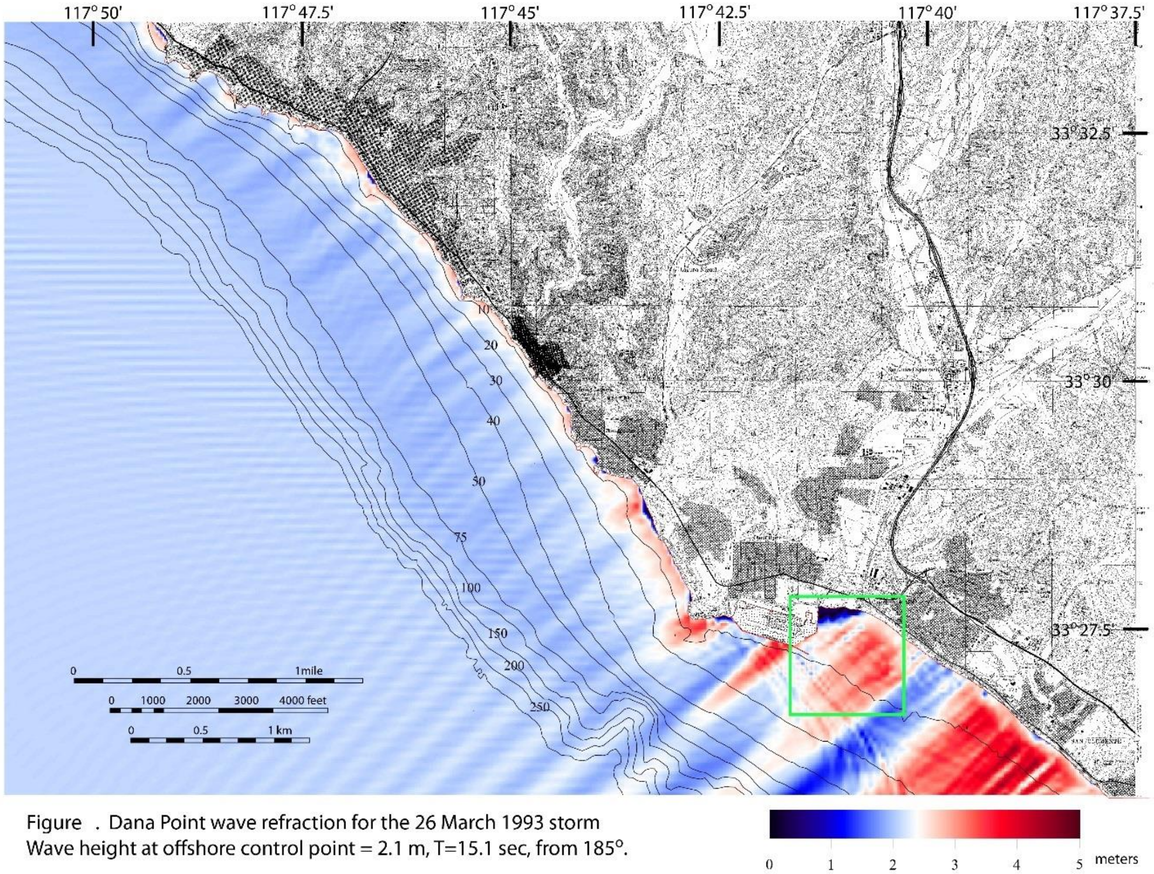


Figure . Dana Point wave refraction for the 26 March 1993 storm
Wave height at offshore control point = 2.1 m, T=15.1 sec, from 185°.

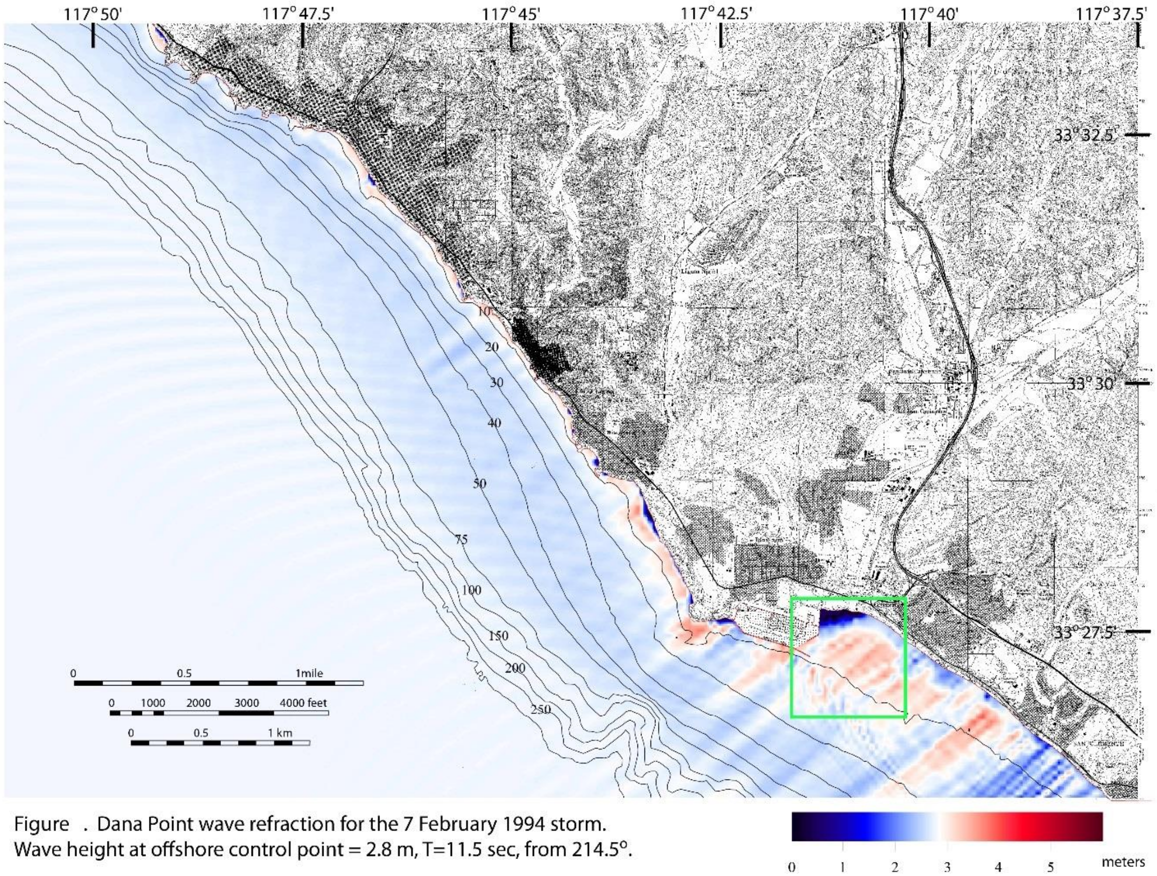


Figure . Dana Point wave refraction for the 7 February 1994 storm.
Wave height at offshore control point = 2.8 m, T=11.5 sec, from 214.5°.

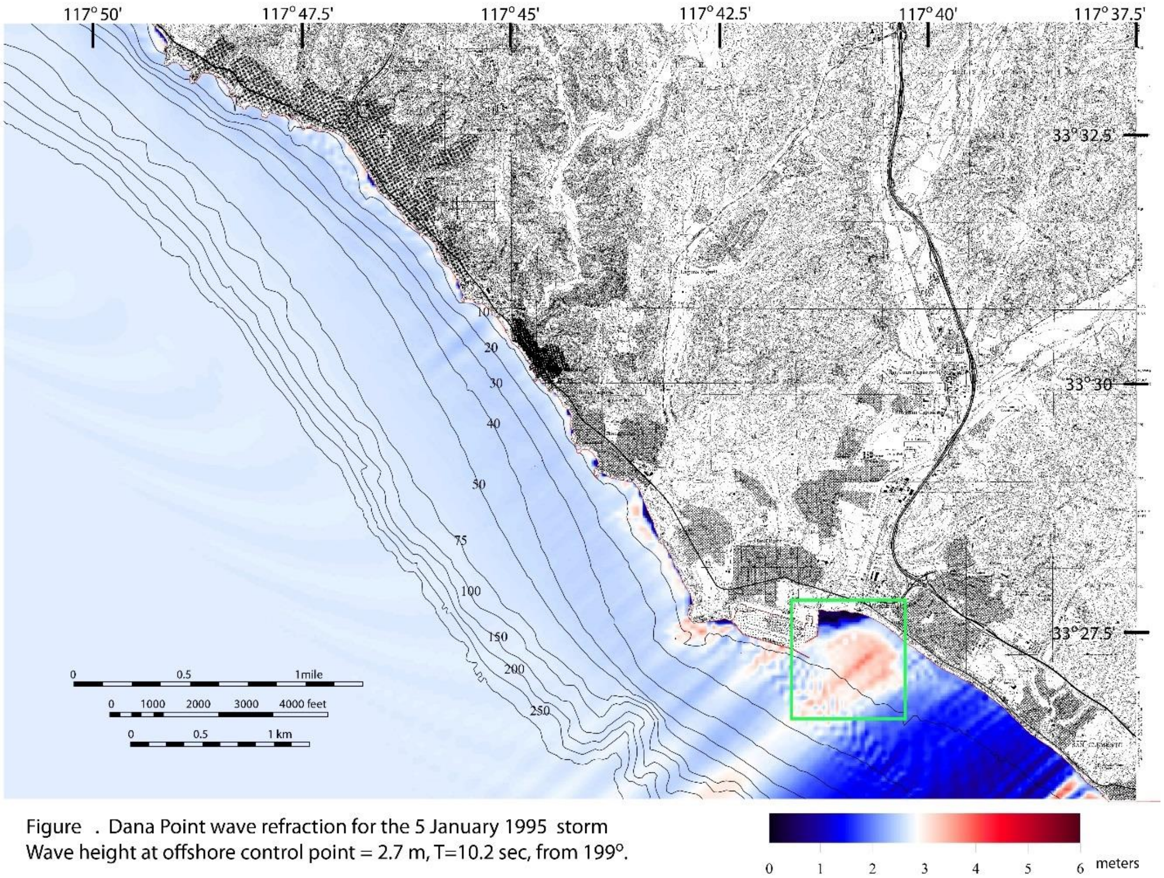


Figure . Dana Point wave refraction for the 5 January 1995 storm
Wave height at offshore control point = 2.7 m, T=10.2 sec, from 199°.

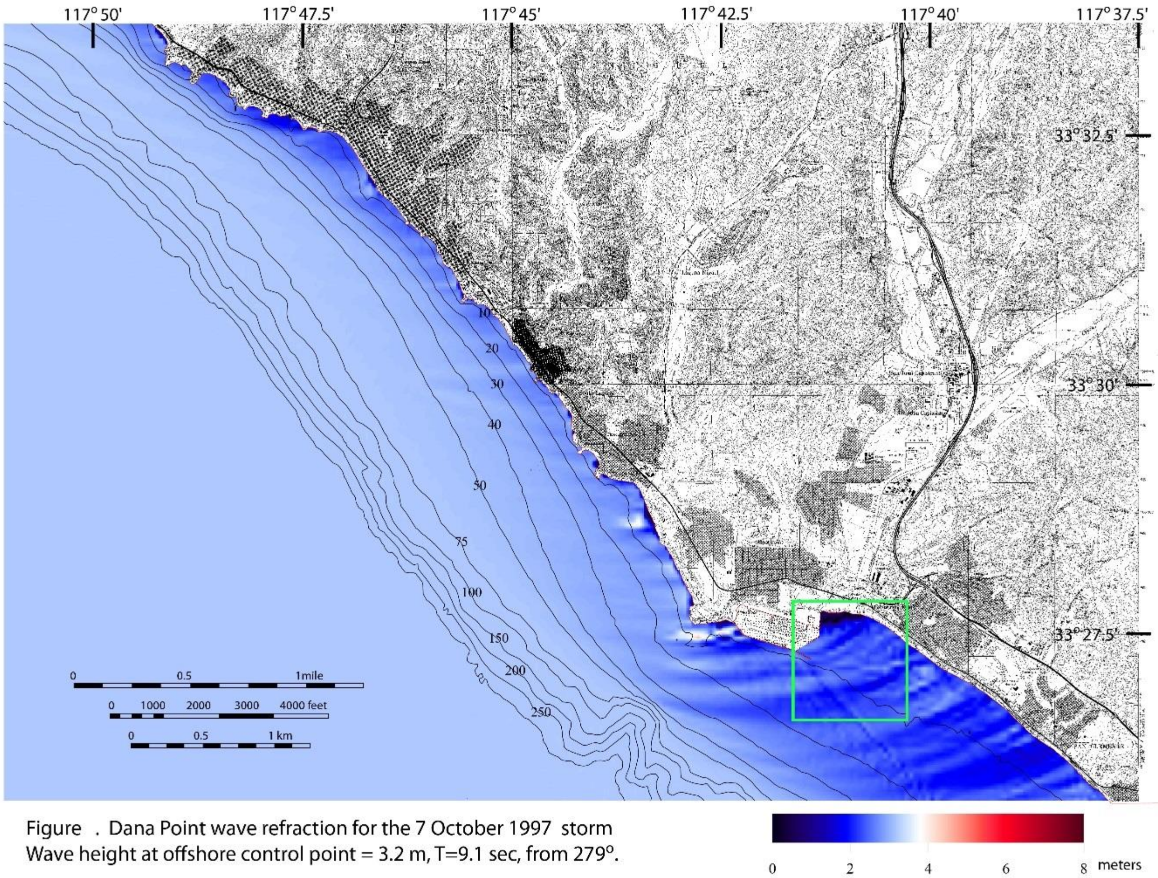


Figure . Dana Point wave refraction for the 7 October 1997 storm
Wave height at offshore control point = 3.2 m, T=9.1 sec, from 279°.

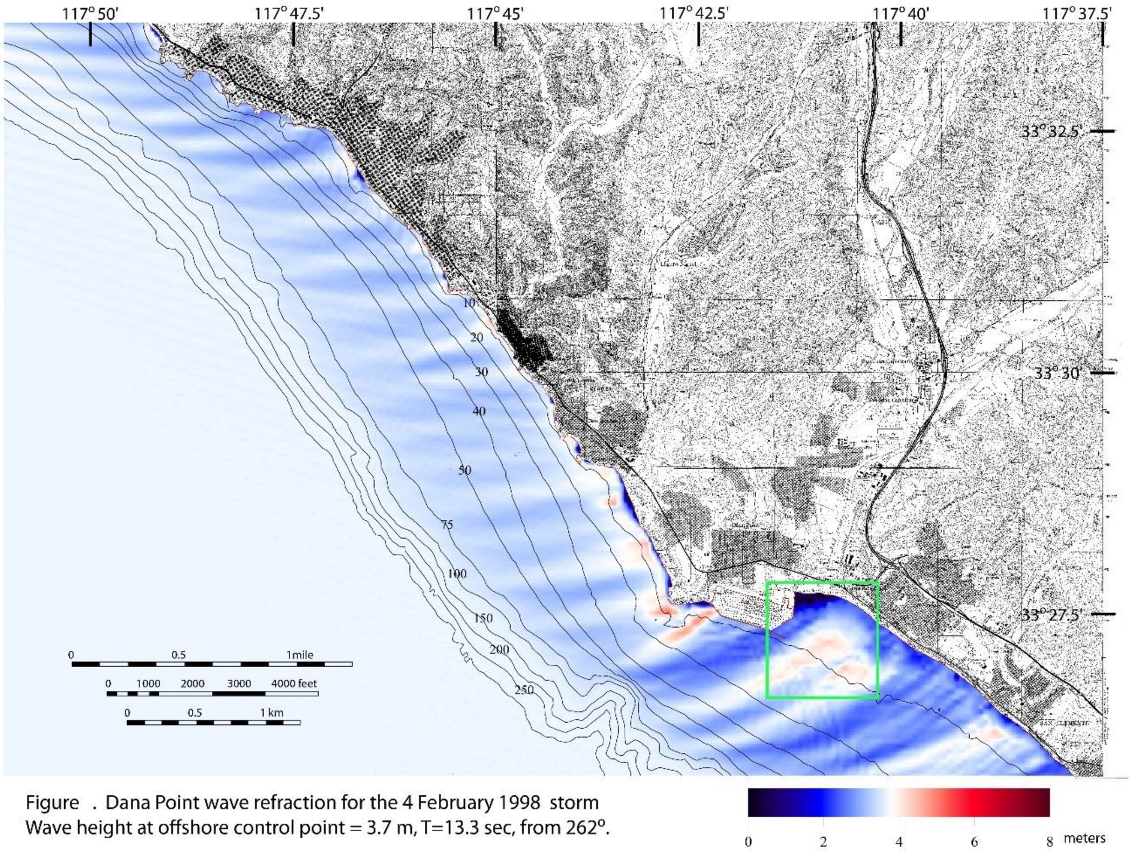


Figure . Dana Point wave refraction for the 4 February 1998 storm
Wave height at offshore control point = 3.7 m, T=13.3 sec, from 262°.

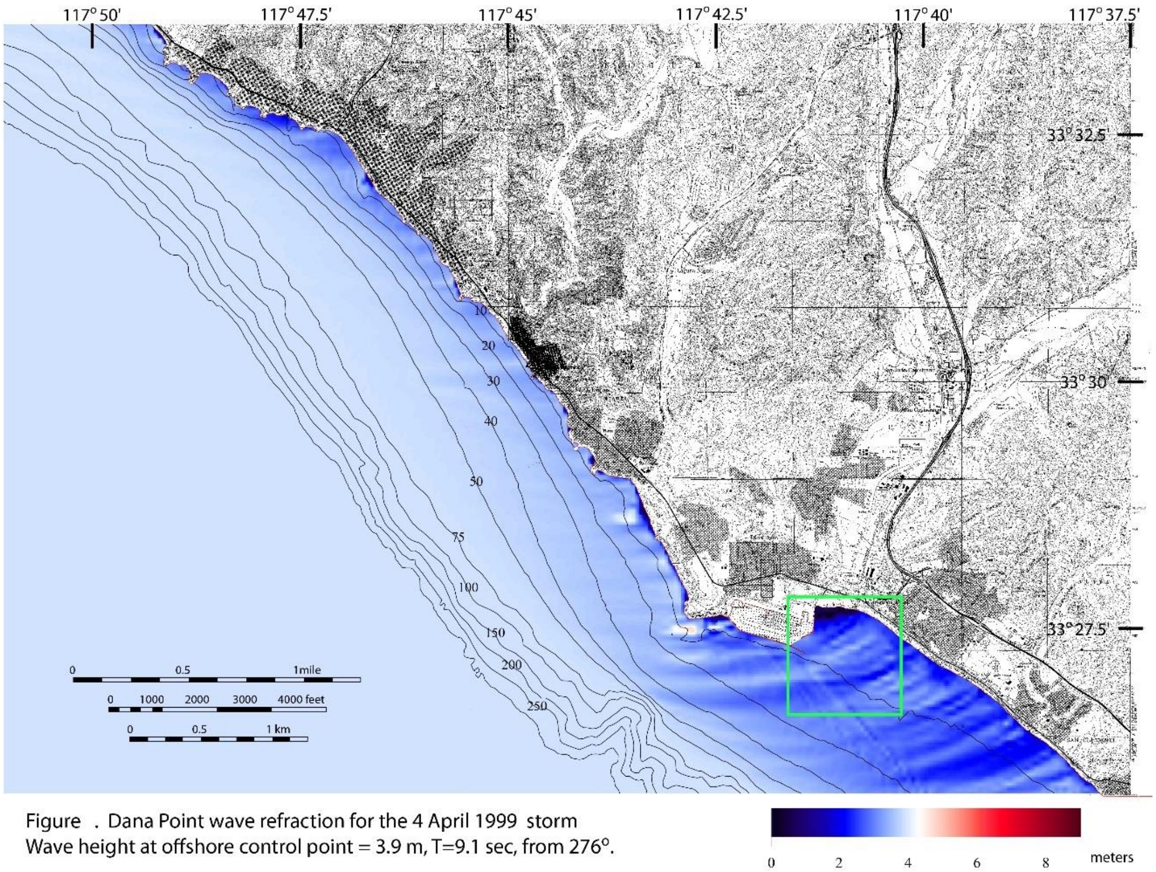


Figure . Dana Point wave refraction for the 4 April 1999 storm
Wave height at offshore control point = 3.9 m, T=9.1 sec, from 276°.

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.

APPENDIX 4.2.5

MARINE BIOLOGY TECHNICAL MEMOS

APPENDIX 4.2.5.1

DIFFUSER ENTRAINMENT MEMO FOR FINAL EIR

6 March 2019

Mark Donovan, PE
GHD
Desalination Program Manager
320 Goddard Way, Suite 200
Irvine CA 92618
mark.donovan@ghd.com

Re: Review of Plumes 18b Modeling Deleterious Diffuser Entrainment
Doheny Desalination Project

Hello Mark:

I reviewed the Plumes 18b report (Jenkins 2019) for the South Coast Water District Doheny Desalination Plant (DDP) and the following are my thoughts.

The results presented in Jenkins Table 3 (buoyant discharge scenarios) and Table 5 (non-buoyant discharge scenarios) include: depth of maximum plume rise, distance to maximum plume rise, volume of water with deleterious entrainment (entrainment mortality), the incremental change in the volume compared to baseline, diameter of the zone of initial dilution (ZID), and the incremental change in ZID diameter.

The buoyant discharge scenarios (Jenkins Table 3) all result in reduced entrainment mortality and smaller ZIDs. Therefore, Jenkins posits that "*no mitigation should be required for DDP (Doheny Desalination Plant) operational scenarios that result in buoyant combined discharges with SOCWA wastewater*."

Table 1. Modeling Summary (note both feet and meters are reported)

	Distance to maximum rise (ft)	Deleterious entrainment volume at maximum rise (mgd)	Incremental reduction in diffuser entrainment (mgd)	ZID diameter (m)	Incremental reduction in ZID diameter (m)
Buoyant scenarios	48.5–68.9	1,702–6,992	33–1,615	63–196	5–188
Non-buoyant scenarios	<1–20.1	67–729	N/A	N/A	N/A

The non-buoyant discharge scenarios (Jenkins Table 5) result in entrainment mortality volumes that range from 67 to 729 million gallons per day (mgd). These volumes "*are to be throughput to the ETM/APF (Empirical Transport Model/Area Production Foregone) calculus to compute the mitigation scaling for DDP diffuser turbulent impact*".

For simplicity, I'll refer to the "volume of deleterious entrainment" as "TM (turbulence mortality) volume". The volumes calculated above are high relative to the actual discharge volumes at the outfall. For example, the baseline discharge of 8 mgd of wastewater results in TM volume at maximum rise of 3,004 mgd, or 376 times the discharge rate. For comparison, when San Onofre Nuclear Generating Station was operational, each unit discharged approximately 1,200 mgd of cooling water (2,400 mgd total for Units 2 and 3).

If ETM/APF is the required approach, the required denominator for proportional entrainment is the source water volume. The APF estimates that we presented in the Draft EIR (Appendix 10.4.1) were based on an estimated source water with dimensions 2 km cross-shore, 25.9 km longshore, and 20 m deep. The longshore distance was based on a current speed of 6 cm/sec. The total source water volume was estimated at approximately 54,779 x 10^6 gallons.

The ETM/APF approach has been used for power plants and desalination facilities, and the focus has been fish eggs and larvae, and target meroplankton such as crabs, squid, and spiny lobster. We have not sampled plankton in the nearshore waters of Dana Point, but performed a year-long plankton study off San Onofre in 2006-7 (MBC 2007). The most abundant fish larvae during studies at San Onofre were Northern Anchovy (*Engraulis mordax*), unidentified anchovies (*Engraulidae*), Queenfish (*Seriphus politus*), and clinid kelpfish (*Gibbonsia* spp). The most abundant fish eggs were Engraulid eggs, unidentified fish eggs, and Sciaenid/Paralichthyid/Labrid eggs. The table below summarizes known egg sizes and hatching lengths for relevant taxa (from Moser 1996), and the percent contribution of each taxon to the egg/larval total in entrainment samples (MBC 2007).

Table 2. Sizes of fish eggs and larvae, and contribution to totals off San Onofre.

Species	Egg diameter (mm)	% contribution to egg total	Hatch length (mm)	% contribution to larvae total
Northern Anchovy	1.23–1.55*	42.8[†]	2.5–3.0	38.5
Deepbody Anchovy	0.6–0.9		1.5–2.5	20.3[††]
Queenfish	0.73–0.78	1.3[‡]	~1.6	5.9
Spotted Kelpfish			4.5	5.8[‡‡]
Giant Kelpfish	1.2–1.4		5.1–6.2	0.6

* Eggs of N. Anchovy are elongate. These are the lengths in the longest dimension.
† Engraulid eggs include N. Anchovy (*E. mordax*) and Deepbody Anchovy (*A. compressa*).
†† Engraulid larvae (unidentifiable to species).
‡ Sciaenid eggs (unidentifiable to species).
‡‡ *Gibbonsia* spp larvae (unidentifiable to species).

In the case of the DDP, it is assumed that TM is limited to organisms <1 mm in size, which would exclude fish larvae and some fish eggs.

This preliminary analysis uses the same assumptions described above, but depth is now considered to be 31 m (centered on the diffuser section). Because the focus is now on organisms <1 mm, we are now analyzing zooplankton instead of fish eggs and larvae (we assume phytoplankton are not of concern). Zooplankton are distributed throughout the water column and migrate vertically to various degrees (Mullin 1986). Zooplankton can be divided into microzooplankton (smaller than ~300 μm) and macrozooplankton (larger than ~300 μm). Microzooplankton feed on particulate organic sources, and consist of protozoans and juvenile stages of metazoan plankton, such as copepod nauplii and early copepodites (Dawson and Pieper 1993). Macrozooplankton include organisms such as gelatinous zooplankton, chaetognaths, copepods, cladocerans, and ostracods.

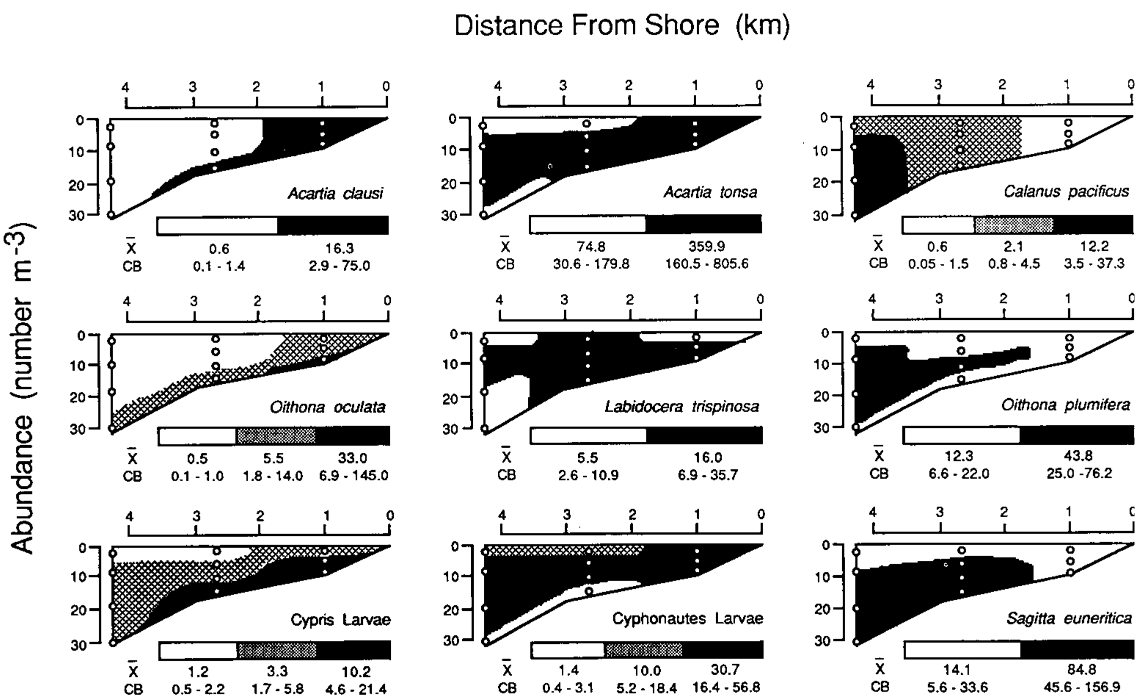


Figure 1. Mean cross-shore abundance profiles of nine zooplankton taxa off San Onofre, 1976 to 1980. From Barnett and Jahn (1987).

As mentioned earlier, the buoyant discharge scenarios all result in smaller TM volumes. The following estimates were calculated for the non-buoyant discharge scenarios, and are based on a source water volume of 87,235 x 10^6 gallons and a larval duration of one day. Note that these are not based on any empirical biological data, and we have no data to characterize spatial or temporal patterns of zooplankton abundance in the immediate project area. This data will ultimately be required for preparation of the Marine Life Mortality Report (per the Ocean Plan, III.M.2.e.1.a [Marine Life Mortality Report]).

Table 3. Probability of Mortality (P_M) and Area Production Foregone (APF) Estimates for DDP based on TM volumes from Jenkins (2019) and a larval duration of one day. The P_M and APF estimates were calculated using the TM volume at the bottom hit of the plume, which was larger than the maximum rise volume for both scenarios.

Non-buoyant Scenario	Wastewater + brine discharge rates (mgd)	TM volume (mgd) at maximum rise of plume	TM Volume (mgd) at bottom hit of plume	Probability of Mortality (P_M)	Area Production Foregone (acres)
1	0 + 3	78.09	120.78	0.00138	3.54
3	0 + 5	95.4	189.7	0.00217	5.57

Potential effects to zooplankton from entrainment in power plants and desalination facilities have not been analyzed in recent studies in southern California. The California Energy Commission published several reports that attempted to summarize standard collection and analysis methods for power plant entrainment studies. The report Assessing Power Plant Cooling Water Intake System Entrainment Impacts, Steinbeck et al (2007) determined:

"*Entrainment affects all types of planktonic organisms, but most studies do not assess holoplankton (phytoplankton and zooplankton that are planktonic for their entire life) because their broad geographic distributions and short generation times reduce the effects of entrainment on their populations. In contrast, the potential for localized effects on certain fish populations is much greater, especially for power plants located in riverine or estuarine areas where a large percentage of the local population may be at risk of entrainment (Barnthouse et al. 1988, Barnthouse 2000). Although the potential for similar effects exists for certain invertebrate meroplankton (for example, crab and clam larvae), taxonomy of early larval stages of many invertebrates is not sufficiently advanced to allow for assessments at the species level.*"

EPRI (2007) summarized analysis of plankton for the SWRCB as follows:

"*The entrainment performance standard for entrainment reduction in the EPA Rule focuses on addressing impacts to fish and shellfish rather than lower tropic levels such as phytoplankton and zooplankton. There are several reasons why there is a low potential for impacts to phytoplankton and zooplankton and why it made sense for EPA to focus on effects on fish and shellfish. EPA recognized the low vulnerability of phytoplankton and zooplankton in its 1977 draft §316(b) guidance (USEPA 1977). The reasons include the following*:

- *The extremely short generation times—on the order of a few hours to a few days for phytoplankton and a few days to a few weeks for zooplankton;*

- *Both phytoplankton and zooplankton have the capability to reproduce continually depending on environmental conditions; and*

- *The most abundant phytoplankton and zooplankton species along the California coast have populations that span the entire Pacific, or in some cases all of the world's oceans. For example, Acartia tonsa, one of the common copepod species found in the nearshore areas of California has a distribution that includes the Atlantic and Pacific coasts of North and South America and the Indian Ocean*."

From the CalAm Draft EIR (ESA 2017):

"*The minimum and maximum discharge velocities (7.4 ft/sec (2.26 m/sec) and 14.8 ft/sec (4.51 m/sec)) modeled across all scenarios for the proposed MPWSP (see Appendix D1) closely approximate the discharge velocities calculated by Foster. Foster (2013) concludes that, at these very small eddy scales: "Overall, the area of high shear impacted by the diffusers is relatively small and transit times through the region short. Thus, it seems reasonable to expect that, while the larvae that experience the highest shear will most likely experience lethal damage, the overall increase in mortality integrated over the larger area will be low.*"

When the environmental effects for San Onofre Nuclear Generating Station were being evaluated in the 1980s, the Marine Review Committee determined intake losses of zooplankton of about 1,350 metric tons per year were not a substantial adverse effect, so no mitigation was required (Ambrose et al. 1990). However, SCE was required to mitigate for losses to fish and kelp, and is prepared to double the size of the mitigation reef off San Clemente to achieve compliance with mitigation requirements (CSLC 2019).

The point of these references is to note that there has been little interest in analysis of potential effects to holoplankton in the last 30 years or so. For CalAm, they actually did a plankton survey in Monterey Bay, but ended up with the conclusion above that the effects would be minimal given the short duration and small area considered.

Please let me know if you would like any more information.

Respectfully,

MBC *Aquatic Sciences*

Shane Beck
President

Cc: K. Thomas (Kimley-Horn), D. Vilas (MBC)

References:

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APPENDIX 4.2.5.2

BRINE DISCHARGE MEMO FOR FINAL EIR

7 March 2019

Mark Donovan, PE
GHD
Desalination Program Manager
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Re: Review of Dense-Discharge Associated Impacts for the Doheny Desalination Project

Hello Mark:

This memo was prepared to review a range of impacts related to the discharge of dense (negatively buoyant) discharge scenarios presented in the Plumes 18b report (Jenkins 2019) prepared for the South Coast Water District Doheny Desalination Plant (DDP).

Based on Jenkins (2019) there are two separate impacts to the marine environment from the discharge of the dense plume: shear stress mortality as the plume is released from the diffuser ports, and the introduction of a sinking, concentrated brine into the marine environment. Shear stress impacts as a result of a dense discharge were reviewed in MBC Aquatic Sciences' (MBC's) memo *Review of Plumes 18b Modeling Deleterious Diffuser Entrainment Doheny Desalination Project* dated 6 March 2019 and will be discussed further later. This memo will determine the area of exposure of the benthic habitat to elevated salinity levels and later combine the two impacts to determine area of impacts as a starting point to evaluate appropriate mitigation.

The California Ocean Plan (SWRCB 2015) allows for an area of initial mixing of concentrated brine discharges as described below:

> *Discharges shall not exceed a daily maximum of 2.0 parts per thousand (ppt) above natural background salinity measured no further than 100 meters (328 ft) horizontally from each discharge point. There is no vertical limit to this zone.*

This area is defined as the Brine Mixing Zone (BMZ). For the Doheny Desalination Project the 328 feet (ft) regulatory limit of the BMZ is displayed in Figure 1. This figure illustrates the BMZ at 328 ft in all directions from the discharge pipe for the 1,488 ft of the pipeline on which diffuser ports are located. The area of the allowed BMZ for the project is 20.1 acres.

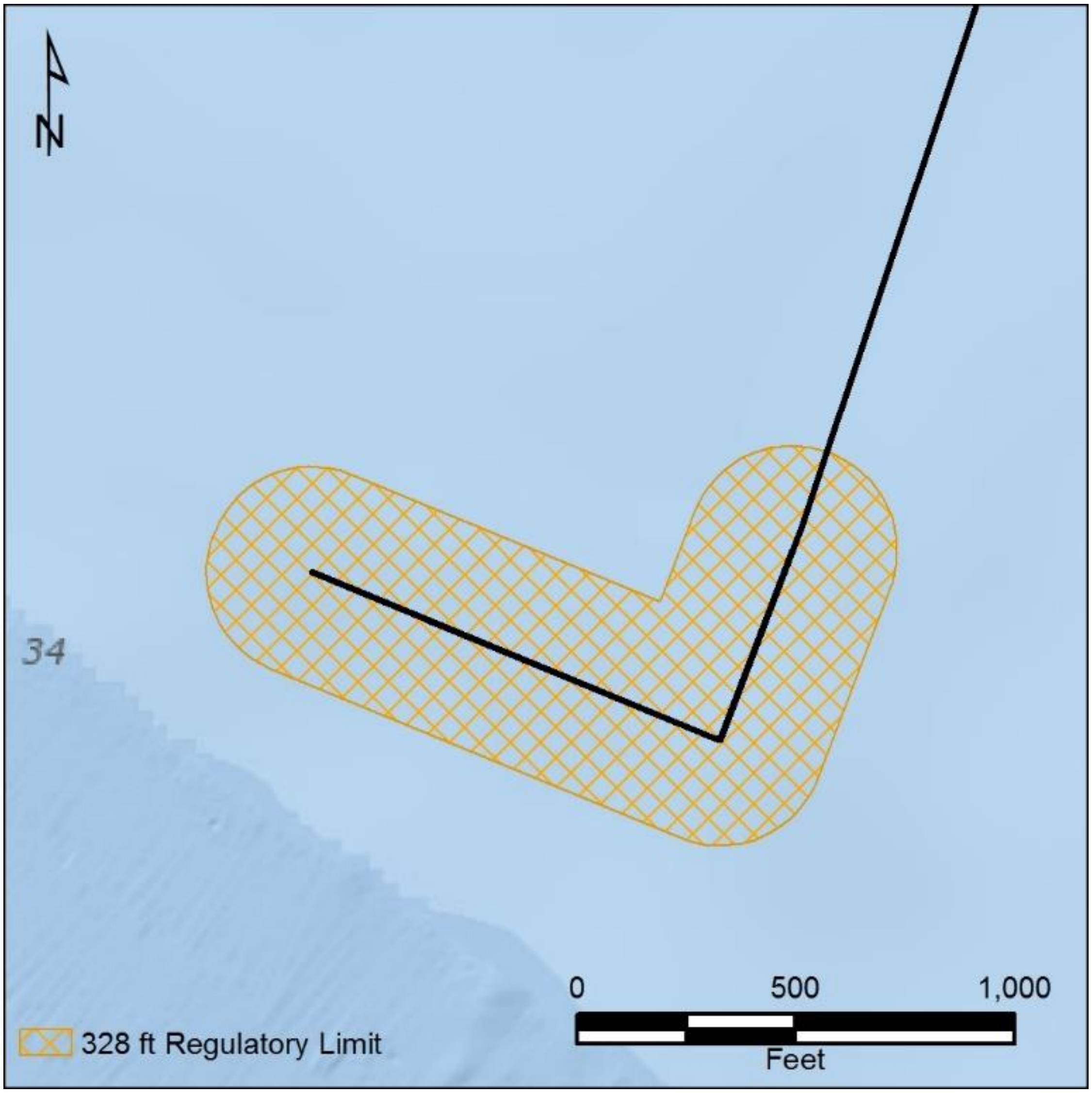

Figure 1. The 328 foot regulatory limit of the BMZ at the project location.

Jenkins (2019, Table 5) modeled eight brine mixing scenarios which would result in a dense plume. Of these, two scenarios, the discharge of 3 million gallons per day (MGD) and 5 MGD of 67 ppt brine with no wastewater dilution, have a remote chance of occurring in the future. Jenkins calculated the distance at which the 2 ppt above ambient salinity is met for the 3 MGD discharge is less than 0.6 ft from the discharge pipe, and for the 5 MGD discharge the limit is met less than 0.7 ft from the pipe (Figure 2). In a worst-case scenario, the discharge of 15 MGD of 67 ppt brine with no wastewater dilution, the salinity mixing meets the 2 ppt limit less than 2.5 ft from the discharge pipe.

Using GIS tools, MBC determined the area of exposure of the benthic habitat to salinities in excess of 2 ppt above ambient for both of the low-likelihood discharge scenarios and for the worst-case scenario (Table 1). To be conservative the area of the pipe was included in in the calculation of these exposure areas. For both low-likelihood dense discharge mixing

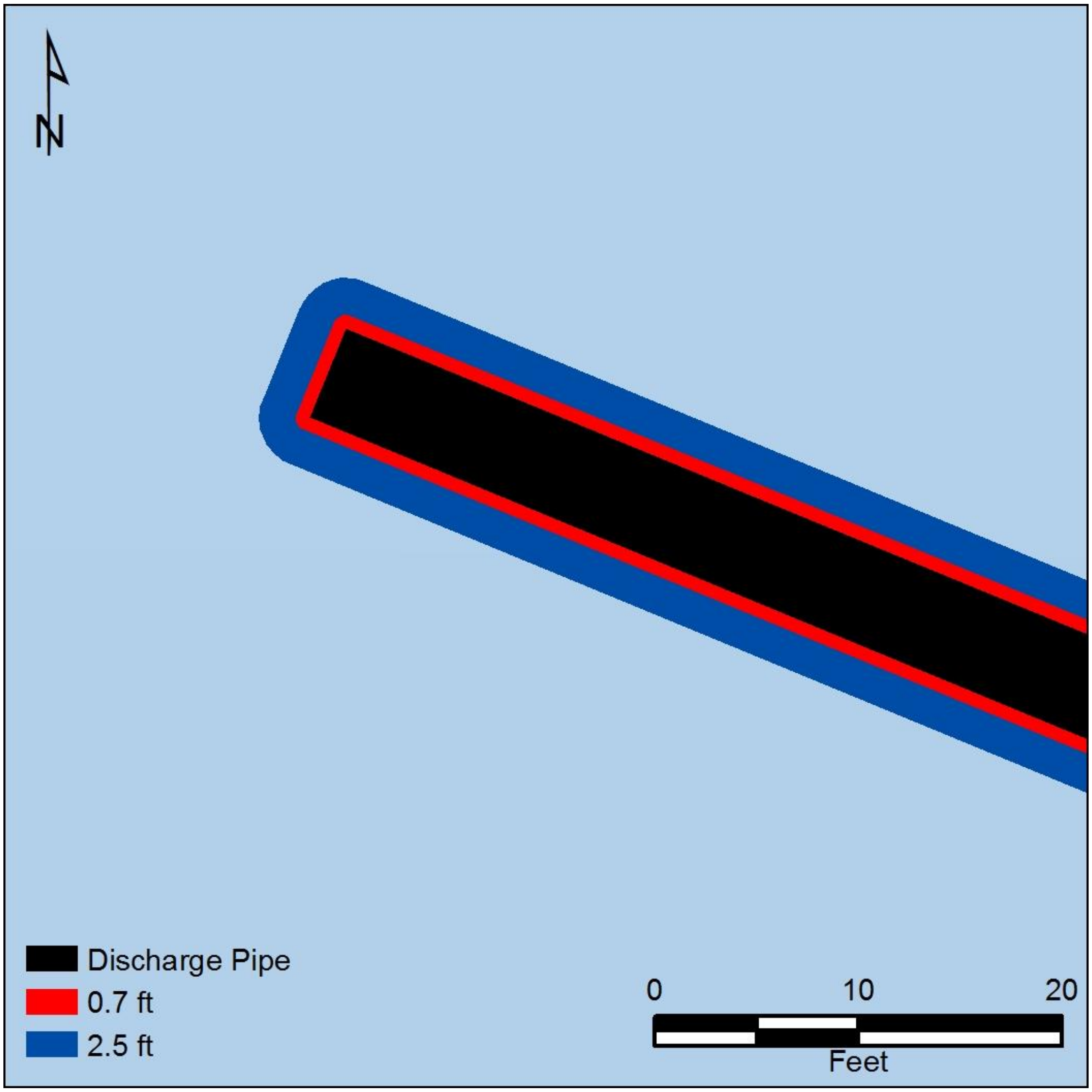

Figure 2. Distance from the discharge pipe for compliance with the 2 ppt regulatory mixing requirement. The red line at 0.7 ft from the pipe approximates the mixing zones for both of the low likelihood mixing scenarios of 3 and 5 MGD of 67 ppt brine with no dilution. The blue line at 2.5 ft from the pipe is the worst-case scenario of 15 MGD of 67 ppt brine with no dilution Note: The pipeline diameter is 4.75 ft.

scenarios the area of benthic exposure >2 ppt above ambient salinity is about 0.2 acres, 1% of the area allowed for this project by the Ocean Plan (SWRCB 2015). For the worst-case scenario the area is still less than one-third of an acre. These area totals are considered the BMZs for the respective scenarios.

Because of the relatively small area of the BMZ for both low-likelihood dense discharge mixing scenarios we will not further parse out detrimental impacts related to acute toxicity or osmotic stresses to benthic organisms that reside within the BMZ. Instead, in these cases we propose basing mitigation on the area of the entire BMZ. We do not suggest that this

Table 1. Area (acres) of benthic exposure >2 ppt above ambient salinity for the two low-likelihood discharge scenarios and for the worst-case scenario.

Dense Discharge Scenario (Jenkins 2019, Table 5)	Wastewater + brine discharge rates (MGD)	Horizontal distance (feet) to within 2 ppt of ambient salinity (Jenkins 2019, Table 5)	Area (acres) of benthic exposure >2 ppt above ambient salinity (BMZ)
1	0 + 3	0.566	0.1999
3	0 + 5	0.653	0.2058
6	0+15	2.466	0.3296

should be set as a precedent, and a case-by-case analysis is recommended, but for this model determining the areas of differential impacts within the BMZ would not substantially change the final mitigation requirements.

As mentioned above, benthic exposure (BMZ) is one of two impacts to the marine environment from the discharge of a dense plume identified by Jenkins (2019). The second impact is stress-related mortality to small, water-column organisms from mechanical mixing of the discharge into the receiving water. Impact areas for stress-related mortality associated with the two low-likelihood dense discharge mixing scenarios were evaluated by the Area of Production Foregone (APF) method in the MBC 6 March 2019 memo. The APF and BMZ area will be considered additive for purposes of determining an area for the basis of evaluating preliminary mitigation requirements. The full area of impact for the two low-likelihood dense discharge mixing scenarios are presented in Table 2. (The worst-case scenario is not further included.) Combined impact area for the 3 MGD discharge scenario is less than four acres and for the 5 MGD discharge scenario less than six acres.

The intent of this memo is to present a methodology and a result for the cumulative area of impact to the marine environment for the low-likelihood dense plume discharge scenarios. Should analysis of the low-likelihood dense plume discharge scenarios move forward, methodology for proposed mitigation for dense-discharge impacts will be presented in subsequent documents.

Table 2. Combined BMZ and APF impact areas (acres) for the determination of mitigation requirements for the two low-likelihood dense plume discharge scenarios.

Dense Discharge Scenario (Jenkins 2019, Table 5)	Wastewater + brine discharge rates (MGD)	Area (acres) of benthic exposure >2 ppt above ambient salinity (BMZ)	Area (acres) Production Foregone (APF)	Combined BMZ and APF area (acres)
1	0 + 3	0.20	3.54	3.74
3	0 + 5	0.21	5.57	5.78

Please let me know if you would like to discuss further.

Cordially,

MBC *Aquatic Sciences*

David Vilas
Senior Scientist

Cc: K. Thomas (Kimley-Horn), D.S. Beck (MBC)

References:

Jenkins, S. 2019. Plumes 18b Modeling Deleterious Diffuser Entrainment Doheny Desalination Project. Prepared for Mark Donovan, GHD, 15 January 2019.

MBC Aquatic Sciences (MBC). 2019. Review of Plumes 18b Modeling Deleterious Diffuser Entrainment Doheny Desalination Project. Memo to Mark Donovan, GHD, dated 6 March 2019.

State Water Resources Control Board (SWRCB). 2015. California Ocean Plan.

Attachment A: Comment Letter O1 Exhibits

- *Exhibit D – Water Well Standards*
- *Exhibit E – Water Well Standards*
- *Exhibit F – IDA Technical Paper (Dennis Williams, 2015)*
- *Exhibit G – Extended Pumping and Pilot Test (MWDOC, 2014)*
- *Exhibit H – CalEEMod User Manual (2017)*

EXHIBIT D

Water Well Standards

CHAPTER II. STANDARDS

Section 23. Requirements for Destroying Wells.

A. *Preliminary Work.* Before the well is destroyed, it shall be investigated to determine its condition, details of construction, and whether there are obstructions that will interfere with the process of filling and sealing. This may include the use of downhole television and photography for visual inspection of the well.

1. *Obstructions.* The well shall be cleaned, as needed, so that all undesirable materials, including obstructions to filling and sealing, debris, oil from oil-lubricated pumps, or pollutants and contaminants that could interfere with well destruction are removed for disposal.

   The enforcing agency shall be notified as soon as possible if pollutants and contaminants are known or suspected to be in a well to be destroyed. Well destruction operations may then proceed only at the approval of the enforcing agency.

   The enforcing agency should be contacted to determine requirements for proper disposal of materials removed from a well to be destroyed.

2. Where necessary, to ensure that sealing material fills not only the well casing but also any annular space or nearby voids within the zone(s) to be sealed, the casing should be perforated or otherwise punctured.

3. In some wells, it may be necessary or desirable to remove a part of the casing. However, in many instances this can be done only as the well is filled. For dug wells, as much of the lining as possible (or safe) should be removed prior to filling.

B. *Filling and Sealing Conditions.* Following are requirements to be observed when certain conditions are encountered:

1. *Wells situated in unconsolidated material in an unconfined groundwater zone.* In all cases the upper 20 feet of the well shall be sealed with suitable sealing material and the remainder of the well shall be filled with suitable fill, or sealing material. (See Figure 9A, of Bulletin 74- 81.)

2. *Well penetrating several aquifers or formations.* In all cases the upper 20 feet of the well shall be sealed with impervious material.

In areas where the interchange of water between aquifers will result in a significant[Note 22] deterioration of the quality of water in one or more aquifers, or will result in a loss of artesian pressure, the well shall be filled and sealed so as to prevent such interchange. Sand or other suitable inorganic material may be placed opposite the producing aquifers and other formations where impervious sealing material is not required. To prevent the vertical movement of water from the producing formation, impervious material must be placed opposite confining formations above and below the producing formations for a distance of 10 feet or more. The formation producing the deleterious water shall be sealed by placing impervious material opposite the formation, and opposite the confining formations for a sufficient vertical distance (but no less than 10 feet) in both directions, or in the case of "bottom" waters, in the upward direction. (See Figure 9B.)

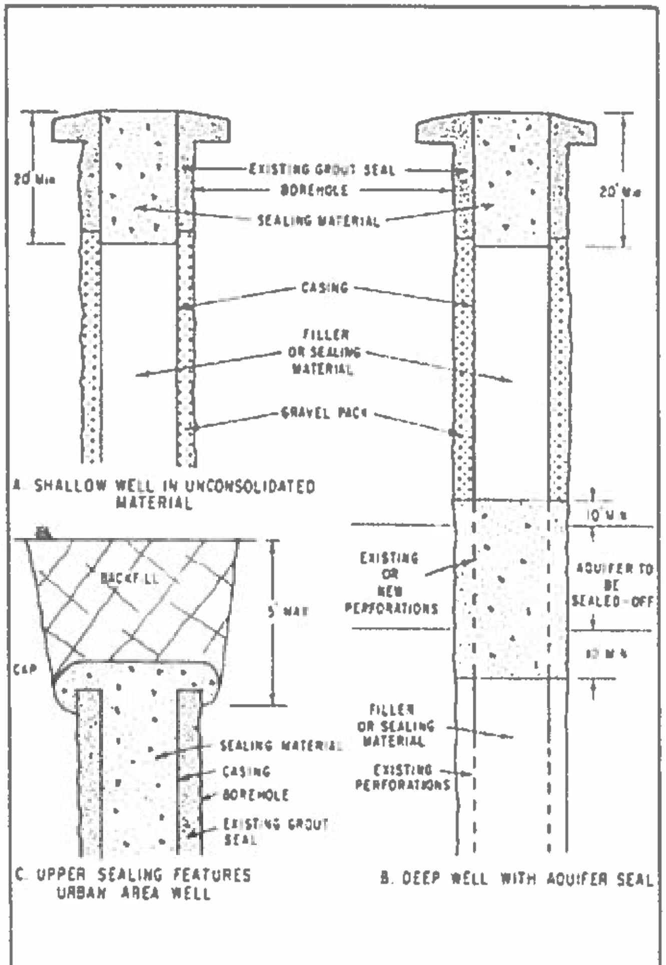

Figure 9. PROPERLY DESTROYED WELLS

In locations where interchange is in no way detrimental, suitable inorganic material may be placed opposite the formations penetrated. When the boundaries of the various formations are unknown, alternate layers of impervious and pervious material shall be placed in the well.

3. *Well penetrating creviced or fractured rock.* If creviced or fractured rock formations are encountered just below the surface, the portions of the well opposite this formation shall be sealed with neat cement, sand-cement grout, or concrete. If these formations extend to considerable depth, alternate layers of coarse stone[Note 23] and cement grout or concrete may be used to fill the well. Fine grained material shall not be used as fill material for creviced or fractured rock formations.

4. *Well in noncreviced, consolidated formation.* The upper 20 feet of a well in a noncreviced, consolidated formation shall be filled with impervious

material. The remainder of the well may be filled with clay or other suitable inorganic material.

5. *Well penetrating specific aquifers, local conditions.* Under certain local conditions, the enforcing agency may require that specific aquifers or formations be sealed off during destruction of the well.

C. *Placement of Material.* The following requirements shall be observed in placing fill or sealing material in wells to be destroyed:

1. The well shall be filled with the appropriate material (as described in Subsection D of this section) from the bottom of the well up.

2. Where neat cement grout, sand-cement grout, or concrete is used, it shall be poured in one continuous operation.

3. Sealing material shall be placed in the interval or intervals to be sealed by methods that prevent free fall, dilution, and/or separation of aggregate from cementing materials.

4. Where the head (pressure) producing flow is great, special care and methods must be used to restrict the flow while placing the sealing material. In such cases, the casing must be perforated opposite the area to be sealed and the sealing material forced out under pressure into the surrounding formation.

5. In destroying gravel-packed wells, the casing shall be perforated or otherwise punctured opposite the area to be sealed. The sealing material shall then be placed within the casing, completely filling the portion adjacent to the area to be sealed and then forced out under pressure into the gravel envelope.

6. When pressure is applied to force sealing material into the annular space, the pressure shall be maintained for a length of time sufficient for the cementing mixture to set.

7. To assure that the well is filled and there has been no jamming or "bridging" of the material, verification shall be made that the volume of material placed in the well installation at least equals the volume of the empty hole.

D. *Materials.* Requirements for sealing and fill materials are as follows:

1. *Impervious Sealing Materials.* No material is completely impervious. However, sealing materials shall have such low permeability that the

volume of water passing through them is of small consequence.

Suitable impervious materials include neat cement, sand-cement grout, concrete, and bentonite clay, all of which are described in Section 9, Subsection D, "Sealing Material" of these standards; and well-proportioned mixes of silts, sands, and clays (or cement), and native soils that have a coefficient of permeability of less than 10 feet per year.[Note 24] Used drilling muds are not acceptable.

2. *Filler Material.* Many materials are suitable for use as a filler in destroying wells. These include clay, silt, sand, gravel, crushed stone, native soils, mixtures of the aforementioned types, and those described in the preceding paragraph. Material containing organic matter shall not be used.

E. *Additional Requirements for Wells in Urban Areas.*

In incorporated areas or unincorporated areas developed for multiple habitation, to make further use of the well site, the following additional requirements must be met (see Figure 9C):

1. A hole shall be excavated around the well casing to a depth of 5 feet below the ground surface and the well casing removed to the bottom of the excavation.

2. The sealing material used for the upper portion of the well shall be allowed to spill over into the excavation to form a cap.

3. After the well has been properly filled, including sufficient time for sealing material in the excavation to set, the excavation shall be filled with native soil.

F. *Temporary Cover.* During periods when no work is being done on the well, such as overnight or while waiting for sealing material to set, the well and surrounding excavation, if any, shall be covered. The cover shall be sufficiently strong and well enough anchored to prevent the introduction of foreign material into the well and to protect the public from a potentially hazardous situation.

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EXHIBIT E

Water Well Standards

CHAPTER II. STANDARDS

Part III. Destruction of Wells

Section 20. Purpose of Destruction.

A well that is no longer useful[Note 21] (including exploration and test holes) must be destroyed in order to: 1. Assure that the groundwater supply is protected and preserved for further use. 2. Eliminate the potential physical hazard.

Section 21. Definition of "Abandoned" Well.

A well is considered 'abandoned' or permanently inactive if it has not been used for one year, unless the owner demonstrates intention to use the well again. In accordance with Section 115700 of the California Health and safety Code, the well owner shall properly maintain an inactive well as evidence of intention for future use in such a way that the following requirements are met:

(1) The well shall not allow impairment of the quality of water within the well and groundwater encountered by the well.

(2) The top of the well or well casing shall be provided with a cover, that is secured by a lock or by other means to prevent its removal without the use of equipment or tools, to prevent unauthorized access, to prevent a safety hazard to humans and animals, and to prevent illegal disposal of wastes in the well. The cover shall be watertight where the top of the well casing or other surface openings to the well are below ground level, such as in a vault or below known levels of flooding. The cover shall be watertight if the well is inactive for more than five consecutive years. A pump or motor, angle drive, or other surface feature of a well, when in compliance with the above provisions, shall suffice as a cover.

(3) The well shall be marked so as to be easily visible and located, and labeled so as to be easily identified as a well.

(4) The area surrounding the well shall be kept clear of brush, debris, and waste materials.

If a pump has been temporarily removed for repair or replacement, the well shall not be considered 'abandoned' if the above conditions are met. The well shall be adequately covered to prevent injury to people and animals and to prevent the entrance of foreign material, surface water, pollutants, or contaminants into the well during the pump repair period.

Section 22. General Requirement.

All "abandoned" wells and exploration or test holes shall be destroyed. The objective of destruction is to restore as nearly as possible those subsurface conditions which existed before the well was constructed taking into account also changes, if any, which have occurred since the time of construction. (For example, an aquifer which may have produced good quality water at one time but which now produces water of inferior quality, such as a coastal aquifer that has been invaded by seawater.)

Destruction of a well shall consist of the complete filling of the well in accordance with the procedures described in Section 23 (following).

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EXHIBIT F

YIELD AND SUSTAINABILITY OF LARGE SCALE SLANT WELL FEEDWATER SUPPLIES FOR OCEAN WATER DESALINATION PLANTS

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Abstract:

There is no theoretical upper limit of the yield and sustainability of slant wells used as a source of feed water supply to ocean desalination plants. Research and field testing over the past nine years suggest that slant wells extracting water from subsea alluvial aquifers can provide a high yielding and long-lasting sustainable water supply when designed, constructed and maintained properly. Furthermore, the total yield is a function of scale and the reliability is guaranteed by the ocean source. Slant wells are angled wells completed in aquifers beneath the ocean floor and receive a high percentage of recharge from both vertical leakage (through the seabed) as well as horizontal flow from offshore aquifers. Natural filtration in the subsea permeable deposits results in low turbidity and reduction or elimination of seawater reverse osmosis (SWRO) pretreatment. Environmental advantages include lack of impacts to fish and marine life and no surface visibility. Maximum supply limitations are a function only of the permeability of near shore and offshore aquifers, the areal and vertical extent of these deposits, the availability of land, and potential adverse impacts. Slant wells are merely vertical wells drilled on an angle. As such, sustainability of flow is assured by the same routine maintenance programs developed over the past seven decades and routinely utilized in vertical wells. Using the dual-rotary method of construction and incorporating a telescopic design, 4 mgd slant wells can be constructed with angles ranging from a few degrees to a few tens of degrees and achieve lengths of 1,000 ft or more. Variable angles allow targeting production from specific aquifers and longer well screen lengths result in higher production than vertical wells. A slant well layout may be comprised of a single well or group of wells within the same wellhead area (i.e., pod). Multiple slant well arrays may be constructed and "linked" together until the cumulative total discharge rate meets the feed water supply demand. Interference between wells governs the number and spacing and geohydrologic considerations and land availability govern limitation on spatial extent. For example, for typical California coastal aquifers, a feed water supply of nine slant well pods with each pod containing three 4.32 mgd slant wells can yield approximately 117 mgd from a two mile reach of coastline. The Monterey Peninsula Water Supply Project has recently completed a 724 ft test slant well at an angle of 19 degrees below horizontal north of Monterey, CA. The well is currently undergoing long-term testing to develop well and aquifer parameters and potential impacts for the 24 mgd full scale project which will consist of ten slant wells including standby capability. Routine maintenance employing mechanical cleaning on the order of every three to five years provides a long-term sustainable feed water supply. The current misconceptions that slant wells yield only low amounts of supply and that they contribute to seawater intrusion is false. Experience gained has shown that the primary constraint to development of subsurface feed water supplies is permitting. This paper discusses research and experience on slant well and well field design and upward scalability for large SWRO desalination feed water requirements exceeding 200 mgd.

INTRODUCTION

1.1 Background

The number of subsurface intakes throughout the world is relatively small compared to open ocean intakes; averaging approximately 12 mgd per facility as compared to approximately 52 mgd per facility for open ocean intakes [1], [2]. Slant well feed water supplies for SWRO desalination plants is an emerging technology. Originating out of the necessity to explore subsea aquifers near Dana Point, CA, the first test slant well was constructed in 2006. Since then, a number of subsurface intakes for SWRO have been and are continuing to be evaluated along the California coast ranging in size from small systems (< 10 mgd) to very large systems (> 150 mgd). As of this writing, a 724 ft test slant well completed in March of 2015 near Monterey, California as part of the Monterey Peninsula Water Supply Project (MPWSP) is currently undergoing long-term test pumping.

Slant wells are simply vertical wells drilled on an angle and produce water from near shore and subsea aquifers. Drawing seawater from subsea and near shore aquifers provides natural filtration from suspended organic matter and sediment, particularly during storm surges and heavy precipitation. Slant wells receive recharge from vertical leakage through the sea floor (i.e., benthic zone) and horizontal flow from subsea and near shore aquifer systems. Field tests show that the engineered artificial filter pack surrounding the screened portion of the slant well intake results in very low turbidity (i.e., low SDI indices) which minimizes the need for RO pre-treatment. A slant well feed water supply typically consists of shallow angled wells[1] in a beach or near-coastal environment. The supply may consist of a single slant well, an array of wells or multiple arrays of wells grouped together into "pods"[2] extending beneath the ocean. As long as there are permeable subsea deposits, available land, and no undesirable impacts, there is no theoretical upper limit to the quantity of feed water which can be produced from slant wells. Environmentally sensitive, slant well systems are buried systems completed below the land surface eliminating both impingement and entrainment issues to marine life as well as undesirable aesthetic impacts (i.e., no visual impacts on the surface).

1.2 Subsurface Intakes - The Preferred Technology for SWRO Intakes in California

In a recent amendment to the Water Quality Control Plan for Ocean Waters of California (i.e., "Ocean Plan"), the California State Water Resources Control Board [3], recommended to "Establish subsurface intakes as the preferred technology for seawater intakes." In accordance with the Ocean Plan amendment, a number of desalination projects along the coast of California have evaluated or are in the process of evaluating the feasibility of subsurface intakes for feed water supply (see Figure 1).

Figure 1. Projects along the California coast which have considered or are evaluating the feasibility of subsurface intake systems

[1] For purposes of this paper, the word angled well and slant well both refer to a non vertical well and are used interchangeably.
[2] A slant well pod is the common wellhead area for multiple slant wells with each well in the pod having varying azimuth angles.

Currently, only ten small desalination facilities are in operation along the California coast with 15 more in the feasibility phase and with a cumulative fresh water output ranging from 260–367 mgd [3].

KNOWLEDGE GAINED FROM THE DOHENY AND MONTEREY TEST SLANT WELL PROJECTS

The Doheny Ocean Desalination Project[3] near Dana Point, California has been summarized extensively in previous documents [4], [5], [6], [7], [8], [9], [10]. A few years after construction of a test slant well in 2006, an approximate two year extended pumping test ensued from 2010-2012. Since then, improvements on the technology have been made and applied to the recently completed MPWSP test slant well north of Marina in Monterey County, California. The Doheny test slant well represents the first successful high capacity slant well completed with an artificial filter pack beneath the ocean floor. At 23 degrees below horizontal and a total lineal length[4] of 350 ft, it was the longest dual rotary-drilled artificially filter packed slant well until recently, when the Monterey test slant well was drilled to a total MD of 724 ft. Continuing work on both the Doheny and Monterey projects is still in progress which will include additional testing, predictive ground water modeling, and final design of the full scale projects.

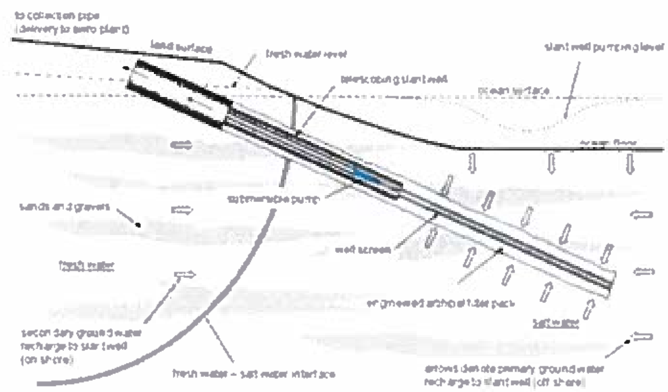

Figure 2. Telescoping Well Design

1.3 Telescoping Well Design

During the two year pilot testing, the Doheny test slant well produced approximately 3 mgd with relatively stable drawdowns. When it was constructed in 2006, it was test pumped at approximately 2,100 gpm[5] and displayed a well efficiency of 95%. During the extended pilot testing the well efficiency dropped from the original value of 95% in 2006 to 52% in 2012 [10].

Figure 3. Monterey Test Slant Well (19 degrees 724 ft)

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[3] Formerly the South Orange Coastal Ocean Desalination Project (SOCOD)

[4] In angled wells, the term lineal length or "Measured Depth" (MD) is used. MD is the length of the well bore as determined by a measuring stick as compared to true vertical depth (TVD) which is a straight vertical line from the surface to the bottom of the borehole (or anywhere along its length).

[5] The pump used for the two year extended pumping test was a high speed, high capacity pump (2480 rpm) producing 2,200 gpm.

This loss of well efficiency was expected due to the inability to fully develop the well during construction. Specifically, due to limited funding, the Doheny test slant well project was completed with a uniform 12-inch diameter casing and screen without a larger diameter pump house chamber. Consequently, the largest diameter submersible test pump that could be installed in the well was ten inches with a maximum discharge rate of 1,700 gpm. The standard industry practice is to develop a well at 1.5 times the design discharge rate, or approximately 3,200 gpm for a design discharge rate of 2,100 gpm. Figure 2 shows a typical telescoping well design and Figures 3 and 4 show the telescoping well design used in the Monterey test slant well north of the town of Marina in Monterey County, California.

1.4 Larger Pump House Casings

Due to the pump house casing limitation experienced at Dana Point and the inability to fully develop the well, the MPWSP test slant well included a larger diameter pump house casing. The Monterey test slant well has an 18 in. pump house casing which can accommodate placement of large development pumps with capacities over 3,000 gpm. Properly developed wells constructed using corrosion resistant materials such as 2507 Super Duplex Stainless Steel minimize well deterioration due to corrosion and biofouling. As such, these design improvements result in less frequent well rehabilitation with intervals estimated at between 3–5 yrs. Redevelopment will include the use of a high capacity air lift/swab assembly as part of the on-going maintenance process.

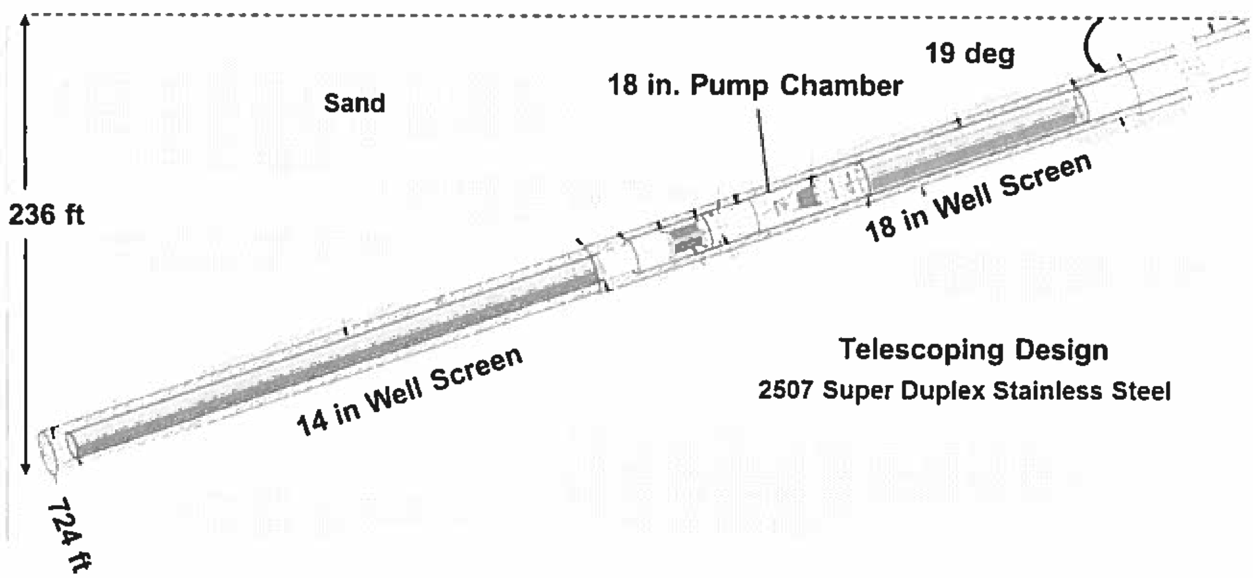

Figure 4. Telescoping design showing larger pump house casing used in the Monterey test slant well

1.5 Pumping Out Old Marine Ground Water

Geochemical tracers used to quantify water sources to the Doheny test slant well during an almost two year pumping test (2010-2012) were used to estimate slant well connectivity to the ocean and relevant amounts of water sources [8]. Testing found that old marine ground water is slightly acidic, anoxic, and enriched with dissolved iron and manganese. Dissolved iron and manganese concentrations increased in the pumped water to a peak of 11 milligrams per liter (mg/L) and decreased to 5 mg/L by the end of the test. It was estimated that the concentration of dissolved iron in the old marine ground water exceeds 41 mg/L. Test results support the increased capture of shallow, young marine ground water. Natural isotope data showed after one year of pumping, recharge to the slant well consisted of a mixture of brackish ground water (which showed a decreasing trend), ocean water (which showed an increasing trend), and old marine ground water which initially increased and then slightly decreased as it was being removed from the aquifer. This reflected the fresh/salt interface being induced to migrate toward the well. The geochemical data combined with a three-dimensional variable density flow and solute transport model predicted that the old marine ground water would be fully removed from the subsea aquifer within approximately one year at the full scale production rate of 30 mgd. Furthermore, upon reaching steady state conditions, (approximately one year), and after removal of the old marine ground water, the source of water to the feed water supply wells was predicted to consist of 95% "younger" ocean water (with very low levels of dissolved iron/manganese, ~ 2 µg/L), and 5% brackish ground water (~2 mg/L of dissolved iron/manganese), resulting in a blended concentration of approximately 0.10 mg/L. Results from the Doheny project suggest that the project may be constructed in two stages:

1- Initial Stage: Well field, conveyance and disposal system to pump out old marine ground water[6],
2- Final Stage: Construction of the project once the feed water quality is known.

Comparison of iron and manganese results from the Doheny test slant well and the current Monterey test slant well shows that the old marine water present in the Dana Point area is not found in the Monterey area. Iron and manganese concentrations from the MPWSP test slant well are very low suggesting that the subsea environment containing old marine ground water may not be present in coastal aquifers (such as Monterey), and may only be associated with subsea paleochannels such as in Dana Point.

SLANT WELL HYDRAULICS

1.6 Universal Drawdown Equation (UDE)

Development of an equation to calculate the drawdown distribution in the vicinity of angled wells was developed out of the necessity to understand water level distributions around slant wells without having to develop a distributed parameter ground water flow model. Conventional drawdown equations for vertical or horizontal wells are inadequate to properly describe the drawdown distribution in the vicinity of slant wells. Williams [11] used the principle of superposition combined with standard well hydraulics to develop universal drawdown equations (UDE) which calculate the drawdown distribution in the vicinity of angled production wells with inclination angles ranging from 0 degrees (horizontal wells), to 90 degrees (vertical wells). The method is computationally simple and, other than the normal assumptions for standard well equations, only requires that the calculated drawdown represent the drawdown which would be measured in a fully penetrating observation well. Solutions using the UDE

[6] During this phase, pilot plant testing would be undertaken to finalize feed water quality for treatment process design.

are developed for confined, unconfined and semi-confined (leaky) aquifers. Figures 5 and 6 below illustrate the variables used in the UDE and their notations.

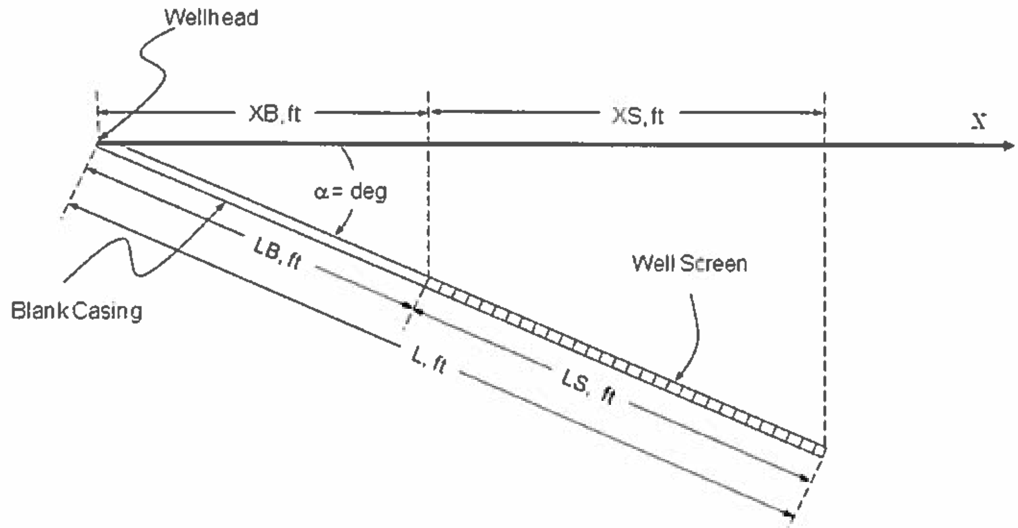

Figure 5. Cross section of an angled well showing notation used in the UDE

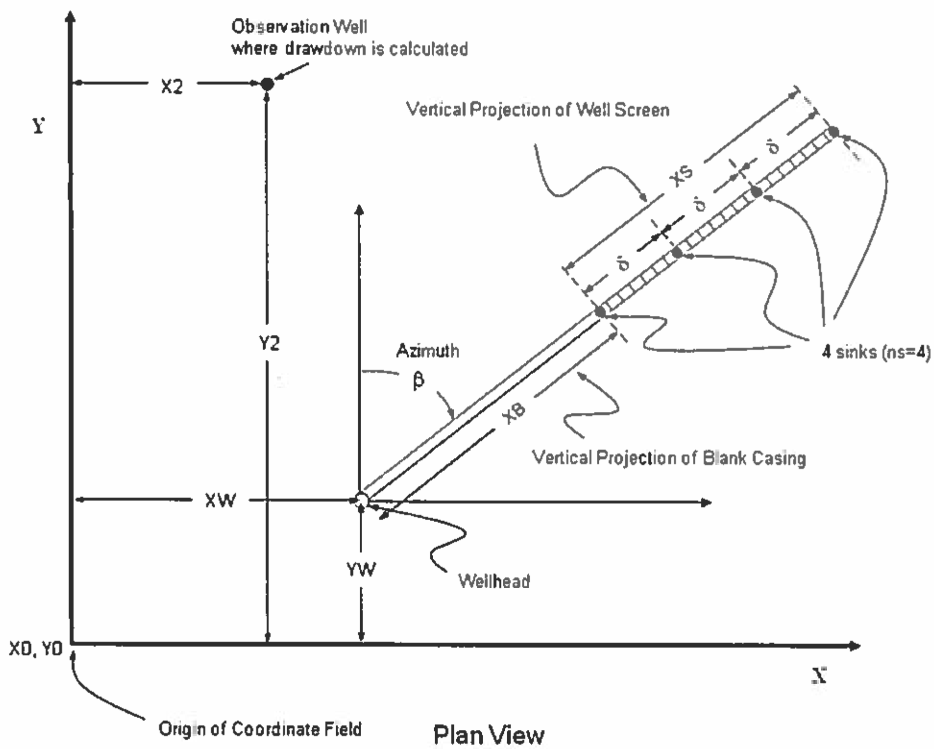

Figure 6. Plan view of angled well and notations used in the UDE

The following equation calculates the drawdown distribution around an angled well for a confined aquifer using Jacob's equation [11]:

$$s = (264\ Q/T)\ [\ \log(0.3Tt/S) - (2/ns)\log(RP_1 \times RP_2 \times RP_3 \times_{...} RP_{ns})\ ] \qquad (1)$$

where:

- s = drawdown, ft
- Q = well discharge rate, gpm
- T = aquifer transmissivity, gpd/ft
- S = aquifer storativity, fraction
- t = time since pumping started, days
- ns = number of sinks in the vertical projection of the well screen
- RP_i = horizontal distance from point where drawdown is desired to the "ith" sink, ft

1.7 Slant Wells Completed Beneath the Ocean

The drawdown solution in the vicinity of a slant well completed in subsea aquifers behaves exactly like a leaky artesian aquifer. The benthic zone (i.e., the zone of the sea floor and a few feet below), is generally a lower permeability zone than underlying subsea aquifers and behaves as a semi-pervious layer. This semi-pervious layer (i.e., leaky layer) has a vertical hydraulic conductivity of K' and a thickness of b' consistent with the leakance term of K'/b' as defined by Hantush [12]. Thus, the UDE concept of superposition may be applied to subsea leaky aquifers using the Hantush-Jacob leaky aquifer equation [13]. Figure 7 illustrates the concept of sea-floor leakage.

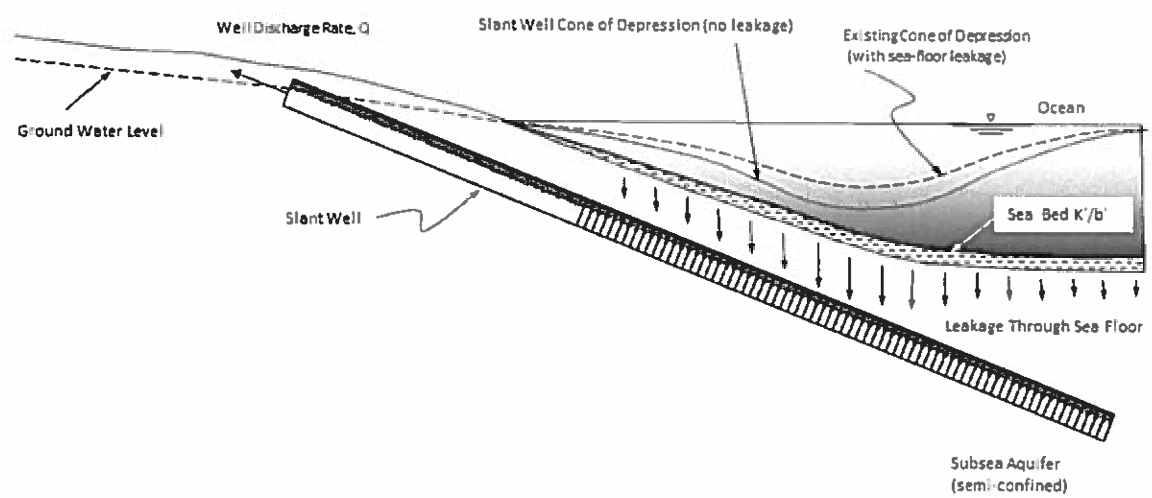

Figure 7. Induced infiltration to slant wells from vertical leakage through the sea floor

1.8 Slant Well and Vertical Well Production Comparison.

The cone of depression in the vicinity of production wells is a function of aquifer hydraulic properties, discharge rate, and time since the start of pumping. Vertical wells have a concentric cone of depression with the highest drawdown being in the vicinity of the well and declining outward. In angled wells, the cone of depression is ellipsoidal and the drawdown distribution "bowl "shaped centered around the vertical projection of the well screen. As such, for the same aquifers, slant wells produce more water than vertical wells for the drawdown available above the top of the well screen. Specifically, in slant wells, the formation loss (i.e., drawdown in the aquifer), is "spread out" over the vertical projection of the well screen length. In vertical wells, it is concentrated in a logarithmic cone centered on a point. Mathematical support for this statement can be seen by comparing the non-steady state equation in a confined aquifer for a vertical well (i.e., Jacob's equation), with that of an angled well (UDE-Jacob – see eq. 6 in [11] with ns=4). When the discharge rate is varied in both cases (vertical and slant), and limiting the maximum drawdown to the top of the well screen, slant wells have discharge rates approximately 1.5 to 2 times greater than vertical wells[7].

$$Q2 / Q1 = \log(B) / [\log(B) - 0.5 \log(XS^4 / 144)] \quad (2)$$

where:

Q1	=	vertical well discharge rate, gpm
Q2	=	slant well discharge rate, gpm
B	=	$(0.3 \times T \times t) / S$, ft^2
T	=	transmissivity, gpd/ft
S	=	storativity
t	=	time, days
XS	=	vertical projection of slant well screen, LS x $\cos(\alpha)$, ft
α	=	slant well angle, degrees below horizontal

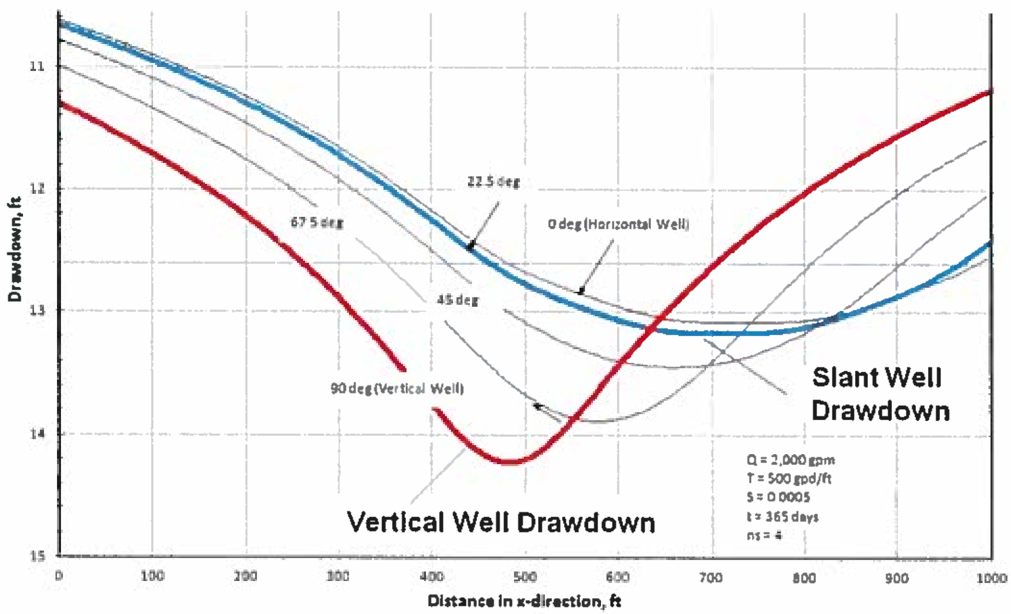

Figure 8. Vertical Well and Slant Well Drawdown Comparisons

1.9 Large-Scale Well Field Hydraulics (UDEM)

An extension of the principle of superposition used to calculate the drawdown distribution around nonvertical wells [11] is presented here to calculate the drawdown distribution around multiple slant

[7] Comparisons are for the same drawdowns and a typical range of aquifer parameters and slant well angles. Also, this comparison is for the laminar flow loss component of the total drawdown only.

wells. The UDEM stands for *Universal Drawdown Equation for Multiple Wells* and is merely a calculational algorithm used to develop regional drawdowns around multiple slant wells and slant well fields. The UDEM calculates the drawdown distribution in the vicinity of multiple slant wells by algebraically adding drawdown distributions from individual wells over a finite difference grid network. The UDEM is useful in the initial planning and layout of slant wells and well pod spacings as well as a first approximation to potential impacts to sensitive habitat and inland ground water resources. As the UDEM obeys the basic assumptions of the underlying equations, regional boundary conditions need to be included by further refinement using a formal three dimensional ground water flow and solute transport model.

1.9.1. Leaky Aquifer Approximation

As slant wells producing from subsea aquifers receive vertical leakage through the sea floor as well as lateral recharge, to properly simulate drawdown effects, the UDEM calculates drawdowns using the Hantush-Jacob leaky aquifer equation [13]. However, when calculating drawdowns for a large number of slant wells with multiple sinks over a large grid network, the calculation may be quite laborious and time consuming due to the approximation of the leaky aquifer well function. This was overcome by using a site specific relationship between the non leaky and leaky aquifer solutions. Specifically:

$$\Delta = a \times \exp(b \times r) \tag{3}$$

where:

Δ	= non leaky aquifer drawdown - leaky aquifer drawdown, ft
a, b	= constants from best fit equation
exp(x)	= the exponential function, also denoted as e^x
e	= base of the natural logarithm (e = 2.718)
r	= distance from pumping well (or sink), ft

For most problems of practical interest, the exponential relationship between Δ and r yields excellent correlation.

UPWARD SCALABILITY OF SLANT WELL FEEDWATER SUPPLIES

1.10 Siting, Permitting, Access and Maintenance

Large scale slant well feed water supplies need a number of permits including land acquisition and access which are dependent upon environmental and operational factors, which if not complied with, could prohibit the project altogether. For example, many of these projects are limited to a maximum percentage of slant well recharge derived from inland water supplies (i.e., basin water vs. ocean water recharge). If this percentage is exceeded, expensive mitigation or provision of supplemental supplies may be required, adding to the overall cost of the project. Other factors affecting wellhead placement may include setbacks due to coastal erosion, a 100-year flood event, sea level rise and proximity to sensitive habitat [14], [15], [16]. Each slant well location should also consider well construction footprints and access to the drilling site and equipment staging area (during construction and routine maintenance).

1.11 Environmental concerns

Environmental factors during construction and operation are primarily concerned with adverse impacts to the natural environment (e.g., sensitive ecological or environmental areas inhabited by a particular species of animal, plant, or other type of organism). In areas of sensitive vegetation, fish habitat or other wildlife, well drawdowns (i.e., ground water level changes), from pumping may restrict placement or hinder construction and maintenance. Other environmental impacts may include visual impacts of facilities during construction or after completion, such as unsightly facilities on the beach or in near shore areas where recreational or other high uses occur.

1.12 Common Wellhead Areas

To minimize unnecessary infrastructure in conveying feed water to the desalination plant site, multiple slant wells can be constructed in close proximity to one another (i.e., common wellhead area or slant well "pod"). The common wellhead areas also minimize disturbance and access during both construction and routine maintenance.

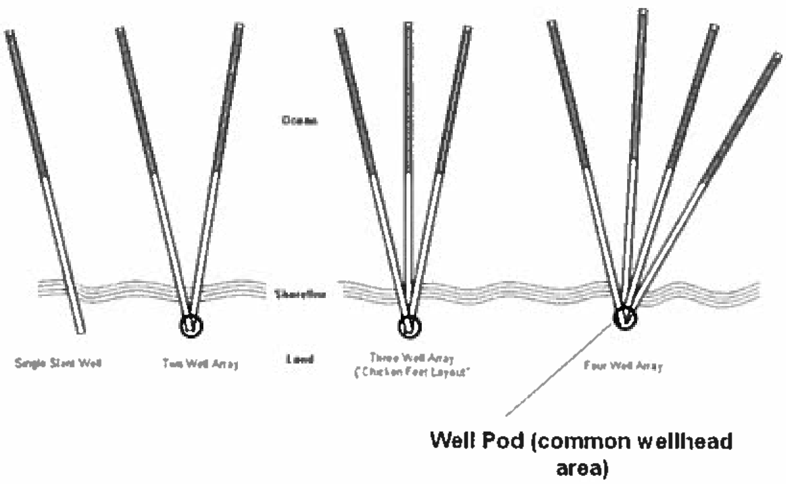

Figure 9. Multiple slant wells from common wellhead areas

1.13 Seawater Intrusion Control

Wells pumping at the coast or from subsea aquifers beneath the ocean floor do not contribute to seawater intrusion. On the contrary, slant wells help prevent seawater intrusion through creation of an extraction trough that intercepts seawater as shown on Figure 10. Modeling studies of full scale slant well projects in California (e.g., Dana Point, Monterey, Cambria, and San Diego) show that the slant well pumping trough acts as a seawater intrusion control mechanism.

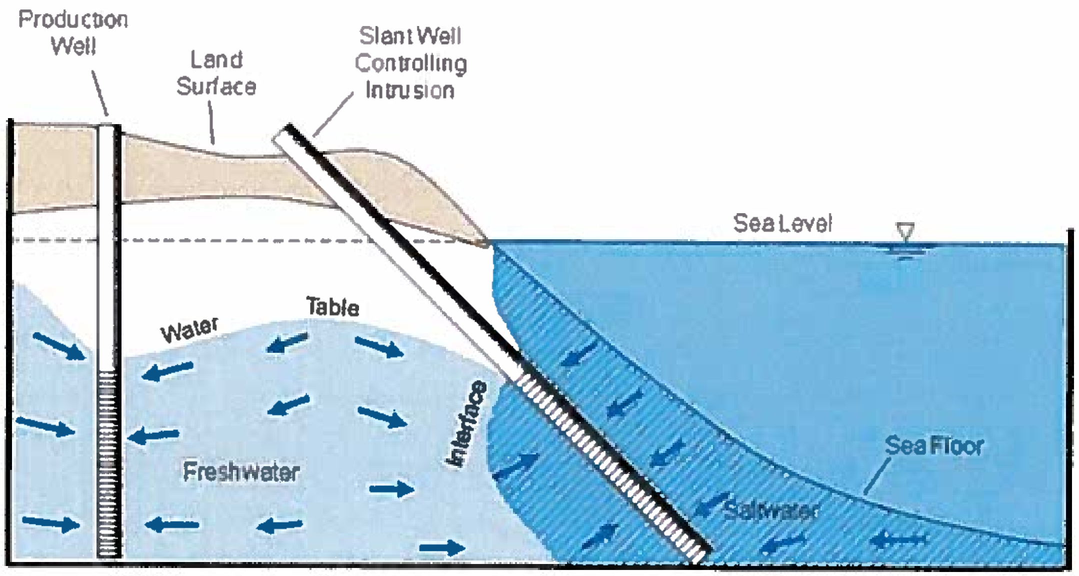

Figure 10. Slant Wells Intercepting Seawater Providing Seawater Intrusion Control

1.14 Large Scale Feed Water Supplies - 78 mgd, 117 mgd, and 233 mgd

For typical California coastal aquifers, a feed water supply of nine slant well pods with each pod containing three 4.32 mgd slant wells can yield approximately 117 mgd from a two mile reach of coastline.

The following interference drawdown plots were generated using the UDEM and field data from a site along the coast of California. The following parameters were used in the calculations:

Slant well length (L)	= 1,000 ft
Length of well screen (LS)	= 860 ft
Angle below horizontal α	= 15 degrees
Transmissivity (Kb)	= 246,000 gpd/ft
Storativity (S_s b)	= 0.045
Saturated thickness(b)	= 223 ft
Slant Well Discharge Rate (Q)	= 3,000 gpm
Time since start of pumping (t)	=365 days
Sea bed leakance (K'/b')	= 0.014/day
Number of sinks per slant well (ns)	= 4
Number of wells per pod	= 3

Figures 11, 12 and 13 illustrate the concept of modular addition of slant well pods to achieve a feed water production of 78 mgd, 117 mgd, and 233 mgd respectively.

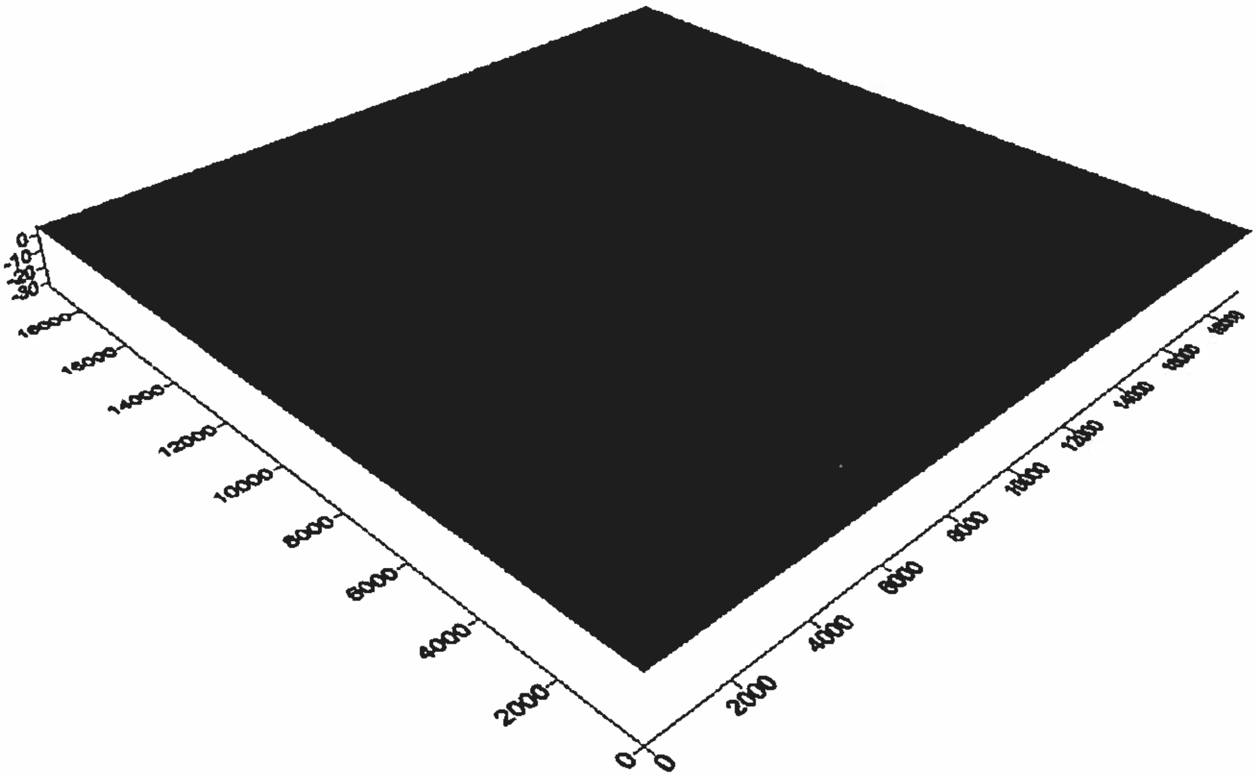

Figure 11. 78 mgd SWRO slant well feed water supply. Six slant well pods and 18 wells. Drawdown distribution in Subsea Aquifers, ft.

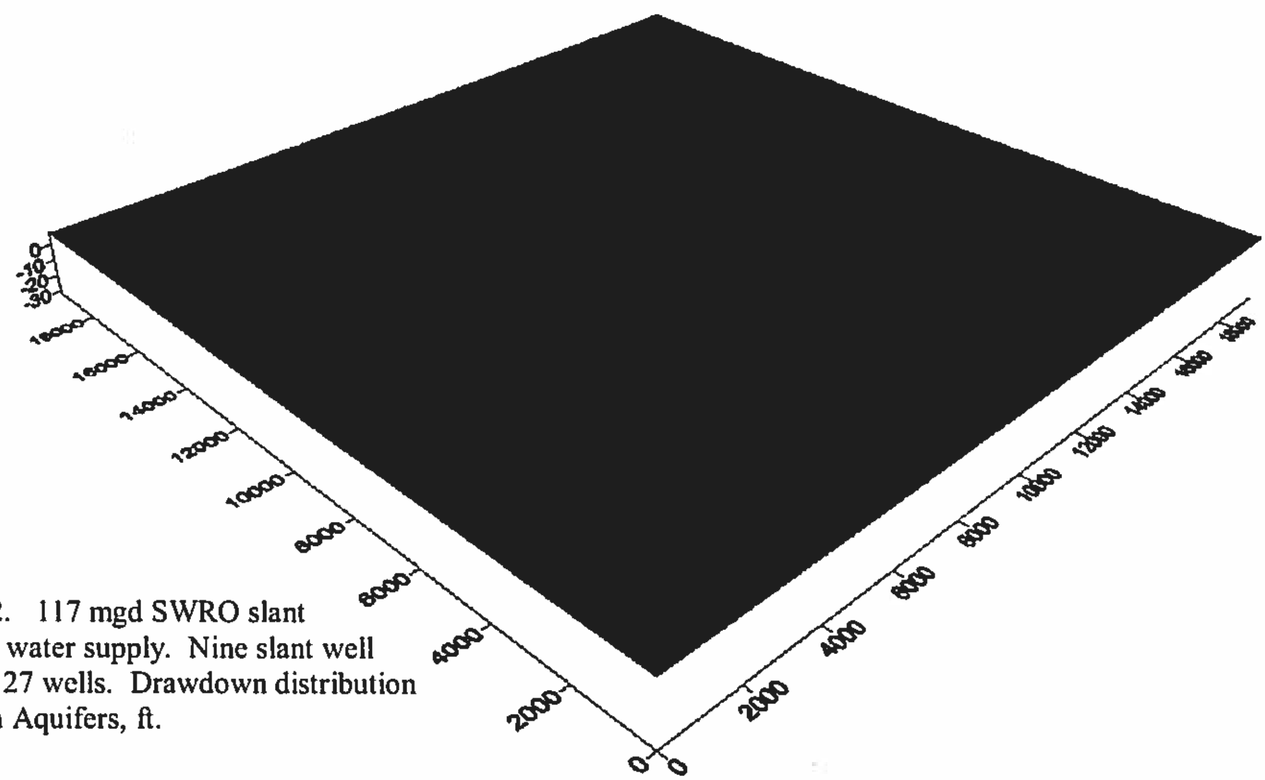

Figure 12. 117 mgd SWRO slant well feed water supply. Nine slant well pods and 27 wells. Drawdown distribution in Subsea Aquifers, ft.

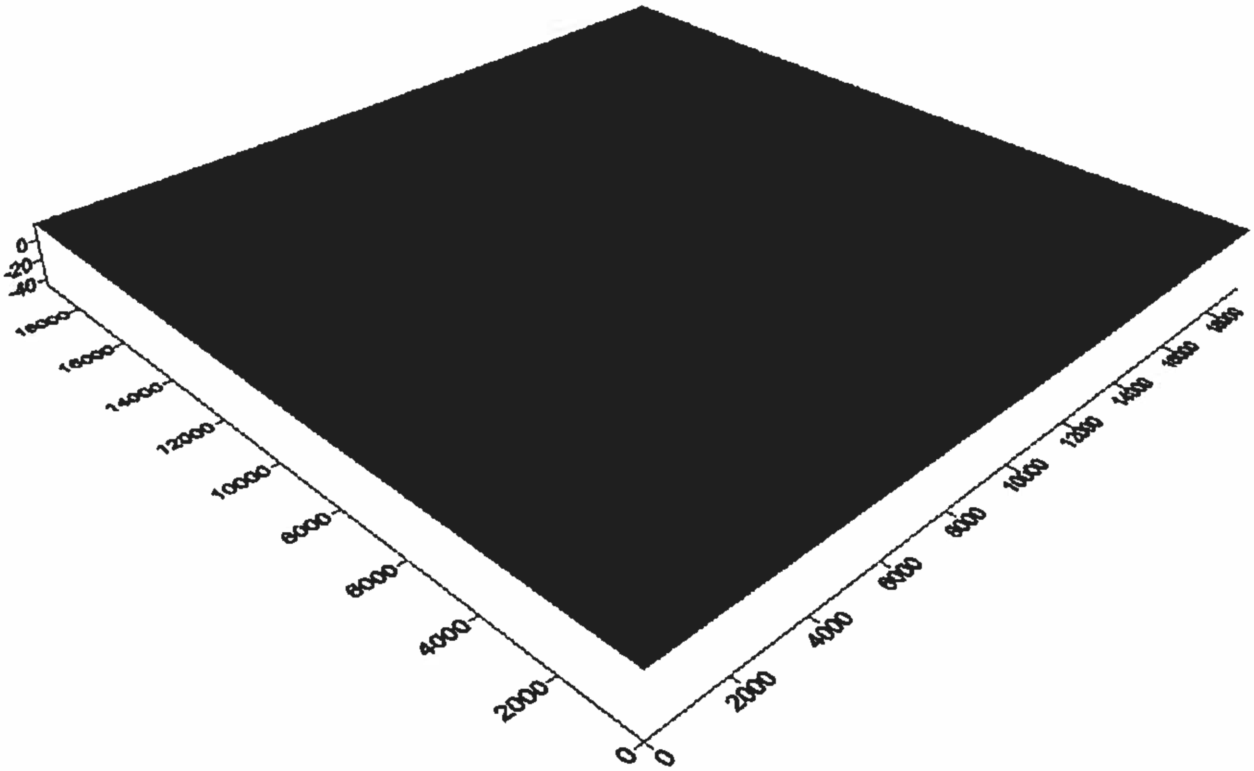

Figure 13. 233 mgd SWRO slant well feed water supply. Eighteen slant well pods and 54 wells. Drawdown distribution in Subsea Aquifers, ft.

SUSTAINABILITY OF SUPPLY

In order to maintain feed water production, planned rehabilitation of a slant well subsurface supply will periodically be necessary. All wells (vertical and angled), need redevelopment from time to time to maintain performance. This periodic redevelopment typically consists of mechanical and/or chemical redevelopment using the same "tried and true" methods developed in the water well industry for vertical wells over the past 70 yrs. As access to the wellhead area is required, provision must be made during siting to minimize disturbance during routine maintenance. This is especially important if the well is sited in an environmentally sensitive area, or in areas where recreation or other high uses exist (e.g., on a State Beach). The frequency between rehabilitation depends on both site-specific conditions and operational schedules. However, it is generally expected to range between approximately three to five years for properly constructed and developed slant wells with corrosion resistant casing and screen.

1.15 Maintaining Well Efficiency

In order to maintain feed water production, planned rehabilitation should be performed when the well efficiency shows an unacceptable decline. Well efficiency is defined as:

$$E = (BQ / s_w) \times 100 \qquad (4)$$

where:

E = well efficiency, %

B = formation loss coefficient, ft/gpm

Q = well discharge rate, gpm

s_w = drawdown in the pumping well, ft

The formation loss coefficient and well efficiency can be calculated from variable rate pumping tests (i.e., step drawdown tests – [17] which is a straight-forward procedure involving at least three different discharge rates. Periodically, step drawdown testing can be performed, efficiency calculated and comparisons made against historical values.

As a general rule, when well efficiencies decline to 50% of the maximum value (at the design production rate), it is a good idea to take the well out of service, perform a video inspection and develop a rehabilitation plan. Based on limited data from the Doheny test slant well, it is expected that in wells properly designed, developed and consisting of corrosion resistant steels, the frequency between well rehabilitation would be on the order of three to five years. However, depending on other constituents in the ground water (e.g., iron and manganese), rehabilitation frequency may vary.

MONTEREY PENINSULA WATER SUPPLY PROJECT

Slant wells are drilled using the dual-rotary method of drilling with angles below horizontal typically ranging from 10 degrees to 30 degrees. A telescoping well design allows construction of slant wells up to 1,000 ft or more with an artificial filter pack and half moon screen pattern[8] typically yielding 3-4 mgd/well. In the Monterey area north of the town of Marina, a 724 ft long test slant well 19 degrees below horizontal has recently been constructed as the first phase of testing of the MPWSP. Prior to construction of the test slant well, a number of exploratory borings were made to define the near-shore aquifers. The test slant well is currently pumping 3 mgd and is being monitored daily for coastal and inland water level and salinity impacts. The full scale feed water supply of 24 mgd will be met using an array of ten slant wells including two for standby capability. Slant well angles can vary depending on site conditions to allow targeting specific aquifer thicknesses. In the case of the Monterey area, aquifers being tested include the shallow Dune Sand and the deeper 180-FTE aquifers. Field testing has validated theoretical analysis and show that shallow angled slant wells have higher discharge rates than vertical wells for the same aquifer thickness due to their increased aquifer efficiency (i.e., broader cones of depression than vertical wells). Data from the long term testing is being used to refine a three dimensional variable density ground water flow and solute transport model which is being used to predict coastal and inland impacts from the full scale project.

SUMMARY

- There is a current misconception that subsurface intakes using slant wells are limited to small scale facilities typically less than 3 mgd. Research and field testing over the past nine years

[8] The half moon screen pattern denotes perforation of only the lower portion of the well screen pipe. This allows for some settlement of the artificial filter pack over the crown of the pipe preventing fine-grained aquifer sands from coming into direct contact with the perforations.

have shown that in typical coastal aquifers in California, slant well feed water supplies can provide approximately 50 mgd of feed water supply per mile of coastline.

- Maximum yield of slant well intakes for SWRO feed water supplies is only limited by the availability of coastline and potential adverse impacts to riparian and onshore resources.

- With improved drilling technology and telescopic designs, slant well lengths can reach and exceed 1,000 ft with individual well yields of 4 mgd and greater.

- Since the first SWRO slant well was constructed off the coast of Dana Point, California in 2006, continuing research and field testing has led to larger scale systems currently in the planning and testing stage. A 724 ft test slant well is currently under long-term testing off the coast of Monterey as part of the first phase of the Monterey Peninsula Water Supply Project.

- Comparison of results from the Doheny test slant well pumping test and the current Monterey test slant well pumping shows that old marine water present in the Dana Point area is absent in the Monterey area. This suggests that widespread subsea coastal aquifers such as present in the Monterey area do not have the same conditions as subsea paleochannels.

- Permitting is the number one constraint to development of subsurface feed water supplies.

- Coastal erosion, 100-yr floods and sea level rise must be considered in the siting and layout of slant well feed water supply systems.

- The Dual Rotary drilling method is a proven technology for construction of artificially filter packed slant wells under the ocean.

- Telescoping slant well design allows for larger pump house casings, proper development and yields of 3-4 mgd per well.

- Slant wells completed in subsea aquifers typically produce over 95% of their supply from ocean water sources (vertical leakage through the sea floor) and lateral flow from subsea aquifers.

- Regular maintenance on the order of every three to five years may be necessary to maintain slant well feed water supply production. Well efficiency declines can be monitored from routine step drawdown testing.

- Slant well maintenance is not complex nor is it more difficult than that which is required for conventional vertical wells and typically includes mechanical and chemical rehabilitation.

- The drawdown distribution in the vicinity slant wells is the algebraic sum of drawdowns for a finite number of point sinks distributed along the vertical projection of the well screen.

- Simplified solutions for the drawdown distribution in the vicinity of slant wells and slant well fields can be developed using the UDEM.

- Variable density ground water flow and solute transport models can predict impacts from full-scale project pumping as well as provide estimates of the time to pump out old marine ground water found in paleochannels.

- Due to slant well geometry, slant wells produce 1.5 to 2 times as much flow as vertical wells for the same available drawdown.

- Silt density indices, one of the major design parameters in desalination feed water supply, are typically < 1 for properly designed and constructed slant wells.

- Coupon testing in a seawater environment show materials such as 2507 Super Duplex Stainless Steel provide long life and minimal corrosion for slant well casing and screens.

- Pumping troughs created by large scale slant well feed water intercept seawater providing seawater intrusion control.

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EXHIBIT G

MUNICIPAL WATER DISTRICT OF ORANGE COUNTY

South Coast Water District

LAGUNA BEACH COUNTY WATER DISTRICT

Moulton Niguel Water

CITY OF SAN CLEMENTE CALIFORNIA

CITY OF SAN JUAN CAPISTRANO CALIFORNIA

UNITED STATES ENVIRONMENTAL PROTECTION AGENCY

DEPARTMENT OF WATER RESOURCES STATE OF CALIFORNIA

U.S. DEPARTMENT OF THE INTERIOR BUREAU OF RECLAMATION

FINAL SUMMARY REPORT
DOHENY OCEAN DESALINATION PROJECT
PHASE 3 INVESTIGATION

Extended Pumping and Pilot Plant Test
Regional Watershed and Groundwater Modeling
Full Scale Project Conceptual Assessment

PREPARED BY

MUNICIPAL WATER DISTRICT OF ORANGE COUNTY

JANUARY 2014

Final Summary Report
Doheny Ocean Desalination Project
Phase 3 Investigation

Extended Pumping and Pilot Plant Test
Regional Watershed and Groundwater Modeling
Full Scale Project Conceptual Assessment

Prepared by

Municipal Water District of Orange County

January 2014

This report was printed on recycled paper.

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MWDOC also wishes to acknowledge the funding support and valuable contributions provided by the MWDOC staffs of the participating agencies throughout the project and their help and guidance provided in numerous technical advisory committee meetings and workshops, without which this project would not have been possible: City of Can Clemente (Dave Rebensdorf), City of San Juan Capistrano (Keith Van Der Maaten and West Curry), Laguna Beach County Water District (Renae Hinchey, Rich Mathis), Moulton Niguel Water District (Joone Lopez, Matt Collings), and South Coast Water District (Betty Burnett, Dave Youngblood, Mike Dunbar, Joe Sovella).

We also wish to acknowledge the project team for all their hard and dedicated work in making this investigation a great success: Carollo Engineers (Gary Meyerhofer, Brandon Yallaly, and Andrew Wiesner), WMI (Dr. Mark Williams), Intuitech (Eric Hoopes), INDAR (Christian Wehrli), SPI (Gerry Filteau, Silvana Ghiu and Mark Donovan), and Geoscience Support Services (Dr. Dennis Williams, Dr. Johnson Yeh, Diane Smith, and Russ Kyle).

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GLOSSARY

AFY	acre-feet per year.
Alluvial/Alluvium	A geologic term describing beds of sand, gravel, silt, and clay deposited by flowing water through which groundwater can readily flow.
Aquifer	A geologic formation or group of formations which store, transmit, and yield significant quantities of water to wells and springs.
Anoxic	A common condition in older natural groundwater where the water is completely devoid of any dissolved oxygen.
ARB	California Air Resources Board
California Ocean Plan	The water quality control plan for the ocean that is established and periodically updated by the State Water Resources Control Board. The plan sets out the standards under which wastewater discharge permits are based upon.
dFe/dMn	Reduced, divalent iron and manganese occur in the dissolved form, primarily as hydroxides in anoxic waters.
D.O.	Dissolved oxygen
Drawdown	The change in hydraulic head or water level relative to a background condition.
Dual Rotary Drill Rig	A water well drilling rig that combines the ability to drill and construct an outer casing to protect the open hole without the use of drilling muds.
DWR	California Department of Water Resources
Evapotranspiration	The combined loss of water from a given area by evaporation from the land and transpiration from plants.
Fault	A fracture in the earth's crust, with displacement of one side of the fracture with respect to the other. Faults may be impervious to the flow of water due to the grinding of adjacent formation materials into very fine sediments.
Fe/Mn	Iron and manganese
gpm	gallons per minute
Groundwater	Water contained in interconnected pores located below the water table in an unconfined aquifer or located in a confined aquifer.
He/Tr	Helium and Tritium isotopes

LBCWD	Laguna Beach County Water District
MET	Metropolitan Water District of Southern California
MGD	million gallons per day
mg/l	milligrams per liter
MNWD	Moulton Niguel Water District
MWDOC	Municipal Water District of Orange County
Natural Isotope Tracer	Naturally occurring radioactive isotopes provide information about a groundwater's age, which refers to the last time the water was in contact with the atmosphere. They can be used to evaluate the sources of pumped groundwater over time.
NTU	nephelometric turbidity units, a measurement of turbidity and clarity of water.
O&M	Operation and maintenance
OTE	Operations, testing and evaluation
R & R	Repair and Rehabilitation
Ranney or Radial Well	A horizontal well built from a central large shaft with radial intakes horizontally pushed out into the formation, usually spaced equidistantly around the circumference of the shaft. These types of wells allow water to be drawn from the lower portion of river or stream channels to maintain yield during dry periods.
RO	Reverse Osmosis. A treatment process that uses high pressure to force water through very fine membranes.
SDCWA	San Diego County Water Authority
SDG&E	San Diego Gas & Electric
SCWD	South Coast Water District
SDI	Silt Density Index, a measure of the suspended solids in water commonly used to measure the clogging potential of feedwater to reverse osmosis membrane systems.
SJBA	San Juan Basin Authority
Slant Well	A water supply well-constructed at a relatively flat angle.

SOCOD	South Orange Coastal Ocean Desalination Project. Former name of the Doheny Ocean Desalination Project.
SOCWA	South Orange County Wastewater Authority
SWP	State Water Project
TDS	Total Dissolved Solids
UCI	University of California Irvine
UF	Ultra Filtration
USBR	United States Bureau of Reclamation
USEPA	United States Environmental Protection Agency
USGS	United States Geological Survey
WHOI	Woods Hole Oceanographic Institute
µ	Micron

A. Project Information

1. Type: Ocean Desalination Feasibility Investigation
2. Title: Phase 3 Doheny Ocean Desalination Project – Extended Pumping and Pilot Plant Test, Regional Watershed and Groundwater Modeling, and Full Scale Project Conceptual Assessment
3. Start Date: January 11, 2008
4. End Date: December 31, 2013
5. Grant and Funding Information:
    a. California Department of Water Resources, Prop 50 Grant Agreement No. 4600007435 for $1,500,000.
    b. U.S. Environmental Protection Agency, STAG Grant Agreement No. XP-00T40501-0, for $848,000.
    c. U.S. Bureau of Reclamation, WaterSmart Grant R10AP35290 for $499,000
    d. Project Participants (South Coast Water District, City of San Clemente, City of San Juan Capistrano, Moulton Niguel Water District) Local Funding totaling $3,300,000.
6. Grantee and Managing Agency: Municipal Water District of Orange County
7. Contact: Mr. Karl W. Seckel, PE, Program Manager; Mr. Richard B. Bell, PE, Project Manager and Principal Engineer
8. Phase 3 Total Project Cost: $6,147,000.

B. Executive Summary

The Municipal Water District of Orange County (MWDOC) in partnership with five participating agencies, investigated the feasibility of slant wells to extract ocean water for the planned Doheny Ocean Desalination Project (aka Dana Point and South Orange Coastal Ocean Desalination (SOCOD) Project). The Phase 3 Extended Pumping and Pilot Plant Test, Regional Watershed and Groundwater Modeling and Full Scale Project Conceptual Assessment work were initiated in January 2008. The five participating agencies provided technical review and elected official decision-maker direction through a project governing committee structure. MWDOC provided overall project management, project development and permitting, technical support work, and staffed the committee.

Project Location and Development of the Doheny Ocean Desalination Project

The Phase 3 test facilities are located in Doheny State Beach in Dana Point, California. The test facilities consisted of the Test Slant Well, submersible pump, control vault, two monitoring wells, conveyance lines, the Mobile Test Facility, electrical service, and a temporary diffuser for discharge to the surf zone.

The full scale project would produce 15 MGD of drinking water (95% operational load factor = 15,961 AFY) and would be situated on a nearby 5-acre parcel being reserved for the project by South Coast Water District. The project site is crossed by the two regional imported supply pipelines and the adjacent San Juan Creek Ocean Outfall has sufficient brine disposal capacity. The major technical issue for the project was to determine the most cost-effective method to produce ocean water.

Figure 1A - Schematic of Test Slant Well

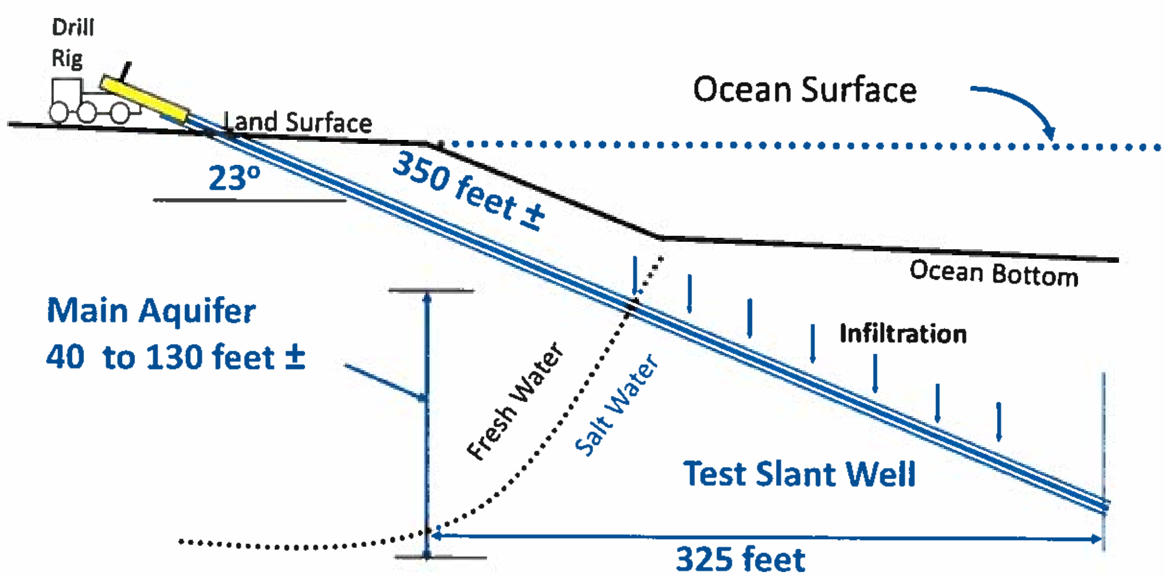


Figure 1B - Schematic of Doheny Desal Project Layout

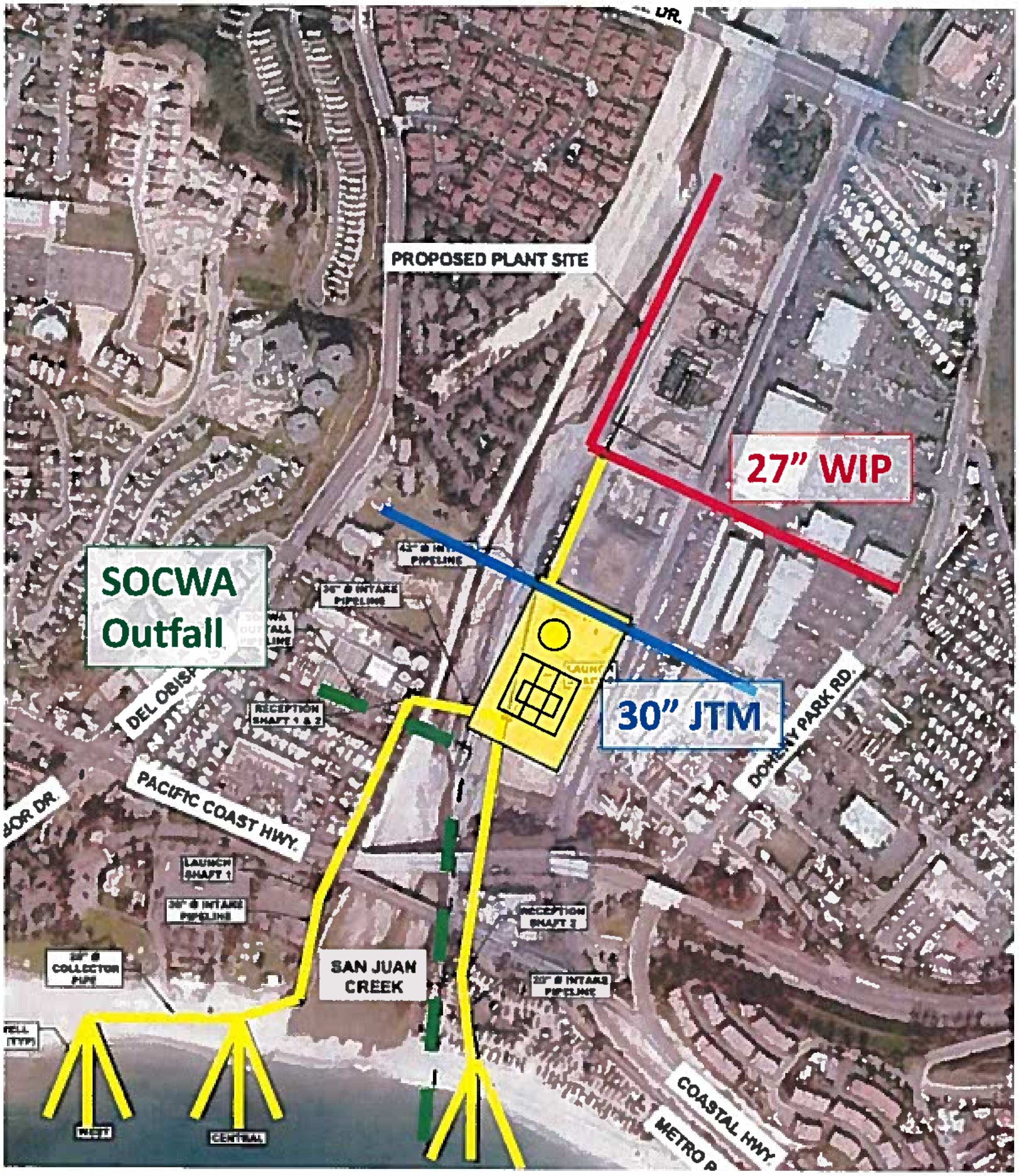


Figure 2 - Schematic of Test Facility

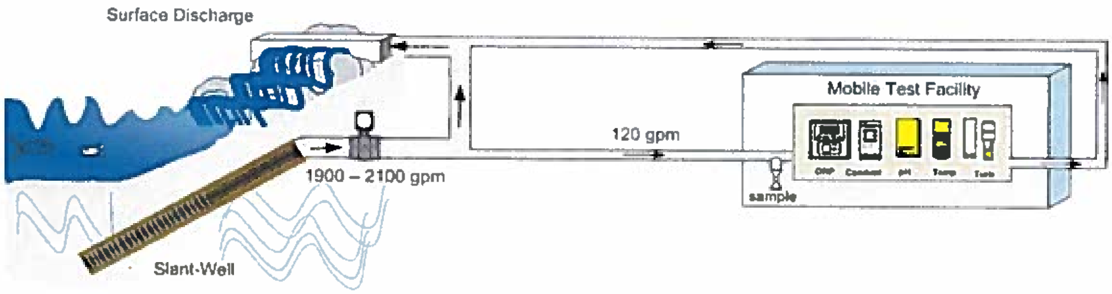
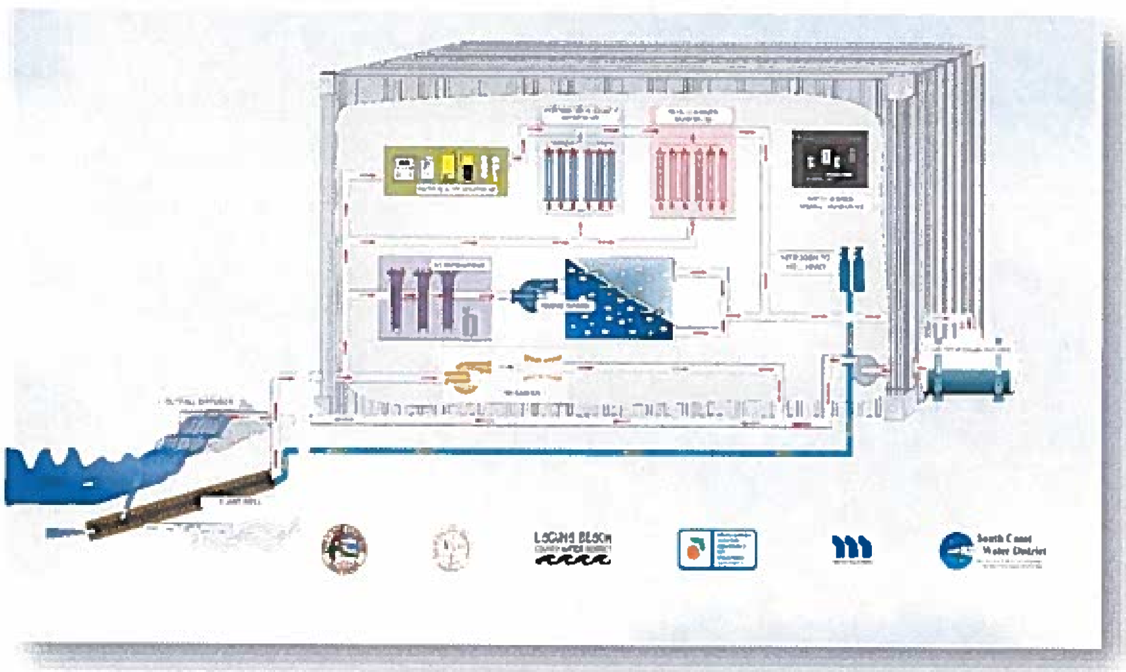


Figure 3 - Layout of Test Facilities

In 2003/04, MWDOC undertook preliminary studies to assess alternative approaches to produce ocean water in the vicinity where San Juan Creek discharges to the ocean in Dana Point. Options included a conventional open intake, a subsurface infiltration gallery, and various types of beach wells. A flat continental shelf in this location would require that a conventional open intake be situated about 7,000 feet offshore to provide sufficient depth for protection of the intake. Due to the

Figure 4 - Mobile Test Facility (MTF)



expected high cost and difficult permitting for an open intake system and based on early discussions with the California Coastal Commission staff, a decision was made to investigate the feasibility of constructing a subsurface intake system using a horizontal or angled well construction method. Infiltration galleries were deemed infeasible due to high costs, ocean floor impacts, clogging, decreasing yields and maintenance challenges. Radial wells (aka Ranney Wells) were deemed infeasible due to high costs, a long construction period that would exceed the 8-month off-season construction window allowed by State Parks, limitations on the ability to gravel pack the laterals, and the limitation to extend the laterals at significance distance out under the ocean.

To investigate the feasibility of a subsurface slant well intake, a phased hydrogeology and subsurface well technology investigation was undertaken. In 2004/05, four exploratory boreholes were drilled along the beach to a depth of 188 feet below the ground surface. The boreholes encountered highly permeable alluvium throughout their depth. In 2005/06, after a thorough review of several technologies it was determined that the most cost-effective approach for this location was the use of slant beach wells constructed with a dual rotary drill rig from the beach out under the ocean. A test slant well was deemed necessary to evaluate the aquifer response, water quality, and aquifer filtration. Groundwater modeling was also necessary to evaluate the impacts of the project draw on the groundwater basin associated with San Juan Creek and to determine the potential capacity of a slant beach wellfield.

In 2005/06 with grant funding support from the California Department of Water Resources, U.S. EPA and U.S. Bureau of Reclamation and MWDOC, a demonstration Test Slant Well was permitted, designed and constructed and a short-term aquifer pumping test was performed. Initial groundwater modeling indicated a full scale slant wellfield could produce about 30 million gallons per day at acceptable drawdowns to wells in the local vicinity. The results from this demonstration well were encouraging and it was then determined that an extended pumping and pilot plant test was necessary.

Phase 3 Extended Pumping and Pilot Plant Test – AN OVERVIEW

The extended pumping and pilot plant test required the installation of a submersible pump, vault with control valves, a diffuser for surf zone discharge of the pumped water, conveyance lines to and from a mobile test facility, and electrical service. MWDOC conducted the planning, environmental documentation and permitting with the assistance of consultants. The mobile test facility was designed by Dr. Mark Williams and the submersible pump was designed by Bayard Bosserman under contracts to MWDOC. The Mobile Test Facility was procured from Intuitech and the submersible pump was procured from INDAR. The remainder of the test facility infrastructure was designed by Carollo Engineers and awarded to and constructed by SCW Contractors. This work was conducted in 2008 to 2010.

Separation Processes (SPI) was the contractor selected for the extended pumping and pilot plant Operations, Testing and Evaluation (OTE) work. They were awarded the work through a competitive proposal/interview process that consisted of staff from the participating agencies and outside experts. The OTE work consisted of pumping the test slant well for a period over 21 months to evaluate the performance of the pump, well and aquifer and to determine water quality produced from the marine aquifer, filtration performance of the aquifer, and corrosion and microbial fouling potential. In addition, the work included iron/manganese pretreatment pilot tests.

The testing work found that the pump and aquifer performed exceptionally well. The well experienced some sand clogging that was due to insufficient well development which was a result of a decision to construct the test slant well with only a 12-inch internal diameter (to reduce costs) and to utilize a high speed submersible pump that would enable a shorter test duration at high pumping rates to adequately stress the aquifer. This problem should not occur in the full scale project as proper and full development would be provided and the well would be equipped with a lower speed production pump.

Over the extended test period, the salinity increased from 2,500 mg/l to over 17,000 mg/l, which was fairly close to what was predicted by the initial variable density groundwater model. It is estimated, that under constant pumping it would have eventually reached about 32,000 mg/l when fully connected with the ocean assuming 95% ocean water at 33,700 mg/l (average of analyses during Phase 3) and 5% brackish groundwater at 2,200 mg/l. The increase in salinity showed that ocean water was slowly being pulled into the well over the test period. A major and unexpected finding was the high level of dissolved iron and manganese contained in the pocket of old marine groundwater that lies under the ocean. This water was anoxic (devoid of oxygen) and slightly acidic, and was found to be about 7,500 years old. From the groundwater modeling work, it was estimated that under full production capacity, the old marine groundwater would be mostly pumped out and replaced by ocean water within a year or so. However, further work is needed to zero in on this time estimate.

The pump out of the old pocket of marine groundwater will likely significantly reduce or potentially eliminate the need for iron/manganese pretreatment. There is also some uncertainty whether the pumped water would remain anoxic under full scale production. In all other respects, the produced water showed a very low silt density index (average around 0.5 units) and turbidity (averaged around 0.1 NTU), indicating excellent filtration by the aquifer which eliminates the need for conventional pretreatment filtration and saves costs.

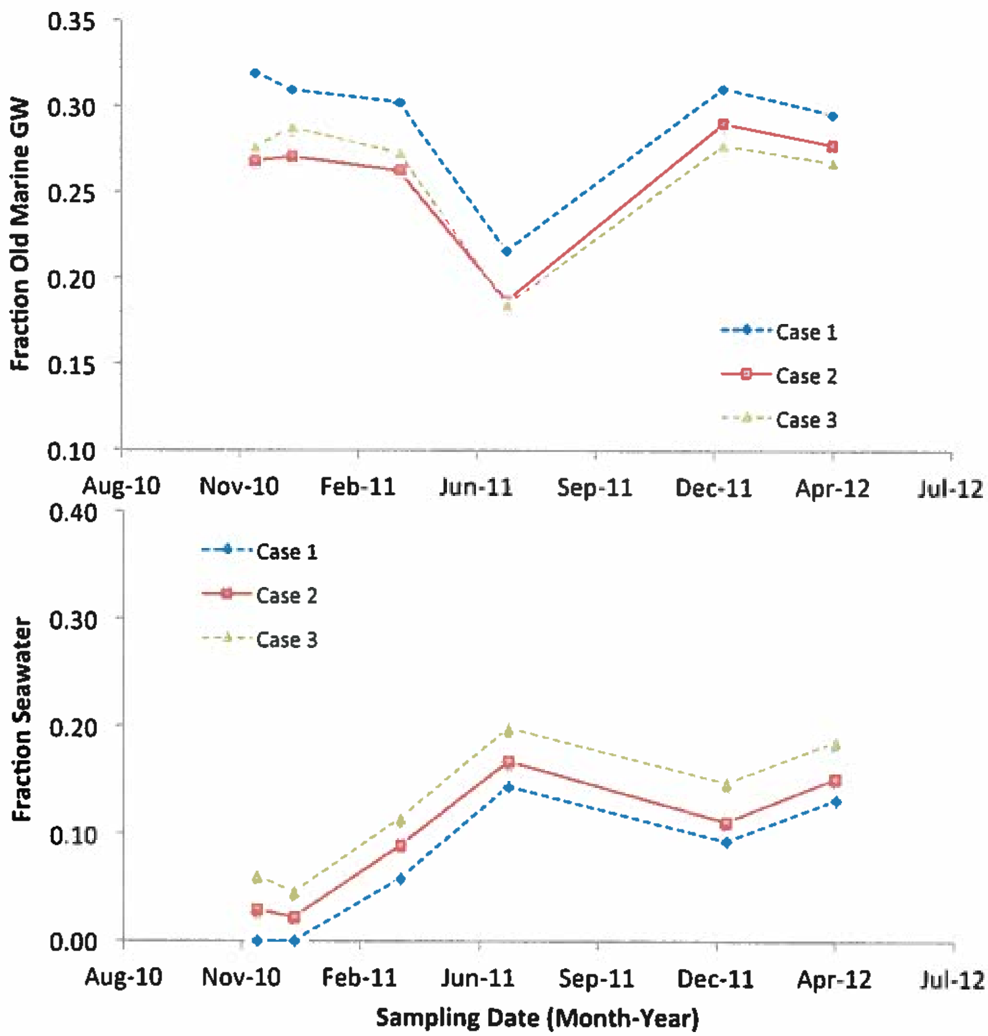
In addition, the produced water showed no presence of bacterial indicator organisms which were found to be present in high concentrations in the ocean and seasonal lagoon. Initial pump out of the brackish groundwater showed higher levels of TOC (Total Organic Carbon) which decreased with increasing production of marine groundwater and ocean water. During the initial period of pump out, a higher level of groundwater bacteria were observed which steadily decreased to extremely low levels. Biofilm growths by the end of the test were found to be less than 10 µ in thickness, a level of no concern for biofouling.

Pumped well water was run directly to the test RO units continuously for over four months. No fouling or performance deterioration was observed during the test or in the post-membrane autopsy as all the dissolved iron and manganese was easily removed as anoxic conditions were maintained throughout the test period.

A pilot plant study was conducted to test advanced iron/manganese removal pretreatment systems. The tested pretreatment processes were oxidized pressure filtration and pre-oxidized UF membrane filtration. Column tests were performed to determine the best media, oxidants, and dosages. Oxidation and sedimentation tests were also performed to evaluate approaches for use during well development to meet discharge requirements. The results showed that the oxidized advanced media filtration process provided higher levels and consistency of removal. A final decision on whether pretreatment would be required must wait until the initial period of pump out of the old pocket of marine groundwater is accomplished. It is recommended that prior to final design, that a final pilot plant test be conducted on the produced water after it has stabilized and the old pocket of marine groundwater has been pumped out.

To determine how much ocean water was being recharged into the aquifer and pumped, natural isotope testing and analyses were conducted throughout the test. This work utilized a multiple tracer approach to quantify the groundwater source captured by the slant well intake. Tracers included natural isotopes of radium, helium, tritium and radiocarbon. Three iterations of a mixing model that utilized the multiple tracer dataset were performed. The model runs suggested ocean water recharge capture was 14-20% by the end of the test with the remainder being a mixture of old marine and brackish groundwater. At the beginning of the test the capture was 0-6%. The 6% range in the model estimates can be narrowed by sampling of the old marine groundwater (see Figure 5).

Figure 5 - Natural Isotope Model - Slant Well Source Production



If the pumping test were to have continued, the old marine groundwater would have been most likely fully pumped out of the offshore formation and replaced by ocean water. Under steady state pumping conditions, there is a high probability that the pumped water would contain very low levels of dissolved iron/manganese. This would result from a combination of the infiltration and plug flow movement of the oxic and slightly alkaline ocean water into and through the aquifer that is reduced to either slightly oxic or anoxic groundwater as a result of microbial activity that consumes dissolved oxygen depending on the amount of available organic carbon. Furthermore, given the observed levels of dissolved Fe and Mn in the old marine groundwater, it is unlikely that their in-situ precipitation from any boundary mixing of oxygenated seawater recharge flows would have a measurable impact on the aquifer permeability at the expected Fe and Mn concentrations, especially under the plug flow conditions that would largely occur. Further, the accumulation of Fe (and Mn) oxides is likely present within the upper shallow aquifer where there is a likely redox boundary where iron precipitation would occur under groundwater ocean discharge conditions. With pumping, ocean water would flow down into the aquifer.

There are two likely locations for precipitation: (1) in the shallow zone of the terrestrial-marine groundwater interface before the water discharges into the ocean and (2) in the shallow sediments on the ocean side of the ocean water interface, where wave and tide driven pore water exchange drive high pH and oxygen rich groundwater into the aquifer. Altogether, under steady-state pumping conditions, this zone would likely contribute little iron to the ocean water that would infiltrate and move through the aquifer to the wellfield. The presence of organic carbon and aerobic bacteria in the shallow seafloor sediments utilizes the oxygen in the ocean water rendering it anoxic, as demonstrated over the extended pumping test. Further evaluation of the organic carbon content in the shallow sediments and sources should be evaluated to determine if the anoxic condition of the recharged ocean water would be maintained over the long run.

Initial Pump Out and Disposal of Old Marine Groundwater

The alluvial channel within the continental shelf offshore of San Juan Creek was submerged by the ocean following the end of the last ice age. Under current conditions, subsurface outflows from San Juan Creek discharge out under and up into the ocean within the area shoreward of the saltwater interface. On the ocean side of this interface, the ocean filled alluvium groundwater has remained isolated since its inundation about 7,500 years ago. We have termed this "older" ocean groundwater as "old marine groundwater".

Testing found that the old marine groundwater is slightly acidic, anoxic and enriched with reduced, divalent, dissolved iron and manganese. Dissolved iron and manganese concentrations increased by the end of the test to a peak of about 11 mg/l and 5 mg/l, respectively. Their concentrations in the old marine groundwater may range from 11 mg/l to as high as 30 mg/l, but the current range is inconclusive due to a lack of offshore aquifer water quality and microbial community conditions.

Water quality and isotope testing provided data to estimate the relative mix by source of the pumped groundwater over the test period. Based on the natural isotope data/model, the pumped water was first mostly brackish groundwater which then steadily decreased as ocean water steadily increased from zero to about 17%, and old marine groundwater. The fraction of old marine groundwater started out at zero, reached an apparent maximum of about 29% before decreasing and in time would have been fully replaced by replaced by recharged "young" ocean water. See Figure 6 for an illustration of how the change in source water would occur over time. Under the full production rate of 30 mgd ocean water recharge would be greatly accelerated from what was observed under the Phase 3 test of 3 mgd.

As illustrated, the source of water being pumped out will continually change in make up until it reaches a steady state condition. For the full scale project, initial modeling suggested that under steady state conditions the extracted well water would reach about 5% brackish groundwater and about 95% ocean water ("young" marine groundwater).

The Phase 3 test data is planned to be utilized in the calibration of a fine grid coastal groundwater flow, variable density, and geochemical model. The fine grid model will help to better predict pumped water quality over time and by source, to evaluate drawdown effects, and seawater intrusion and controls.

Under the full scale project, during the period of initial pumping when the pocket of old marine groundwater is being pumped out and replaced by "young" ocean water, there are two major questions:

(1) How long will it take to pump out the pocket of old marine groundwater?

(2) What is the best approach for handling the old marine groundwater?

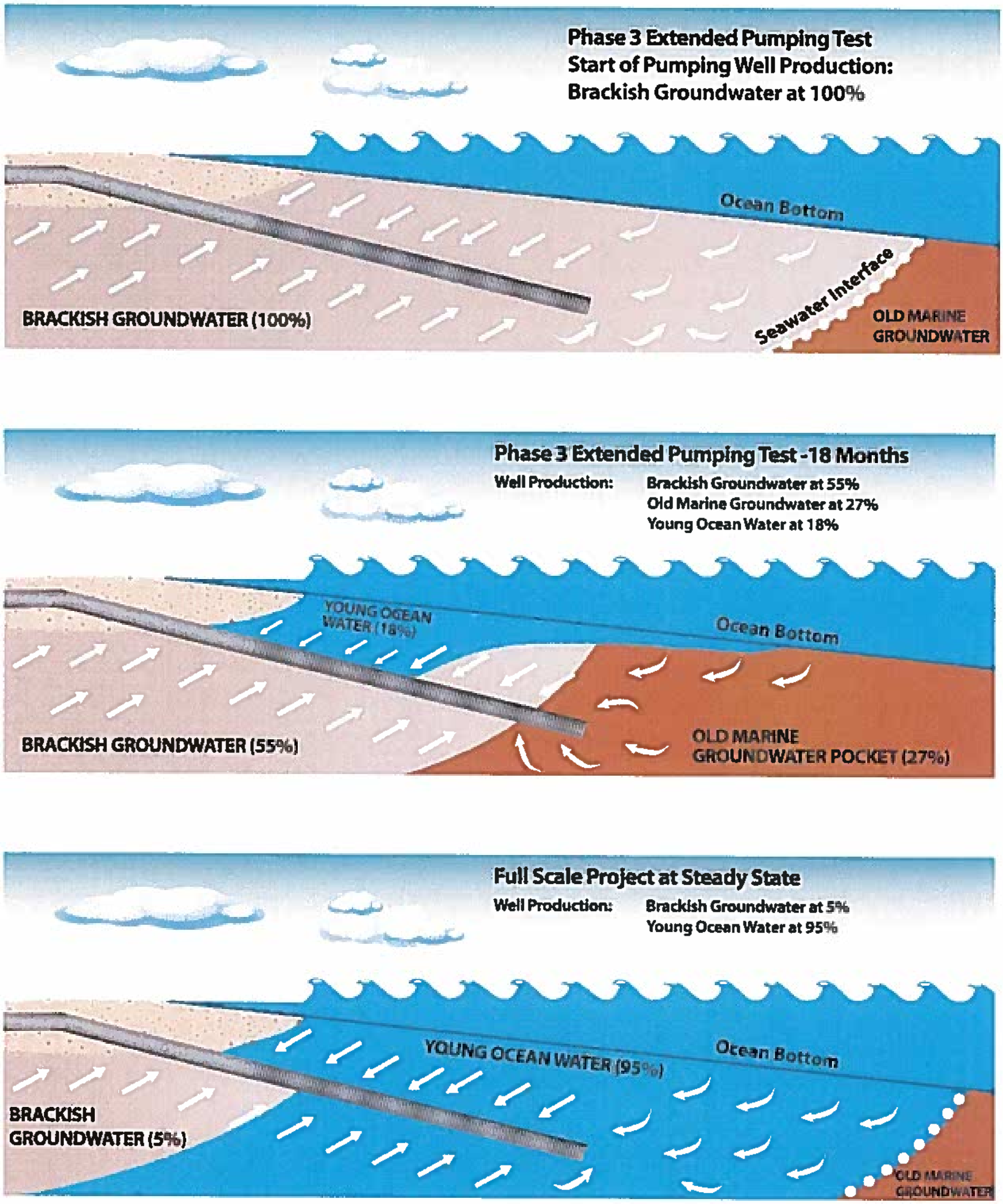
We see two basic approaches for construction of the full scale 30 mgd slant well intake capacity project: (1) include in the desalination plant an iron/manganese pretreatment unit (capital cost estimated at $50 million), or (2) pump out the old pocket of marine groundwater before completing the design and construction of the desalination plant, since it is expected that levels will drop significantly under steady state conditions to levels which will either significantly reduce or avoid the need for Fe/Mn removal.

In addressing the first approach, Arcadis (Malcolm Pirnie) assumed that the steady state iron concentration would remain constant at 6 mg/l and developed capital and O&M cost opinions for handling this amount of dissolved iron. This approach assumes a constant high level of iron/manganese throughout the project life. This is unlikely the case.

It should be noted that during the Phase 3 test, the iron concentration in the pumped water reached 11 mg/l and was fairly constant for several months. However, when considering the full scale project slant well intake production rate of 30 mgd, based on initial modeling, it would be expected that the old marine groundwater would be pumped out in about one year, reducing the concentration of iron/manganese in the feedwater to very low levels. As previously noted, the fine grid, variable density, geochemical model will aid in better understanding the old marine groundwater pump out time as well as aiding in understanding changes in water quality during the pump out period and what might be expected under steady state conditions.

For the second approach to be feasible, we need to better know how long it will take to pump out the old marine groundwater until it is fully replaced with "young" ocean water and reaches steady state conditions. During the Phase 3 test, the iron levels increased steadily and then stayed relatively constant after reaching about 10 mg/l after 8 months of pumping and then slightly increased to 11 mg/l near the end of the test; the increasing amount of "young" ocean water and the slightly decreasing fraction of old marine groundwater kept the iron concentrations relatively flat over the last year of the test. The isotope data showed a slightly decreasing fraction of old marine groundwater being pumped over the test, as the "young" ocean water recharged the marine aquifer area where brackish groundwater had discharged out under the ocean. The location of the seawater interface was previously estimated at about 1,100 feet offshore under 2005 wet hydrologic conditions and lower basin pumping. For comparison, it is worth noting that the estimated volume of the brackish water from the shoreline to the saltwater interface was about 1200 AF (at a specific yield of 10 percent) under 2005 conditions and over the Phase 3 test the pumped volume of brackish water was estimated at about 3,600 AF out of a total volume of 5,286 AF by a salinity model that used actual test data (see Figure 6).

Figure 6 - Illustration of Slant Well Source Water Production vs. Time



Modeling will be required to evaluate the change in fraction of source water reaching the full scale project wells as a function of pumping rate and duration. Based on the earlier Phase 2 modeling, it had been roughly estimated that the old marine groundwater could be fully pumped out within about a year or so at the much higher 30 mgd production rate. The fine grid model will improve this estimate. At steady state after pump out of the old marine groundwater, the wells were predicted to produce about 95% "young" ocean water and 5% brackish groundwater.

The blended concentration at steady state is expected to be low from the large dilution of the "young" ocean water component. The iron/manganese concentrations at steady state are largely dependent on the concentration of iron/manganese in the brackish groundwater reaching the wells and if there is any trace amount of old marine groundwater remaining. Ocean water in the vicinity of the project is fully oxidized and would be expected to have a very low level of iron/manganese (levels are higher near the shoreline and decrease offshore away from San Juan Creek). As the ocean water is recharged into the aquifer, it is anticipated that the ocean water will pick up some dissolved Fe. Under steady state conditions, the produced water is expected to have a dissolved iron concentration around 0.10 mg/l assuming brackish groundwater iron at 2.0 mg/l. At this low total iron concentration the RO membrane should not have a problem removing any oxidized portion of the dissolved iron/manganese in the produced water. However, some chemical conditioning may be required to minimize cleaning. If higher concentrations occur, higher oxidized media filtration rates than assumed by the Arcasdis cost estimate could be used to remove iron/manganese at much lower capital and O&M cost.

If an injection barrier is found to be necessary to reduce drawdown impacts, in time both the injected and slant wellfield produced water would likely be largely free of dissolved iron/manganese.

Further fine grid flow, variable density and geochemical modeling is necessary to provide a better estimate of the pump out time, to estimate produced water quality over time, and to estimate pumped water quality under typical or steady state conditions. Offshore hydrogeology borehole lithology and water quality data and geophysical surveys for alluvial channel structural data will be necessary to fine tune these estimates during the project design, but are expensive to obtain. With operational data, the best method of handling the old marine groundwater iron/manganese loads can then be determined.

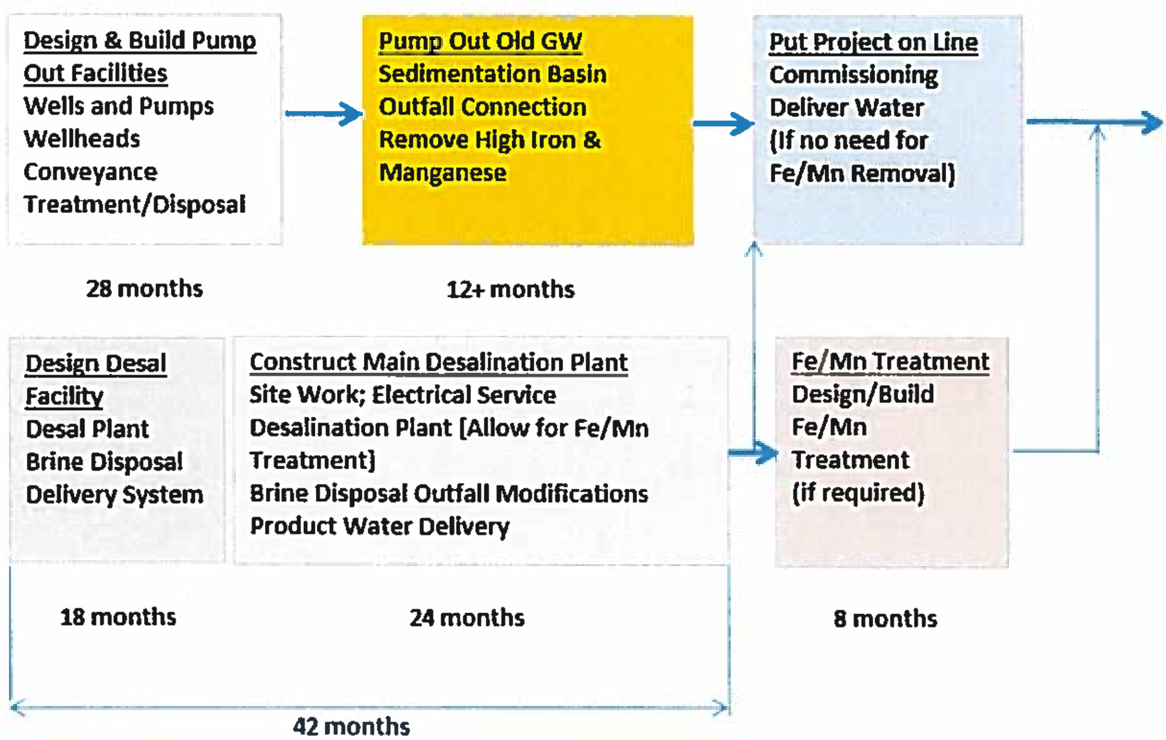
Assuming that the old marine groundwater can be pumped out in about a year or so under full scale production at 30 mgd, the second approach would be preferred. This approach would require that the project be constructed in two stages: (1) wellfield, conveyance and disposal system constructed and operated to pump out the old marine groundwater, complete pilot plant testing to finalize feedwater quality for treatment process design, and (2) complete construction of the remainder of the project. This may be necessary in any event due to the unknown steady state pumped water quality.

During the initial period of pump out of the old marine groundwater, it would be necessary to install a system to remove iron/manganese to levels that can meet discharge requirements through the SOCWA ocean outfall. The current NPDES permit does not have an iron/manganese numerical discharge limitation, but does have limits on settleable solids and turbidity, which would be impacted by the discharge of oxidized iron/manganese. This operation would require permitting through SOCWA and under its NPDES discharge permit.

To meet discharge requirements, iron/manganese will need to be reduced to acceptable levels in a cost-effective manner. During the Phase 3 iron/manganese pilot plant testing work, data were obtained on the effectiveness of oxidizing soluble iron/manganese followed by sedimentation to reduce the iron/manganese load. It was found that chlorine addition was necessary to provide effective oxidation followed by sedimentation at 15 minutes detention, which nearly fully removed all the iron and manganese. The cost for this short-term operation, for one year would include the costs for outfall use, slant well pumping energy, outfall O&M, ocean monitoring, and treatment equipment with chemicals and O&M. The cost for one year of operation is estimated around $4.5 Million. If a longer period is required, a second year is estimated to cost about $3.5 M. Compared to the cost of installing a full scale iron/manganese removal plant at $50 Million, the two stage approach is warranted.

Figure 7 "Full Scale Project Design and Construction Staged Implementation" illustrates the sequence for the major design and construction activities for the full scale project following the recommended approach to pump out the old marine groundwater prior to a decision on Fe/Mn treatment.

Figure 7 - Full Scale Project Design and Construction Staged Implementation



Regional Watershed and Groundwater Modeling

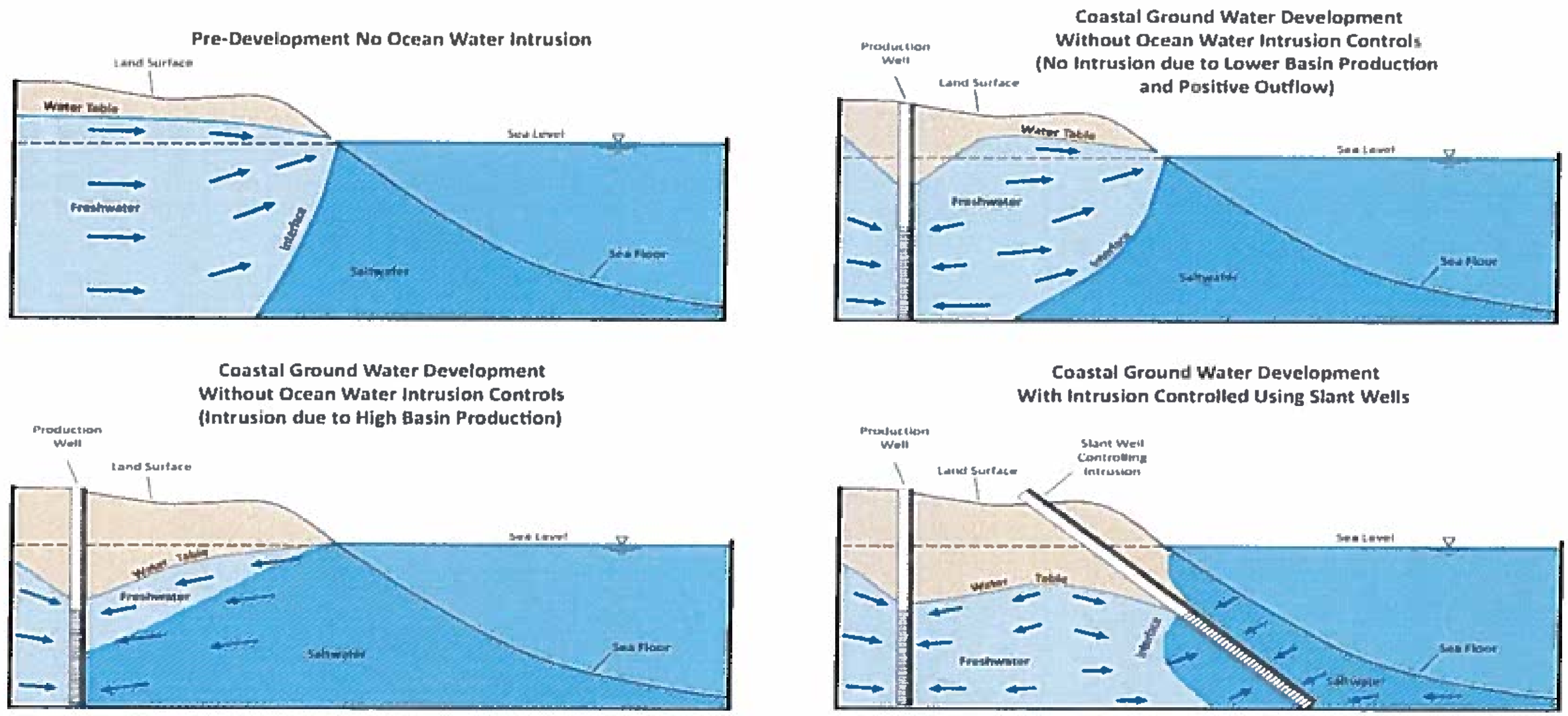
In this location, the paleo San Juan Creek alluvial channel extends out under the ocean within the continental shelf for about three miles. This paleo-channel offers a permeable connection to the ocean. The slant wells would tap into this alluvial structure to pull in filtered ocean water. Under steady state conditions, about 5% of the pumped water would be pulled in from the landward portion of the aquifer, which is brackish groundwater. Groundwater development of the Lower San Juan Basin has occurred over the last several years with the construction of two groundwater recovery desalter plants. To determine the Doheny Desal project impact on the basin and the desalter plant wells, it was necessary to develop analytical models to evaluate drawdown and groundwater take impacts on the basin.

To determine these impacts, a regional surface watershed and groundwater model was developed to determine the basin operable yield using a 64 year hydrology record (1947-2010) which included a 31 year dry period. The first tasks were to determine the basin operable yield without the ocean desalination project. This work which required nearly three years of effort, determined that the lower basin total storage capacity is about 46,000 acre-feet, about 12% less than previously estimated by DWR in 1972 and that the actual volume of water in storage in 2010 was about 30,000 af. The modeling also showed that basin yields over an extended dry and average periods would be about 8,040 AFY and 9,150 AFY, respectively, less than previously believed. Over the 64 year hydrology, it was found that basin storage levels would drop to about 25% of capacity during the long dry period and would refill relatively rapidly under average and wet periods. The model also indicated that seawater intrusion would occur over both dry and average conditions and would reach the SCWD wells in 9 to 12 years, assuming the higher production levels at the long-term sustainable yield levels, rendering them inoperable if additional desalination process treatment were not constructed. Accounting for the seawater intrusion would reduce the yields noted above by 300-400 AFY. Further work is necessary to refine these estimates.

As previously noted, about 5% of the 30 mgd slant well field production (about 1,660 AFY) would be basin brackish groundwater. In addition, the slant well field would provide seawater intrusion control through a coastal trough created from pumping. To mitigate the drawdown and take impacts on impacted producers, make-up water from the desalination project up to 1,660 AFY could be provided to them, less the amount that the basin would otherwise have to use to curtail production to avoid seawater intrusion impacts. Also, seawater intrusion control benefits that would be provided by the Doheny Desal Project should greatly reduce or fully avoid SJBA seawater intrusion control costs.

Future detailed coastal groundwater and geochemical modeling are required to fine tune drawdown impacts and to predict pumped water quality over time. This work will also evaluate physical mitigation using injection wells to create an artificial barrier by raising groundwater levels in the coastal area. This analysis will help to determine the least cost mitigation approach. Other work by the SJBA will investigate the ability to augment the groundwater supplies through stormwater conservation and recycled water and means to protect against seawater intrusion. The two monitoring wells constructed by MWDOC in Doheny State Beach should be maintained and used to monitor for seawater intrusion under upstream groundwater operations.

Figure 8 - Illustration of Seawater Intrusion and Extraction Control



Full Scale Project Conceptual Assessment

The full scale Doheny Desal Project will consist of five major components: (1) feedwater supply system, (2) power supply, (3) desalination plant, (4) brine disposal and (5) system integration. Following is a brief description of each major system component.

Feedwater Supply System. At this time, it is expected that 30 MGD of ocean water supply can be drawn from a slant beach well system consisting of nine wells constructed in three clusters of three wells each along the mouth of the paleo-channel of San Juan Creek along Doheny State Beach. The wells will be fully buried and will extend out under the ocean. Seven wells will be fully operational with two standby wells for operating flexibility and redundancy. The slant wells, wellhead vaults, submersible pumps, power supply, instrumentation cables, nitrogen feed lines, and conveyance pipelines will all be fully buried. Since the wells will be constructed on Doheny State Beach, the construction and maintenance periods are restricted to the off-peak recreational use season, September 15 to May 15.

The wells will be constructed from the beach upslope of the ordinary high water line near the back of the sandy beach, at a 23 degree angle from horizontal, fully penetrating the offshore paleo-channel alluvial deposits. The preferred construction method is Dual Rotary Drilling which avoids the need for drilling muds by advancing an outer pipe shield casing that also prevents cave ins. The well lengths will be approximately 520 feet, consisting of about 280' of 24-inch diameter blank pump housing and 240' of 12 to 16-inch diameter well screen. The long pump housing permits maximum drawdown and yield.

The wells will be constructed in arrays of three wells each with a single construction location and common well vault. The three vaults will be buried to a depth of about five feet below the beach. The vaults will contain the well headers, distribution pipeline, well spools for well cleaning, control valves, flow meters, check valves, isolation valves, nitrogen gas feed lines, and power and instrument cable connections. The nitrogen gas is required to prevent air being pulled into the well in order to minimize any potential oxidation of dissolved iron and manganese prior to the treatment processes.

Preliminary vault drawings are shown in Figure 9. Acoustical damping of the submersible pump noise to very low levels on the beach may be required.

Conveyance from the slant wells to the Desalination Plant site will be by pipeline/tunneling. Preliminary alternative alignments were identified in the Boyle Engineering Corporation Engineering Feasibility Study (March 2007). Two candidate alignments were recently laid out and costs estimated by Kiewit. A collection pipeline to each of the three well vaults will parallel the shoreline and then combine into a single line to cross under PCH and/or cross under San Juan Creek and then to the Desalination Plant. Excavation and microtunneling construction methods, with launch and reception shafts for construction under the beach, PCH and San Juan Creek will be required. The conveyance system will terminate at the Desalination Plant at the Feedwater Supply High Pressure Pumping Station. This pumping station must be in-line without a wet well to prevent air entrainment and oxidation of iron/manganese which is expected in the feedwater at low concentrations, at least during the initial start-up period.

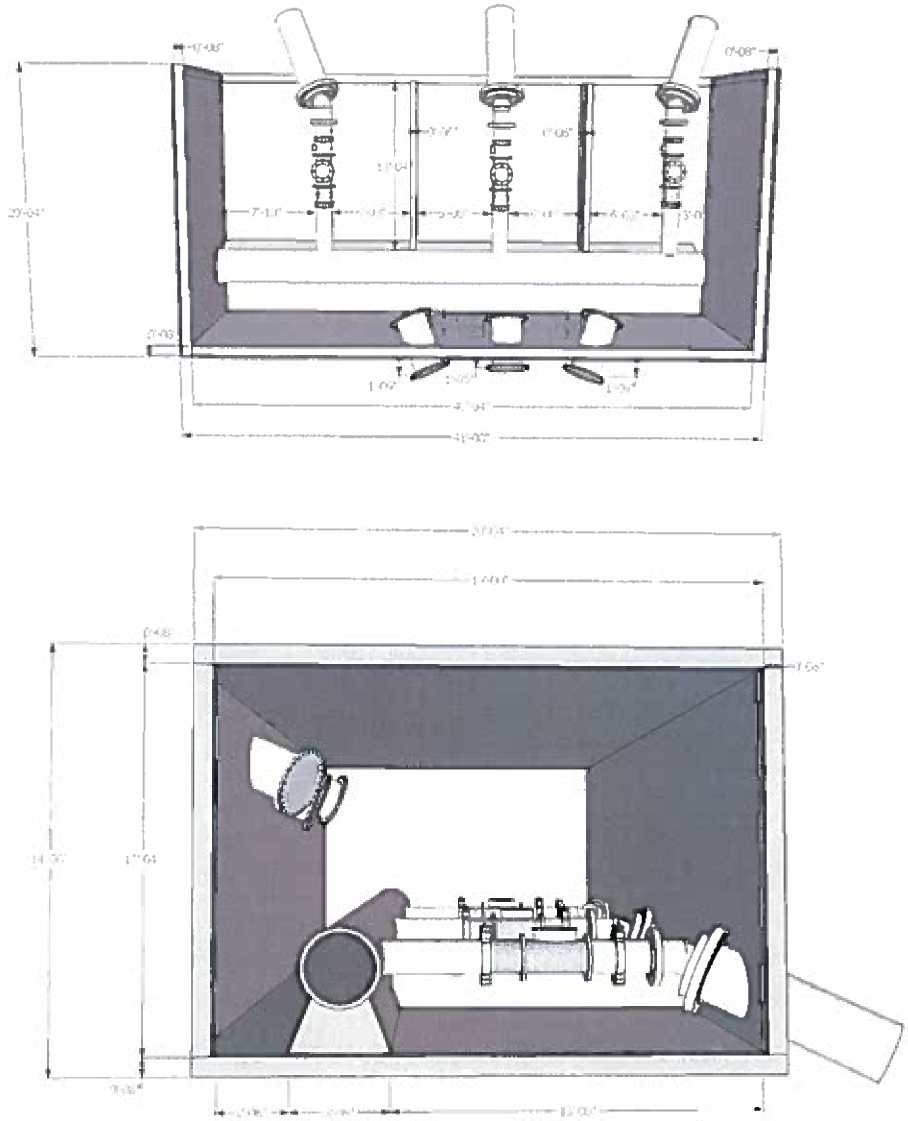


Figure 9 - Top and Side Views of Conceptual Wellhead Vault

Power Supply. Electrical service to the facility will be provided by San Diego Gas & Electric Company. SDG&E prepared an "Engineering Study for Electric Service at the Dana Point Ocean Desalination Plant" dated March 2007. An updated study will be required and is being discussed at this time. Based on an estimated load of 8.3 MW, one to two 12kV transmission circuit feeds would be extended to the plant site, with transformer, panels, cables and meter. About 1,000 feet in new trenches for 4-5" conduits would be required to extend existing feeds to the plant site. Additional facilities and equipment to step voltage down to 4kV or lower voltages would be the responsibility of the project and would be placed on the desalination plant site. The capital cost of these facilities is about $700,000 with the bulk of the power supply costs being built into the rates by SDG&E. The full options for power service will need to be evaluated. In addition, it may be possible to enter into a "demand shedding" agreement with SDG&E

for short-term "called" interruptions in the power supply to help them manage loads during peak demand periods. In exchange, a discount on the energy rate is provided. These options have not been fully explored at this time. Clearwell storage and/or reservoir storage would be used to maintain supplies during the few hours of "load shedding".

Renewable energy capabilities at the site and within the ocean are quite limited. Solar panels may be placed on the building roofs, but would only support minimal energy needs. Wave energy is considered infeasible in this location. Third party wheeling of renewable energy sources developed outside of the area is not available to water utilities at this time. Further, it would be expected that the costs for these types of renewable projects would be higher than what the electrical utility can develop. If the same requirements are placed on the project as incurred by the Poseidon Resources project, offset energy would be required to make the project carbon neutral with imported water deliveries. The cost of providing this mitigation is modest, estimated at about $50,000 per year.

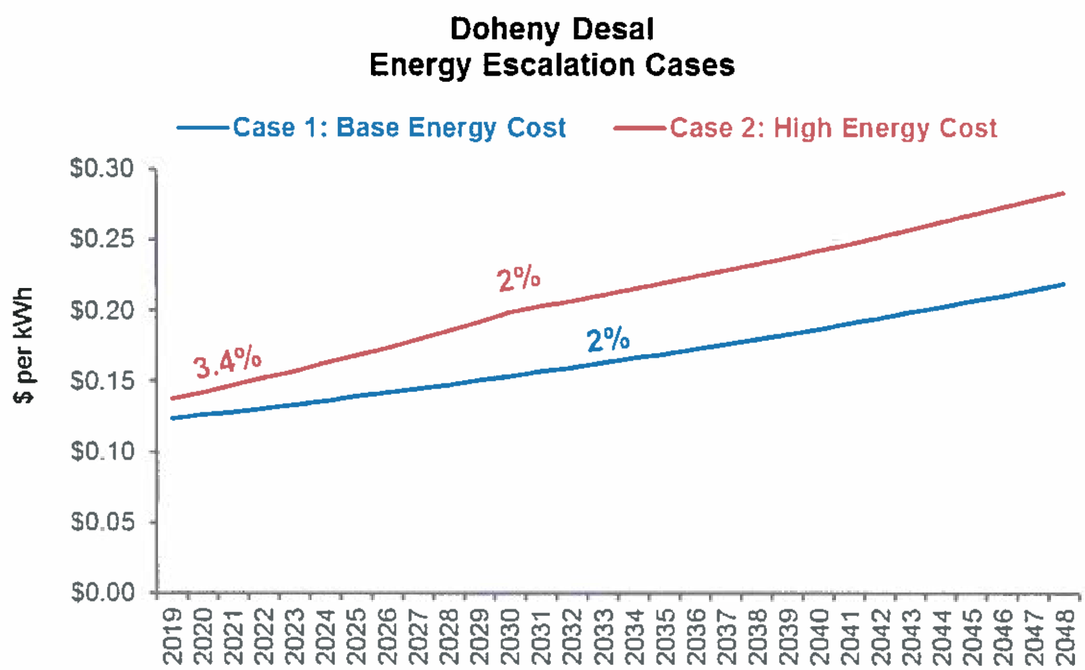
Projected Cost of Electricity for the Plant. Electricity charges are projected to bump up over the next 7 years and then level off due to several coincidental factors. There are three main causes for the bump up in rates: (1) California's mandate to achieve 33 percent renewable energy by 2020 which includes solar, wind and ocean generation, energy storage, and new transmission and distribution facilities, (2) phase out of once-through cooling systems and retirement of older inefficient generation facilities, and (3) closure of the San Onofre Nuclear Generation Station. Long-term estimates of electrical energy costs to supply the plant are difficult to make in California given the uncertainty in how far California will pursue renewable energy goals beyond the 2020 mandate, the effect of future increased distributed user generation and storage systems, long-term natural gas fuel prices, efficiency standards and usage, future population and economic growth drivers, and general inflation.

For the Doheny Desal economic analysis, two rate projection scenarios were evaluated. These rate projections were developed by SDCWA in July 2012 for their energy cost analysis for the Carlsbad Desalination Project and are considered applicable at this time. It should be recognized that actual energy prices will likely be higher or lower than the forecasts. It should be remembered that the Doheny Desal would be a base-loaded 24-7, 365 day per year operating facility. Recent changes by SDG&E in their cost of service have favored these types of facilities compared to typical residential customers, which has resulted in a lowering of the rates. The two cases analyzed are:

- Base Case 1 – Assumes significant RPS (renewable portfolio standard) and AB 32 implementation with electricity cost escalation at 2% annually through 2030 (5 successive 6% rate case increases from July 2012 – actual rate effective in July 2012 was 10.5¢ per kwhr) and then at 2% thereafter. The first bump up in rates occurred in late September 2013 when the AL-TOU rate increased from 10.54¢ to 11.54¢ per kwhr, a 9.5% increase in 15 months (7.6% annualized rate of increase).
- Higher Rate Scenario Case 2 – Assumes high RPS/AB 32 implementation with electricity costs escalation at 3.4% annually through 2030 (6 successive 10+% rate case increases from July 2012) and then reversion thereafter to 2%.

Figure 10 below shows a comparison of the two rate forecasts. Since energy costs account for about 30% of the project cost, the issue of future energy costs needs to be carefully tracked. Depending on future regulatory policy, renewable technology advancements, and shale gas production and natural gas prices, self-generation or investments in outside projects to deliver the energy to the site may be viable options, but competing with SDG&E at their cost of energy and based on the level of reliability they bring will be difficult.

Figure 10



Desalination Plant. The Desalination Plant site is a 5-acre parcel situated on the east side of San Juan Creek just north of PCH on land owned by South Coast Water District. This parcel is situated within the jurisdictional boundary of the California Coastal Commission under the category of "Appeal Jurisdiction". The parcel is currently rough graded to an elevation of approximately 22 feet msl. A geotechnical study is required to determine the design measures to reduce geotechnical hazards from either an earthquake, flood or tsunami. It is anticipated that the site will need to be raised to provide flood control protection with an allowance for sea level rise. 100 or 200 year storm flood protection and flow criteria will need to be determined for protection of the site. In addition, it is anticipated that the site will need to be excavated, compacted and stabilized to provide an adequate foundation for the facility structures.

The Desalination Plant will consist of the following main system components: (1) Electrical Service Sub-Station and Equipment, (2) High Pressure Feedwater Supply Pumping Station, (3) possible Pretreatment Facilities, (4) Reverse Osmosis Desalination Building and Equipment, (5) Post-Treatment Facility, (6) Concentrate Brine Holding Storage and Discharge Connection to the adjacent San Juan Creek Ocean Outfall, (7) a potable clearwell reservoir and (8) a booster pumping station. The site will also consist of roads, parking areas and other related storage, equipment, chemical storage and feed system, and related appurtenances. The structures will need to be constructed in an architecturally pleasing style fitting to the area and will be constructed to be energy efficient with possible solar roof panels and/or green roofs and other related "green" energy systems.

The plant will receive feedwater at 30 MGD. Due to the limitations on yield, it is recommended that a recovery rate of 50% be designed in order to yield 15 MGD of product water. Energy recovery pressure exchanger devices will be utilized to recover 95% of the energy in the high pressure brine stream.

Subject to regulatory and economic feasibility, the Doheny Desal project may be designed to recover the RO concentrate streams from the City of San Juan Capistrano and South Coast Water District groundwater recovery plants by using those flows as feedwater. It is estimated that both of these plants will be enlarged from their current combined 6 MGD capacity to 10 MGD in the future, producing about 2 MGD of brine at a concentration of approximately 10,000 mg/l. This could result in an increased Doheny Desal Project plant yield by up to 1 MGD. This approach appears promising as it would reduce costs to both the City of San Juan Capistrano and South Coast Water District and to the Doheny Desal Project. The feasibility of an integrated brine recovery plan should be evaluated.

Post-Treatment for the RO permeate will be required to stabilize the water so that it is not corrosive to the distribution system. The standard method is to add in lime to the permeate to produce a stabilized water. Some locations, such as Israel now also require the addition of magnesium to achieve a more balanced cation mix. One option that will be considered for regulatory and economic feasibility is to further condition the water with about 1 MGD of brackish water, potentially from one of the SCWD wells, treated for removal of dissolved iron and manganese, disinfected and blended back with the permeate. This will allow production of water that more closely resembles in quality imported water, including providing a more natural blend of cations (calcium, magnesium, potassium) and anions (carbonate, bicarbonate, chloride, sulfate). Additional stabilization with respect to calcium carbonate saturation will be required.

Product water quality criteria will be developed for the desalination system. Key considerations are the level of bromide and boron in the product water. A second pass system at a minimum of 40% capacity is being planned to lower bromide to acceptable levels that prevent accelerated decay of chloramine disinfection residuals in the finished water. Boron levels will also be reduced when achieving the bromide levels. This will provide a product water that is fully protective of ornamental landscape plants.

Brine Concentrate Disposal. The waste brine concentrate from the Reverse Osmosis unit process will be co-disposed with treated municipal wastewater in the adjacent San Juan Creek Ocean Outfall. Due to the diurnal flow pattern of the wastewater flows, a regulatory storage basin at the desalination plant will be required. The concentrate will have a concentration of approximately 66,000 mg/l and will be combined with wastewater having a concentration about 800 mg/l. The current average dry weather municipal wastewater flow in the outfall is 17 MGD. It is anticipated that this flow rate will decrease in the future with additional upstream recycling.

The SWRCB (State Water Resources Control Board) is in the process of amending its California Ocean Plan for Ocean Desalination Intakes and Brine Disposal. When the plan is amended it is anticipated that more stringent requirements for brine discharges will be required.

The ocean outfall diffusers may need to be modified to meet the new SWRCB Ocean Plan Amendment requirements. Modifications might include new diffusers, such as tidal or rosetta valves, or other diffuser devices to increase initial dilution to meet new regulatory requirements. The San Juan Creek Ocean Outfall has an estimated hydraulic capacity of 85 MGD. Plant operations and brine disposal will be ceased only during major storms when total wastewater and infiltration/inflow rates exceed the ability to discharge the brine. This is a rare event and only occurs during very wet years when the collection system trenches are saturated and when stormflows greater than an estimated 25 year intensity occur.

The existing outfall requires structural improvements at the ocean junction structure and at the surge chamber connection from the Latham Plant to the outfall where it joins with the Santa Margarita Water District land outfall on the east side of San Juan Creek. These improvements would be undertaken by South Orange County Wastewater Authority as they are needed for wastewater disposal. The brine concentrate line would connect to the surge chamber structure which is located adjacent to the project site. Flow and water quality monitoring will be required for the discharge. SOCWA approval is required. For project participants not discharging wastewater to the San Juan Creek Outfall, it will be necessary to acquire capacity in the system. The current San Juan Creek Ocean Outfall capacity and ownership are shown in the following Table 1. Cost allowances for the outfall capacity have not been included in the Project Cost Estimate because final capacity selection by agencies have not yet been made and nor has an engineering study been completed, which needs to be held off until the new SWRCB Ocean Plan Amendments are finalized.

Table 1 – SOCWA San Juan Creek Ocean Outfall – Agency Ownership

Agency	Ownership Percentage (%)	Capacity Ownership (mgd)	
		80 mgd	85 mgd
Moulton Niguel WD	15.51	12.42	13.18
San Clemente	16.62	13.30	14.13
San Juan Capistrano	11.08	8.86	9.42
Santa Margarita WD	44.32	35.46	37.67
South Coast WD	12.47	9.98	10.60
	100.00	80.00	85.00

Ref: SOCWA Hydraulic Capacity Evaluation, Carollo Engineers, June 2006

System Integration. The project water will be pumped into the Joint Transmission Pipeline and the Water Importation Pipeline. The hydraulic grade line is approximately 450 feet in both pipelines. Both pipelines cross near the Desalination Plant site on South Coast Water District property, requiring short pipelines to the two points for interconnection. Connections to Laguna Beach County Water District will require a small pump station addition at the existing SCWD/LBCWD interconnection station. Some additional provisions to assure maintenance of the disinfection residual at sag points may be required.

Conceptual Level Cost Opinion

Arcadis (Malcolm Pirnie) prepared a conceptual level cost opinion update for the project in 2011. The cost estimate was modified for the RO system cost, based on cost reviews provided by three firms.

Operation and Maintenance costs were estimated for labor, replacements and repairs, chemicals and feed systems, maintenance materials, and energy. These costs are shown in Table 2. Without energy, the O&M costs are estimated at about $5.8 million per year which is equal to $363/AF. Energy costs are estimated at $7.1 million per year which is equal to $446/AF. Total O&M, plus energy is estimated at $809/AF.

The overall adjusted project capital cost opinion was $152,800,000 (2012$) for the case without iron/manganese removal as shown on the following Table 3. The reviewers had more recent bid data and recommended reducing the RO system cost by 20% ($8 million). The costs include a 25% contingency ($22.6 million) and 15% for professional services ($18.8 million).

The unit cost of water from the project, in current dollars, assuming high iron and manganese removal is not required, is estimated at:

- $1,611 per AF without the MET subsidy of $250 per AF
- Capital at $588 per AF (includes contingency and professional services)
- O&M at $363 per AF
- Energy at $446 per AF
- Land Lease at $47 per AF
- GW Mitigation at $167 per AF for take of 1,660 AFY on average

- Accounting for the MET subsidy results in a cost of water of $1,361 per AF (2012 dollars)
- For comparison purposes, MET avoided water costs in 2013 (Tier 1 + Capacity Charge Readiness to Serve Charge) amounts to $953 per AF

More detailed cost information is shown in the subsequent cost and economic analysis section.

Areas of greatest cost uncertainty are: (1) electrical energy and (2) brine disposal. The projected rate of increase in electrical energy costs over the next decade is a major uncertainty due to a combination of factors: implementation of AB32 and renewable energy, elimination of coastal power plants once through cooling systems, and the shutdown of the San Onofre Nuclear Generation Station (SONGS). These costs will need to be closely followed and incorporated into the project economic analysis.

Brine disposal costs for purchase of capacity in the San Juan Creek Ocean Outfall for those needing new or additional capacity are not yet included in the costs. The costs to modify the outfall diffuser to allow meeting discharge requirements are unknown at this time and no estimates have been included. A placeholder for modifications to the outfall junction structure at $2 million has been included. The outfall costs may further increase if significant recycling depletes the wastewater discharge. Evaluation of new diffuser systems and the performance of the system under the forthcoming SWRCB brine disposal regulations will need to be undertaken to determine the cost for brine disposal. This work also will require brine dispersion modeling and possibly some marine biology assessments.

Table 2 - Full Scale Doheny Desal Project O&M Cost Opinion

Excluding Electrical Energy Malcolm Pirnie (2011)	
	No Pretreatment
Labor	$1,260,000
Replacements/Repairs (Includes RO membranes & other)	$1,937,000
Chemicals/Feed Systems	$1,300,000
Maintenance Materials	$750,000
Other	$550,000
Subtotal O&M	$5,797,000
O&M $/AF	$363
Energy	$7,112,900
Energy $/AF	$446
Total - $/AF	$809

Notes

1. Average Labor rate updated to $105,000/year (OCWD GWRS O&M labor cost plus benefits)
2. Malcolm Pirnie assumed 12 FTE no Pretreatment
3. Replace First Pass RO Membranes every 3 years and Second Pass every 5 years; plus includes all other equipment replacements.
4. Energy at 4,228 kwhr/af and 10.5¢/kwhr
5. O&M increases to $421 per AF if high iron and manganese treatment is required.

Table 3 - Doheny Ocean Desalination Project Capital Cost Opinion

South Orange Coastal Ocean Desalination Project Conventional Design-Bid-Build Project Cost Opinion (Oct 2011)				
Major Activity Cost Item	**Description/Sub-Activities**	**Estimated Schedule (Months)**	**Case 1 Fe/Mn Pretreatment**	**Case 2 No Pretreatment**
PRE-CONSTRUCTION PHASE				
Preliminary Engineering Work	Engineering Work and Support for Environmental and Permitting Work	24	$750,000	$750,000
CEQA/NEPA Work	Baseline Environmental Monitoring	12	$300,000	$300,000
	Prepare and Process EIR/EIS	18	$500,000	$500,000
Additional Studies & Investigations	Outfall Modeling & Modification Engineering	15	$250,000	$250,000
	San Juan Creek Property Geotechnical and Site Investigations	15	$100,000	$100,000
	Offshore Geophysical Investigation	12	$400,000	$400,000
	Offshore Hydrogeology/Downcoast Drilling/Testing Investigation	12	$3,600,000	$3,600,000
	Power Supply Plan	12	$100,000	$100,000
Permitting and Approvals	Agency Meetings (Parks, CDPH, RWQCB, ACOE, CCC, SLC etc)	24	$400,000	$400,000
	Permit Applications Supporting Technical Data/Analyses			
	Permit Applications Preparation and Submittals			
	Permit Processing and Approvals			
JPA Formation, Legal/Financial Advisors	JPA Formation Legal and Financial Advisor	12	$300,000	$300,000
Design/Construction Team Selection	RFP Development and Design Engineer Selection	12	$300,000	$300,000
SUBTOTAL UP FRONT ACTIVITIES COST	Subtotal		$7,000,000	$7,000,000
	Contingency at 20%		$1,400,000	$1,400,000
	Total		**$8,400,000**	**$8,400,000**
DESIGN & CONSTRUCTION PHASE		30		
Design/Construction Project Costs	Intake and Raw Water Conveyance		$44,759,000	$44,759,000
	Pretreatment for Fe/Mn Removal		$43,300,000	$0
	RO Treatment		$53,534,000	$53,534,000
	Post Treatment		$15,636,000	$15,636,000
	Miscellaneous (Brine, SDGE, State Parks, Mitigation)		$11,648,000	$11,648,000
	Subtotal Construction Contractor Cost		**$168,877,000**	**$125,577,000**
	Base Construction Contractor Cost		$138,503,250	$102,991,000
	Contingency (25%) **(1)**		$30,373,750	$22,586,000
	Prof Services (Design & Construction Phases at 15%)		$25,331,550	$18,836,550
	Subtotal Contstruction Cost		**$194,208,550**	**$144,413,550**
Total Project Duration and Capital Cost		**70**	**$202,608,550**	**$152,813,550**

(1) Cost of pump-out and treatment of high iron and manganese laden water prior to start of operations estimated at $4.5 million, assumed part of contingency

Cost Comparison to Imported Water and Economic Analyses

Local projects that develop new sources of supply provide both source and system reliability benefits. In the case of ocean desalination, there is also a water quality benefit derived by production of desalinated water that has lower salts and hardness than the imported supply. Typically, when evaluating new projects, the cost of the new supply is first compared to the projected cost of MET water. The desalination supply will offset MET water purchases and in time these costs are projected to be less than imported water costs resulting in a net positive savings (benefit #1). In addition, ocean desalination improves system reliability (benefit #2), provides a drought proof supply (benefit #3) and provides improved water quality (benefit #4). The question is how to more accurately account for these benefits. Since the local agency drought benefit is reduced under the current approach taken in MET's Water Supply Allocation Plan and water quality benefits are derived by the end-user through longer water fixture life, the analysis conducted focused only on the direct supply and reliability benefits.

The unit costs were favorably compared to the projected costs of imported water, showing a possible cross over in about 10 years after start of operations. The investment cost was also favorably compared to the value of system reliability provided by the project when compared to alternative emergency reservoir costs and capabilities.

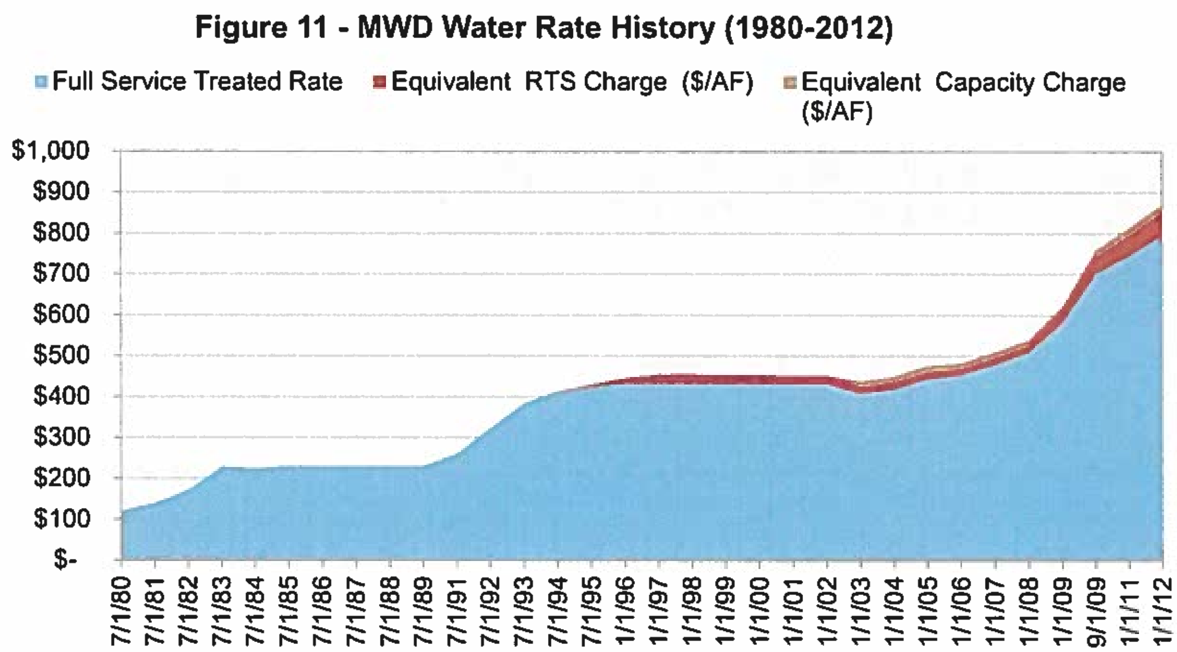
Cost of MET Water. MET has recently updated the projected cost of water to 2017. MET staff believes the near-term projection of rates is a reasonable estimate. Many factors that will result in upward pressure on MET rates have been reflected in these projections including a lower water sales assumption. The effect of a lower water sales assumption by MET is more conservative and, hence, is able to provide more flexibility for covering unexpected rate impacts in the future. Discussions with MET staff indicated that out-year projections beyond 2017 would best be covered by looking at a range of escalation factors from 3 percent on the low side to 6 percent on the high side.

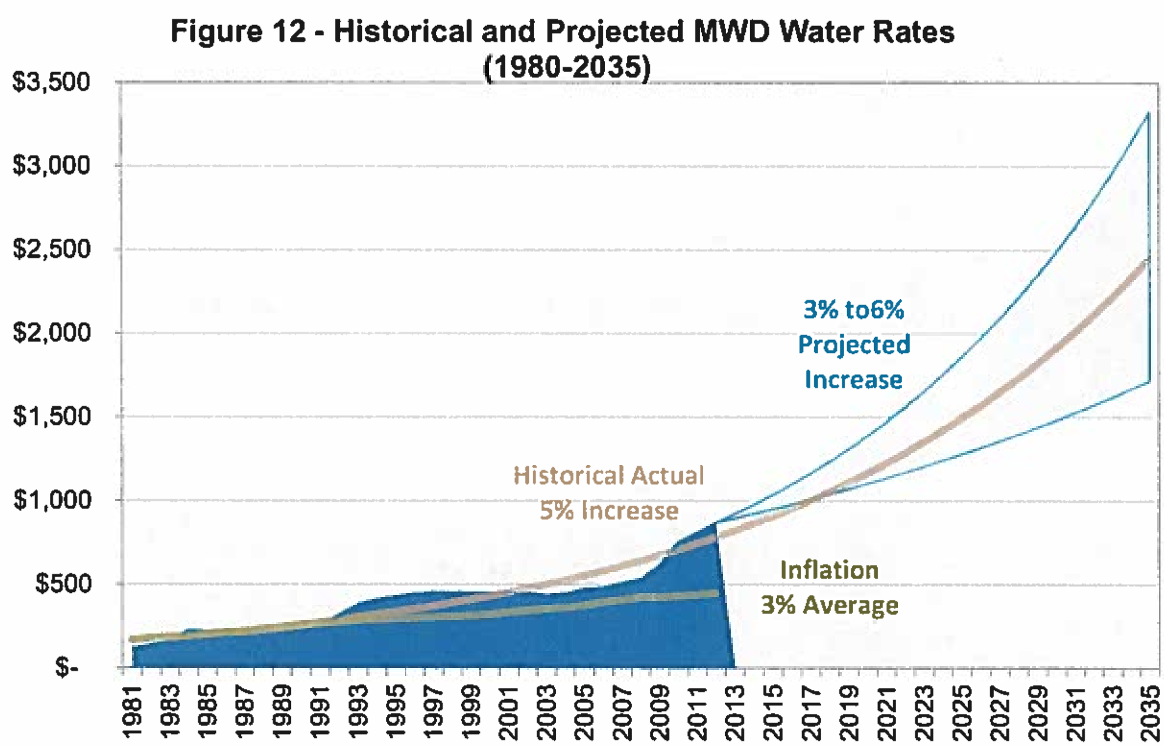
The future cost of water from MET is sensitive to a number of variables, making it difficult to develop an accurate long-term projection. Following are potential factors that could impact rates into the future:

- Energy Costs – The impact of California's Global Warming and Solutions Act (AB 32) on electricity prices is not factored in and is unknown at this time. Higher energy rates are forecasted due to several factors: AB32 mandated requirement for a higher mix of renewable energy sources, replacements and expansions in the Statewide electrical transmission system, phase out of Once-Thru-Cooling coastal power plants, and the shutdown of the SCE SONGS Plant (San Onofre Nuclear Generation Station) and its replacement. MET and the State Water Project Contractors are also facing a particular nuance of the AB 32 legislation whereby the electricity they import from out-of-state for Colorado River Aqueduct and State Water Project pumping may be assessed by California Air Resources Board as an "energy generator" in the state. MET staff is in the process of negotiating a method to provide relief and at this time ARB has indicated that they may provide MET some allowances, but not to the SWP. The impact of this decision could impact MET costs on the order of several million dollars per year.

- Bay-Delta Conservation Plan (BDCP) – A portion of the future costs of the BDCP have been factored into the near-term forecasts with the remaining portion of the costs to be included in the escalation range. The most recent estimate of costs for the fix, assuming MET pays for about 25%, is the cost of water for capital amortization and O&M costs estimated around $200 per AF on the MET water rate. Depending on what actually occurs, the costs could likely be either higher or lower, but would probably tend to cluster towards a higher cost. These are factored in between now and 2026 when the project is expected to start-up. Inflation is not included in these costs.
- MET Rehabilitation and Repair (R&R) Costs of Infrastructure (PAYGO funding) – MET has over $6 billion of investments in the ground not including their share of the SWP. These assets require periodic R&R or replacement. MET's asset management analysis completed several years ago estimated that the R&R program can be achieved at an annual cost of $125 M per year. This program is funded annually through the Pay-As-You-Go (PAYGO) funding, which is still considered sufficient at this time. When inflation picks up, the spending over time will have to correspondingly increase to keep in step with the R&R and replacement needs.
- SWP R&R – It is widely reported that the SWP is not maintained in nearly as good a condition as the MET system. Currently, the SWP is limited by facility conditions to about 70% of the delivery capacity of the SWP and hydropower generation has been reduced because of the failure at the Oroville facilities. MET has included some additional costs of future requests for SWP R&R funding in their budget (higher than what the State is requesting). This may or may not be sufficient to cover the deficiencies in the SWP needs. The SWP contracts expire in 2035 and as the contracts are renewed, it is possible that the renewed contracts will allow for additional levels of R&R and replacement funding without rate increases when the original debt of the SWP is fully repaid. MET and DWR are currently looking at options for the SWP R&R needs.
- Treatment Costs – The full capital and O&M costs associated with the ozone retrofit project at all five of MET's treatment plants are fully captured in the near-term projected water rates.
- Pension/Health Costs – A portion of the (not all) MET pension costs are already built into the rate projections. Other Post Employment Benefits (OPEB) have about a $500 million unfunded liability. MET believes they can eliminate the exposure with an annual contribution of about $50 M per year over the next 10 years. This is not fully reflected in the near term water rates. The other possibility is that by setting a more conservative assumption on water sales, any excess revenue, should it occur, could be used to fund this liability.
- The most recent population projections for the MET service area show an increase of 7.5 million by 2060. This increase in population will require additional new water supply at an increased cost to the region. The share of these costs between MET and the retail suppliers is the subject of future decisions.
- MET staff is examining methods to increase their fixed revenue. One such method is to change the basis of future AV tax revenue so that the percentage of tax levy remains fixed into the future at the current level rather than having the tax levy transition to zero between now and 2035 as planned. The additional tax levy, if successful, would tend to hold rates down in the future because of the estimated $80 million or so in fixed revenue that would accrue each year.

Figures 11 and 12 provide a summary of historical and projected MET water rates. Note the stair step pattern seen in the historical chart. This pattern is caused by water sales, costs and reserve variations.





Discussions with MET staff indicate that outyear cost projection beyond 2017 ranging from an annual escalation of about 3% per year on the low side to about 6% per year on the high side can be expected. Discussions with various sources in the industry note more cost pressures pushing rates towards the higher side of this range although recent discussions with MET staff indicate the potential that MET costs will trend towards the lower side of the range over both the near and mid-term, depending on future inflation rates and other potential unexpected costs.

Sensitivity Modeling. A sensitivity analysis approach was utilized to set up an economic analysis which would allow various input assumptions to be tested to understand the effects on both the cost of water from the Doheny Desal Project and to evaluate the project cost cross over point with MET rates (the point in time when the project cost would be less than imported water costs). This allows an analysis of the potential net present value difference between Doheny Desal and MET water rate scenarios. Figure 9 presents the "base case" analysis. The model provides the ability to vary the following parameters:

- Cost and escalation assumptions for Doheny Desal, the level of contingency assumed and whether or not pre-treatment facilities for iron and manganese will be needed

- Energy consumption and cost information can be varied. Two periods of energy escalation were provided, 2012 to 2030 and then after 2030 to allow the rate assumptions to be tested

- General inflation rates

- Project financing assumptions including the bond interest rate and whether any grant funds will be provided

- For the economic analysis, the Present Value factor can be modified

- A place-holder for land costs and an escalation factor is provided

- The MET rates are hard coded into the analysis through 2017 and then an escalation rate is used for rates beyond 2017

- The calculation summary provides the capital and O&M cost breakdown

- The Net Present Value function calculates the difference between the project rate and the MET rate and provides a present value to 2012 dollars. The purpose of this calculation is to understand the amount of costs above the MET rates up to the point of cross over and then it also quantifies the amount of costs less than the MET rate after the cross over and summarizes the full 30-year Net Present Value (positive = savings).

- A Reliability Benefit is the last input function. This is a measure of the system reliability benefit for the project. There are good reasons for investing in a project, even if the initial cost of water from the project may be above the cost of MET water. These include the reliability provided by having a local production facility able to supply system needs during an outage of the imported system in the event of a major earthquake or other cause and through an extended drought, as the desalination supply is independent of hydrology. The project would provide a significant emergency supply, system reliability benefit to protect the area from an outage of the imported water system as well as a drought supply benefit.

Discussion of Economic Assumptions in Table 4. Nine different economic scenarios were run to test the sensitivity of the assumptions in the sensitivity model, and the results can be found in Table 4. The findings indicated that the Doheny Desal Project supply cost is generally competitive with projected imported water costs. When considering the system reliability benefit of avoided investment in other local projects, the project provides a substantial cost savings and economic value to the community. The cross over point and net present value savings is most sensitive to future MET rates escalation assumption, e.g. higher MET rates improve the project comparisons. The detailed presentations of the nine sensitivity cases are included in the Appendix. The nine scenario runs include the following assumptions:

- **Reliability Benefit**. A project benefit is the ability to continue providing water into the local system in the event of an outage of the import system. The ocean is analogous to an emergency reservoir. Santa Margarita WD recently constructed the Upper Chiquita Reservoir Project at a cost of $50 M. This facility can provide emergency water supply at 23 cfs for about 2 weeks. The Doheny Desal Project can supply 23 cfs continuously. For a one month outage, the desal project provides the same emergency supply as two Upper Chiquita Reservoirs. The cost of two reservoirs would be about $100 M, which is the equivalent emergency reliability benefit that would be provided by the Doheny Desal Project assuming a 30 day outage. The value increases with the length of outage. Taking this benefit into account by amortizing it at the same rate and period as the overall project results in lowering the "cost" line (shown below by a second "project cost line" by about $385 dollars per AF (amortized cost of $100M). Accounting for the second benefit does not truly lower the cost of the project, but it does help identify and account for the emergency supply value of the project and the avoided cost of new reliability projects.

- **Fe/Mn Treatment.** The basis for the iron/manganese pretreatment system cost estimate was the assumption that Fe/Mn concentrations would remain at 6 mg/l throughout the project life, resulting in a capital cost for the oxidized filtration system at $50 million. Based on our expert panel review, it is expected that the old marine groundwater which is high in Fe/Mn would be pumped out in about a year, leaving just the 5% contribution from the brackish groundwater which has Fe/Mn concentrations around 2 mg/l. Under this scenario, the steady state Fe/Mn concentration would be 0.10 mg/l, not 6 mg/l. At this low level, pretreatment is not likely necessary, or if it is the costs would be substantially below the $50 million estimate as much higher loading rates could be utilized in the oxidized media filters. Also, use of an injection barrier along the coast to mitigate the project's take of brackish groundwater would eliminate in about a year or so the Fe/Mn contribution from brackish groundwater, thus eliminating any need for Fe/Mn removal.

- **Energy Scenario.** For the base case, energy costs have been escalated at 2% per year and have been projected at that same rate based on studies by SDG&E and others before the shutdown of the SONGS and increase in renewable requirement to 33% by 2020. For the high energy rate escalation scenario, 3.4% was used out to 2030 and 2% thereafter, based on work done by SDCWA.
- **Project Financing.** Project financing was assumed at an interest rate of 4.5% (current municipal AA bond rates). It is likely the project could receive a low interest loan from the State Water Resources Control Board State Revolving Fund that would further reduce the interest rate (at one-half of the State's prior year's general obligation bond rates).

- **Additional Benefits.** The project would also provide seawater intrusion control and water quality benefits to the basin, avoiding the need for a dedicated seawater intrusion control barrier. The project supports optimum utilization of the San Juan Basin without the basin having to incur the cost for seawater intrusion control. The basin benefits have not been factored into the economic analysis. This benefit was NOT specifically addressed in this analysis and is likely better to be accounted for in any future mitigation discussions.

Figure 13 – Doheny Ocean Desalination Project Economic Analysis – Base Case

Doheny Ocean Desalination Project - Economic Analysis - DRAFT VERSION 1.8

Input

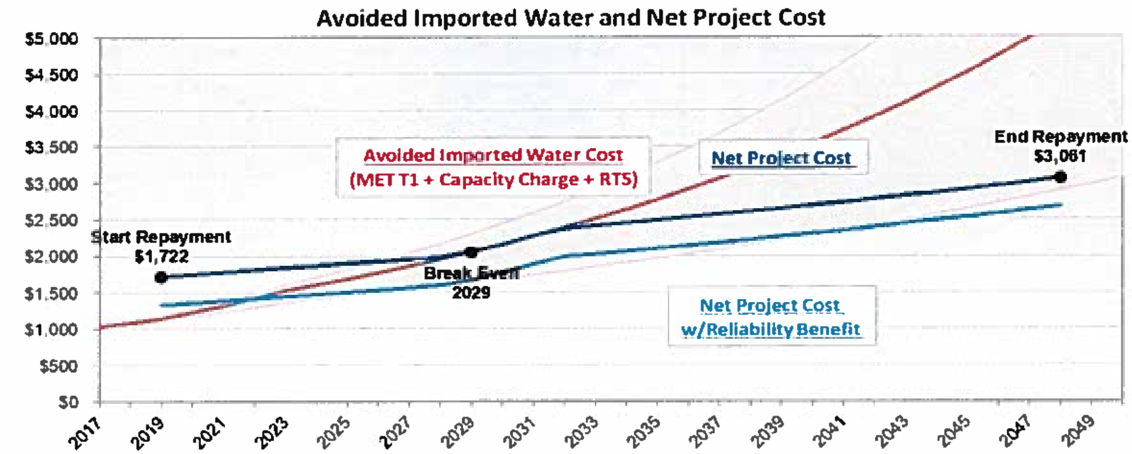
Capital Cost	
Fe/Mn Treatment?	No
Contingency of Const.	25%
Start of Construction	2017

O&M Costs		
Energy Costs		
Energy Usage (kWh)		4,228
Base Energy Rate ($/kWh)		$ 0.105
Energy Escalation 2012 - 2030		2.0%
Energy Escalation after 2030		2.0%
Land Cost		
Land Cost per AF (2011)	$ 47	
Lease Escalation Rate	2.5%	
General Inflation Rate		
Current - 2020	2.5%	
2021 - Future	2.5%	

Project Financing		
Grant Amount		$
Bond Repayment	Select →	30
Bond Interest Rate	4.50%	
PV Discount Rate	4.00%	

MET Avoided Costs	
Esc. Rate After 2017	5.0%
Melded Rate- % Tier 1	100%

Output



Project Capital Costs	
Capital Cost (2012)	$130,227,550
Grants	$0
Contingency	$22,586,000
Total Cost (2011)	$152,813,550
Total Cost Escalated to Midpt. of Const.	$181,647,290

	Current Year	First Year
Costs per AF	2012	2019
Capital Cost	$588	$699
O&M	$363	$489
Energy	$446	$522
Annual Cost	$1,397	$1,709
Annual Cost w/ Land	$1,444	$1,767
Mitigation Costs	$167	$205
Total w/ MET Subsidy	$1,361	$1,722

Net Present Value	
Up to Break Even	($31,436,811)
Break Even to End	$72,613,733
Total Savings	$41,176,923

Additional Reliability Benefit	
Storage Reservoir	$100,000,000

Table 4 - Summary of Economic Analyses

Case	Description	Fe/Mn Treat.?	Energy Scenario	MET Esc.	Cross Over Year	30 Year PV Savings	With Reliability Added
1	Base Case – Expected w/ 4.5% Finance	No	Base	5%	2029	$41 M	$141 M
2	With Fe/Mn	Yes	Base	5%	2032	$-6 M	$94 M
3	High Electrical Costs	No	High	5%	2032	$7 M	$107 M
4	Expected with $15 M Grant	No	Base	5%	2028	$55M	$155 M
5	Low Interest Rate at 2.5%	No	Base	5%	2026	$72M	$172M
6	Base w/Low MET Costs	No	Base	3%	2046	$-7M	$93M
7	Fe/Mn with High Energy	Yes	High	5%	2035	$-10 M	$90 M
8	Fe/Mn with Low MET Costs	Yes	Base	3%	2048	$-10M	$90M
9	Low Interest & Low MET Costs	No	Base	3%	2040	$-5M	$95M

Cost Comparison to the Poseidon Resources Huntington Beach Project

Comparison of the cost of ocean desalination projects from location to location can be difficult, especially when comparing a public project to a private project. Typically, public financing offers cost advantages compared to private equity financing. Private projects can be crafted in a manner to take on additional responsibilities and risks when they are providing water to public entities. Site characteristics can also vary and result in cost differences from project to project.

For the Doheny Desal Project, there are several site and other factors that make the costs very competitive:

- For the size of the Doheny Desal Project, slant wells are less expensive than open intakes which also require pretreatment systems to remove sediments and organic materials. Slant wells provide highly filtered water via the natural filtration process provided by the marine aquifer, thus avoiding the cost of having to construct and operate conventional pretreatment strainers, filtration and solids handling/disposal facilities. It has been determined from the results of the extended pumping test that the use of a slant well intake system will avoid the need for conventional pretreatment costs estimated at $56 million in capital and about $1 million in O&M costs, thus reducing the costs compared to other sites by more than $300 per AF.

- Co-disposal with wastewater through an existing outfall with sufficient hydraulic capacity avoids construction of a new brine discharge line and should make compliance with brine discharge easier to meet.

- System integration is relatively simple as the regional pipelines cross the desalination plant site and the pumping lift is relatively moderate at 450 feet. The savings of this integration system when comparing to other locations can be over $100 per AF or more.

- Public financing costs are typically lower than private financing

For the Huntington Beach site:

- Quite a bit of work has been done at the site and the engineering and permitting for moving forward with a construction project is nearly complete.

- Initially, the project can use the existing intake and outfall system. Uncertainties exist with the need for potential regulatory driven future changes to the intake and outfall systems. Use of the open ocean intakes also requires investments for the pre-treatment of the water.

- System integration is more complex than at the Doheny site.

- The methodology for capital recovery is on an escalated basis at 2.5% per year and has the result of lowering the early year costs and increasing the later year cost. This is an appropriate technique for phasing the costs of the project with future escalation; however, it results in a "different" cost compared to equalized annual debt recovery. The approximate first year impact is a decrease of about $300 per AF. If Doheny Desal used the same technique, the first year cost would be about $180 per AF lower.

- The costs also include repayment of private equity at considerably higher interest rates than available to public financed projects, project development costs, profit, and franchise tax and related payments. However, Poseidon has also agreed to take on much of the construction and performance risks for providing potable drinking water that meets specific quality criteria at the purchased water price.

The Poseidon Huntington Beach project unit cost as of February 2013 is around $1,800 per AF, including all costs and assuming a contribution from MET of $250 per AF. The Doheny Desal Project cost, **assuming an escalation of debt repayment similar to the Huntington Beach Project at 2.5%**, is currently estimated around $1,200/AF including all costs and assuming a contribution from MET of $250 per AF. Most of the differential in costs between the two projects can be explained by the factors noted above with the exception that:

- Poseidon found that their early cost estimates were overly optimistic compared to what was finally agreed upon. We will not have a more detailed estimate for Doheny until additional work is completed

- The element of "risk" taken on by Poseidon is not able to be defined as a cost per AF value.

Conclusion and Recommendations

The project is awaiting decisions by the project participants, SJBA and MWDOC on the next activities for the Project. The only work scheduled at this time is the upcoming Foundational Action Plan work; each of the Phase 3 Participants are now considering what their interest and role will be in that work. Key remaining issues for the project include how best to mitigate the drawdown and take impacts from the project on the San Juan Basin, the produced water quality from the slant wellfield over time, energy costs, and project costs. The groundwater basin and project mitigation alternatives questions will be answered through the work to be undertaken through the MET Foundational Action Program proposed work. This work includes groundwater basin management planning and additional project groundwater modeling work that will be completed over the next year or two by both SJBA and several of the Doheny Desal partners. This work will be important in formulation of the final project concepts and configuration.

Over the past several years of work, a great deal of information on the basin and the project has been developed. Our understanding of the basin and the project interaction has evolved over these years but additional information, study and project development work remain necessary. With respect to the groundwater basin, the necessary work falls under the following areas:

- Complete project impact analysis using a more detailed coastal model
- Evaluate alternative project mitigation measures – providing make-up water from the project or injecting recycled water along the coast to mitigate the drawdown and take impacts of the project on the basin.

- Evaluate seawater intrusion control effectiveness with a more detailed, coastal model

- Evaluate any project impacts to the seasonal coastal lagoon water levels

- Coordinate and track work with the SJBA on its implementation of the Groundwater Management Plan Recommended Alternative No. 6 and opportunities for coordinated and/or joint facility development and use.

The work has resulted in a "lot of new news" and a better understanding of the relationship among these various parameters. At this time, both the work to be conducted by the SJBA and several of the Doheny Desal partners needs to occur to focus in on the final projects configuration.

At any time, the pre-design CEQA and permitting work could be started. The critical path items are the environmental baseline monitoring, offshore geotechnical work, and preliminary engineering for the ultimate project, or the schedule could include a waiting period to finish the work at hand. Discussions with the five Doheny Desal Participants regarding how they would like to move forward will be occurring over the next several months.

The Participants recommended staff develop a "watch" list of issues that could ultimately impact the cost and/or feasibility of the Project. The following Table 5 identifies issues to keep within our monitoring efforts as we move forward.

Table 5 Doheny Desal Cost Impact "Watch" List
These are issues that could impact the ultimate cost of water from the Doheny Desal Project and so should be reviewed from time to time for their status and impact to the project assessment:
1. Financing has been at record low levels.
2. Outside funding may be available from State or Federal sources, either via grants or legislative actions; the State Revolving Fund and anticipated Water Infrastructure Finance and Innovation Authority (WIFIA) funding and 2014 State Bond are examples.
3. Technology Improvements can lower the costs of desalination.
4. The bidding environment has been at record low levels; many companies are interested in getting involved in ocean desalination in the U.S. and California.
5. The cost of energy is difficult to predict in the State of California due to implementation of AB 32, related regulatory policies and programs, hydraulic fracking and natural gas prices, changes in solar energy technology and costs, etc.
6. Iron and manganese pretreatment may be necessary (the costs have been estimated) but at what level is uncertain at this time.
7. The State Water Resources Control Board Ocean Plan Amendment is pending and the cost implications are unknown. New regulations could impact brine discharge through the SOCWA outfall.
8. Other regulatory issues that might arise during permitting.
9. Future costs will be higher due to inflation but are uncertain on a real dollar basis with improvements in technology and increased competition.
10. Mitigation costs with the San Juan Groundwater Basin have to be negotiated – a placeholder has been included in the conceptual level cost opinion.
11. Fisheries issues (e.g., southern Steelhead) in San Juan Creek and the Seasonal Coastal Lagoon due to groundwater drawdown may need to be worked out.
12. Design/Build and Operate, and Design/Build/Operate delivery mechanisms could offer savings in life cycle project costs compared to the conventional Design, Bid, Build, Operate method.
13. As other projects in California get up and operating, relevant knowledge can be transferred to the project.
14. Drought supply shortages and an increasingly greater public recognition of the value of water may spur increased public and political support and willingness to pay for improved supply reliability.

C. Goals and Objectives

The three main goals for Phase 3 were:

- Conduct an extended pumping and pilot plant test to determine the performance of the well and aquifer, to determine water quality over time, and to determine the pretreatment effectiveness of the aquifer

- Evaluate the project impacts and mitigation approaches on the groundwater basin using a regional watershed and groundwater model by first estimating the basin yield and its performance without the project and then determine the effect on the basin with the project.

- Conduct a conceptual level assessment of the full scale project and its costs.

To support the overall goals of the Phase 3 work, 10 specific objectives were developed:

1. Obtain long-term well performance, salinity, and drawdown data and use in validating and refining the groundwater model that will be used in aiding in the design of the feedwater supply system and evaluating project impacts. Conduct natural isotope testing on the extracted water to quantify the sources of water pumped from the well over the extended test period.

2. Collect and analyze slant test well water quality to determine the character of groundwater produced over the extended pumping period. Assess how water quality may change over time as the well pulls in offshore marine groundwater and ocean water. Evaluate how potential changes in ocean water quality, such as red tides, may influence the produced well water. This information will also help to validate the existing SEAWAT groundwater model predictive capability and develop source water quality specification that can be used for project environmental review and permitting.

3. Conduct corrosion studies to determine appropriate materials for the wells, pumps, and system piping and valves.

4. Evaluate the effectiveness of using a nitrogen blanket in the test slant well headspace to minimize introduction of air into the well. This step is intended to control microbiological growth and oxidation/precipitation of dissolved iron and manganese in the produced well water and to facilitate evaluation of any oxygenated ocean water entry into the well over the test period.

5. Conduct studies to identify and measure the extent of microbiological growth over the extended pumping period on the well and selected materials, which are anticipated to result from both brackish and ocean water influences. Determine the speciation of natural organisms that may grow in the well/conveyance facilities and evaluate control approaches as necessary.

6. Evaluate the pretreatment effectiveness of the aquifer and well through the use of standardized testing procedures (e.g., silt density index (SDI), turbidity, pilot unit RO membrane performance); evaluate microbial, colloidal, and particulate fouling; and determine and test any additional pretreatment that may be necessary.

7. Conduct an extended "Under the Influence of Surface Water" study for determining if the well production is affected by San Juan Creek water quality, evaluate applicable California Department of Public Health (DPH) treatment requirements, and develop testing protocols with DPH review.

8. Test RO process performance using test slant well water initially without pretreatment then with the addition of pretreatment, if necessary.

9. Develop a regional watershed model to generate streamflows and a groundwater model to determine groundwater basin yield over an extended period of time including a dry period and to determine the impact of the project on the basin and mitigation approaches.

10. Conduct conceptual level assessment of the full scale project to develop an opinion of probable construction and O&M costs.

The Phase 3 investigation accomplished all of the above objectives.

D. Phase 3 Project Implementation

MWDOC was responsible for carrying out the implementation of the Phase 3 test project. This work included:

Environmental Documentation

A consultant was retained who prepared the project description and mitigated negative declaration for the Phase 3 facilities construction and their operation and maintenance, publication, processing and adoption. This work was done by Chambers Group, an environmental consulting firm.

Permitting and Approvals

This work included the preparation of information and special studies for the permit applications, the permitting process, including agency meetings, and execution of the permits. The following permits and approvals were required and issued: (1) California Department of Parks and Recreation (Right of Entry Permit), (2) State Lands Commission (amended lease), (3) California Regional Water Quality Control Board (NPDES Discharge Permit and a Water Quality 401 Certification), (4) California Department of Fish and Game (Streambed Alteration Agreement), (5) U.S. Army Corps of Engineers (404 Outfall Nationwide Permit), and (6) California Coastal Commission (Coastal Development Permit).

Design, Procurement and Construction of the Test Facilities

This work included consultant selection and design, procurement and construction of the test facilities. The test facilities were designed, procured, or constructed under the direction of MWDOC, who served as the project manager. This work included: (a) well inspection and redevelopment, (b) design and procurement of a submersible pump, (c) installation of the submersible pump, (d) design and procurement of a Mobile Test Facility, and (e) design and construction of appurtenant test facility infrastructure (placement of the Mobile Test Facility, pipelines, conduits, control and metering vault, outfall diffuser and electrical service).

These facilities were located entirely within Doheny State Beach. GEOSCIENCE/Boart Longyear provided the well work and Carollo Engineering provided the design and construction observation services for the test facility. Williams McCaran, Inc. designed the Mobile Test Facility, which was then procured by MWDOC. MWDOC procured this item due to its long-lead time in manufacturing and special features that were required for the Phase 3 extended pumping and pilot plant test. This also allowed MWDOC to control overall quality of the facility. MWDOC also solicited bids as part of this effort. Intuitech, a company specializing in assembling pilot water and wastewater process test equipment, manufactured the test facility. Prior to installation at Doheny State Beach, Intuitech performed shakedown testing using a freshwater supply to make sure that all process equipment, instrumentation, and electrical equipment was functioning properly. This work was observed by WMI to ensure all work was completed in compliance with the design.

Pilot Facilities Start-up and Operation

After installation and construction of the test facilities, SPI was selected to operate the test facility and to conduct the various testing work over the extended pumping test.

Remove/Destroy/Abandon Test Facilities and Restore Site

Participant funds are being reserved to eventually remove the test facilities and restore the project site. Currently, an agreement with State Parks allows the test facility to remain in place. Permits are also maintained. The temporary facilities that will eventually be removed are: (1) the mobile test facility (this is planned to be salvaged and moved to the full scale plant site for use during start up and for future testing work); (2) test slant well submersible pump, wellhead, discharge piping and outfall diffuser; (3) temporary electrical and instrument conduits run from the test facility to the wellheads and; (4) the meter and electrical conduit supply to the test facility. Additionally, the test horizontal/slant well and nested monitoring well MW1 located on the beach will be abandoned or destroyed if there is no future use for these facilities. MW1 is expected to be transferred to San Juan Basin Authority which will require a long-term use agreement with State Parks.

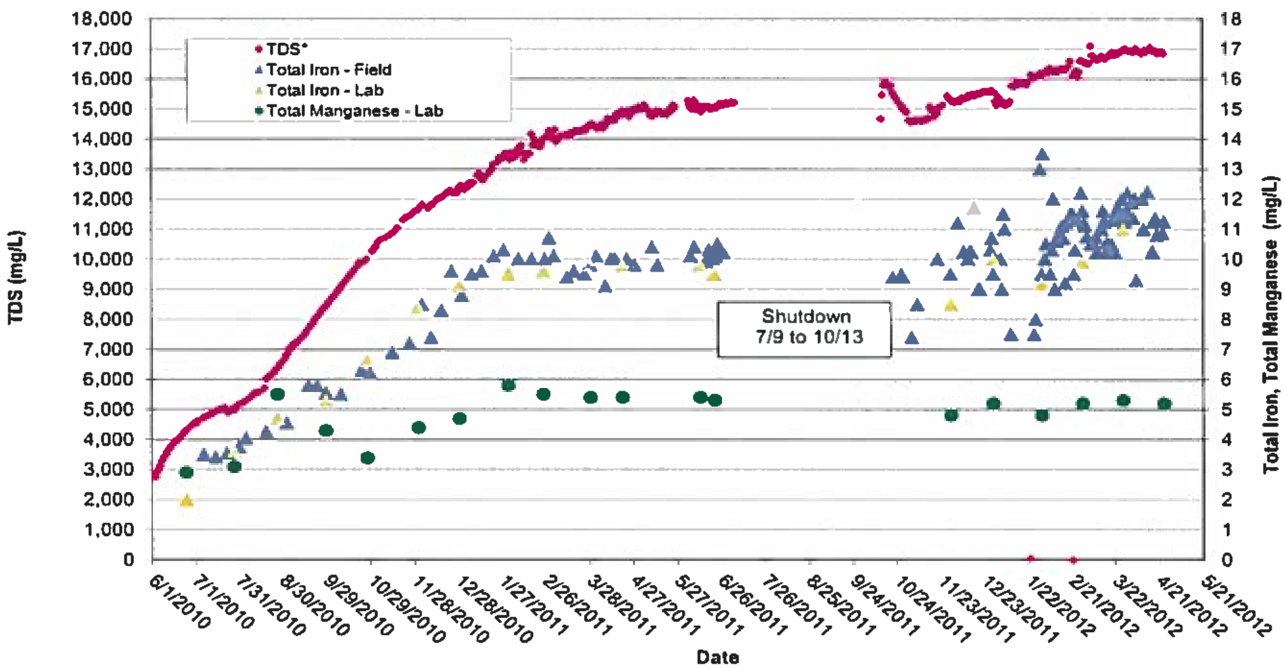
E. Project Results – What Was Learned

Following is a summary of results, findings and conclusions gained from the Phase 3 work.

Feedwater Supply

1. Construction and operation of slant wells along Doheny State Beach is feasible.
2. Old Marine groundwater was encountered and was found to be enriched with dissolved iron and manganese and remained anoxic (without oxygen) throughout the nearly two year extended pumping test. This test showed a continuing increase in salinity and of ocean water (from isotope data) being pulled into the well. See Figure 14.

Figure 14 - Slant Well TDS, Total Iron and Total Manganese



* Note: TDS calculated by 0.65 x Conductivity

3. We believe the pocket of old marine groundwater will be pumped out over time. Geochemical modeling or offshore geophysics and borings are required to more accurately estimate the time required to pump out the old water.
4. The Marine Aquifer provides excellent filtration as evidenced by nearly two years of pumping and testing data.

5. The natural isotope study provided excellent information on the rate of connection to the ocean and the data can be used to refine the coastal groundwater model calibration. The data clearly showed an increasing trend in the amount of ocean water being pumped (which is a good trend).
6. The corrosion study recommends 2507 Super Duplex Stainless Steel for the wells. This was the material used to construct the test submersible pump.
7. The microbial biofouling study showed very low levels of microbial biofilm growth.
8. The slant wellfield configuration is expected to consist of 3 clusters of 3 wells located along Doheny State Beach for a total of nine wells. Preliminary study indicates that the wells would be about 520 feet long at an angle of about 23 degrees. The actual wellfield configuration, well and wellhead design, and wellfield capacity needs to be determined. In the future, the offshore geophysics survey will be needed for both the coastal groundwater model update and wellfield configuration design work.
9. The slant wellfield can be permitted as a water supply. The subsurface intake is regarded favorably by the regulatory agencies based on verbal comments and staff reports by the Coastal Commission for other projects. Further, the State Water Board draft Ocean Desalination Policy is also supporting a slant well subsurface intake approach. Using a subsurface intake will save significant permitting time and costs. Drawdown impacts on the lagoon are expected to be minor. Environmental baseline monitoring is required to support the environmental impact report and permitting activities.
10. Based on work being conducted by West Basin MWD, an open ocean intake system may also be feasible with the use of wedge wire screens. However, conceptual work indicates that it will be a very expensive proposition to construct a "new intake" structure via tunneling if pursued at the Doheny site. Another potential option is to put the intake in the easterly basin in Dana Point Harbor, but limited depths and fueling operations would make this option problematical. This approach was not investigated.

Lower San Juan Basin Groundwater Yield and Integrated Operations

1. The 2007 preliminary groundwater model has been significantly improved through development of a basin wide surface water flow model and updated groundwater model for the Lower San Juan Basin completed in April 2013. This work was developed in close cooperation with San Juan Basin Authority (SJBA) and with their Groundwater Management Plan development work.

2. The groundwater model has been recently re-calibrated to a reasonable level of accuracy for planning purposes over the more recent period, 2004-2010, a period with higher groundwater pumping than under historical operations.

3. Groundwater production in the basin during the period 2004-2010 averaged 5,370 AF per year. Under this level of production, groundwater discharges to the ocean from rising water and subsurface outflow were estimated at 1,880 AFY. The near-term pumping by San Juan Capistrano and South Coast in the Lower San Juan Basin will increase over these historical levels which will

significantly reduce the rising water and subsurface outflow losses. Continued increased pumping can result in seawater intrusion.

4. Without the Doheny Desal Project, the 2013 modeling results indicate that net basin water supply on average came out to 9,150 afy and during a repeat of the 30-year dry period the supply would decrease to 8,040 afy. These values include ocean water intrusion, rising groundwater outflow to the ocean, subsurface outflow to the ocean and change in basin storage. Under this run, ocean water intrusion began to occur; the South Coast wells were turned off after nine years when the salinity reached 2,600 ppm. It is likely these basin yield values are over estimated by about 300-400 AFY as the modeled pumping amounts results in seawater intrusion. The breakdown of this analysis is shown below in Table 6:

Table 6 - Groundwater Modeling Production Analysis – Base Case (2i/2j)
Pumping Water Level Constraint with Salinity Constraint

	Groundwater Pumping Yield (afy)	
Producer	Dry	Average
City's GWRP Wells	5,808	6,690
City's Other Wells	823	942
Subtotal City	6,631	7,632
SCWD	559	664
Private Wells	850	850
Total	8,040 afy	9,146 afy

5. With the Doheny Desal Project intake production at 30 mgd, the groundwater modeling indicates that on average about 5% of the slant well production (1.5 mgd, 1,660 afy) will be San Juan Creek brackish groundwater. This estimate was made by averaging the Doheny Desal draw on the basin of 1,495 afy in dry periods and 1,820 afy in average periods, averaging about 1,660 afy.

6. The modeling indicates that South Coast Water District wells (the wells in the basin closest to the ocean) would be potentially impacted by a drop in groundwater elevation between 15' to 20' with slant wellfield production level at 30 mgd. The drawdown impacts to the City of San Juan Capistrano wells further up in the basin would be approximately 1 to 3 feet.

7. The 30 mgd slant wellfield production level will protect the SCWD wells and the lower basin (e.g., Latham WWTP) from ocean water intrusion.

8. The leaking underground storage tanks at the gasoline stations in the vicinity are in the process of being cleaned up and are not expected to impact the project start up. Continued coordination with the Orange County Heath Care Agency (OCHCA) and oversight is required.

9. Drawdown impacts to the San Juan Creek seasonal lagoon at the ocean interface will likely be small as the lagoon is underlain by a shallow highly permeable aquifer and an areal extensive clay layer. The seasonal lagoon receives ocean water recharge as well as streamflow from storms and urban runoff. A more detailed coastal groundwater model will be needed in the future to assess this impact as well as intrusion through the shallow aquifer.

Desalination Facility, Product Water Quality and System Integration

1. The desalination facility site (5 acres) is proposed to be located just north of PCH on existing South Coast Water District property. South Coast Water District has generally reserved the site for the project. Negotiations for use of the plant site will have to be completed. The current cost estimate has a placeholder lease cost for the site. The site will require geotechnical work to prepare the foundation for location of a new plant. The rough grade of the site will need to be raised to protect against flooding including an allowance for sea level rise.

2. Product water quality will be driven by the level to which bromide and boron need to be reduced. A bromide level of 0.3 mg/l will provide adequate protection for disinfection residual stability. This requires about a 40% second RO pass. This will also produce a boron level around 0.5 mg/l which will be protective for ornamental plants. Typical second pass RO configurations for plants range from 30% to 100%.

3. System integration is relatively low in cost, as both imported water pipelines cross near the Plant site. The water would be boosted out of a clearwell reservoir to a 450 foot hydraulic grade line to match with the imported water system (Joint Regional Water Supply System (JRWSS) and Water Importation Pipeline (WIP)). Additional pumping of about 110 feet would be required to supply the water to the Laguna Beach 400 zone from the SCWD 290 zone.

Brine Disposal

1. The San Juan Creek Ocean Outfall has adequate capacity to dispose 15 mgd of brine flow from the Doheny Desal Project. The outfall has a capacity of about 85 mgd and present day average daily dry weather flow is about 17.5 mgd; the current permitted capacity is 30 mgd. In the future the average daily dry weather flow will likely decrease with additional recycling and water use efficiency measures.

2. The brine disposal point of connection would be into the surge chamber junction, located adjacent to the Desalination Facility site.

3. A brine disposal study needs to be undertaken with South Orange County Wastewater Authority (SOCWA) to determine if any modifications are necessary to the outfall and its diffuser for compliance with SOCWA's National Pollution Discharge Elimination Standard (NPDES) permit. The study would need to evaluate ranges of blending with wastewater for co-disposal of 0% up to about 50%.

4. Non participants in the SOCWA outfall will have to acquire capacity from agencies with excess capacity.

5. The SWRCB is in the process of amending its California Ocean Plan which will include new regulations and standards for brine disposal. This amendment is expected to be completed either late this year or in early 2014.

Energy Supply and GHG Offsets

1. The project will have an electrical load of about 8.2 megawatts (MW). The project is estimated to consume 4,228 kilowatt-hours (kwhr) of electrical energy per acre-foot (AF) of produced water, including the pumping lift for system integration. For comparison purposes, imported water delivered to the area from the East Branch of the SWP through the Water Importation Pipeline uses a net of about 3,440 kwhr/af.

2. An electrical service study by SDG&E was completed in 2007; we are working with SDG&E to update this study. As of this time we don't have any response from SDG&E on the cost of the new work or time required to complete the update.

3. SDG&E is embarking on a $500 million reliability upgrade to their electrical distribution system in its Orange County service area.

4. The SDG&E reliability improvements include a new enlarged San Juan Capistrano substation. This should reduce the cost of running a 12 kV service to the Desalination Facility (the previous study ran the 12 kV line from the Laguna Niguel substation).

5. SDG&E has indicated that their worst case power outage would be for 12 hours. Based on this, no back-up power would be required for this short of an outage. This does not include any electrical reliability issues that have arisen with the recent SONGS plant closure.

6. SDG&E offers programs to shed load for electrical cost savings. The two main programs are their Critical Peak Pricing and Base Interruptible schedules. These will be further explored to reduce costs to the project.

7. A new law allows an agency, not a Joint Powers Authority (JPA), to build and wheel up to 3 MW of renewable energy through the PUC regulated agency grid. However, typically these costs are higher than grid energy from SDG&E.

8. SDG&E service environmental impacts could be covered under the Doheny Desal Project EIR.

9. SDG&E indicated that 2 years are required to design and construct their service facilities.

10. Energy costs will increase due to reliability improvements, expansion of the State's transmission and distribution system, meeting renewable energy targets of 33 percent by 2020, phase out of power plants using Once Thru Cooling (OTC) technology, impact of SONGS closure and replacement power, and general rate increases. However, natural gas fuel costs continue to stabilize the cost of energy from natural gas fired power plants. Predicting future energy costs with a reasonable degree of certainty is difficult at this time. Future decisions on SONGS replacement (assumed) and consumer liability by the PUC and SDG&E have not yet been made and no projections are available.

11. Greenhouse gas (GHG) offsets will likely be required by the State Lands Commission and Coastal Commission. Without any mitigation, the annual cost for GHG offsets is not expected to be significant, at about $50,000 per year at today's market rate.

Project Costs and Economics

1. Project capital cost is estimated at $153 million ($2012).

2. Capital and Project Unit Costs ($/AF) are lower than other desalination projects due to the attractive project location: slant wells avoid pretreatment costs compared to an open intake system, land is available near the coast, outfall capacity is available, system integration and pumping lift costs are very low, and SDGE is investing $500 million to improve electrical service reliability to the area (which should slightly reduce the electrical service cost to the Doheny Desal Project). Slant well intakes have unit costs per capacity similar to open intake systems, but can be built at lower capacities at much reduced capital cost than open intakes, which are best suited to large scale plants.

3. Estimated project unit costs (at this time) in 2012 dollars without grants or low interest loans are:

    - $1,611 per AF without the MET subsidy of $250 per AF
    - Capital at $588/AF (includes a 25% contingency and a 15% allowance for professional services)
    - O&M at $363/AF
    - Energy at $446/AF
    - Land at $47/AF
    - GW Mitigation at $167/AF for take of 1,660 afy on average
    - Total of all costs = $1,611 per AF.
    - Accounting for the MET subsidy results in a cost of water to the local agencies in 2012 dollars of $1361 per AF
    - For comparison purposes, MET avoided water costs in 2013 (Tier 1 + Capacity Charge + Readiness to Serve Charge) amounts to $953/AF.

4. Projected imported and desalination water costs cross about 8 to 10 years out (or further depending on the assumptions used) from which point on the desalination water costs would be

lower than imported water costs. Nine different economic scenarios were run to test the sensitivity of the assumptions. The most sensitive assumption was the out-year escalation of MET water rates (a higher MET escalation makes the Doheny Desal Project look more favorable and a lower escalation of MET rates is not favorable to the economics of the project).

5. One of the scenarios included higher energy cost escalation, which would increase the cost of the project. Current energy escalation costs are somewhat speculative. Future work should focus on refining the energy costs inputs to the project.

6. The system reliability benefit of the project has been estimated at about $100 Million when valued on the cost of storage at Upper Chiquita Reservoir Project. The project also provides benefits during droughts and helps prevent water shortages during emergency situations – these last two benefits have not been captured in the economic analysis.

F. Conclusions Regarding Slant Wells

Water supply wells when properly designed, constructed and developed can last for 75 years or more. There is no difference with Slant Wells as these will be built using tried and true water well technology along with the design and construction experience and innovations gained from the construction and operation of the Test Slant Well. We expect the Slant Wells to perform very well over the long- term and expect a useful life of 75 years.

Well Production Capacity

Based on the Test Slant Well pumping test at 2,100 gpm and recent groundwater modeling, we expect the full scale wells will be able to produce 3,000 gpm. Drawdowns, including well interference, will be approximately 90 feet vertically from mean sea level to the pumping water level in the well to produce the 30 mgd from seven pumping wells with two wells on rotational standby. The aquifer thickness is about 200 feet along the coastline, which is sufficient to allow the expected drawdowns and well yield. Should a problem occur during the summer when beach access is restricted there will be two standby wells that can then be turned on to continue uninterrupted production at the 30 mgd level. Drawdown impacts to wells in the San Juan groundwater basin will only be significant to the most nearby wells owned by South Coast Water District.

Well Design, Construction and Development

Design and construction of the full scale slant wells will need to be approached similarly to conventional water well design and drilling, but since the wells will be relatively flat in slope, additional care must be taken in gravel placement and well development. The design and construction will be aided through the experience gained in design and construction of the Test Slant Well. A key to the long-term success of the wells will be to provide thorough development work to assure minimum levels of sand clogging to the gravel pack. Sand clogging can occur over time in a well when it is not properly designed, constructed and/or developed. Causes include too large of well screen slot spacing, too large of gravel size in the gravel pack, gaps in the gravel pack, and most commonly, insufficient development of the well. The well screen and gravel pack size can be properly sized assuming the well designer has good technical capability and experience. Improper well development can occur due to insufficient swabbing, bailing and/or air lifting and due to insufficient development pumping rate and time.

For the full scale slant wells development, the development pumping rate needs to be around 1.5x the production rate with development pumping over a sufficient period of time to allow complete removal of entrainable fines from the near borehole formation. Assuming the full scale well capacity at 3,000 gpm, the development pumping rate should be specified at 4,500 gpm.

To assure adequate development pumping, procurement of high speed 4,500 rpm pump(s) in advance of the construction will be required. Well contractors typically do not stock submersible pumps of this capacity that would be able to fit into the well. Contractors often use suction development pumping, but this option will not be possible, as these pumps are limited to a suction or drawdown of 32 feet and a greater lift will be required. The designed drawdown will be approximately 45 feet below sea level (lower low water) and the wellhead floor elevation will be approximately minus 2 feet MSL, a differential of 43 feet, exceeding suction limits.

Another consideration in the construction of the nine wells is the ability to complete the work within the 8-month winter time window. This will likely require three well drilling crews working concurrently. The advantage of three wells drilled from a single site is the time and cost savings from moving the drill site. The well driller will need to possess well in advance of construction three large dual rotary drill rigs (DR-40) and trained crews. Sufficient lead time will need to be provided to acquire any additional rigs from the manufacturer.

Well and Pump Materials and Corrosion Protection

The Slant Wells will be constructed with Super Duplex 2507 Stainless Steel, an alloy which showed very little corrosion over the extended pumping test and which is considered suitable for achieving a long useful life for the well. Over the nearly two year extended pumping test, this alloy showed no corrosion. It is used in many ocean desalination projects worldwide. Super Duplex 2507 will not support biofouling iron bacteria that are common in carbon steel cased wells. It is considerably less costly than AL-6XN, another superior stainless steel used in ocean applications.

Long-Term Aquifer Performance

Over the nearly two-year extended pumping test, the step drawdown test indicated no observable change in aquifer losses. Aquifer loss can occur in certain types of aquifers that are susceptible to biochemical in-situ encrustation or precipitation, especially in limestone formations. For the alluvial aquifer system offshore of San Juan Creek this condition will not occur.

During the initial start up pumping period, the wells will pump out the old (age 7500 years) marine groundwater that is anoxic and enriched with dissolved iron and manganese. As the wells pump, the ocean water, which is oxic and has only trace levels of iron and manganese, will slowly recharge the aquifer and flow towards the well. No mixing will occur along the boundary of the marine groundwater and recharge front of ocean water, except for trace convective diffusion effects which will have no observable effect on aquifer permeability due to any minimal oxidation along the front as the masses in the boundary zone are insignificant.

The oxic ocean water will slowly become less oxic as microbial activity consumes the available organic carbon and dissolved oxygen as the recharging ocean water flows through the aquifer to the wells. Since the ocean water will have some dissolved oxygen over part of its flow course to the wells, this oxic condition will not cause any further dissolution of iron and manganese minerals that might remain in the sediments. Likely all of the iron and manganese mineral oxides in the original sediments were fully dissolved out of the formation since the time the ocean flooded these sediments, some 7,500 years ago ("old marine groundwater"). Over the extended pumping test, the well was pulling in about 20% ocean water, which became anoxic by the time it reached the well. This ocean recharge most likely entered the well near its upper screens that are only 50 feet below the ocean floor. Sufficient organic carbon was available to the naturally occurring aerobic bacteria in the seafloor sediments. The travel path to the remainder of the screens is longer and will allow for further uptake of any dissolved oxygen in the recharging water. The San Juan Creek and lagoon produce significant organic carbon loads which are swept out to the ocean by periodic storms. This condition is likely to indefinitely continue into the future.

Within the aquifer, where the ocean water groundwater flow and brackish groundwater flow boundary occurs, there will be a small mass reaction over time along this boundary due to slowly varying heads and tidal forces that will result in some convective diffusion along the boundary area which would cause some iron oxide precipitation within this brackish/ocean water flow boundary. However, the masses are quite small compared to the volume of the alluvium pore space that it would take a very long time to seal this flow boundary with iron oxy-hydroxide precipitates. The effect would be to reduce the amount of brackish groundwater that would enter the wells, which is a desirable outcome.

The project microbiologist, Dr. Sunny Jiang from UCI studied biofouling rates over the two year extended pumping test. Biofouling rates were found to be very low with biofilms less than 10 µm in thickness on the stainless steels. She does not expect much biofouling activity in the full scale wells.

Under the initial period of pump out, a large portion of the pumped water was brackish groundwater. This water has a much higher TOC than the old marine groundwater and ocean water. Initial levels of naturally occurring bacterial growths were fairly high but declined dramatically as the TOC levels dropped significantly as the ocean water was pulled into the well. It is uncertain what impact if any the project will have on the seasonal lagoon associated with San Juan Creek, as this area is underlain by an extensive 4-foot plastic clay layer that minimizes drawdown effects on water levels in the lagoon. The reverse condition is also true – the lagoon should have very little if any effect on the water quality produced from the slant wells.

Well Oxidation Control

The wells will be designed to be fed nitrogen gas into the headspace in the well above the pumping water level to prevent oxygen transfer into the water. This was used successfully over the Phase 3 extended pumping test and performed quite well.

Well and Pipeline Cleaning

If the ocean water that enters the wells contains some dissolved oxygen it will then mix with any anoxic brackish groundwater that has dissolved iron and manganese that enters the well. Once the mixing is initiated the oxidation reaction times are fairly rapid. If the DO levels are above about 1 ppm, this will lead to oxidation during the movement of water through the pipeline to the plant of dissolved iron and manganese. Under this condition, some accumulations of iron deposits along the walls in the upper well screen area, through the pump column, and along the conveyance pipeline can be anticipated. A mitigation design measure is to size the conveyance system to maintain high velocities around 8 to 9 fps, within a reasonable headloss, to help to scour and minimize iron deposition accumulations.

The submersible pumps will be serviced or replaced once every 5 to 10 years along with well inspection and any required maintenance. It may be necessary to acquire a dual rotary drill rig with angled set up to allow for less costly well maintenance, as the mobilization costs can be high as these rigs are often kept out of state as they are frequently used in the mining industry. In the future, the merits of this approach should be evaluated.

Phase 3 Final Reports

Separately published Project reports from Phase 3 are listed below in Table 7.

Table 7 - Phase 3 Final Reports

#	Title	Author	Issued
1.	Project Summary Report	MWDOC	Final Jan 2014
2.	Volume 1 – Phase 3 Project Development Report	MWDOC & Carollo Engineers	Final Sep 2013
3.	Volume 2 – Pilot Plant Operations, Testing, Evaluation Report	SPI	Final Aug 2013
4.	Volume 3 – Phase 3 San Juan Basin Regional Watershed and Groundwater Models Report	Geoscience	Final Nov 2013
5.	Pilot Testing of Slant Well Seawater Intakes and AWT Pretreatment Technologies for Control and Removal of Iron and Manganese	SPI	Final July 2013
6.	Expert Panel Workshop Report: Offshore Hydrogeology/Water Quality Investigation Scoping, Utilization of Slant Beach Intake Wells for Feedwater Supply	Dr. Susan Paulson, Flow Science and MWDOC	Final Oct 2012
7.	Final Report: Desalination Corrosion Study	Dr. Joseph King, Engineering Materials	Final May 2012
8.	Natural Isotope Tracer Study: Test Slant Well Phase 3 Extending Pumping Test	Matthew A. Charette, Ph.D. - Coastal Groundwater Consulting & WHOI	Final Nov 2012
9.	TECHNICAL MEMORANDUM: Aquifer Pumping Test Analysis and Evaluation of Specific Capacity and Well Efficiency Relationships, SL-1 Test Slant Well	Geoscience	Final Sept 2012
10.	Microbial Testing – Phase 3 Extended Pumping Study	Dr. Sunny Jiang, UCI	Final Nov 2012

Appendix

Project Photographs

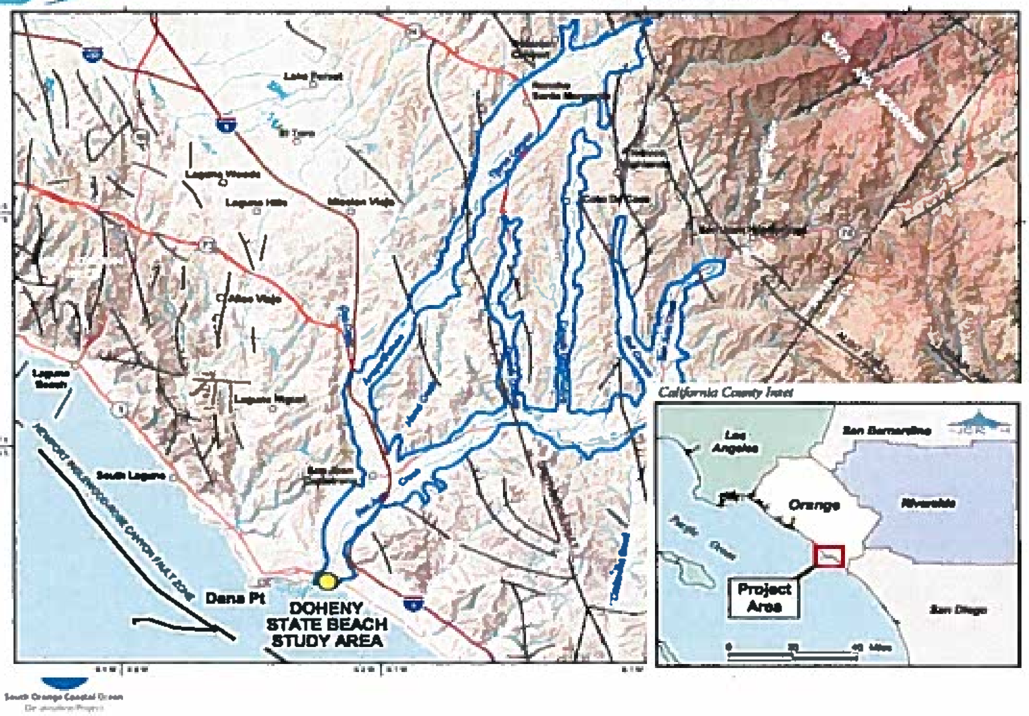
Groundwater Modeling Exhibits

Project Economic Analyses Scenarios

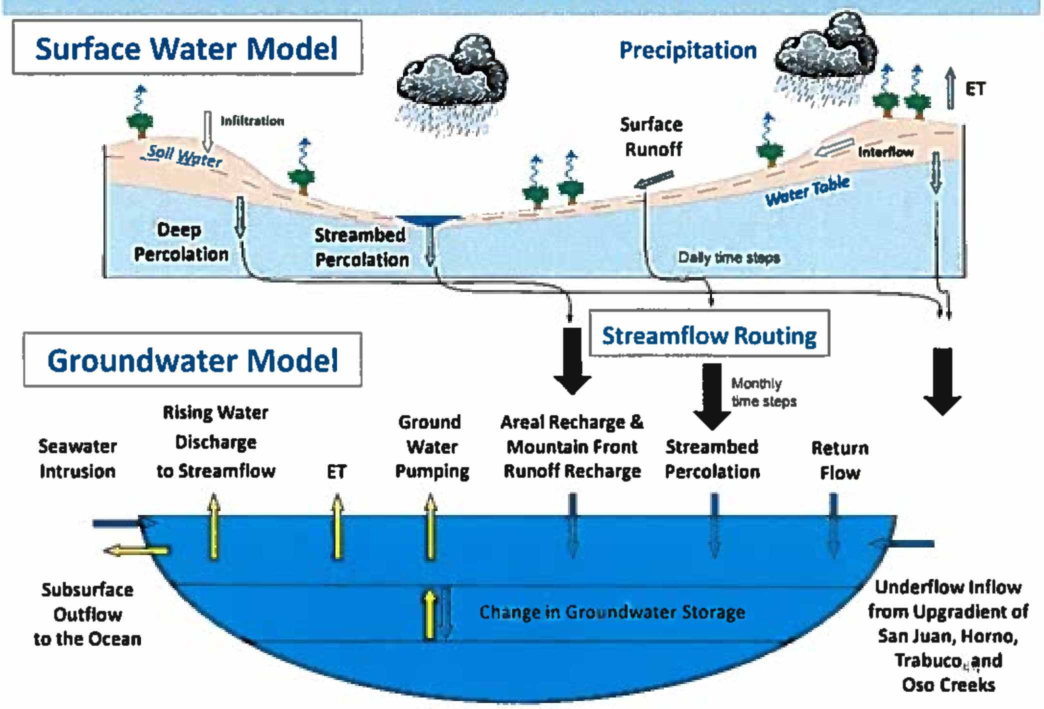
View of Slant Well and Test Facility Site Doheny State Beach

Mobile Test Facility

San Juan Groundwater Basin



Surface Water Model/Groundwater Model Interface



Project Economic Analyses Cases

Economic Analysis – Case 1 Base
No Fe/Mn Pre-treatment (with MITIGATION costs)

Doheny Desalination Project - Economic Analysis - Draft Version 1.8

Input

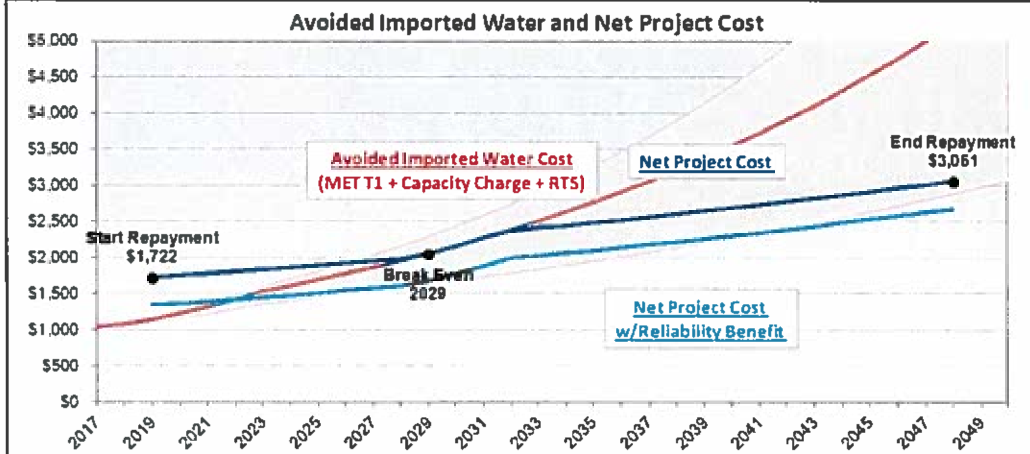
Capital Cost	
Fe/Mn Treatment?	No
Contingency of Const	25%
Start of Construction	2017

O&M Costs	
Energy Costs	
Energy Usage (kWh)	4,228
Base Energy Rate ($/kWh)	$ 0.105
Energy Escalation 2012 - 2030	2.0%
Energy Escalation after 2030	2.0%
Land Cost	
Land Cost per AF (2011)	$ 47
Lease Escalation Rate	2.5%
General Inflation Rate	
Current - 2020	2.5%
2021 - Future	2.5%

Project Financing	
Grant Amount	$
Bond Repayment	Select → 30
Bond Interest Rate	4.50%
PV Discount Rate	4.00%

MET Avoided Costs	
Esc. Rate After 2017	5.0%
Melded Rate- % Tier 1	100%

Output



Project Capital Costs	
Capital Cost (2012)	$130,227,550
Grants	$0
Contingency	$22,586,000
Total Cost (2011)	$152,813,550
Total Cost Escalated to Midpt. of Const.	$191,647,290

Costs per AF	Current Year 2012	First Year 2019
Capital Cost	$588	$699
O&M	$363	$489
Energy	$446	$522
Annual Cost	$1,397	$1,709
Annual Cost w/ Land	$1,444	$1,767
Mitigation Costs	$167	$205
Total w/ MET Subsidy	$1,361	$1,722

Net Present Value	
Up to Break Even	($31,436,811)
Break Even to End	$72,613,733
Total Savings	$41,176,923

Additional Reliability Benefit	
Storage Reservoir	$100,000,000

Economic Analysis – Case 2
Base Case with Fe/Mn Pretreatment (with MITIGATION costs)

Doheny Desalination Project - Economic Analysis - Draft Version 1.8

Input

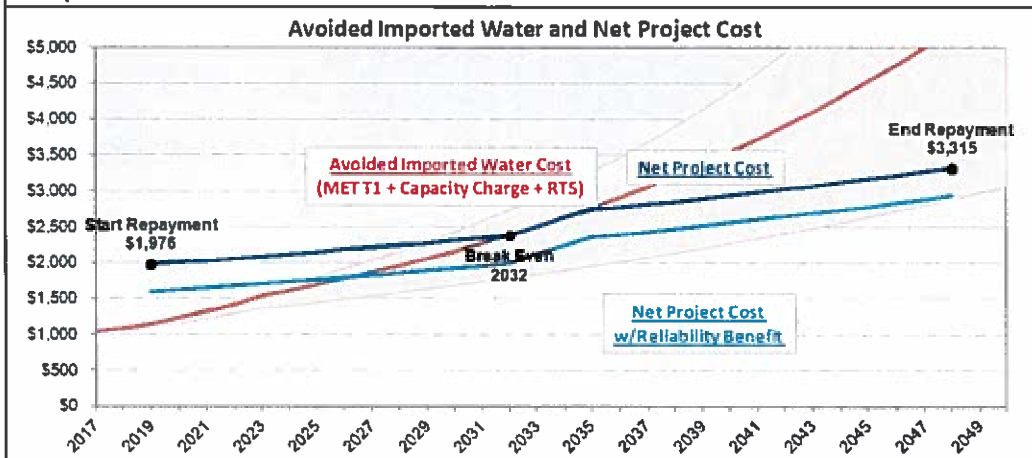
Capital Cost	
Fe/Mn Treatment?	Yes
Contingency of Const	25%
Start of Construction	2017

O&M Costs	
Energy Costs	
Energy Usage (kWh)	4,259
Base Energy Rate ($/kWh)	$ 0.105
Energy Escalation 2012 - 2030	2.0%
Energy Escalation after 2030	2.0%
Land Cost	
Land Cost per AF (2011)	$ 47
Lease Escalation Rate	2.5%
General Inflation Rate	
Current - 2020	2.5%
2021 - Future	2.5%

Project Financing	
Grant Amount	$
Bond Repayment	Select → 30
Bond Interest Rate	4.50%
PV Discount Rate	4.00%

MET Avoided Costs	
Esc. Rate After 2017	5.0%
Melded Rate- % Tier 1	100%

Output



Project Capital Costs	
Capital Cost (2012)	$172,234,800
Grants	$0
Contingency	$30,373,750
Total Cost (2011)	$202,608,550
Total Cost Escalated to Midpt. of Const.	$240,837,897

Costs per AF	Current Year 2012	First Year 2019
Capital Cost	$779	$926
O&M	$421	$489
Energy	$446	$522
Annual Cost	$1,646	$1,937
Annual Cost w/ Land	$1,693	$1,994
Mitigation Costs	$196	$231
Total w/ MET Subsidy	$1,640	$1,976

Net Present Value	
Up to Break Even	($59,808,624)
Break Even to End	$53,308,026
Total Savings	($6,500,598)

Additional Reliability Benefit	
Storage Reservoir	$100,000,000

Economic Analysis – Case 3
No Fe/Mn; High Electrical (with MITIGATION costs)

Doheny Desalination Project - Economic Analysis - Draft Version 1.8

Input

Capital Cost	
Fe/Mn Treatment?	No
Contingency of Const.	25%
Start of Construction	2017

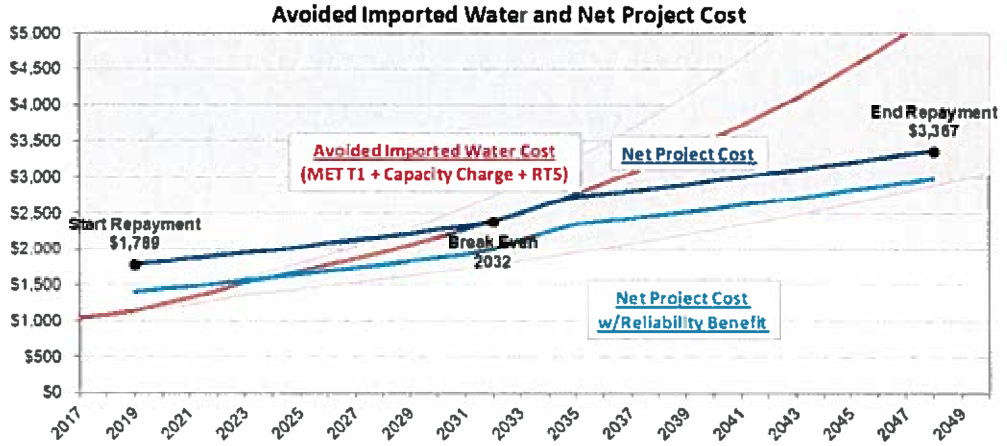
O&M Costs		
Energy Costs		
Energy Usage (kWh)		4,228
Base Energy Rate ($/kWh)	$	0.105
Energy Escalation 2012 - 2030		3.4%
Energy Escalation after 2030		2.0%
Land Cost		
Land Cost per AF (2011)	$	47
Lease Escalation Rate		2.5%
General Inflation Rate		
Current - 2020		2.5%
2021 - Future		2.5%

Project Financing		
Grant Amount		$
Bond Repayment	Select →	30
Bond Interest Rate	4.50%	
PV Discount Rate	4.00%	

MET Avoided Costs	
Esc. Rate After 2017	5.0%
Melded Rate- % Tier 1	100%

Output

Avoided Imported Water and Net Project Cost



Project Capital Costs	
Capital Cost (2012)	$130,227,550
Grants	$0
Contingency	$22,586,000
Total Cost (2011)	$152,813,550
Total Cost Escalated to Midpt. of Const.	$181,647,290

	Current Year	First Year
Costs per AF	2012	2019
Capital Cost	$588	$699
O&M	$363	$489
Energy	$446	$582
Annual Cost	$1,397	$1,769
Annual Cost w/ Land	$1,444	$1,827
Mitigation Costs	$167	$212
Total w/ MET Subsidy	$1,361	$1,789

Net Present Value	
Up to Break Even	($45,288,833)
Break Even to End	$52,460,361
Total Savings	$7,171,528

Additional Reliability Benefit	
Storage Reservoir	$100,000,000

Economic Analysis – Case 4
Base Case with $15M Grant; No Fe/Mn (with MITIGATION costs)

Doheny Desalination Project - Economic Analysis - Draft Version 1.8

Input

Capital Cost	
Fe/Mn Treatment?	No
Contingency of Const.	25%
Start of Construction	2017

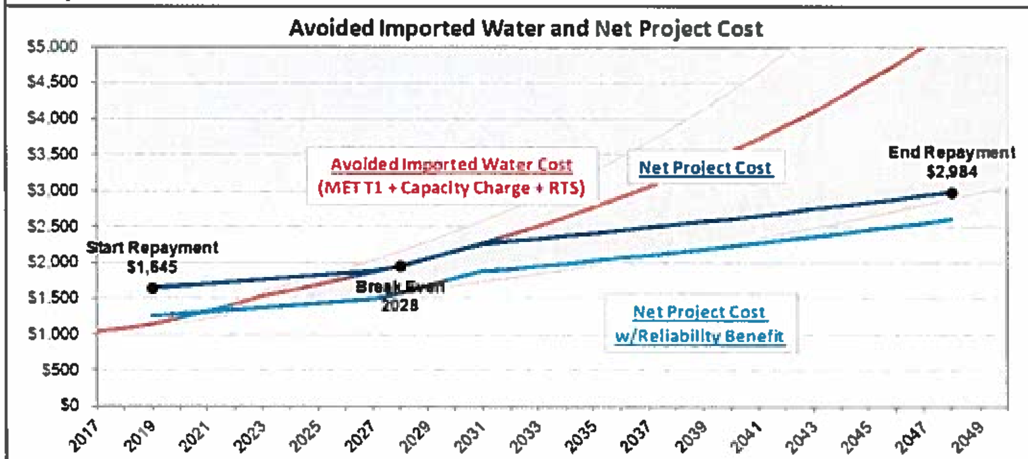
O&M Costs		
Energy Costs		
Energy Usage (kWh)		4,228
Base Energy Rate ($/kWh)	$	0.105
Energy Escalation 2012 - 2030		2.0%
Energy Escalation after 2030		2.0%
Land Cost		
Land Cost per AF (2011)	$	47
Lease Escalation Rate		2.5%
General Inflation Rate		
Current - 2020		2.5%
2021 - Future		2.5%

Project Financing		
Grant Amount		$ 15,000,000
Bond Repayment	Select →	30
Bond Interest Rate	4.50%	
PV Discount Rate	4.00%	

MET Avoided Costs	
Esc. Rate After 2017	5.0%
Melded Rate- % Tier 1	100%

Output

Avoided Imported Water and Net Project Cost



Project Capital Costs	
Capital Cost (2012)	$130,227,550
Grants	$15,000,000
Contingency	$22,586,000
Total Cost (2011)	$137,813,550
Total Cost Escalated to Midpt. of Const.	$153,817,004

	Current Year	First Year
Costs per AF	2012	2019
Capital Cost	$530	$630
O&M	$363	$489
Energy	$446	$522
Annual Cost	$1,339	$1,641
Annual Cost w/ Land	$1,386	$1,698
Mitigation Costs	$161	$197
Total w/ MET Subsidy	$1,297	$1,645

Net Present Value	
Up to Break Even	($24,278,982)
Break Even to End	$79,452,496
Total Savings	$55,173,514

Additional Reliability Benefit	
Storage Reservoir	$100,000,000

Economic Analysis – Case 5
Low Interest Rate; No Fe/Mn (with MITIGATION costs)

Doheny Desalination Project - Economic Analysis - Draft Version 1.8

Input

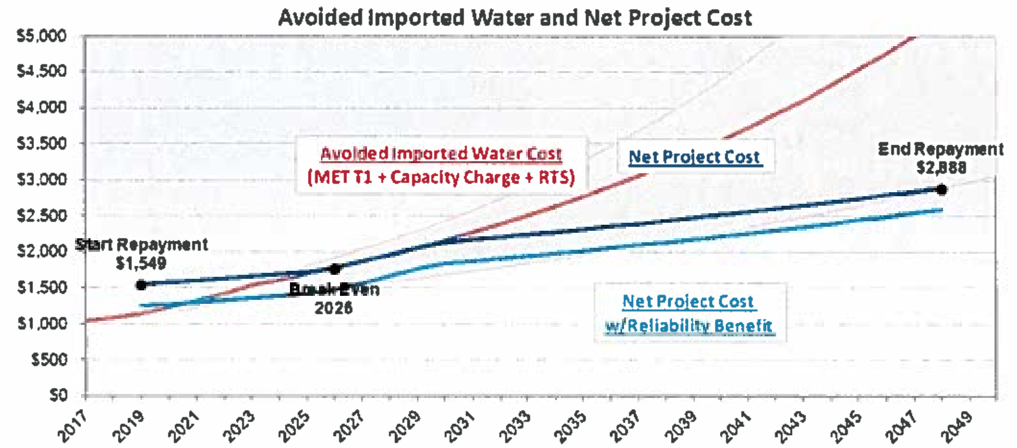
Capital Cost	
Fe/Mn Treatment?	No
Contingency of Const.	25%
Start of Construction	2017

O&M Costs		
Energy Costs		
Energy Usage (kWh)		4,228
Base Energy Rate ($/kWh)	$	0.105
Energy Escalation 2012 - 2030		2.0%
Energy Escalation after 2030		2.0%
Land Cost		
Land Cost per AF (2011)	$	47
Lease Escalation Rate		2.5%
General Inflation Rate		
Current - 2020		2.5%
2021 - Future		2.5%

Project Financing		
Grant Amount		$
Bond Repayment	Select —	30
Bond Interest Rate	2.50%	
PV Discount Rate	4.00%	

MET Avoided Costs	
Esc. Rate After 2017	5.0%
Melded Rate- % Tier 1	100%

Output



Project Capital Costs	
Capital Cost (2012)	$130,227,550
Grants	$0
Contingency	$22,586,000
Total Cost (2011)	$152,813,550
Total Cost Escalated to Midpt. of Const.	$181,647,290

	Current Year	First Year
Costs per AF	2012	2019
Capital Cost	$457	$544
O&M	$363	$489
Energy	$446	$522
Annual Cost	$1,266	$1,554
Annual Cost w/ Land	$1,313	$1,612
Mitigation Costs	$152	$187
Total w/ MET Subsidy	$1,216	$1,549

Net Present Value	
Up to Break Even	($16,445,195)
Break Even to End	$88,913,956
Total Savings	$72,468,761

Additional Reliability Benefit	
Storage Reservoir	$100,000,000

Economic Analysis – Case 6
Base with Low MET Escalation; No Fe/Mn (with MITIGATION costs)

Doheny Desalination Project - Economic Analysis - Draft Version 1.8

Input

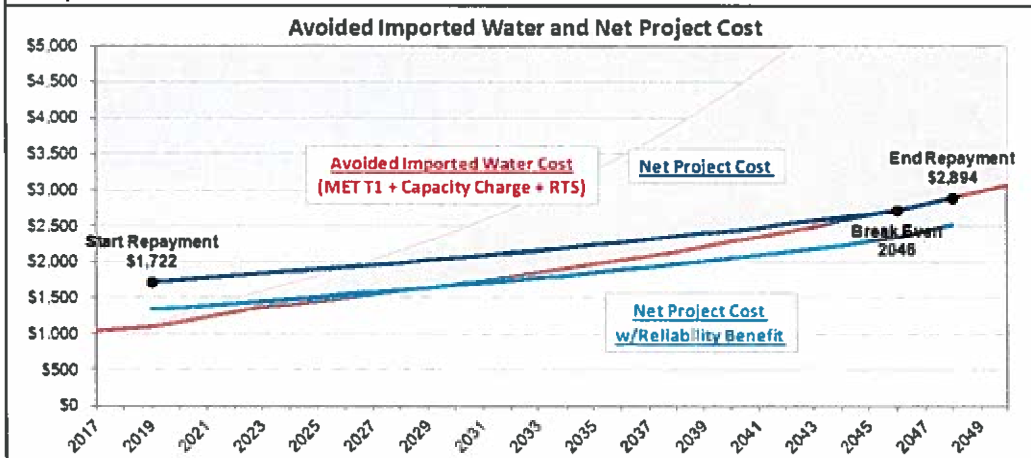
Capital Cost	
Fe/Mn Treatment?	No
Contingency of Const.	25%
Start of Construction	2017

O&M Costs		
Energy Costs		
Energy Usage (kWh)		4,228
Base Energy Rate ($/kWh)	$	0.105
Energy Escalation 2012 - 2030		2.0%
Energy Escalation after 2030		2.0%
Land Cost		
Land Cost per AF (2011)	$	47
Lease Escalation Rate		2.5%
General Inflation Rate		
Current - 2020		2.5%
2021 - Future		2.5%

Project Financing		
Grant Amount		$
Bond Repayment	Select —	30
Bond Interest Rate	4.50%	
PV Discount Rate	4.00%	

MET Avoided Costs	
Esc. Rate After 2017	3.0%
Melded Rate- % Tier 1	100%

Output



Project Capital Costs	
Capital Cost (2012)	$130,227,550
Grants	$0
Contingency	$22,586,000
Total Cost (2011)	$152,813,550
Total Cost Escalated to Midpt. of Const.	$181,647,290

	Current Year	First Year
Costs per AF	2012	2019
Capital Cost	$588	$699
O&M	$363	$489
Energy	$446	$522
Annual Cost	$1,397	$1,709
Annual Cost w/ Land	$1,444	$1,767
Mitigation Costs	$167	$205
Total w/ MET Subsidy	$1,361	$1,722

Net Present Value	
Up to Break Even	($72,772,358)
Break Even to End	$65,495,638
Total Savings	($7,276,720)

Additional Reliability Benefit	
Storage Reservoir	$100,000,000

Economic Analysis – Case 7
High Electrical & Fe/Mn Pre-Treatment (with MITIGATION costs)

Doheny Desalination Project - Economic Analysis - Draft Version 1.8

Input

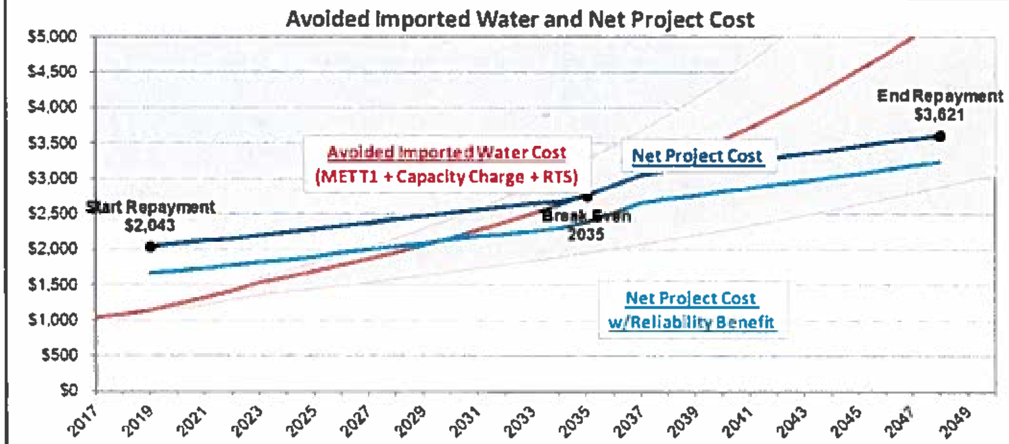
Capital Cost		
Fe/Mn Treatment?	Yes	
Contingency of Const.	25%	
Start of Construction		2017

O&M Costs		
Energy Costs		
Energy Usage (kWh)		4,259
Base Energy Rate ($/kWh)	$	0.105
Energy Escalation 2012 - 2030		3.4%
Energy Escalation after 2030		2.0%
Land Cost		
Land Cost per AF (2011)	$	47
Lease Escalation Rate	2.5%	
General Inflation Rate		
Current - 2020	2.5%	
2021 - Future	2.5%	

Project Financing		
Grant Amount		$ -
Bond Repayment	Select →	30
Bond Interest Rate	4.50%	
PV Discount Rate	4.00%	

MET Avoided Costs		
Esc. Rate After 2017	5.0%	
Melded Rate- % Tier 1	100%	

Output



Project Capital Costs	
Capital Cost (2012)	$172,234,800
Grants	$0
Contingency	$30,373,750
Total Cost (2011)	$202,608,550
Total Cost Escalated to Midpt. of Const.	$240,837,897

	Current Year	First Year
Costs per AF	2012	2019
Capital Cost	$779	$926
O&M	$421	$489
Energy	$446	$582
Annual Cost	$1,646	$1,997
Annual Cost w/ Land	$1,693	$2,054
Mitigation Costs	$196	$238
Total w/ MET Subsidy	$1,640	$2,043

Net Present Value	
Up to Break Even	($78,811,738)
Break Even to End	$68,285,103
Total Savings	($10,526,635)

Additional Reliability Benefit	
Storage Reservoir	$100,000,000

Economic Analysis – Case 8
Low MET Escalation with Fe/Mn Pre-Treatment (with MITIGATION costs)

Doheny Desalination Project - Economic Analysis - Draft Version 1.8

Input

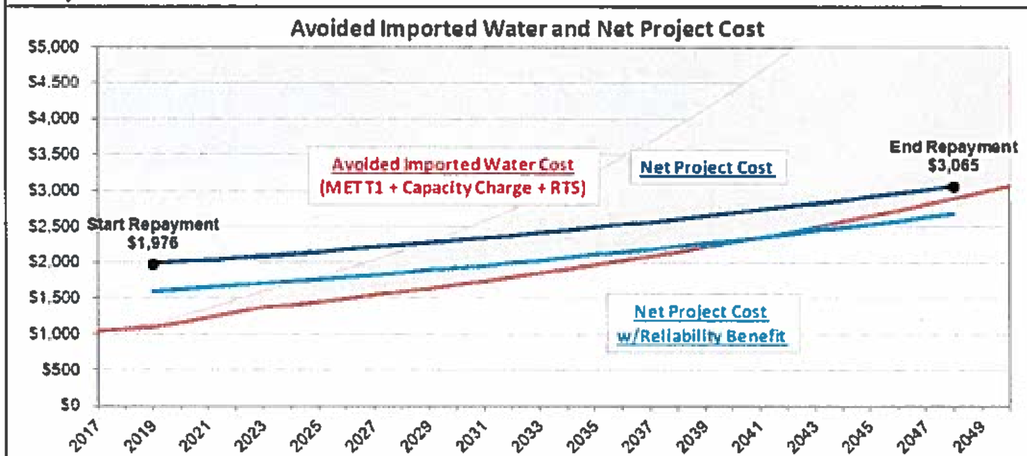
Capital Cost		
Fe/Mn Treatment?	Yes	
Contingency of Const.	25%	
Start of Construction		2017

O&M Costs		
Energy Costs		
Energy Usage (kWh)		4,259
Base Energy Rate ($/kWh)	$	0.105
Energy Escalation 2012 - 2030		2.0%
Energy Escalation after 2030		2.0%
Land Cost		
Land Cost per AF (2011)	$	47
Lease Escalation Rate	2.5%	
General Inflation Rate		
Current - 2020	2.5%	
2021 - Future	2.5%	

Project Financing		
Grant Amount		$ -
Bond Repayment	Select →	30
Bond Interest Rate	4.50%	
PV Discount Rate	4.00%	

MET Avoided Costs		
Esc. Rate After 2017	3.0%	
Melded Rate- % Tier 1	100%	

Output



Project Capital Costs	
Capital Cost (2012)	$172,234,800
Grants	$0
Contingency	$30,373,750
Total Cost (2011)	$202,608,550
Total Cost Escalated to Midpt. of Const.	$240,837,897

	Current Year	First Year
Costs per AF	2012	2019
Capital Cost	$779	$926
O&M	$421	$489
Energy	$446	$522
Annual Cost	$1,646	$1,937
Annual Cost w/ Land	$1,693	$1,994
Mitigation Costs	$196	$231
Total w/ MET Subsidy	$1,640	$1,976

Net Present Value	
Up to Break Even	($125,500,571)
Break Even to End	$115,260,666
Total Savings	($10,239,905)

Additional Reliability Benefit	
Storage Reservoir	$100,000,000

Economic Analysis – Case 9
Low MET Escalation with Low Interest (with MITIGATION costs)

South Orange Coastal Desalination Project - Economic Analysis - DRAFT VERSION 1.8

Input

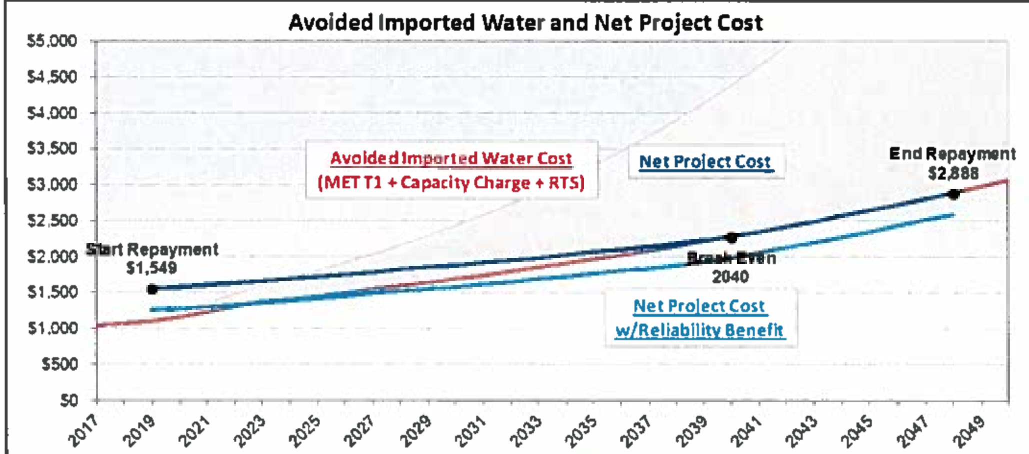
Capital Cost		
Fe/Mn Treatment?	No	
Contingency of Const.	25%	
Start of Construction		2017

O&M Costs		
Energy Costs		
Energy Usage (kWh)		4,228
Base Energy Rate ($/kWh)		$ 0.105
Energy Escalation 2012 - 2030		2.0%
Energy Escalation after 2030		2.0%
Land Cost		
Land Cost per AF (2011)	$ 47	
Lease Escalation Rate	2.5%	
General Inflation Rate		
Current - 2020	2.5%	
2021 - Future	2.5%	

Project Financing		
Grant Amount		$
Bond Repayment	Select --	30
Bond Interest Rate	2.50%	
PV Discount Rate	4.00%	

MET Avoided Costs	
Esc. Rate After 2017	3.0%
Melded Rate- % Tier 1	100%

Output



Project Capital Costs	
Capital Cost (2012)	$130,227,550
Grants	$0
Contingency	$22,586,000
Total Cost (2011)	$152,813,550
Total Cost Escalated to Midpt. of Const.	$181,647,290

	Current Year	First Year
Costs per AF	**2012**	**2019**
Capital Cost	$457	$544
O&M	$363	$489
Energy	$446	$522
Annual Cost	$1,266	$1,554
Annual Cost w/ Land	$1,313	$1,612
Mitigation Costs	$152	$187
Total w/ MET Subsidy	$1,216	$1,549

Net Present Value	
Up to Break Even	($40,765,490)
Break Even to End	$35,505,320
Total Savings	($5,260,170)

Additional Reliability Benefit	
Storage Reservoir	$100,000,000

EXHIBIT H

California Emissions Estimator Model®

User's Guide

Version 2016.3.2

Prepared for:
California Air Pollution Control Officers Association (CAPCOA)

Prepared by:
BREEZE Software, A Division of Trinity Consultants
in collaboration with South Coast Air Quality Management District and the California Air Districts

Date:
November 2017

Acknowledgements

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Table of Contents

List of Appendices

1 Introduction

This User's Guide (Guide) to the California Emission Estimator Model (CalEEMod)® is meant to give the user an introduction on how to use the program as well as to document the detailed calculations and default assumptions made in associated appendices. The purpose of CalEEMod is to provide a uniform platform for government agencies, land use planners, and environmental professionals to estimate potential emissions associated with both construction and operational use of land use projects. It is intended that these emission estimates are suitable for quantifying air quality and climate change impacts as part of the preparation of California Environmental Quality Act (CEQA) documents. In addition, individual districts may rely on the model's emission estimates to show compliance with local agency rules.

CalEEMod utilizes widely accepted methodologies for estimating emissions combined with default data that can be used when site-specific information is not available. Sources of these methodologies and default data include but are not limited to the United States Environmental Protection Agency (USEPA) AP-42 emission factors, California Air Resources Board (CARB) vehicle emission models, studies commissioned by California agencies such as the California Energy Commission (CEC) and CalRecycle. In addition, some local air districts provided customized values for their default data and existing regulation methodologies for use for projects located in their jurisdictions. When no customized information was provided and no regional differences were defined for local air districts, then state-wide default values were utilized. Since resource data and regulations are constantly changing, local agencies should be consulted to determine whether there are any circumstances when updated values should be used in place of the defaults currently incorporated into CalEEMod. A majority of CalEEMod's default data associated with locations and land use is derived from surveys of existing land uses. For any project that substantially deviates from the types and features included in the surveys, site-specific data that are supported by substantial evidence should be used, if available.

The model provides a number of opportunities for the user to change the defaults in the model; however, users are required to provide justification for all changes made to the default settings (e.g., reference more appropriate data sources) in the Remarks box provided at the bottom of the screen before the user will be able to proceed to the next screen. Further, the user should make every effort to ensure that correct data is entered, including the choice and percent reduction of mitigation most applicable to the land use project being evaluated.

1.1 Purpose of Model

CalEEMod provides a simple platform to calculate both construction emissions and operational emissions from a land use project. It can calculate both the daily maximum and annual average for criteria pollutants as well as annual greenhouse gas (GHG) emissions. The output from these calculations can be used in the preparation of quality and GHG analyses in CEQA documents such as Environmental Impact Reports (EIRs) and Negative Declarations. For projects located in the jurisdiction of San Luis Obispo APCD, the model can also calculate the sum of reactive organic gas (ROG) and nitrogen oxide (NO_x) emissions on a rolling quarterly basis. In addition, CalEEMod contains default values for estimating water and energy use which may be useful for preparing hydrology and energy analyses in other sections of a CEQA document. Specifically, the model can aid the user by conducting the following calculations:

- Short-term construction emissions associated with the demolition, site preparation, grading, building, coating, and paving from the following sources:
  - Off-road construction equipment;
  - On-road mobile equipment associated with workers, vendors, and hauling;
  - Fugitive dust associated with grading, demolition, truck loading, and on-road vehicles traveling along paved and unpaved roads. (Fugitive dust from windblown sources such as storage piles and inactive disturbed areas, as well as fugitive dust from off-road vehicle travel, are not quantified in CalEEMod, which is consistent with approaches taken in other comprehensive models.)
  - Architectural coating activities *(including the painting/striping of parking lots)* and paving (ROG).
- Operational emissions for fully built-out land use development from the following sources:
  - On-road mobile vehicle traffic generated by the land uses;
  - Fugitive dust associated with roads;
  - Architectural coating activities (ROG);
  - Off-road equipment (e.g., forklifts, cranes) used during operation;
  - Landscaping equipment;
  - Emergency generators, fire pumps, and process boilers;
  - Use of consumer products, parking lot degreasers, fertilizers/pesticides, and cleaning supplies (ROG);
  - Wood stoves and hearth usage;
  - Natural gas usage in the buildings;
  - Electricity usage in the buildings (GHG only);
  - Electricity usage from lighting in parking lots and lighting, ventilation and elevators in parking structures;
  - Water usage per land use (GHG only); and,
  - Solid waste disposal per land use (GHG only).
- One-time vegetation sequestration changes
  - Permanent vegetation land use changes
  - New tree plantings

- Mitigation adjustments to both short-term construction and operational emissions. Several of the mitigation measures described in CAPCOA's Quantifying Greenhouse Gas Mitigation Measures[1] have been incorporated into CalEEMod.

[1] Available at: http://www.capcoa.org/wp-content/uploads/2010/11/CAPCOA-Quantification-Report-9-14-Final.pdf

2 Program Installation

The program is distributed and maintained by the California Air Pollution Control Officers Association[2]. The most recent version can be downloaded from www.caleemod.com.

2.1 Operating System Requirements

CalEEMod was programmed by Trinity using Microsoft SQL Compact Edition in conjunction with a Visual Basic Graphical User Interface (GUI). CalEEMod requires the following system requirements:

- Microsoft Windows 8 or 10 Operating System with Microsoft .NET Framework 3.5 (includes .NET 2.0 and 3.0)
- Microsoft Windows XP, Vista, or 7 Operating System with Microsoft .Net Framework 4 or higher
- Microsoft SQL Server Compact 3.5 SP2
- Microsoft Access Database Engine 2010 Redistributable, 32-bit
- 300 Mb hard drive space available

2.2 Installation Procedures

To install:

1. Ensure you have the required Microsoft .Net framework installed on your machine. Microsoft .NET Framework 3.5 is available for free from Microsoft at: https://www.microsoft.com/en-us/download/details.aspx?id=21. Microsoft .NET Framework 4.0 or higher is available free from Microsoft at: https://www.microsoft.com/en-us/download/details.aspx?id=17851. Once the file is downloaded, unzip the file anywhere on your computer and run the installation file (setup.exe) and follow the instructions on Microsoft's website to locate the appropriate .msi file.

2. To install Microsoft SQL Server Compact 3.5 SP2, go to https://www.microsoft.com/en-us/download/details.aspx?id=5783. For 32-bit computers, you will need to install SSCERuntime_x86-ENU.msi. For a 64-bit computer, you will need to install both the 32-bit and the 64-bit version of the SQL Server Compact 3.5 SP2 MSI files because the existing SQL Server Compact 3.5 applications may fail if only the 32-bit version of the .msi file is installed on the 64-bit computer.

3. To install 32-bit Microsoft Access Database Engine 2010 Redistributable, go to https://www.microsoft.com/en-us/download/details.aspx?id=13255&751be11f-ede8-5a0c-058c-2ee190a24fa6=True, click on Download, select "AccessDatabaseEngine.exe" (25.3 MB), and click on Next. Once this file is

[2] CalEEMod® 2017 All Rights Reserved by California Air Pollution Control Officers Association.

downloaded, double click on "AccessDatabaseEngine.exe" file and follow the on-screen instructions to finish the installation.

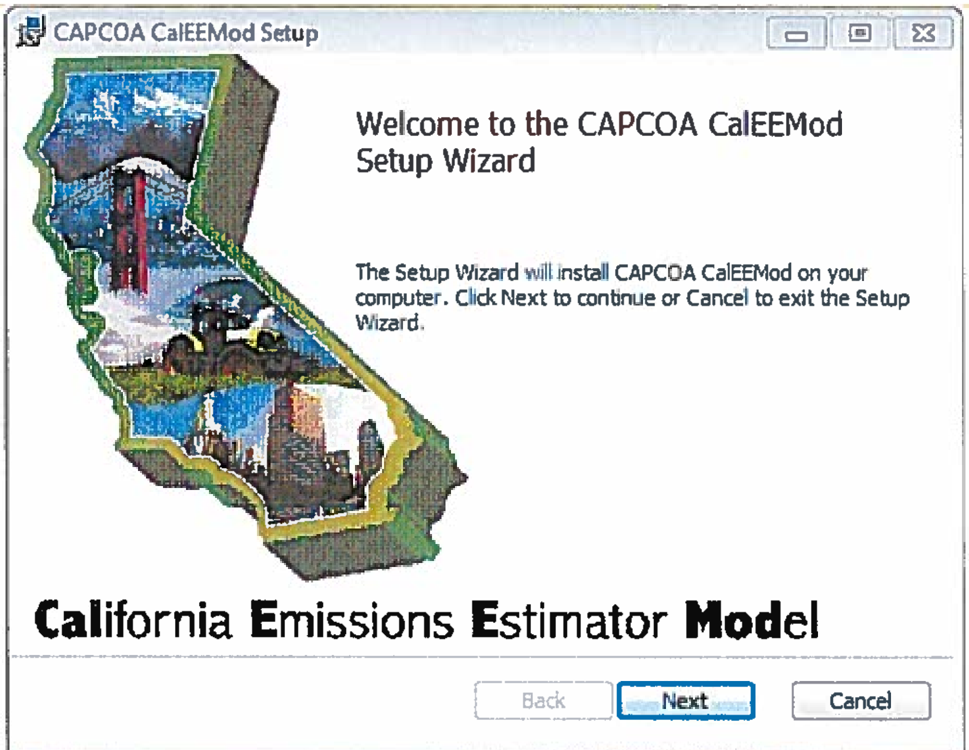
4. From www.CalEEMod.com, download the installation file (CalEEMod.WixSetup 2016.3.2.25.msi), click on the file and follow the instructions. Pages 6 through 8 show screen shots of the CalEEMod Windows Installer XML (WiX) Setup Wizard.

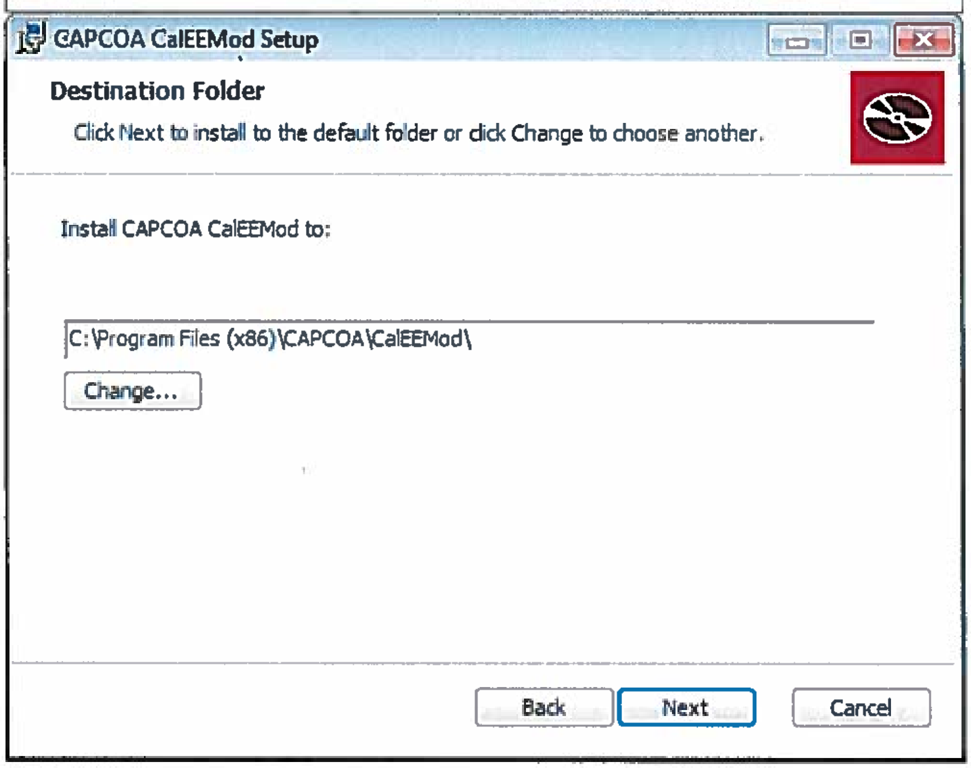
5. CalEEMod version 2016.3.2 can be installed side by side with version 2016.3.1 provided that each version is installed in different folders. For 32-bit computers, the default directory for CalEEMod version 2016.3.2 is C:\Program Files\CAPCOA\CalEEMod; for 64-bit computers, the default directory for CalEEMod version 2016.3.2 is C:\Program Files (x86)\CAPCOA\CalEEMod. If you want to run CalEEMod version 2016.3.2 side by side with CalEEMod version 2016.3.1 but CalEEMod version 2016.3.1 is already installed in C:\Program Files\CAPCOA\CalEEMod on a 32-bit computer or C:\Program Files (x86)\CAPCOA\CalEEMod on a 64-bit computer, click on Change to change the destination folder[3].

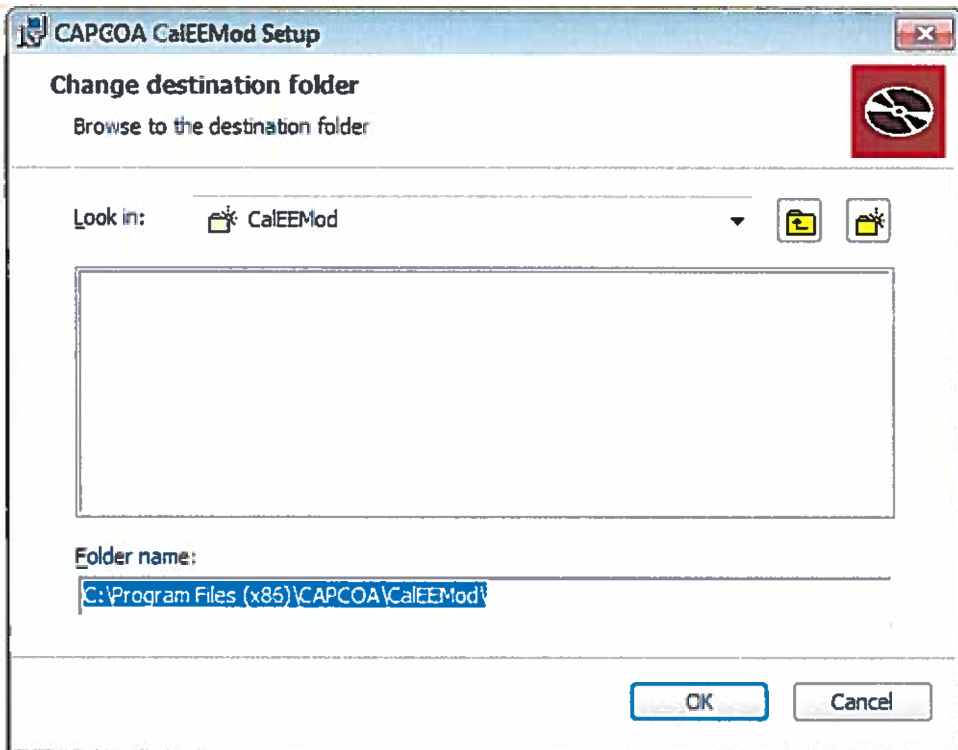
6. Click Next until the installation has completed, then click Finish to exit the installer.

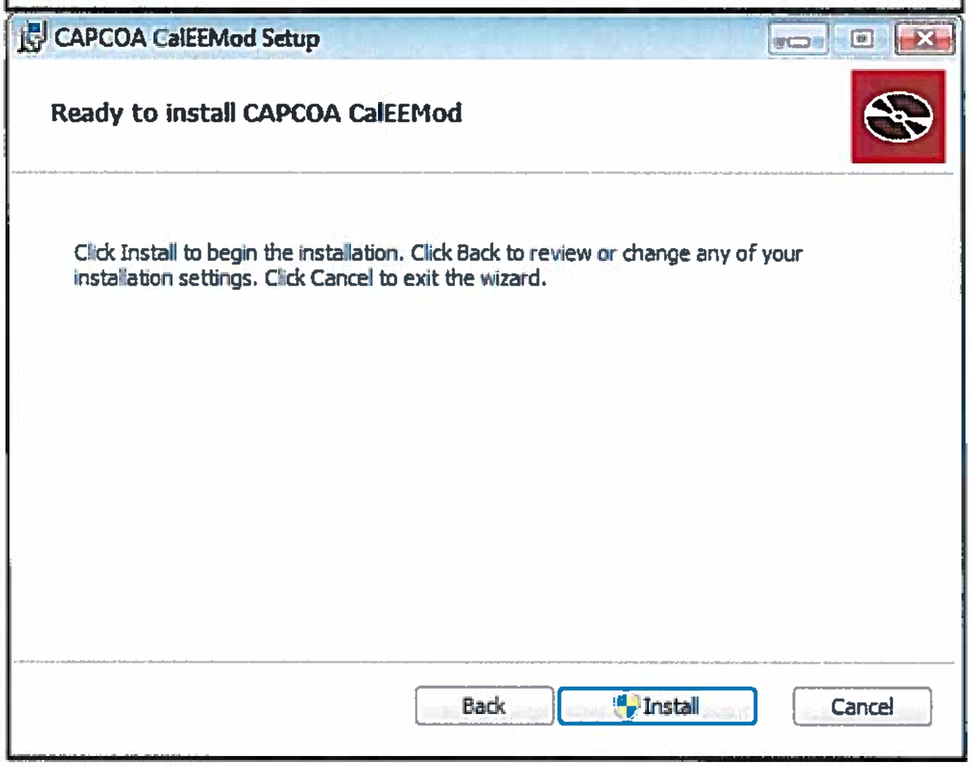
7. If you have any further trouble installing CalEEMod, verify that you have appropriate user privileges and that your computer meets the operating system requirements.

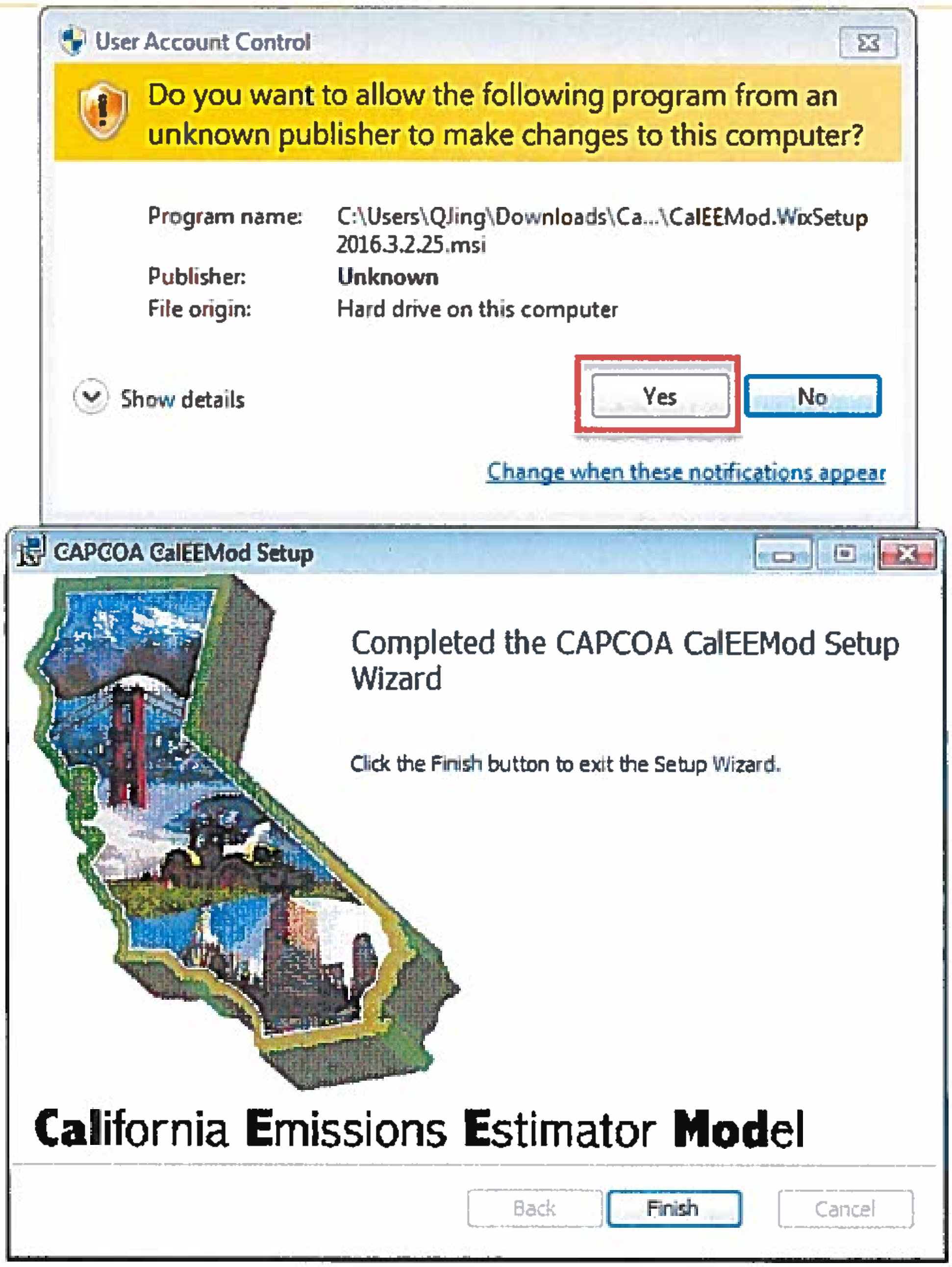
[3] If you use Windows Vista, 7, 8 or 10, file privileges may not allow access rights to some folders during program operations such as C:\Program Files\.


CAPCOA CalEEMod Setup
Welcome to the CAPCOA CalEEMod Setup Wizard
The Setup Wizard will install CAPCOA CalEEMod on your computer. Click Next to continue or Cancel to exit the Setup Wizard.
California Emissions Estimator Model
Back
Next
Cancel


CAPCOA CalEEMod Setup
Destination Folder
Click Next to install to the default folder or click Change to choose another.
Install CAPCOA CalEEMod to:
C:\Program Files (x86)\CAPCOA\CalEEMod\
Change...
Back
Next
Cancel

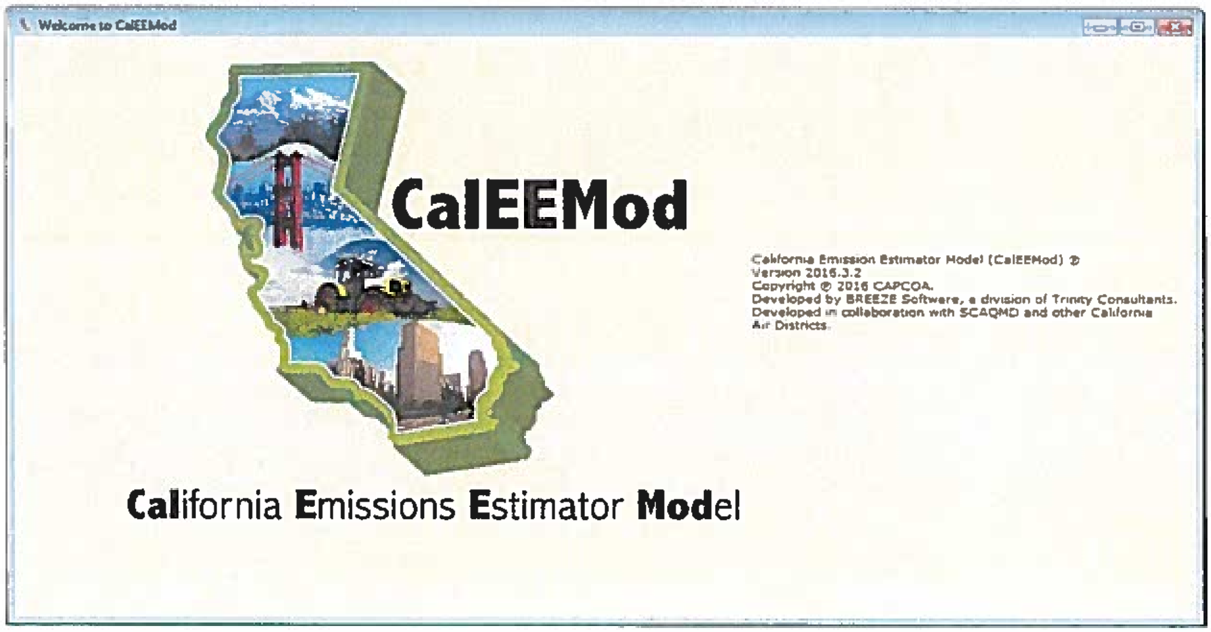

CAPCOA CalEEMod Setup
Change destination folder
Browse to the destination folder
Look in:
CalEEMod
Folder name:
C:\Program Files (x86)\CAPCOA\CalEEMod\
OK
Cancel


CAPCOA CalEEMod Setup
Ready to install CAPCOA CalEEMod
Click Install to begin the installation. Click Back to review or change any of your installation settings. Click Cancel to exit the wizard.
Back
Install
Cancel


User Account Control
Do you want to allow the following program from an unknown publisher to make changes to this computer?
Program name:
C:\Users\QJing\Downloads\Ca...\CalEEMod.WixSetup 2016.3.2.25.msi
Publisher:
Unknown
File origin:
Hard drive on this computer
Show details
Yes
No
Change when these notifications appear
CAPCOA CalEEMod Setup
Completed the CAPCOA CalEEMod Setup Wizard
Click the Finish button to exit the Setup Wizard.
California Emissions Estimator Model
Back
Finish
Cancel

2.3 Starting CalEEMod

After the installation is complete, a CalEEMod short cut icon will be appear on the desktop and CalEEMod will appear in the list of Programs available from the Start Button. To start the model, select CalEEMod from the program files or double click on the CalEEMod short cut icon.



3 Using CalEEMod

3.1 Key Features

CalEEMod is comprised of a linear series of screens with each screen designed with an individual purpose to define features of the project such as project characteristics, construction schedule and equipment, operational activity, mitigation measures, etc. The user will need to input basic information about the project such as location, land use type (e.g., residential, commercial, retail, etc.) and project size and the model will populate later screens with pre-determined defaults. The user may override the defaults to input more accurate, project-specific information as appropriate.

The figure on page 11 identifies some key features of CalEEMod which are described below.

1. Menu Bar: A drop down menu bar is found on all screens. For example, the Home menu controls file features such as New Project, Open Project, Save Project, and Save As Project. The Help menu will link to appropriate information for the relevant screen from this User's Guide. All of the other menus will allow navigation between the screens in any order.

2. Screen Name: Identifies the name of the current screen.

3. Default Button: This button allows the user to restore the program defaults after the user has changed any default values on the screen. User-entered values will be highlighted in yellow to clearly indicate the defaults that have been changed. The user will be prompted to specify whether the default should be restored for the current or last cell on the screen or for the entire screen. The Import csv option will allow the user to load in a .csv file for a specific data grid. Clicking on the Undo button will allow the user to cancel or undo the previous action.

4. Remarks: This section is located at the bottom of each screen and it requires the user to enter comments regarding any defaults that have been replaced with user-defined values. The Remarks section is meant to assist project reviewers to determine or assess the justification for user-defined values entered.

5. Next Button: When the user clicks on this button, the next sequential screen will appear. As the user progresses through the model, later screens will also show a Previous button that will take the user to the previous screen.

6. Data Grid: This is a common box where values for the variables defined across the top are to be filled in with data. The number of rows will automatically be adjusted based on the number of rows of information required to define the information. On some data grids, the last row may have an asterisk (*) and once the user begins adding information to this row, a new row will be added at the end. To delete a row, select the desired row to delete, and hit the delete button on your keyboard. (Deleting information is generally allowed unless the data grid contains a fixed list such as the Pollutant selection list.) Scroll bars (both horizontal and vertical) may also occur on some data grids, as appropriate.

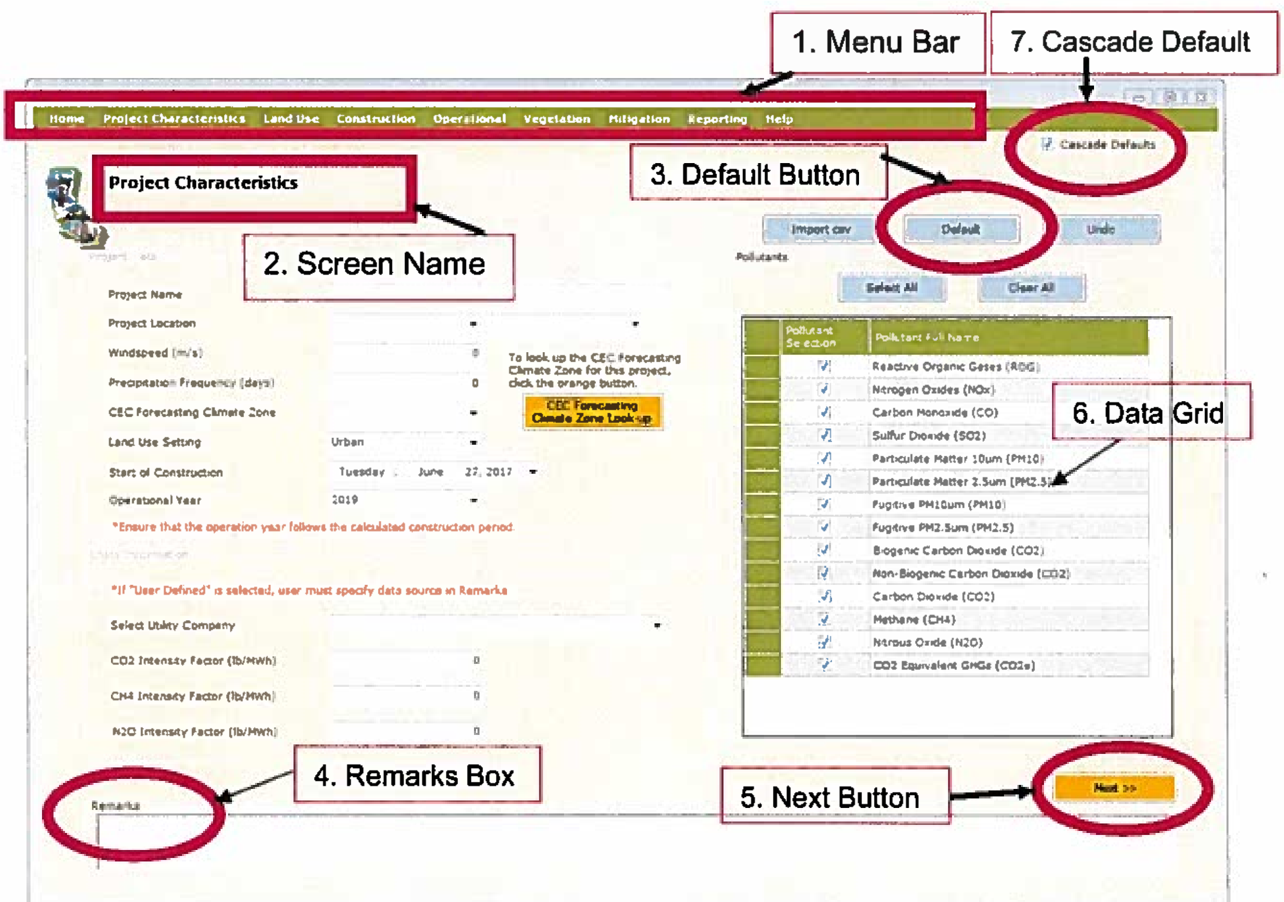
7. Cascade Defaults: CalEEMod has a feature that freezes the automatic downloading of the programmed defaults. Each input screen displays a box called Cascade Default which will be automatically checked to populate defaults in future screens. However, if user unchecks the Cascade Default box, no defaults will be populated in subsequent screens and the user will need to input project-specific data. Unless all the necessary input parameters required for a proper analysis are known, the user should run the model at least once with "Cascade Default" button checked to allow the defaults to be populated. Then, if the user would like to change the project's parameters (e.g., number of dwelling units, building square footage, etc.) without cascading new defaults in later screens, then the user should uncheck the Cascade Default box when in the Land Use screen. This feature may be helpful when the defaults are replaced with project-specific information (e.g., construction schedule, construction equipment, water use, energy use, etc.) and the user would like to evaluate different project scenarios with the same basic project information (e.g., land use type, location, etc.). In addition, by unchecking the Cascade Default box, the following will occur:

   - The defaults in *ALL* subsequent screens will be frozen.
   - Any changes that are made to screens that follow the Land Use screen (e.g., adding a new construction phase) will not cascade defaults relating to that change or add

new tabs (e.g., trips and VMT, dust material movement). Thus, the user will need to manually input project-specific information in order for the impacts to be calculated.

- If any changes to land use type (e.g., from single family housing to a hospital) are made, the subsequent screens will not reflect the new land use type causing some incorrect calculations (e.g., impacts from energy and water use) to be performed.

When changing or adding a land use type, the user should click on the Cascade Default button so the future screens will be populated with appropriate defaults and the correct calculations specific to the changed or added land use type will occur.



3.2 Home

The Home tab on the file menu bar that controls the file saving and opening features. The available options are:

- New Project
- Open Project
- Save
- Save As
- Exit

The user should select Open Project to open a project that has been previously created and saved or New Project to create a new project. Note that opening a previously saved project will remove any information that has been entered into the GUI unless it has been saved to a file. Save will save the currently loaded project database as a Microsoft Excel file and this file can be closed, and then re-opened later. Save As will allow the user to change the name of the saved project file. Exit will close CalEEMod. The Microsoft Excel file can be edited following the format of the save file to quickly make edits outside of the Graphical User Interface (GUI) but the user will still need to use the GUI in order to report the results. This can be most useful in making changes to construction lists. Data for individual tabs can be uploaded as a .csv file in various places in CalEEMod to minimize the data entry.

3.3 Defining a Project

In order to define a project, the user will need to enter information on both the Project Characteristics screen and the Land Use screen. After entering information on these two screens, CalEEMod will populate all of the other information required to calculate unmitigated construction (unless there is demolition, grading, or site preparation) and operation emissions using default data. If demolition, grading, and/or site preparation activities are part of the project, then the user will need to enter additional information on the appropriate construction screens, including but not limited to, the amount of material to be demolished and transported to or from the site. If site-specific information is not needed for the project, the user can skip this part and jump to the Mitigation screen and enter mitigation measures. After completing the Mitigation screen, the user can proceed to the Reporting screen to select the type of report to be generated for the project.

3.4 Altering Default Data

CalEEMod was designed with default assumptions supported by substantial evidence to the extent available at the time of programming. The functionality and content of CalEEMod is based on fully adopted methods and data. However, CalEEMod was also designed to allow the user to change the defaults to reflect site- or project-specific information, when available, provided that the information is supported by substantial evidence as required by CEQA. If the user chooses to modify any defaults, an explanation will be required in the Remarks box found at the bottom of the screen to justify and support the modification before the user will be able to proceed to the next screen. Modifications to defaults and the explanations are noted in the output report. Comments in the Remarks box are also included in the report and alert reviewers of modifications to the defaults. Comments are important because they show the user's justification for the modifications, which allows the reviewers the ability to determine whether or not the modifications are appropriate and sufficiently justified.

3.5 Mitigation

Common construction mitigation measures that impact the calculations in CalEEMod have been incorporated as options for the user to select. It is important to note that compliance with fugitive dust rules vary widely by district and include requirements to reduce dust. Even though the fugitive dust rules contain requirements that when implemented, have the effect of mitigating dust emissions, these requirements are not considered to be mitigation per se. For these reasons, requirements such as percentage adjustments to fugitive dust rules have not been incorporated into the unmitigated fugitive dust calculations.

Several mitigation measures from CAPCOA's Quantifying Greenhouse Mitigation Measures have been incorporated including combinations and caps when using multiple mitigation measures. CalEEMod was designed to include typical mitigation measures that are some of the more effective measures available to development projects. If mitigation measures are not available as options in CalEEMod, the user can alter the inputs in the program to adjust to account for mitigation measures that may be less common. This will require separate runs of CalEEMod files in order to properly account for unmitigated and mitigated scenarios. For more details regarding mitigation, see Subchapter 4.11.

3.6 Reporting

The Reporting tab allows the user to select the type of report (e.g., annual, winter or summer) to present the results of the calculations. The reports can be viewed on screen and then saved as either a Microsoft Excel file or a .pdf file. For more details regarding reporting, see Subchapter 4.11.

4 Detailed Program Screens

4.1 Project Characteristics

The Project Characteristics screen is starting point where the user enters the project name, project location, and selects utility provider, climate zone, and pollutants to be analyzed. The information entered on this screen will trigger project appropriate default data to populate subsequent screens. Any changes entered on this screen will override any previously entered user-defined data and the corresponding default data. The project name will appear in the reports. Each of the information categories on this screen are described in more detail below.

Project Location

To define the region where the project is located, the user is given the option to select Air District, Air Basin, County, or Statewide. The second drop down box will reveal a list of specific locations to the region selected. If the user selects County, It is important to note that there may be some counties that are shared by multiple Air Districts, Air Basins or District-specific sub-regions and the default values (e.g., on-road vehicle emissions, trip lengths, water supply and treatment electricity use, solid waste disposal rates, amount of paved roads, days of landscaping equipment use, architectural coating emissions, and hearth usage) may vary accordingly. Thus, if the user selects County, the user may also be prompted to select the sub-county area. If you are uncertain about what region to choose for your project location, consult your lead agency.

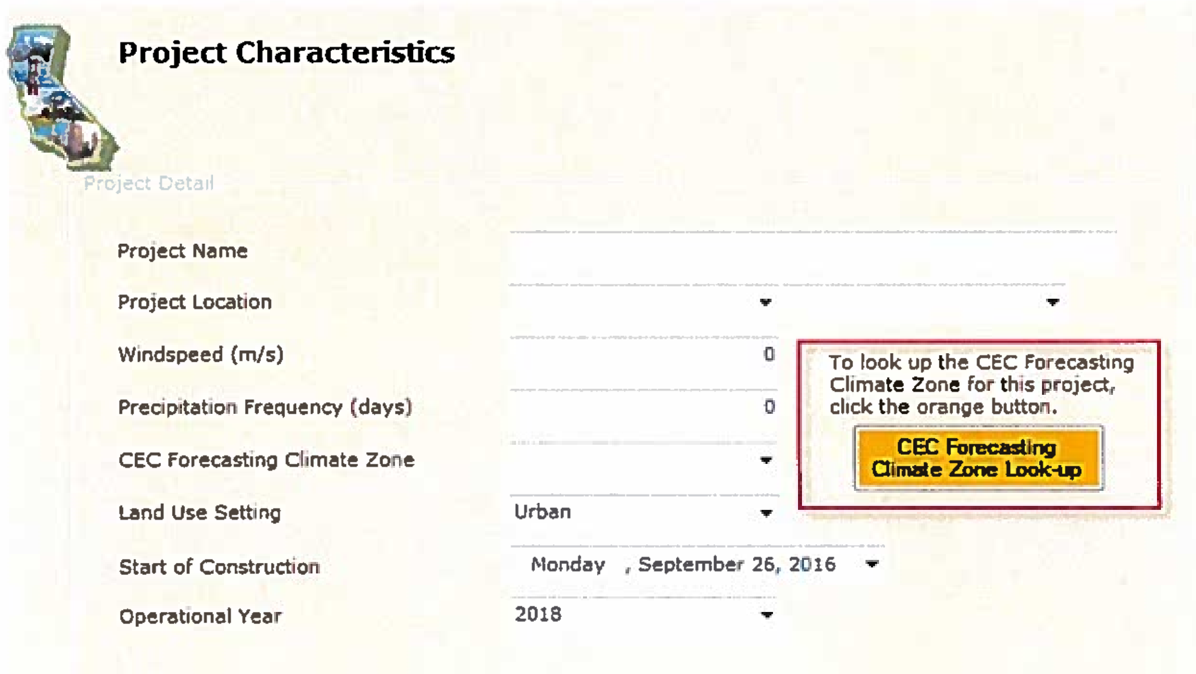
Wind Speed and Precipitation Frequency

Selection of project location will automatically fill in the default wind speed and precipitation frequency. The user can also choose to override this information and enter a different value. The wind speed, in meters per second (m/s), is used in the fugitive dust calculations. Precipitation frequency, e.g. the number of days per year with a precipitation amount measuring greater than 0.01 inches in one day, is used in the fugitive dust calculations.

Climate Zone

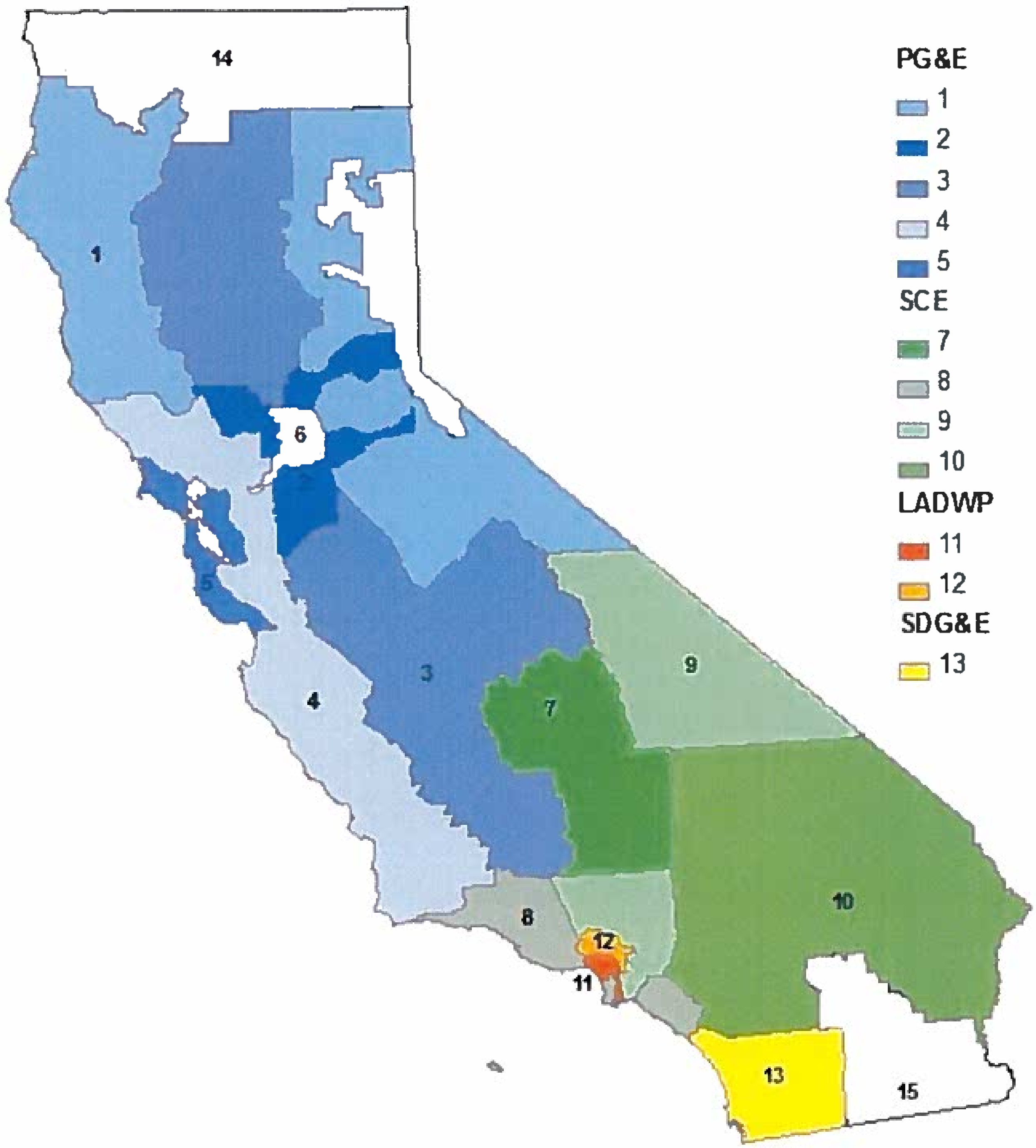
Selection of project location will restrict the climate zones available for the user to choose from based on the climate zones in the project location. The climate zones that have been programmed into CalEEMod are based on the California Energy Commission's (CEC) Forecasting Climate Zones, which are different from the Title 24 Building Climate Zones. The user should determine the correct climate zone by either referring to the figure below or by clicking on the orange button that says "CEC Climate Zone Forecasting Look-up" on the Project Characteristics screen. In addition, the user may also determine the climate zone by city or zip code from the look up tables in Appendix F.

CEC Forecasting Climate Zone Look-up Button



CalEEMod utilizes the Forecasting Climate Zones because the baseline data in the 2002 California Commercial End Use Survey (CEUS) and 2009 Residential Appliance Saturation Survey (RASS), upon which CalEEMod relies, are categorized in this manner. Further information on the calculation of building energy usage, including the application of data specific to the Forecasting Climate Zones, is contained in Appendix E.

CEC Forecasting Climate Zones[4, 5]



[4] Adapted from Figure ES-2 of CEC. 2010. Residential Appliance Saturation Survey. Available at: http://www.energy.ca.gov/2010publications/CEC-200-2010-004/CEC-200-2010-004-ES.PDF

[5] White spaces represent areas served by other electric utilities not included in survey.

Land Use Setting

The Land Use Setting tab is where the user indicates whether the project is located in a rural or urban setting. The user should contact the local air district for the region where the project is located for guidance on the appropriate Land Use Setting to select.

Start of Construction

To indicate when construction of the project will begin, the user will need to insert a date in the Start of Construction field. The date when construction will start triggers a rolling calendar that starts with the construction start date and follows by various construction phases that will be populated with default date ranges in the Construction screen.

Operational Year

CalEEMod is currently designed to key off of one year to initiate the beginning of the full operation of the project. Thus, to indicate when the project will begin operation activities, the user will need to insert a year. CalEEMod will use this year to determine the appropriate emission factors to be used in all operational module calculations. CalEEMod can accommodate the following years for the initial operational year: 2000, 2005, 2010-2035, 2040, 2045, and 2050. To conduct a backcasting analysis by inserting an operational year that occurs in the past, the selection of years is limited to minimize the file size associated with vehicle emission factors. For a project that consists of multiple phases with operation activities occurring over multiple years, the user should run the model multiple times for the various input parameters for each operational year.

Utility Company

From the drop down list, the user will need to select the appropriate utility company that will serve the project location. . When a specific utility is selected, the intensity factors for CO_2, CH_4 and N_2O will be automatically populated with defaults applicable to the specified utility. However, if the utility for the project is not in the drop down list, the user may select User Defined and the user will need to manually enter the various intensity factors. In addition, the user will need to identify the utility in the Remarks section.

The intensity factors are used in various modules to calculate the GHG emissions associated with electricity use. The default values are based on CARB's Local Government Operations Protocol (LGO)[6] for CO_2, updated public utility protocols for CO_2, and E-Grid values for CH_4 and N_2O. Each default CO_2 intensity factor is based on the latest reporting year available for each utility. Appendix D, Table 1.2 provides the default CO_2 intensity factor and reporting year from which the factor was identified for each utility identified in the drop down list. As with other defaults in the model, if a new intensity factor is identified before the defaults in CalEEMod are updated, the user may override the default and provide justification for the change in the Remarks section at the bottom of the Project Characteristics screen.

[6] Available at: http://www.arb.ca.gov/cc/protocols/localgov/localgov.htm

Pollutants

CalEEMod provides a list of pollutants with adjacent check boxes for the user to select. Upon starting a new project, all of the boxes are automatically checked and if the boxes remain checked, all pollutants will be quantified and identified in the reports. If user unchecks any of the boxes, the unchecked pollutants will be excluded from the calculations and the reports. Some of the pollutants may overlap other identified pollutants. For example, carbon dioxide (CO_2) is identified on its own, and it is separated into biogenic and non-biogenic categories. In addition, CO_2 Equivalent GHGs represents, all CO_2 emissions plus methane (CH_4) and nitrous oxide (N_2O) as adjusted by their corresponding Global Warming Potential (GWP) weighted value. The GWPs are based on the 2007 IPCC's Fourth Assessment Report (AR4)[7], and are consistent with 2014 CARB's Scoping Plan Update[8].

Remarks

As previously explained in Subchapter 3.4, if the user chooses to modify any defaults, the user will be required to provide an explanation or justification in the Remarks section for incorporating user defined (e.g., non-default) values before the user will be able to proceed to the next screen. Any remarks that are entered will be included in the reports and will assist a reviewer in understanding the reasons for a change in the default value (e.g., new trip rate based on a project-specific traffic study conducted by traffic engineers).

4.2 Land Use

The Land Use screen is where the user identifies the land use(s) that will occur at the project site. The data in the land use types and subtypes, unit amounts, size metric, lot acreage, square feet and population fields determine the default variables that are used in the calculations. It is important to note that for any project that includes a city park, golf course, or recreational swimming pool land use, the user will be prompted to enter the square footage of the buildings associated with these land uses (e.g., restrooms/changing rooms, pro-shop, etc.). By excluding the entire lot size for these three land use types, and instead only using the square footage of the buildings, the calculations for consumer product use will provide a more accurate representation of where these materials are actually used and avoid incorrectly attributing consumer products use to greenspaces and pool water. For more information on the calculations for consumer product use, see Subchapter 4.5, Section 4.5.2.

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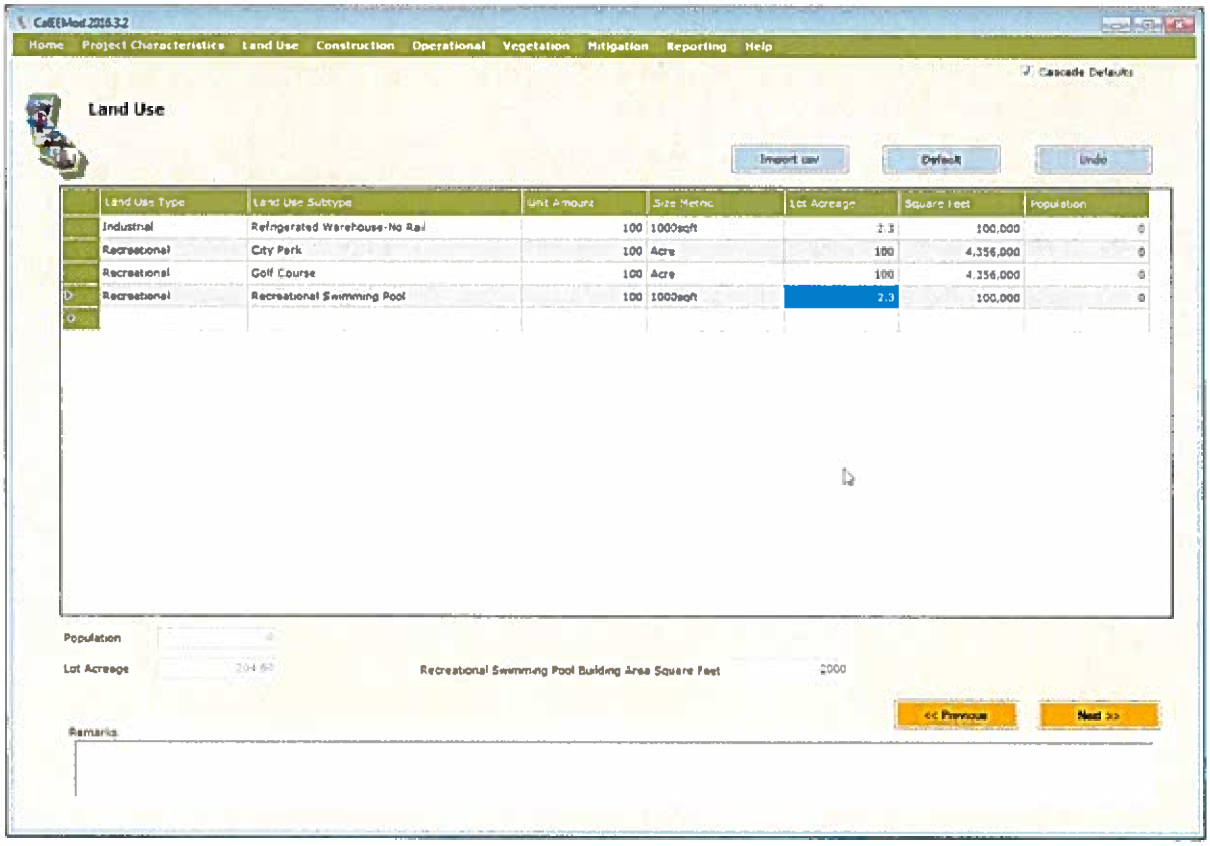
7 Available at: https://www.ipcc.ch/pdf/assessment-report/ar4/syr/ar4_syr_full_report.pdf

8 Available at: http://www.arb.ca.gov/cc/scopingplan/document/updatedscopingplan2013.htm



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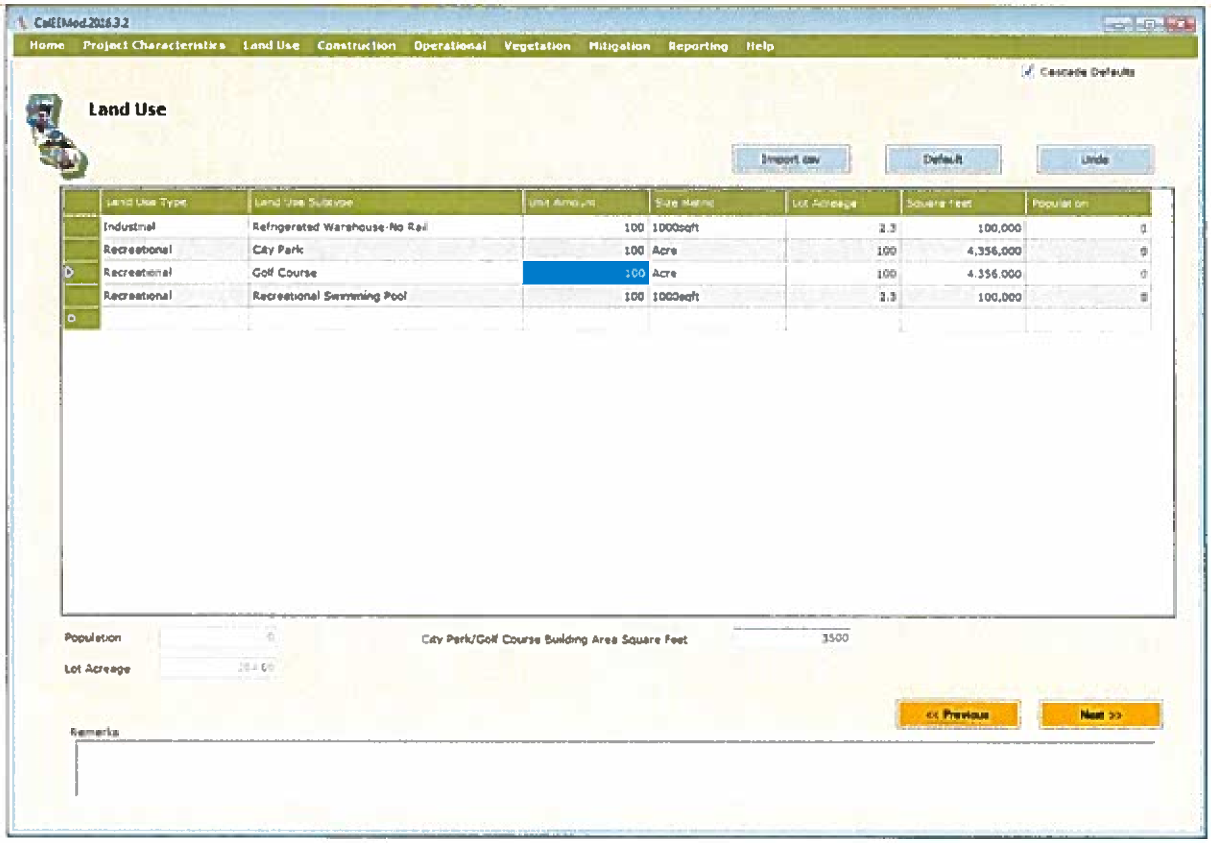
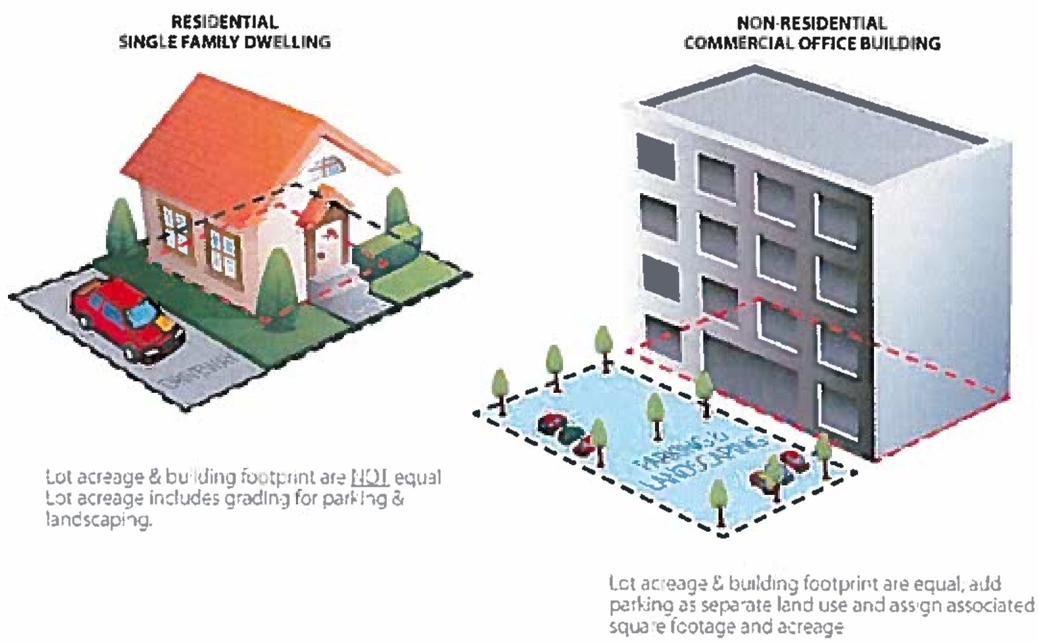


Land Use Type

The Land Use Type tab allows the user to select any of the following primary land use types from a drop down list: Commercial, Educational, Industrial, Parking, Recreational, Residential, and Retail. The 63 different land use types were chosen for inclusion in CalEEMod because each has an established trip rate critical for mobile source calculations.

CalEEMod specifically designates parking areas as a separate land use rather than as a part of an associated non-residential land use (e.g., commercial buildings, retail facilities, etc.). However, no separate parking land use for a driveway or garage needs to be identified for residential land uses because parking is already included in the calculation For more information on how CalEEMod treats parking based on the footprint and lot acreage of residential and non-residential land uses, please refer to the following figure. As depicted, the lot acreage of a residential land use includes the parking and building footprint. For non-residential land uses, the lot acreage is the same as the building footprint, so parking needs to be entered as a separate land use.

CalEEMod Default Lot Acreage for Res and Non Res Land Uses



For the parking land use subtype, two primary options are available: parking lot or parking structure (e.g., garage). There are four types of parking structures: 1) enclosed; 2) enclosed with an elevator; 3) unenclosed; and, 4) unenclosed with an elevator. The reason for these specific descriptions is so that the model properly accounts for energy impacts associated with ventilation and elevator operations.

For land use subtypes that are not listed (e.g., roads, underground parking, pipelines, etc.) or that do not accurately represent the project being analyzed, each land use subtype has a User Defined option that the user can select. If a User Defined land use subtype is selected, there is no default data (including size metric) that will automatically populate the data fields. Instead, the user will need to manually enter the unit amount, size metric, lot acreage, etc. If these fields are left blank, no emissions will be calculated for the User Defined land use subtype. Also, whatever size metric (e.g., per acre, per 1000 square foot, etc.) the user chooses for the User Defined land use subtype needs to be consistently applied to all subsequent default values (e.g., gallons of water used *per acre* or *per 1000 square foot*). An alternative approach to entering a User Defined land use subtype would be to choose a land use subtype that most closely fits the project and allow the model to populate the data fields with the defaults. Then, the user can go back through the model and modify the defaults with any known specific project information and enter the required Remarks to explain why the defaults are modified.

Land Use Subtype

63 land use subtypes have been included in CalEEMod and each has an established trip rate that is used for calculating mobile source emissions. By tabbing over to the next column in a row, the user can select a variety of land use subtypes. The user also has the option to select a User Defined land use subtype; however, as explained previously, there is no default data (including size metric) that will automatically populate the data fields. Instead, the user will need to manually enter the unit amount, size metric, lot acreage, etc. Land use subtypes are based primarily on the land use definitions used for (mobile source) trip generation rate information from the Institute of Transportation Engineers (ITE) 9th edition of the Trip Generation Manual. In some cases similar generalized land uses or surrogate data was mapped to some land use subtypes in order to generate the default data needed for various modules.

Table 1: Land Use Subtype Descriptions

Land Use Subtype	Description[1]	ITE Number
	RESIDENTIAL	
Apartments High Rise	High-rise apartments are units located in rental buildings that have more than 10 levels and most likely have one or more elevators.	222
Apartments Low Rise	Low-rise apartments are units located in rental buildings that have 1-2 levels.	221
Apartments Mid Rise	Mid-rise apartments in rental buildings that have between 3 and 10 levels.	223
Condo/Townhouse	These are ownership units that have at least one other owned unit within the same building structure.	230
Condo/Townhouse High Rise	These are ownership units that have three or more levels.	232
Congregate Care (Assisted Living)	These facilities are independent living developments that provide centralized amenities such as dining, housekeeping, transportation and organized social/recreational activities. Limited medical services may or may not be provided.	253
Mobile Home Park	Mobile home parks consist of manufactured homes that are sited and installed on permanent foundations and typically have community facilities such as recreation rooms, swimming pools and laundry facilities.	240
Retirement Community	These communities provide multiple elements of senior adult living. Housing options may include various combinations of senior adult housing, congregate care, assisted living, and skilled nursing care aimed at allowing the residents to live in one community as their medical needs change.	255
Single Family Housing	All single-family detached homes on individual lots typical of a suburban subdivision	210

Table 1: Land Use Subtype Descriptions

Land Use Subtype	Description[1]	ITE Number
	EDUCATIONAL	
Day-Care Center	A day care center is a facility where care for pre-school age children is provided, normally during the daytime hours. Day care facilities generally include classrooms, offices, eating areas and playgrounds.	565
Elementary School	Elementary schools typically serve students attending kindergarten through the fifth or sixth grade. They are usually centrally located in residential communities in order to facilitate student access and have no student drivers.	520
High School	High schools serve students who have completed middle or junior high school.	530
Junior College (2Yr)	This land use includes two-year junior, community, or technical colleges.	540
Junior High School	Junior High schools serve students who have completed elementary school and have not yet entered high school.	522
Library	A library is a facility that consists of shelved books; reading rooms or areas; and sometimes meeting rooms.	590
Place Of Worship	A church is a building in which public worship services are held. A church houses an assembly hall or sanctuary; it may also house meeting rooms, classrooms and occasionally dining catering or party facilities.	560
University/College (4Yr)	This land use includes four-year universities or colleges that may or may not offer graduate programs.	550
	RECREATIONAL	
Arena	Arenas are large indoor structures in which spectator events are held. These events vary from professional ice hockey and basketball to non-sporting events such as concerts, shows, or religious services. Arenas generally have large parking facilities, except when located in or around the downtown of a large city.	460
City Park	City parks are owned and operated by a city.	411
Fast Food Restaurant W/O Drive Thru	This land use includes fast-food restaurants without drive-through windows. Patrons generally order at a cash register and pay before they eat.	933
Fast Food Restaurant With Drive Thru	This category includes fast-food restaurants with drive-through windows.	934
Golf Course	Golf courses include 9, 18, 27 and 36 hole courses. Some sites may also have driving ranges and clubhouses with a pro shop, restaurant, lounge and banquet facilities.	430
Health Club	These are privately-owned facilities that primarily focus on individual fitness or training. Typically they provide exercise classes; weightlifting, fitness and gymnastics equipment; spas; locker rooms; and small restaurants or snack bars.	492

Table 1: Land Use Subtype Descriptions

Land Use Subtype	Description[1]	ITE Number
High Turnover (Sit Down Restaurant)	This land use consists of sit-down, full-service eating establishments with turnover rates of approximately one hour or less. This type of restaurant is usually moderately priced and frequently belongs to a restaurant chain.	932
Hotel	Hotels are places of lodging that provide sleeping accommodations and supporting facilities such as restaurants; cocktail lounges; meeting and banquet rooms or convention facilities; limited recreational facilities and other retail and service shops.	310
Motel	Motels are places of lodging that provide sleeping accommodations and often a restaurant. Motels generally offer free on-site parking and provide little or no meeting space and few supporting facilities.	320
Movie Theater (No Matinee)	Movie theaters consist of audience seating, single or multiple screens and auditoriums, a lobby and a refreshment stand. Movie theaters without matinees show movies on weekday evenings and weekends only; there are no weekday daytime showings.	443
Quality Restaurant	This land use consists of high quality, full-service eating establishments with typical turnover rates of at least one hour or longer. Quality restaurants generally do not serve breakfast, some do not serve lunch; all serve dinner. This type of restaurant usually requires reservations and is generally not part of a chain. Patrons commonly wait to be seated, are served by a waiter, order from menus and pay for meals after they eat.	931
Racquet Club	These are privately-owned facilities that primarily cater to racquet sports.	491
Recreational Swimming Pool	This is a typical recreational swimming pool that may be associated with community centers, parks, swim clubs, etc.	495
	PARKING	
Enclosed Parking Structure	This is an enclosed parking structure that may be above or below ground. It is not covered in asphalt. This land use will require lighting and ventilation, and will be more than one floor with no elevator.	
Enclosed Parking with Elevator	This is an enclosed parking structure that may be above or below ground. It is not covered in asphalt. This land use will require lighting and ventilation, and will be more than one floor with an elevator.	
Other Asphalt Surfaces	This is an asphalt area not used as a parking lot (e.g., long driveway, basketball court, etc.)	
Other Non-Asphalt Surfaces	This is a non-asphalt area (e.g., equipment foundation, loading dock area, etc.).	
Parking Lot	This is a typical single surface parking lot typically covered with asphalt. This land use will require lighting.	
Unenclosed Parking Structure	This is an unenclosed parking structure that may be above or below ground. It is not covered in asphalt. This land use will require lighting but not ventilation. It will be more than one floor with no elevator.	
Unenclosed Parking with Elevator	This is an unenclosed parking structure that may be above or below ground. It is not covered in asphalt. This land use will require lighting but not ventilation. It will be more than one floor with an elevator.	

Table 1: Land Use Subtype Descriptions

Land Use Subtype	Description[1]	ITE Number
RETAIL		
Automobile Care Center	An automobile care center houses numerous businesses that provide automobile-related services, such as repair and servicing; stereo installation; and seat cover upholstering.	942
Convenience Market (24 Hour)	These markets sell convenience foods, newspapers, magazines and often beer and wine. They do not sell or dispense motor vehicle fuels (e.g., gasoline and diesel).	851
Convenience Market With Gas Pumps	These markets sell or dispense motor vehicle fuels (e.g., gasoline and diesel), convenience foods, newspapers, magazines and often beer and wine. This includes convenience markets with motor vehicle fueling dispensers where the primary business is the selling of convenience items, not the fueling of motor vehicles.	853
Discount Club	A discount club is a discount store or warehouse where shoppers pay a membership fee in order to take advantage of discounted prices on a wide variety of items such as food, clothing, tires and appliances. Many items are sold in large quantities or in bulk.	857
Electronic Superstore	These are free-standing facilities that specialize in the sale of electronic merchandise.	863
Free-Standing Discount Store	Discount stores offer centralized cashiering and sell products that are advertised at discount prices. These stores offer a variety of customer services and maintain long store hours seven days a week.	815
Free-Standing Discount Superstore	The discount superstore is similar to the free-standing discount stores with the addition that they also contain a full-service grocery department under the same roof that shares entrances and exits with the discount store area.	813
Gasoline/Service Station	This land use includes service stations where the primary business is the fueling of motor vehicles. They may also have ancillary facilities for servicing and repairing motor vehicles.	944
Hardware/Paint Store	These stores sell hardware and paint supplies and are generally free-standing buildings.	816
Home Improvement Superstore	These are free-standing facilities that specialize in the sale of home improvement merchandise.	862
Regional Shopping Center	A shopping center is an integrated group of commercial establishments that is planned, developed, owned and managed as a unit. A shopping center's composition is related to its market area in terms of size, location and type of store.	820
Strip Mall	Small strip shopping centers contain a variety of retail shops and specialize in quality apparel, hard goods and services such as real estate offices, dance studios, florists and small restaurants.	826

Table 1: Land Use Subtype Descriptions

Land Use Subtype	Description[1]	ITE Number
Supermarket	Supermarkets are free-standing retail stores selling a complete assortment of food: food preparation and wrapping materials; and household, cleaning items. Supermarkets may also contain the following products and services: ATMs, automobile supplies, bakeries, books and magazines, dry cleaning, floral arrangements, greeting cards, limited-service banks, photo centers, pharmacies and video rental areas.	850
	COMMERCIAL	
Bank (With Drive-Through)	Drive-in banks provide banking facilities for motorists who conduct financial transactions from their vehicles; many also serve patrons who walk into the building.	912
General Office Building	A general office building houses multiple tenants where affairs of businesses commercial or industrial organizations or professional persons or firms are conducted. If information is known about individual buildings, it is suggested that this land use be used instead of the more generic office park.	710
Government (Civic Center)	A group of government buildings that are interconnected by pedestrian walkways.	733
Government Office Building	This is an individual building containing either the entire function or simply one agency of a city, county, state, federal, or other governmental unit.	730
Hospital	A hospital is any institution where medical or surgical care and overnight accommodations are provided to non-ambulatory and ambulatory patients. However, it does not refer to medical clinics or nursing homes.	610
Medical Office Building	This is a facility that provides diagnoses and outpatient care on a routine basis but is unable to provide prolonged in-house medical and surgical care. One or more private physicians or dentists generally operate this type of facility.	720
Office Park	Office parks are usually suburban subdivisions or planned unit developments containing general office buildings and support services, such as banks, restaurants and service stations, arranged in a park-or campus-like atmosphere. This should be used if details on individual buildings are not available.	750
Pharmacy/Drugstore W/O Drive Thru	These are retail facilities that primarily sell prescription and non-prescription drugs. These facilities may also sell cosmetics, toiletries, medications, stationery, personal care products, limited food products and general merchandise. The drug stores in this category do not contain drive-through windows.	880

Table 1: Land Use Subtype Descriptions

Land Use Subtype	Description[1]	ITE Number
Pharmacy/Drugstore With Drive Thru	These are retail facilities that primarily sell prescription and non-prescription drugs. These facilities may also sell cosmetics, toiletries, medications, stationery, personal care products, limited food products and general merchandise. The drug stores in this category contain drive-through windows.	881
Research & Development	R&D centers are facilities devoted almost exclusively to R&D activities. The range of specific types of businesses contained in this land use category varies significantly. R&D centers may contain offices and light fabrication areas.	760
	INDUSTRIAL	
General Heavy Industry	Heavy industrial facilities usually have a high number of employees per industrial plant and are generally limited to the manufacturing of large items.	120
General Light Industry	Light industrial facilities are free-standing facilities devoted to a single use. The facilities have an emphasis on activities other than manufacturing and typically have minimal office space. Typical light industrial activities include printing, material testing and assembly of data processing equipment.	110
Industrial Park	Industrial parks contain a number of industrial or related facilities. They are characterized by a mix of manufacturing, service and warehouse facilities with a wide variation in the proportion of each type of use from one location to another. Many industrial parks contain highly diversified facilities.	130
Manufacturing	Manufacturing facilities are areas where the primary activity is the conversion of raw materials or parts into finished products. It generally also has office, warehouse, and R&D functions at the site.	140
Refrigerated Warehouse-No Rail	This is a warehouse that has refrigeration but no rail spur.	152
Refrigerated Warehouse-Rail	This is a warehouse that has refrigeration and a rail spur.	152
Unrefrigerated Warehouse-No Rail	This is a warehouse that does not have refrigeration and no rail spur.	152
Unrefrigerated Warehouse-Rail	This is a warehouse that does not have refrigeration but has a rail spur.	152

[1] Based on land use descriptions in Institute of Transportation Engineers (ITE) Trip Generation Manual, 9th Edition.

Unit Amount and Size Metric

By tabbing over to the Unit Amount and Size Metric columns, respectively, the user can enter the number of units (e.g., houses, apartments, etc.) and the corresponding size metric (e.g., per 1000 sq. ft., employees, students, etc.). This data combination will be used to populate the lot acreage, square feet and population columns on this screen. For example, a school land use allows the user to define its size by the number of students, building square footage, or number of employees. It is important to note that the square footage, which is used for calculating such impacts as architectural coatings and energy use, relates to the total building square footage and not the building footprint or lot acreage which is used for housing density as well as grading and site preparation calculations.

Lot Acreage

If actual lot acreage data is available, the user should override the default value. However, for a mixed use, multi-story building, the user should not override the square footage default value for each individual land use or the acreage default value assigned to the residential portion or the split between the non-residential land uses if there is no residential portion. The figure below provides an example of a mixed use project and instructions for applying the appropriate square footage and acreage.

Acreage is used to estimate housing density and assign construction default data (e.g., grading, site preparation, etc.). Table 2 contains housing density default data per land use in terms of dwelling units (DU) per acre. By using this data, CalEEMod can estimate the number of acres per dwelling unit (DU) for residential land use. For example, if the user enters 10 apartments in a low rise building, then the lot acreage will be 0.625 acre (10 DU divided by 16 acres/DU). According to the California Energy Commission's Residential Appliance Saturation Survey (RASS), the metric for low rise apartments is 1,000 square feet per DU (see Table 2.1). Similarly, using the same example, the building footprint will be 0.23 acre (10 DU x 1000 sq. ft./DU x 1 acre/43,560 sq. ft.). Thus, the total lot acreage includes the residential footprint plus driveway and landscaping/open space.

After the user has completed entering all of land uses for the project, CalEEMod will add the lot acreage values for each land use and the total will be reflected in the lot acreage text box located at the bottom of the screen. The value in the total lot acreage box cannot be modified by the user.

Example of Mixed Use Project in CalEEMod

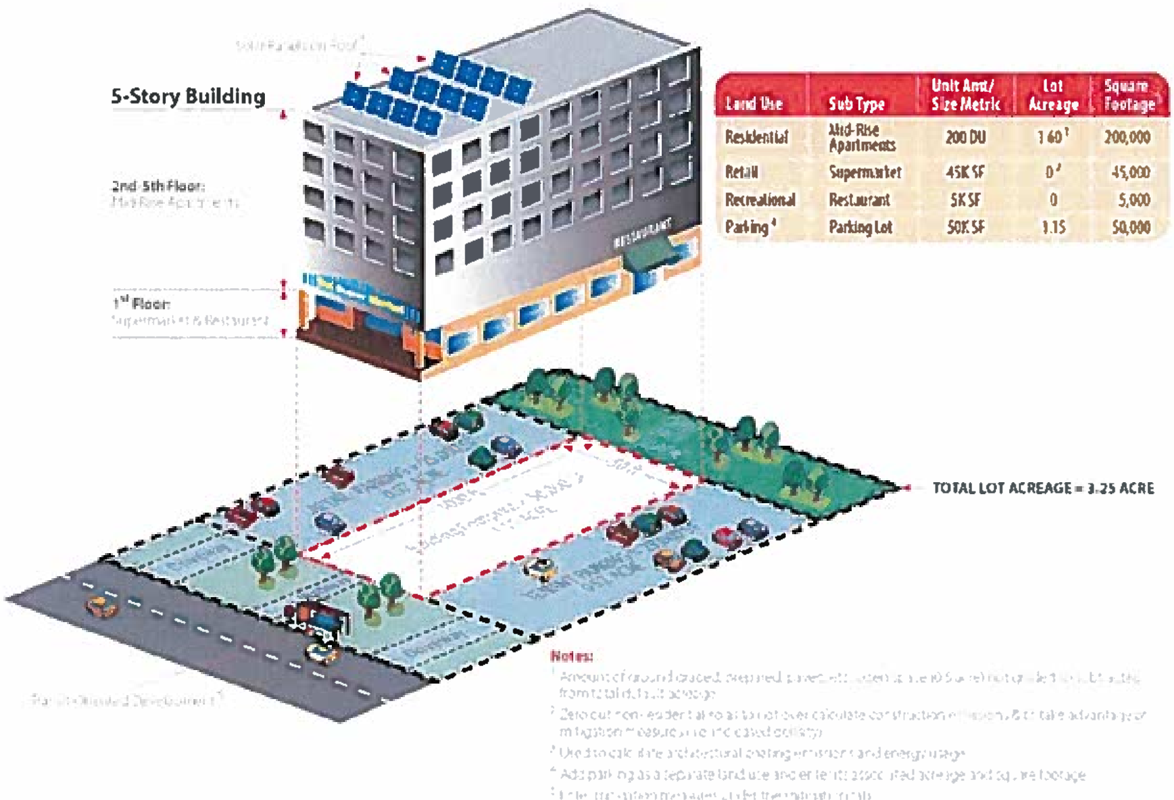


Table 2: Default Housing Density[1]

Land Use Subtype	Density (Dwelling Units/Acre)
Single Family Housing	3
Apartments low rise	16
Apartments mid rise	38
Apartments high rise	62
Condo/townhouse	16
Condo/townhouse high rise	64
Mobile Home Park	8
Retirement Community	5
Congregate care (Assisted Living)	16

[1] Based on the density assumed in ITE Trip Generation 8th Edition

Square Footage

If actual square footage of the total building or building footprint is known, the user should override the default value.

Population

After the completing the tabs for unit amount, size metric, lot acreage, and square footage, the population field will contain a default which represents an estimate of the population for each land use type and subtype selected by the user. If the actual population data is known, the user should override the default value.

After the user has completed entering all of land uses for the project, CalEEMod will add the population values for each land use and the total will be reflected in the population text box located at the bottom of the screen. The value in the total population box cannot be modified by the user.

City Park/Golf Course Building Area Square Feet (text box)

If the user selects a City Park and/or Golf Course land use, a text box will appear at the bottom of the screen that will prompt the user to enter the building square footage of all the buildings that will be located on the City Park and/or Golf Course property (e.g., restrooms/changing rooms, pro-shop, etc.). The user must input site-specific building square footage data because there are no default values for building footprints on these types of land uses. If the building square footage is left blank (e.g., zero square feet), a warning message will appear to remind the user to enter a value in this field.

Recreational Swimming Pool Building Area Square Feet (text box)

If the user selects a Recreational Swimming Pool land use, a text box will appear at the bottom of the screen that will prompt the user to enter the building square footage of all the buildings that will be located on the property (e.g., restrooms/changing rooms, pro-shop, etc.). The user must input site-specific building square footage data because there is no default value for the building footprint on this type of land use. If the building square footage is left blank (e.g., zero square feet), a warning message appear to remind the user to enter a value in this field.

4.3 Construction

After completing the Land Use screen and clicking on the Next button, the Construction screen will appear along with seven tabs/sub-screens that cover the following construction topic areas: Construction Phase; Off-Road Equipment; Dust from Material Movement; Demolition; Trips and VMT, On-Road Fugitive Dust, and Architectural Coatings. To move from one tab/subscreen to another, the user can use the Next and Previous buttons, or click on any of grey tabs. The construction tabs/sub-screens contain default information that was obtained from a survey of construction sites conducted by South Coast Air Quality Management District (SCAQMD). The construction survey data is grouped by construction phase and lot acreage and can be found in Appendix E1. The default construction equipment list and phase length data were determined to be the most appropriate for the size and types surveyed. In addition, some data in the survey was extrapolated to create default values for project sizes that were not in the survey. However, if the user has more detailed site-specific equipment and phase information, the user should override the default values.

4.3.1 Construction Phase

The Construction Phase tab is where the user can enter the type of each construction phase and the date range for each phase. . Default phases are based on the total lot acreage of the project. Depending on the project being modeled, not all phases may be necessary so the user may need to delete phases that are not applicable to the project. For example, not all projects require demolition. In addition, the user may need to add multiple phases of similar types for large projects with staged build out scenarios. It is important to note that if a project has demolition, grading, and site preparation phases, the user will need to provide additional project-specific data on the Demolition and Dust from Material Movement sub-screens.

Phase Name and Phase Type

The Phase Name and Phase Type fields will be automatically populated with the following default construction phases: Site Preparation; Demolition; Grading; Building Construction; Paving; and, Architectural Coating. The inclusion of any of these phases will define the types of calculations and default assumptions for on-road vehicle trips and fugitive emissions that occur in subsequent construction sub-screens. The definitions of the default phase types are as follows:

- *Demolition* involves removing buildings or structures.
- *Site Preparation* involves clearing vegetation (grubbing and tree/stump removal) and removing stones and other unwanted material or debris prior to grading.
- *Grading* involves the cut and fill of land to ensure that the proper base and slope is created for the foundation.
- *Building Construction* involves the construction of the foundation, structures and buildings.
- *Architectural Coating* involves the application of coatings to both the interior and exterior of buildings or structures, the painting of parking lot or parking garage striping, associated signage and curbs, and the painting of the walls or other components such as stair railings inside parking structures.
- *Paving* involves the laying of concrete or asphalt such as in parking lots, roads, driveways, or sidewalks.

Start Date and End Date

The user can enter with the aid of a calendar, the Start Date and End Dates for each construction phase. The default Start Date is the Start of Construction date defined on the Project Characteristics screen. The cells will be automatically populated with a default construction schedule starting with the Demolition phase, with subsequent phases starting the following day after the previous phase's end date. The user may change the defaults to alter the total days estimated for each phase. Because CARB's emission factors vary from year to year, when the user inserts the start and end dates for each construction phase, the model will select the correct emission factors for the year when each piece of off-road equipment will be utilized.

Days per Week

The user can select from a drop down box the number of days per week (either 5, 6, or 7 days) that construction will occur. Five days per week assumes that construction will occur from Monday through Friday, and six days per week assumes that construction will occur Monday through Saturday.

Total Days

The Total Days field is intended to indicate the number of days that it will take to complete a particular construction phase and this field is initially populated with default values. If the End Date or the Days per Week fields are changed, clicking the Total Days field will trigger a recalculation of the Total Days. If the Total Days field for any phase is changed, then once leaving this field, the program will automatically adjust the End Date based on the Start Date for that phase.

4.3.2 Off-Road Equipment

The Off-Road Equipment tab is for the user to select the type and quantity of off-road equipment needed for each construction phase and to define the daily usage schedule. Since equipment lists can be lengthy and vary widely for each construction phase, the user will need to first select the phase from Phase Name drop down list or by clicking on the Previous or Next buttons located next to the phase name, and then select the off-road equipment that will be used for each construction phase. The Off-Road Equipment screen calculates emissions based on the expected off-road equipment engine use for each piece of equipment listed over the duration of the phase length. It is important to note that fugitive emissions from off-road equipment are calculated elsewhere on other construction screens.

After the user enters the Equipment Type, Number of Units, and Hours per Day for each piece of equipment that will be used in any phase, The Horsepower and Load Factor fields will be automatically populated with the default average values from CARB's OFFROAD2011. If equipment-specific information is available, the user can override these default values. In some cases, CARB's OFFROAD2011 emission factors are not available for all years. Thus, if the user selects a construction year that does not have corresponding emission factors, CalEEMod has been programmed to substitute the emissions factors from nearest, lower end (e.g., oldest) year. For example, if construction will occur in year 2037 (a year which does not have emission factors), CalEEMod will substitute the emission factors from year 2035 instead. Since newer equipment tends to have less emissions than older equipment, by selecting emission factors from year 2035 (an older year), the calculations may result in a conservative, slight overestimate of emissions.

If the project requires the use of off-road equipment that is not specifically listed in the drop down list, the user can select from three generalized equipment categories to add customized equipment to the analysis: 1) Other Construction Equipment; 2) Other General Industrial Equipment; and, 3) Other Material Handling Equipment. In addition, the user may choose to select a surrogate equipment type which has a similar horsepower rating and load factor. To include water trucks and cement trucks in the analysis, the user needs to first determine if these trucks are off-road or on-road vehicles. If they are only driven off-road, then the user can select the Off-Highway Trucks category in the Off-Road Equipment screen. If the trucks are driven on-road, the user can account for the on-road emissions by entering this information as Additional Vendor Trips on the Trips and VMT screen (see Subchapter 4.3.5).

4.3.3 Dust from Material Movement

The Dust from Material Movement sub-screen is intended for calculating fugitive dust emissions associated with the Site Preparation and Grading phases (defaults) during construction. This sub-screen calculates the following three types of fugitive dust: 1) fugitive dust from dozers moving dirt; 2) fugitive dust from graders or scrapers leveling the land; and; 3) fugitive dust from loading or unloading dirt into haul trucks. These methods have been adapted from USEPA's AP-42 method for Western Coal Mining. Once the enters the amount of material imported and exported to the site, CalEEMod will estimate the number of hauling trips associated with from material transport activities. The user may define the units in terms of Ton of Debris or Cubic Yards. The user may also select whether the import/export of material is phased (e.g., a the same truck that arrives with material departs with another load of material to export in one round trip or two-one way trips. The calculations for non-phased material import/export trips assume that one truck arrives empty and departs full and a different truck arrives full for a total of two round trips (or four one-way trips). Thus, phasing material import and export trips reduces the number of haul trips.

The Total Acres Graded field represents the cumulative distance traversed on the property by the grading equipment, assuming a blade width of 12 feet. In order to properly grade a piece of land, multiple passes with grading equipment may be required. So even though the lot size is a fixed number of acres, the Total Acres Graded could be an order of magnitude higher than the footprint of the lot and is calculated based on the equipment list (including number of equipment), the number of days need to complete the grading and/or site preparation phase, and the maximum number of acres a given piece of equipment can traverse in an 8-hour workday. For more information regarding how Dust from Material Movement is calculated, including grading rates, see Appendix A, Subchapter 4.3.

4.3.4 Demolition

The Demolition sub-screen is intended for the user to enter the amount of material that is demolished, if a demolition phase is selected by the user as part of the construction project. The user can select the Size Metric to define the amount of demolished material that is expected to be generated during the demolition phase in terms of Ton of Debris or Building Square Footage. With this data, fugitive dust emissions generated during demolition are calculated. The calculation of fugitive dust emissions during demolition is derived from the methodology described in the report prepared for the USEPA by Midwest Research Institute, Gap Filling PM_{10} Emission Factors for Selected Open Area Dust Sources.

4.3.5 Trip and VMT

The Trip and VMT sub-screen is used to provide the number and length (in terms of vehicle miles traveled or VMT) of on-road vehicle trips for workers, vendors, and hauling for each construction phase. Depending on the land use type and subtype combined with the various construction phases, CalEEMod will populate the fields for Number of Trips, Trip Length, and Vehicle Class for worker, vendor and haul trips, respectively, with default values. The vehicle class descriptor HHDT, MHDT means that there is a 50/50 percent mix of heavy-heavy duty trucks and medium-heavy duty trucks. Similarly, the vehicle class descriptor LDA, LDT1, LDT2 means that there is a 50/25/25 percent mix of light duty autos, light duty truck class 1 and light duty truck class 2, respectively. The user may override the defaults and enter different weightings of vehicle fleet mixes. It is important to note that if the user selects a construction year that does not have corresponding EMFAC2014 emission factors for on-road vehicles, CalEEMod has been programmed to substitute the emissions factors from nearest, lower end (e.g., oldest) year. For example, if construction will occur in year 2037 (a year which does not have emission factors), CalEEMod will substitute the emission factors from year 2035 instead. Since newer equipment tends to have less emissions than older equipment, by selecting emission factors from year 2035 (an older year), the calculations may result in a conservative, slight overestimate of emissions.

CalEEMod quantifies the number of construction workers by multiplying 1.25 times the number of pieces of equipment for all phases (except Building Construction and Architectural Coating). For the Building Construction, the number of workers is derived from a study conducted by the Sacramento Metropolitan Air Quality Management District (SMAQMD) which determined the number of workers needed for various types of land uses and corresponding project size. This study and its analysis are included in Appendix E2. For the Architectural Coating phase, the number of workers is approximately 20% of the number of workers needed during the Building Construction phase.

The number of vendor trips during the Building Construction phase is also derived from a study conducted by the SMAQMD. The SMAQMD trip survey during construction counted cement and water trucks as vendor trips (instead of counting them as off-road vehicle trips) and these trip rates were incorporated into the calculations for the Building Construction phase. If the user deletes the Building Construction phase from the analysis, but the project will require water and/or cement trucks, then the user will need to account for these either as vendor trips under another construction phase or under the Off-Road equipment screen.

The default values for hauling trips are based on the assumption that a truck can haul 20 tons (or 16 cubic yards) of material per load. If one load of material is delivered, CalEEMod assumes that one haul truck importing material will also have a return trip with an empty truck (e.g., 2 one-way trips). Similarly, a haul truck needed to export material is assumed to have an arrival trip in an empty truck and a loaded departure truck (e.g., 2 one-way trips). Thus, each trip to import and export material is considered as two separate round trips (or 4 one-way trips). However, if the Phase box is checked, the same haul truck that imported the material will be assumed to be the same haul truck that export material resulting in one round trip (or 2 one-way trips).

4.3.6 On-Road Fugitive Dust

The On-Road Fugitive Dust sub-screen defines the variables that will be used to determine the fugitive dust emissions from on-road vehicles driving over paved and unpaved roads during construction. CalEEMod automatically populates the data fields based on the construction phase. The calculations use emission factors from USEPA's AP-42 for paved roads (January 2011 edition) and unpaved roads (November 2006 edition). Each data field is the same as those defined in the aforementioned AP-42 sections.

4.3.7 Architectural Coatings

The Architectural Coatings sub-screen is intended to calculate ROG emissions associated with painting the interior/exterior of residential and non-residential buildings as well as calculate emissions from parking lot painting or striping. The user may override any of the default interior and exterior surface areas estimated for residential and non-residential buildings. In addition, each of these surface types has a different emission factor indicating the ROG content of the paint in grams per liter (g/L). It is important to note that the parking area square footage is not included in the non-residential interior/exterior square footage when calculating emissions attributable to parking lot striping. See Appendix A, Subchapter 4.7 for the methodology of estimating surface areas to be coated from building square footage.

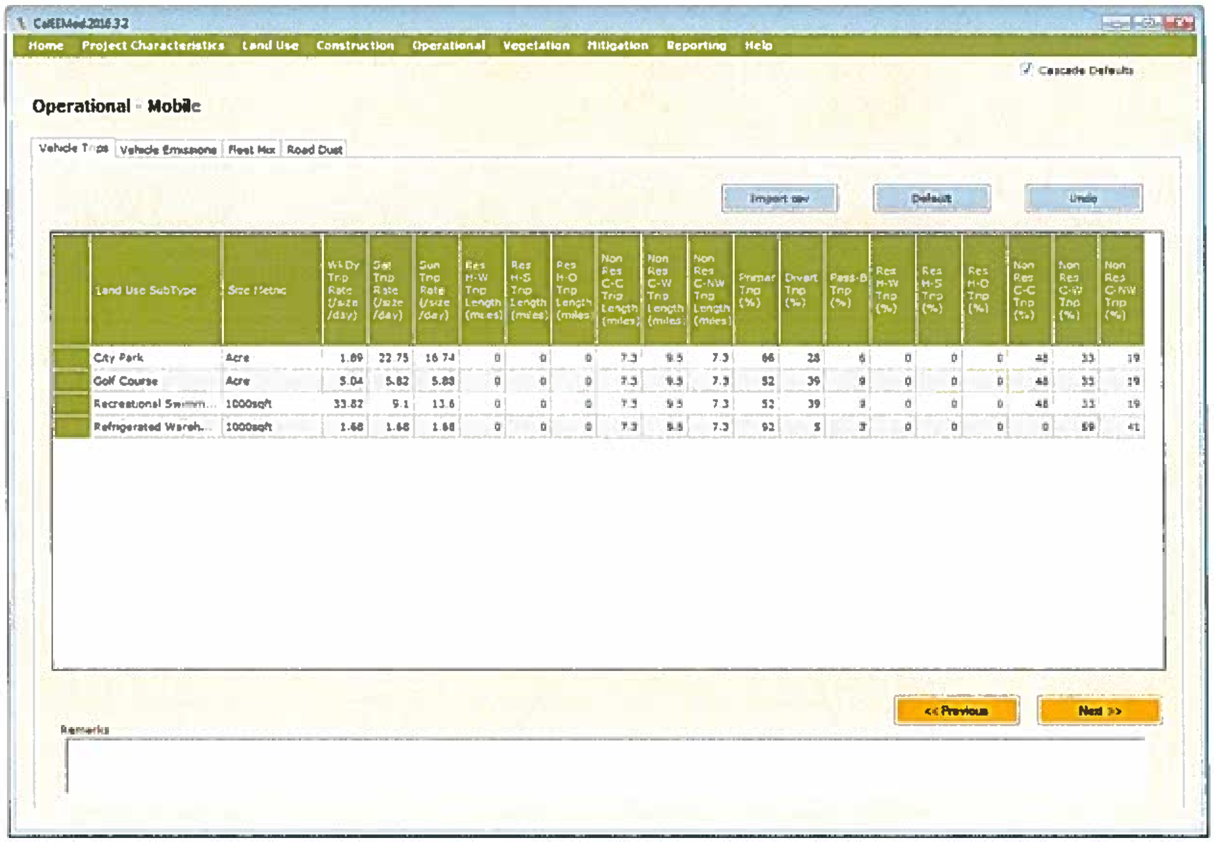
4.4 Operational Mobile

The operational mobile screen is made up of four sub-screens: Vehicle Trips, Vehicle Emissions, Fleet Mix and Road Dust. These screens are used in defining the information necessary to calculate the emissions associated with operational on-road vehicles.

4.4.1 Vehicle Trips

This sub-screen includes the trip rates, trip lengths, trip purpose, and trip type percentages for each land use subtype in the project. The user can edit any of this information by entering a new value in the appropriate cell. Trip rates are in terms of the size metric (thousand square footage or dwelling unit) defined on the land use screen and are listed for weekday, Saturday and Sunday if available. Trip lengths are for primary trips. Trip purposes are primary, diverted, and pass-by trips. Diverted trips are assumed to take a slightly different path than a primary trip and are assumed to be 25% of the primary trip lengths. Pass-by trips are assumed to be 0.1 miles in length and are a result of no diversion from the primary route. Residential trip types are defined as home-work (H-W), home-shop (H-S), and home-other (H-O). Non-residential trip types are defined as commercial –customer (C-C), commercial-work (C-W), and commercial-nonwork (C-NW) such as delivery trips. Appendix A includes the equations and methodology used to calculate motor vehicle emissions from the operation of a project.

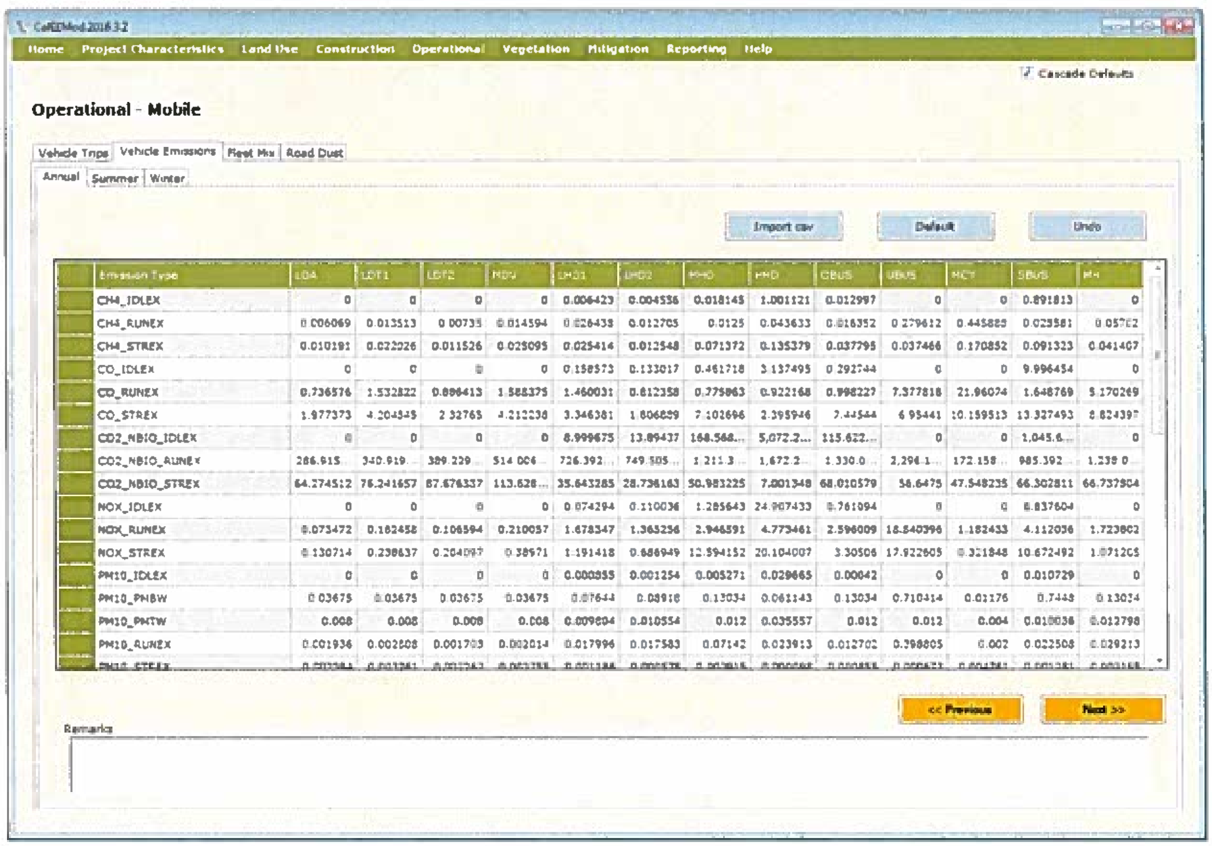
The trip rates are based on ITE 9th edition average trip rates for the respective land use categories.



4.4.2 Vehicle Emissions

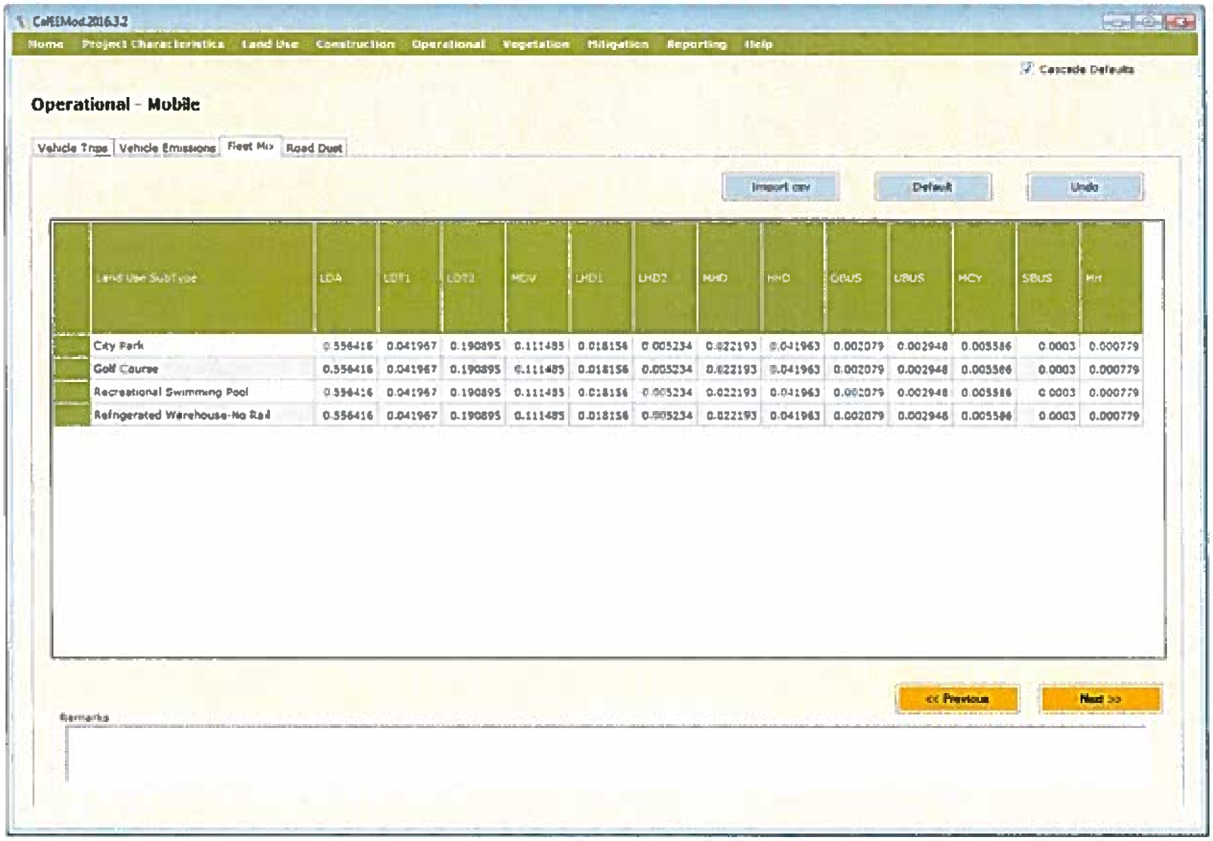
This sub-screen contains the detailed vehicle emission factors based on EMFAC2014. Appendix A includes the description of how these emission factors were derived from EMFAC2014. It is anticipated that most users will not edit data in this sub-screen. There are separate tabs for annual, summer, and winter emissions values. If the user wants to alter the breakdown of fuel types (catalytic, non-catalytic, and other) within a vehicle class, they will have to provide their own data. This will likely be an infrequent change due to CEQA enforceability requirements.

This screen along with the previous screen (Vehicle Trips) and next screen (Fleet Mix) will provide the data for the model to calculate the emissions associated with on-road motor vehicle use. The calculation does not include the fugitive dust emissions from travel over roads as these are associated with the next screen (Road Dust).



4.4.3 Fleet Mix

In CalEEMod Version 2016.3.1, the fleet mix was separated from the Vehicle Emissions screen and a new Fleet Mix screen was created so that users are able to change default fleet mix associated with different land use subtypes.



4.4.4 Road Dust

This sub-screen is used to change any of the default values that are used in the USEPA's AP-42 methods for calculating fugitive emissions from paved and unpaved roads. The defaults for the road dust (e.g., material silt content, material moisture content, and mean vehicle speed) are statewide averages, but the user has the ability to override the defaults if data specific to the project is known. Local jurisdictions can also provide guidance to users as to what default properly reflects known regional road dust parameters.

For the San Luis Obispo region, the user is recommended to provide the following unpaved road dust parameters overriding the statewide defaults if users choose to use USEPA's AP-42 methods:

9.3 for Material Silt Content (%) *(instead of 4.3 statewide default)*

0.1 for Material Moisture Content (%) *(instead of 0.5 statewide default)*

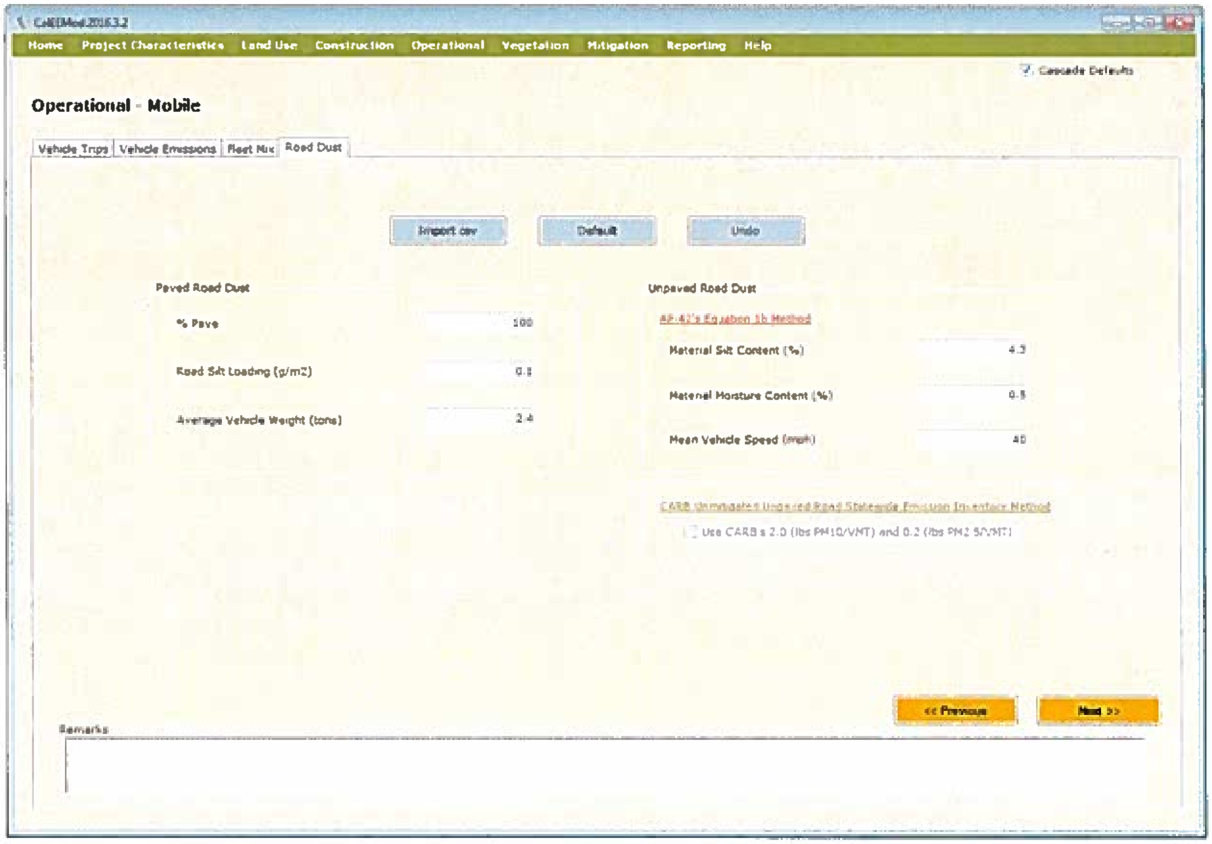
32.4 for Mean Vehicle Speed (mph) *(instead of 40 statewide default)*

In CalEEMod Version 2016.3.1, projects located in San Luis Obispo County APCD and Sacramento Metropolitan AQMD were provided an additional option for the user to select CARB's 2.0 lbs. PM_{10}/VMT[9] as the default unmitigated fugitive dust emission factor for unpaved roads during the operational phase. If this default is selected, an emission factor of 0.2 lbs. $PM_{2.5}$/VMT is also applied based on a 10% $PM_{2.5}$/ PM_{10} ratio[10, 11]. By checking the box, the program will use CARB's emission factor to override the calculated emission factor based on USEPA AP-42. Note: For project locations other than San Luis Obispo County APCD and Sacramento Metropolitan AQMD, CARB's 2.0 lbs. PM_{10}/VMT is not an option that the user can select.

9 Available at: http://www.arb.ca.gov/ei/areasrc/fullpdf/full7-10.pdf

10 Available at: http://www3.epa.gov/ttnchie1/ap42/ch13/related/mri_final_fine_fraction_dust_report.pdf

11 Available at: http://www.arb.ca.gov/app/emsinv/emssumcat_query.php?F_YR=2015&F_DIV=-4&F_SEASON=A&SP=2009&F_AREA=CA#0

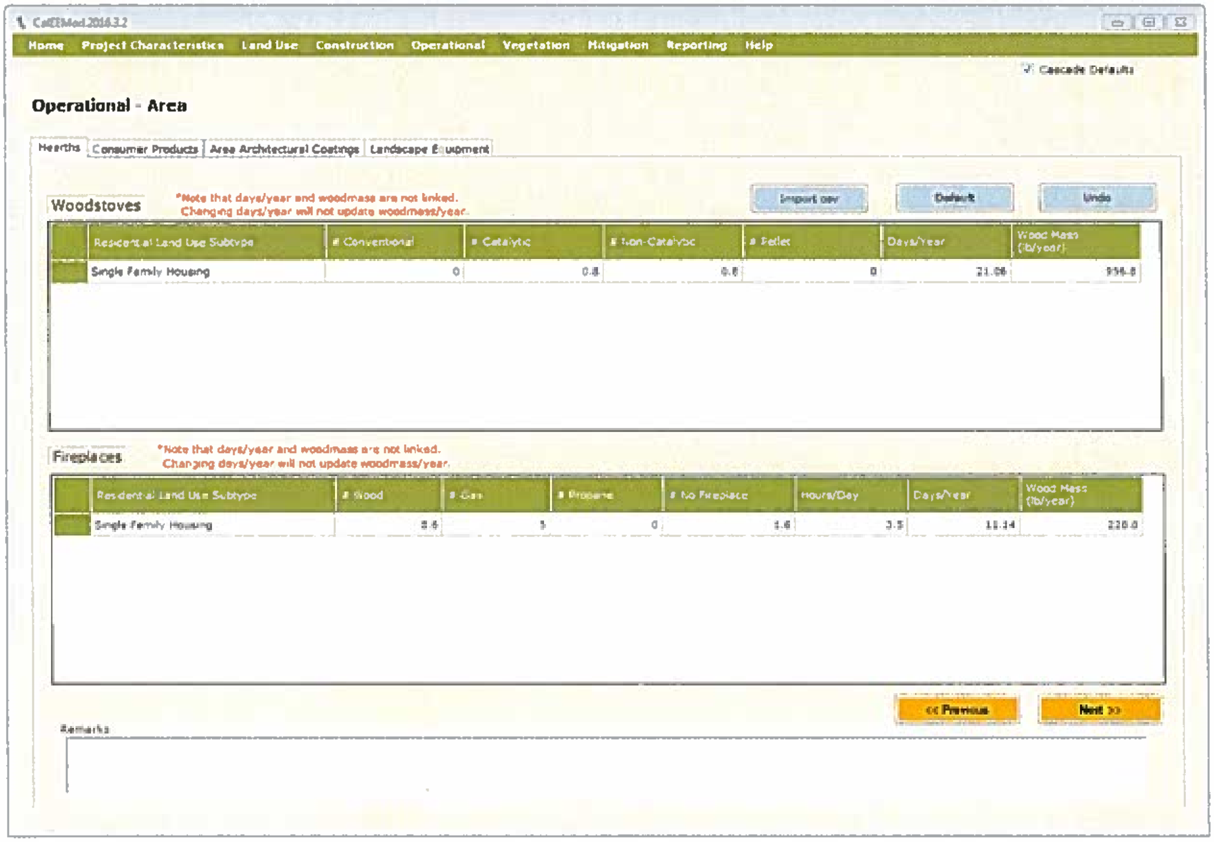


4.5 Area

The area source screen consists of four sub-screens: Hearths, Consumer Products, Area Architectural Coatings, and Landscaping Equipment. Natural gas emission variables from all uses except hearths are included in the energy use screen (described in Section 4.6).

4.5.1 Hearths and Woodstoves

This sub-screen allows the user to enter the number of woodstoves and hearths of various types as well as the usage of these devices. Woodstoves are separate from fireplaces since a home may have both and these devices may have different use patterns. The number of devices that is entered for each device type represents the total number of devices installed in the dwelling units for a particular land use. Appendix A contains the emissions calculation methodology and details of variables that the user cannot override. Some of these emissions may be classified as biogenic and are therefore reported as CO_2-Biogenic. For most locations a default percent of hearths and stoves was provided by air districts and is multiplied through. The number of devices was chosen to include in CalEEMod instead of a percentage to allow for incorporation of various air district rules regarding hearths and woodstoves in new residences without having specialized data entry screens. Commercial land uses by default do not have hearths or woodstoves in CalEEMod. These are included for those cases where they may occur such as in restaurants or hotels.



The San Joaquin Valley jurisdiction has a regulatory limit on the number of hearths depending upon the type and number of residential development. The regulatory limit is generated by CalEEMod but all the input parameters (e.g., unit density, etc.) are necessary to determine the value. Thus, the regulatory limit is disclosed during the reporting stage under the Default Value box in the report. The model, however, calculates emission impacts from the number of hearths inputted on the Area source screen (listed under the New Value column in the report). Therefore, if the user wants to calculate emissions from regulatory limit, the report needs to be run to determine the regulatory limit and the user needs to go back to the Area Source screen to input that value and re-run the report. If the user chooses to calculate emissions from a different number of hearths (e.g., a number of hearths less than the regulatory limit), then that number needs to be inputted on the Area Source screen to properly calculate emissions. Again, the report will provide the regulatory limit under the Default Value column and the user input value under New Value column.

4.5.2 Consumer Products

Consumer products are various solvents used in non-industrial applications which emit ROGs during their product use. These typically include cleaning supplies, kitchen aerosols, cosmetics and toiletries. SCAQMD has developed an emission factor based on the total of all building square footage for both residential and non-residential buildings. Details of how this emission factor was developed can be found in Appendix E. The user can change this emission factor if more relevant data is available. In CalEEMod Version 2016.3.1, ROG emissions from pesticides/fertilizers for City Parks and Golf Courses and ROG emissions from parking surface degreasers were separated from the general consumer products category. Also in CalEEMod Version 2016.3.1, the model also assumes that there would be no ROG emissions from the actual pool surface area for Recreational Swimming Pools because the chemicals used for maintaining pools are not considered to be ROGs. Details of how the ROG emission factors for pesticides/fertilizers and parking surface degreasers were determined can be found in Appendix E.

4.5.3 Area Architectural Coatings

This sub-screen has text boxes for the reapplication rate and coating ROG content for each building surface type and parking surface. The reapplication rate is the percentage of the total surface area that is repainted each year. A default of 10% is used, meaning that 10% of the surface area is repainted each year (i.e., all surface areas are repainted once every 10 years). Daily emissions divide the annual rate by 365 days per year. This is based on assumptions used by SCAQMD in their district rules regarding architectural coatings. Some districts provided details on their coating regulations that phase-in over time, which have been incorporated to the extent feasible, given the general classifications of paint (interior or exterior for residential and non-residential). Coating ROG content from state regulations are used for air districts that did not provide specific architectural coating information. Consult your local air district for suggested values that may be lower than the state regulations.

The ROG contents under the Operational Area Architectural Coatings screen (either CalEEMod defaults or site-specific values defined by users) become the default ROG contents for the Area Mitigation screen. The user may check the box under the Area Mitigation screen and specify a lower ROG content limit.

4.5.4 Landscape Equipment

This sub-screen has two text boxes to show the number of snow days or summer days. In addition, the defaults consider a realistic number of days which the landscaping equipment would be operated. For example, landscaping at commercial facilities typically do not take place during a weekend or during the summer at educational facilities that are not open. The number of days are applied to the appropriate landscape equipment types available in OFFROAD2011 using the average horsepower and load factors of the population mode. The derivation of emission factors used for each equipment type from OFFROAD2011 is described in Appendix A.

4.6 Energy Use

The energy use screen is used to gather the information necessary to estimate the emissions associated with building electricity and natural gas usage (non-hearth). The electricity energy use is in units of kilowatt hours (kWh) per size metric for each land use subtype. Natural gas use is in units of a thousand British Thermal Units (kBTU) per size metric for each land use subtype.

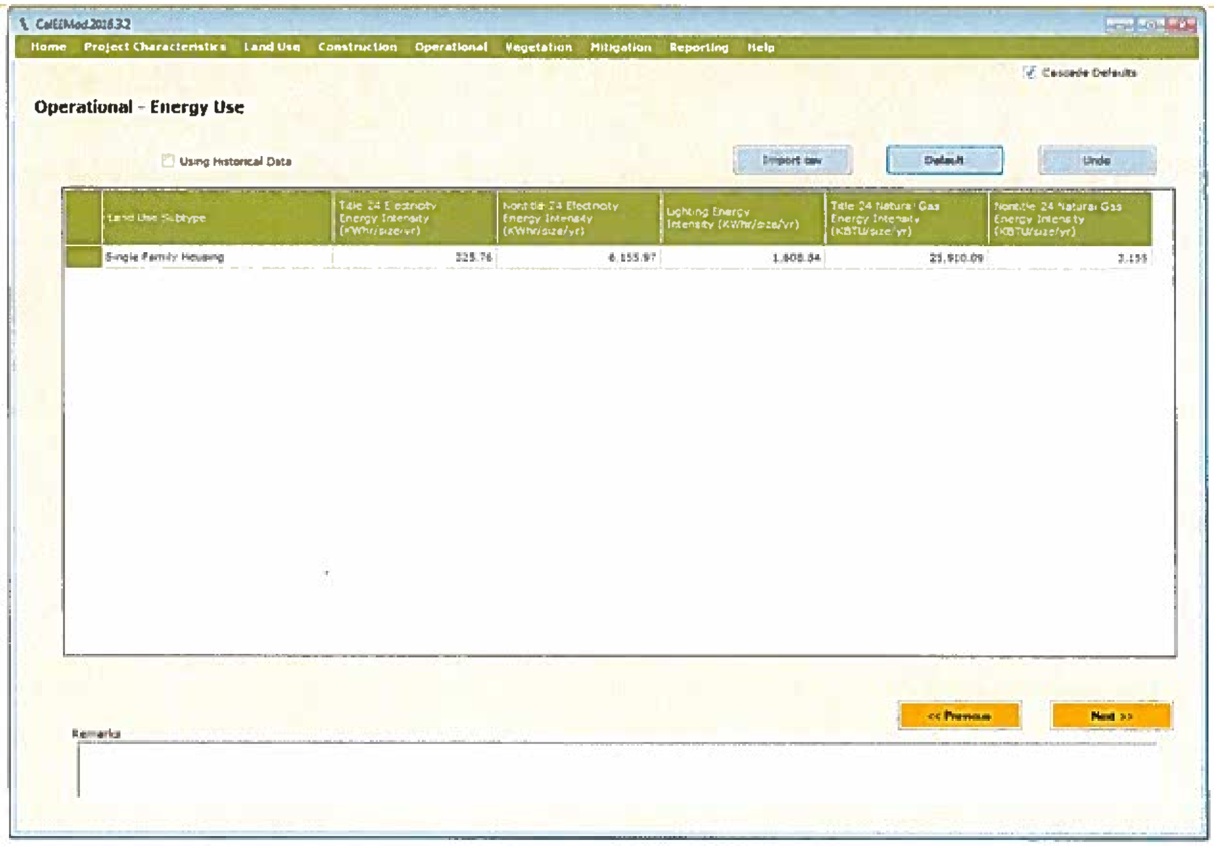
Title 24 of the California Code of Regulations, known as the California Building Standards Code or Title 24, contains energy conservation standards applicable to all residential and non-residential buildings throughout California. With CalEEMod, building electricity and natural gas use is divided into two categories: 1) end uses subject to Title 24 standards; and, 2) end uses not subject to Title 24 standards. The distinction is used when the mitigation measure for exceeding Title 24 standards (BE-1) is applied. Lighting is also a separate category in CalEEMod for which a separate mitigation measures (LUT-1) may be applied for using energy efficient lighting.

For electricity, Title 24 uses include the major building envelope systems covered by Part 6 (California Energy Code) of Title 24 such as space heating, space cooling, water heating, and ventilation. Non-Title 24 uses include all other end uses, such as appliances, electronics, and other miscellaneous plug-in uses. Because some lighting is not considered as part of the building envelope energy budget, and since a separate mitigation measure is applicable to this end use, CalEEMod makes lighting a separate category.

For natural gas, uses are likewise categorized as Title 24 or Non-Title 24, with Title 24 uses including building heating and hot water end uses. Non-Title 24 natural gas uses include cooking and appliances (including pool/spa heaters).

The baseline values are based on the CEC sponsored California Commercial End Use Survey (CEUS) and Residential Appliance Saturation Survey (RASS) studies[12]. For climate zones not included in these surveys, data from the closest climate zone was used as a surrogate. Since these studies are based on older buildings, adjustments have been made to account for changes due to Title 24 building codes as described in Appendix E. The user should select the use historical box if they only want an adjustment to the 2005 standards which were in effect when CARB developed its Scoping Plan 2020 No Action Taken predictions. After selecting the historical button, the user must also click the default button to load the historical default values.

[12] CEC. October 2010. Residential Appliance Saturation Survey. Available at: http://www.energy.ca.gov/appliances/rass
CEC. March 2006. Commercial End-Use Survey. Available at: http://www.energy.ca.gov/ceus/



4.7 Water and Wastewater Use

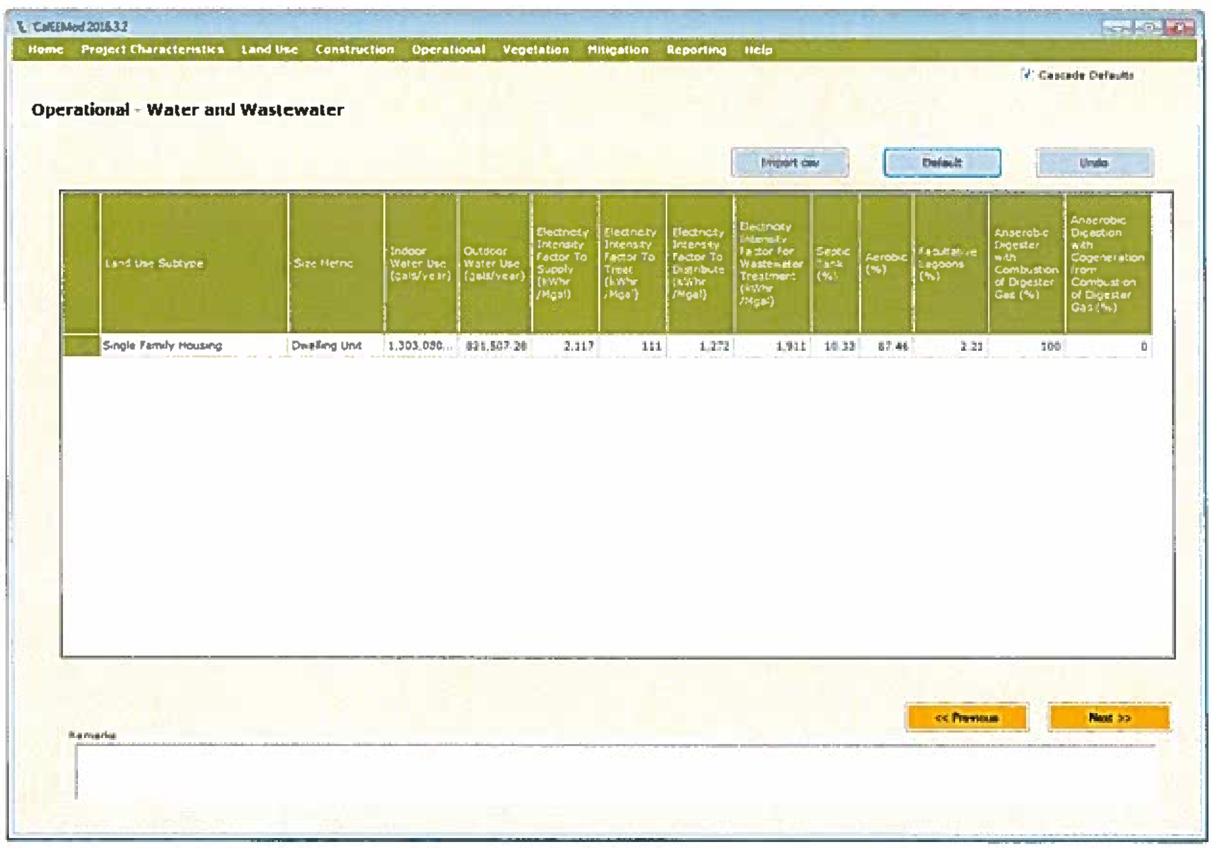
This screen estimates the land uses contribution of GHG emissions associated with supplying and treating water and wastewater. This screen is used to enter the amount of water in gallons used indoors and outdoors for each land use subtype[13]. The indoor water is also used to estimate the amount of wastewater. The electricity intensity factor for various phases of providing water is provided. Depending on the specific water supply used or treatment method used these numbers can vary over a wide range. Supplying water is bringing the water from its primary source such as the ground, river, or snowpack to the treatment plant. Distributing the water is bringing the water from the treatment plant to the end users. The electricity intensity

[13] Gleick, P.H.; Haasz, D.; Henges-Jeck, C.; Srinivasan, V.; Cushing, K.K.; Mann, A. 2003. Waste Not, Want Not: The Potential for Urban Water Conservation in California. Published by the Pacific Institute for Studies in Development, Environment, and Security. Full report available at: http://www.pacinst.org/reports/urban_usage/waste_not_want_not_full_report.pdf. Appendices available at: http://pacinst.org/publication/waste-not-want-not/

Dziegielewski; B.; Kiefer, J.C.; Optiz, E.M.; Porter, G.A.; Lantz, G.L.; DeOreo, W.B.; Mayer, P.W.; Nelson, J.O. 2000. Commercial and Institutional End Uses of Water. Published by the American Water Works Association Research Foundation.

Northern California Golf Association. Improving California Golf Course Water Efficiency. Available at: http://www.water.ca.gov/wateruseefficiency/docs/2004Apps/2004-079.pdf factors are multiplied by the utility GHG emissions intensity factors for the GHGs and are classified as indirect emissions. The default electricity intensity is from the CEC's 2006 Refining Estimates of Water-Related Energy Use in California using the average values for Northern and Southern California[14]. The location will automatically select the appropriate values if using these defaults. Since the electricity can vary greatly based on locations, the user should override these values if they have more specific information regarding their specific water supply and treatment.

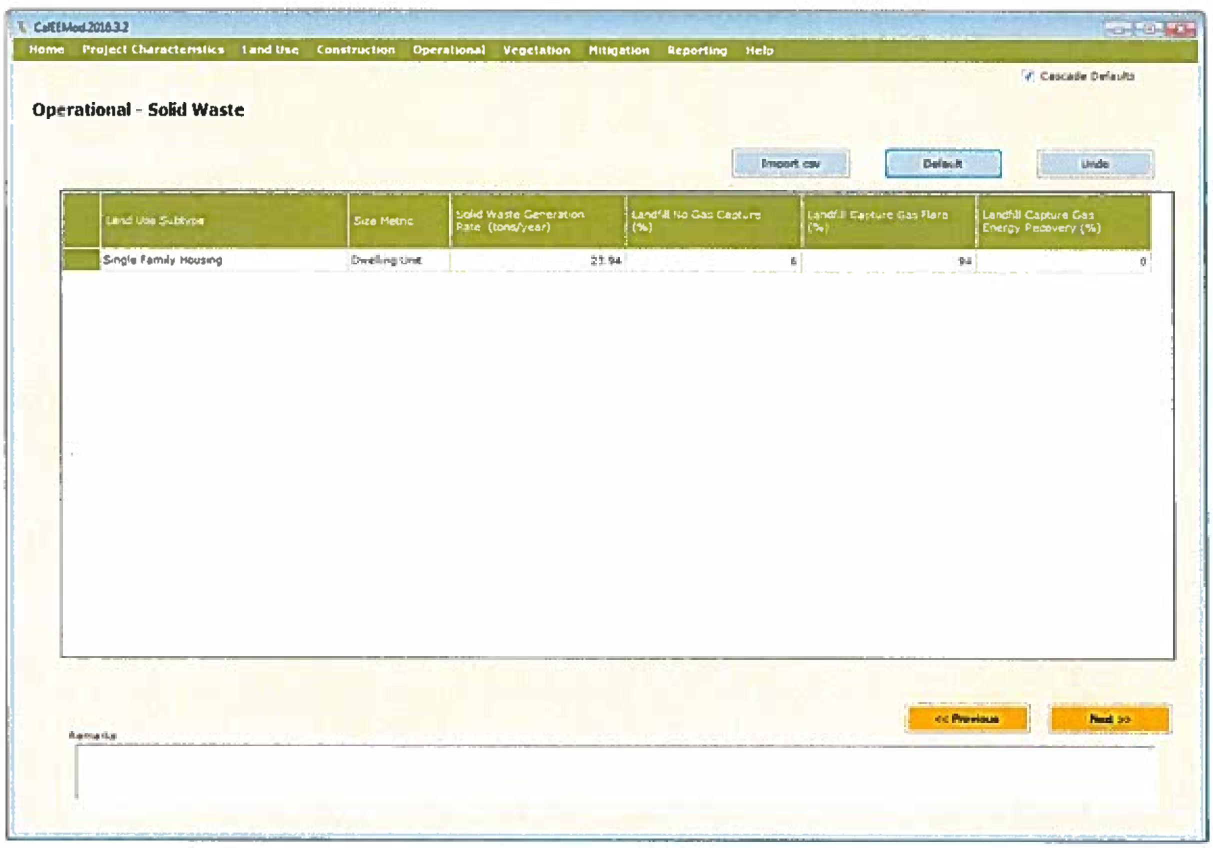
Wastewater may also have direct emissions of GHGs. These depend on the type of wastewater treatment system (e.g., septic, aerobic or lagoons) used and therefore the wastewater treatment type percentages are variables. In addition, the model calculates impacts if the solids are digested either through an anaerobic digester or with co-generation from combustion of digester gas. Each type has associated GHG emission factors. Some of these may be classified as biogenic. Not all of the biogenic emissions are accounted for since there are not adequate emissions factors at this time. Refer to Appendix A on how to properly change the defaults, if necessary, and the methodology used to calculate impacts from wastewater treatment.



[14] CEC-500-2006-118. Available at http://www.energy.ca.gov/2006publications/CEC-500-2006-118/CEC-500-2006-118.PDF.

4.8 Solid Waste

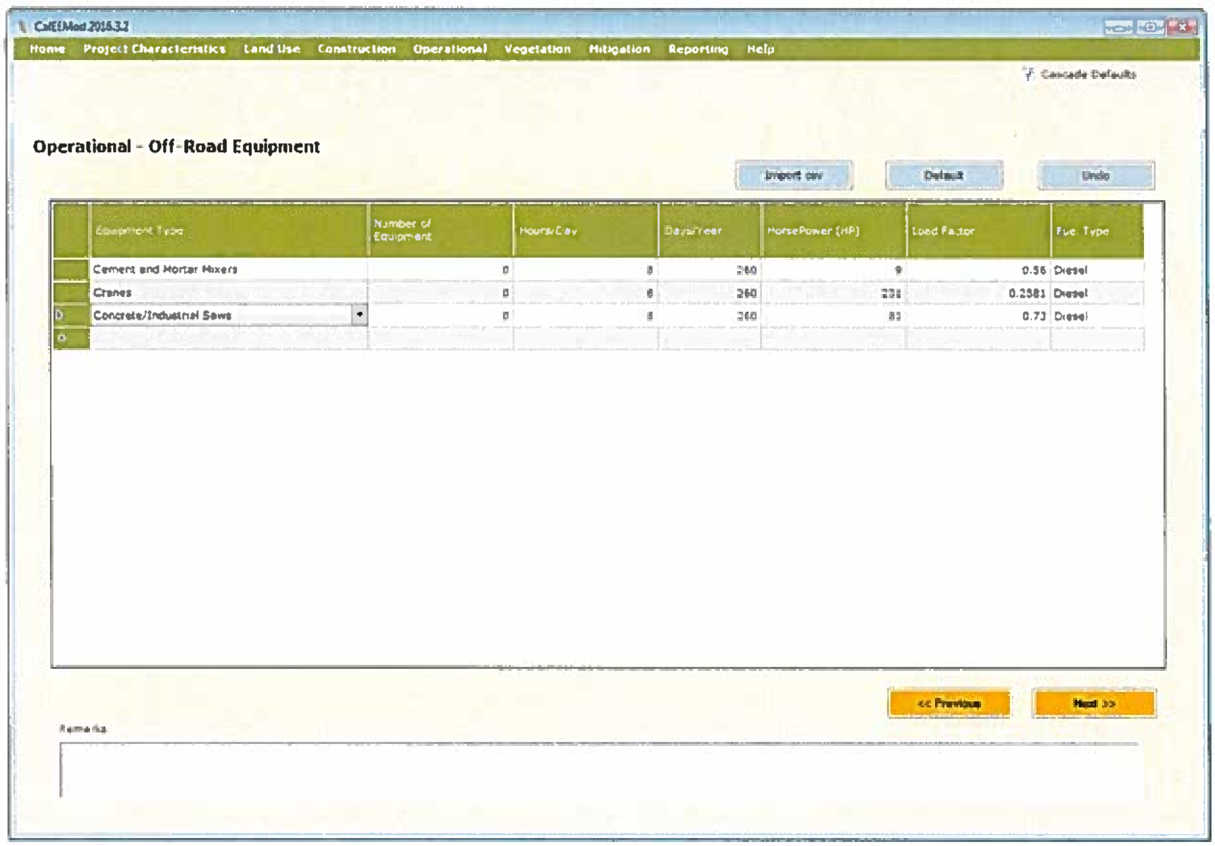
The solid waste screen determines the GHG emissions associated with disposal of solid waste into landfills. In order to estimate the eventual contribution of GHG emissions from solid waste disposed by a land use annually, the total amount of carbon dioxide and methane that would be evolved over the span of many years is calculated. This is based on the IPCC's methods for quantifying GHG emissions from solid waste using the degradable organic content of waste[15]. Waste disposal rates by land use and overall composition of municipal solid waste in California is primarily based on CalRecycle data. The amount of methane emitted depends on characteristics of the landfill, and therefore the default percentage is based on the types of landfills assumed by CARB in their GHG emissions inventories. Portions of these emissions are biogenic. The defaults for the gas capture (e.g., no capture, flaring, energy recovery) are statewide averages except for Santa Barbara APCD which has a 100% landfill capture gas flare. The user has the ability to override the defaults if the gas capture at the landfill to be used by the project is known. Local jurisdictions can also provide guidance to users as to what default properly reflects known regional solid waste gas capture.



[15] IPCC. 2006. 2006 IPCC Guidelines for National Greenhouse Gas Inventories. Volume 5 Waste. Available at http://www.ipcc-nggip.iges.or.jp/public/2006gl/index.html.

4.9 Off-Road Equipment

The Operational - Off-Road Equipment sub-screen allows the user to identify any off-road equipment used during operational activities (e.g., forklifts, cranes, loaders, generator sets, pumps, pressure washers, etc.) at the project site. Because such equipment cannot be assumed to be needed for a particular land use project, a user must provide the data in order for CalEEMod to calculate the resulting emissions from off-road equipment operation. A dropdown list of off-road equipment is provided for the user to identify each piece of equipment. The model requires the following specific information per equipment type. The user would need to provide the number of pieces for each equipment type. The model assumes an operation activity of 8 hours per day and 260 days per year, as well as the horsepower and load factor of the equipment type, but the user has the ability to override the default assumptions with project specific information. Finally, the model assumes diesel fuel, but a dropdown menu is provided to allow the user to choose bio-diesel, compressed natural gas (CNG) or electrical if known, to power the equipment.

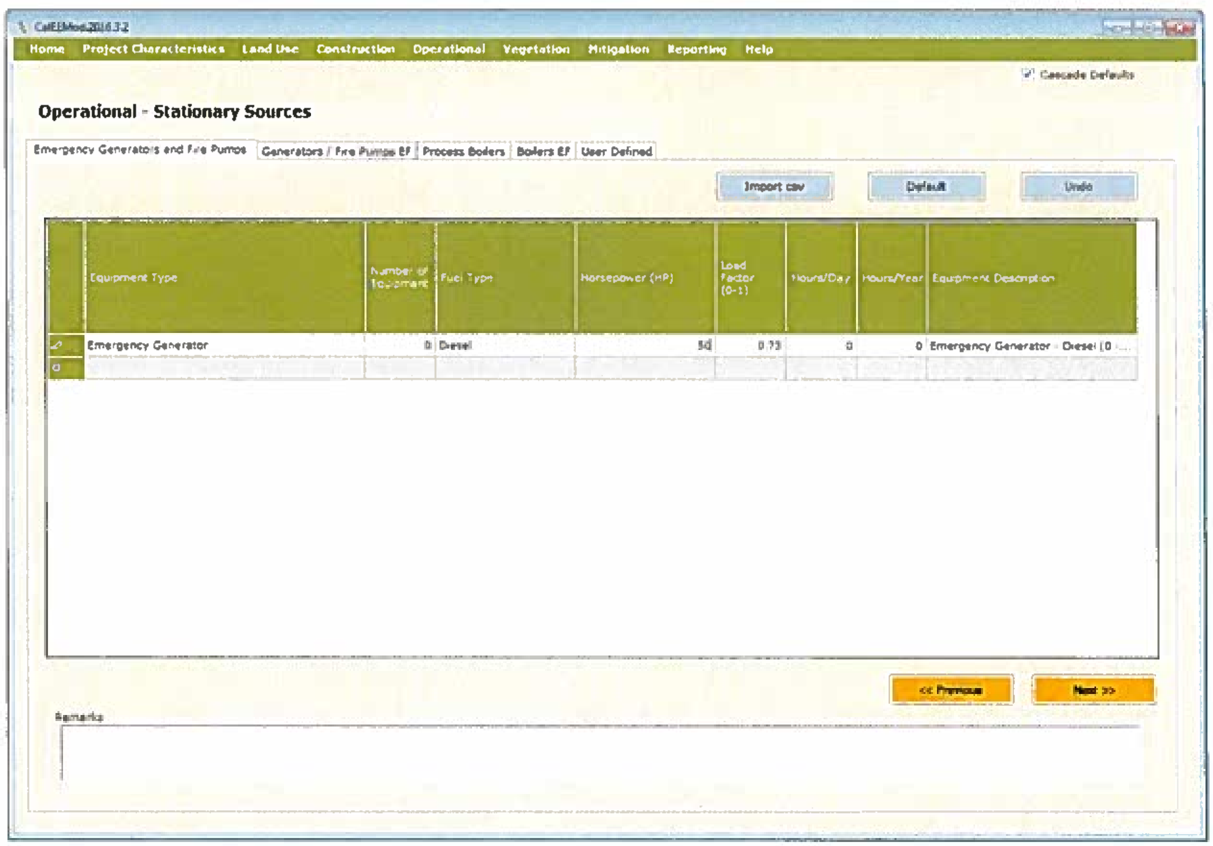


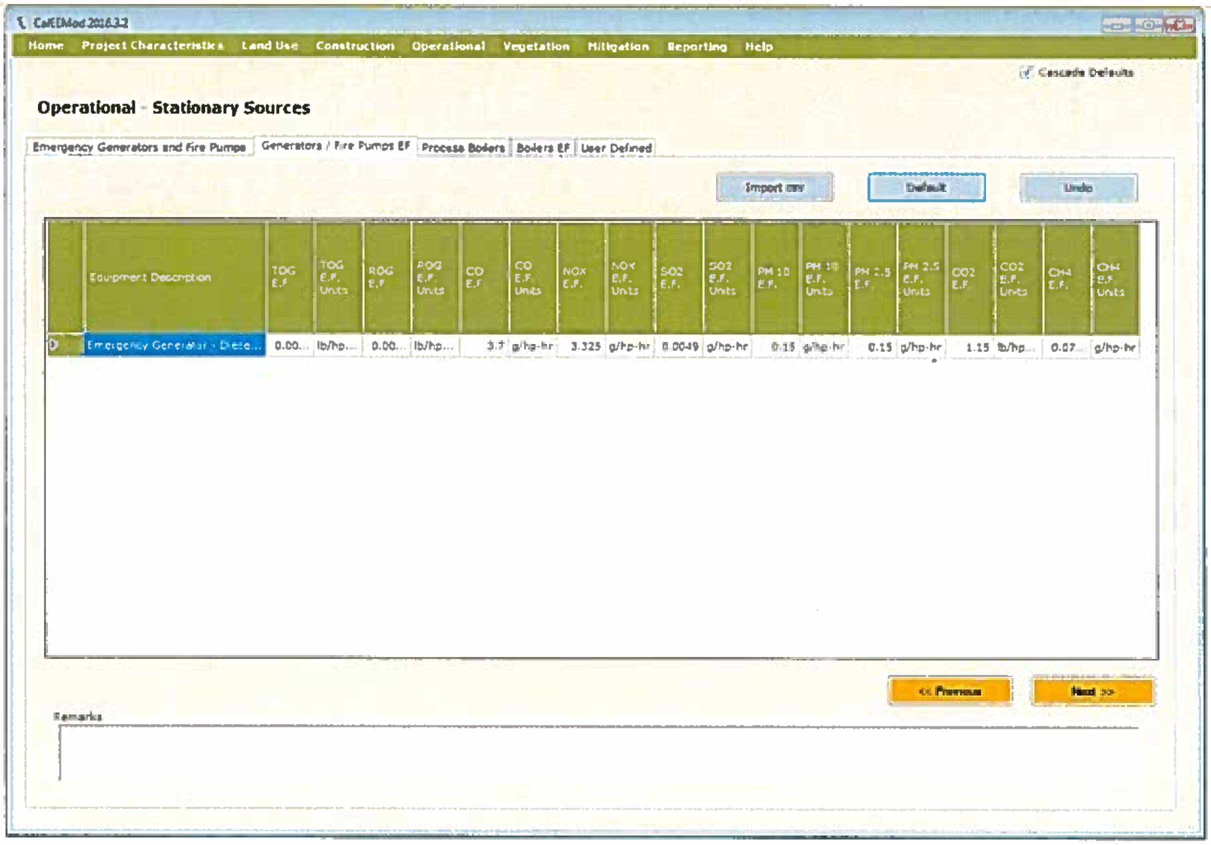
4.10 Stationary Sources

The Stationary Sources screen consists of five sub-screens: Emergency Generators and Fire Pumps and their default emission factors, Process Boilers and their default emission factors, and User Defined Sources. Consult with the local air district to determine if permitted stationary sources should be included in the project analysis using CalEEMod.

4.10.1 Emergency Generator and Fire Pumps and Default Emission Factors

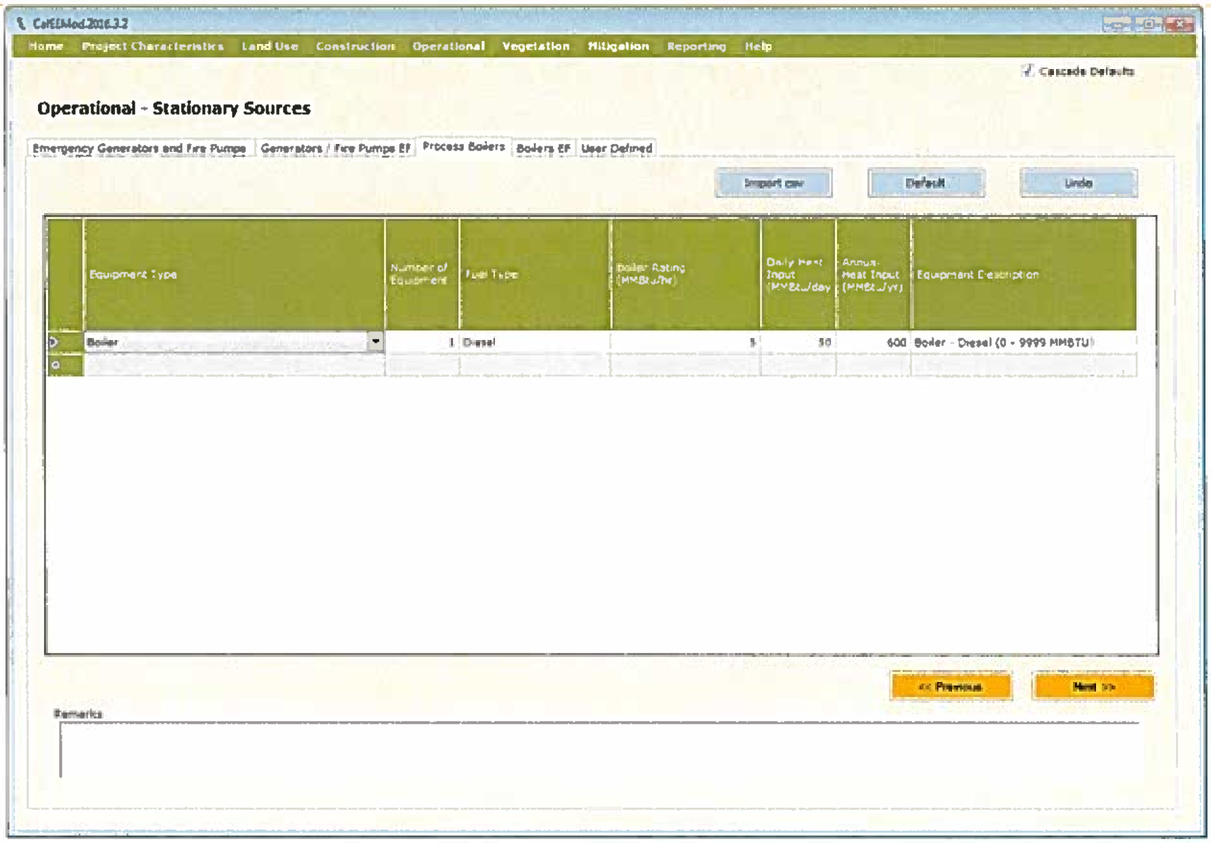
Two sub-screens allow the user to enter emergency power generators and diesel fueled fire pumps and to estimate emissions. This type of equipment operates only for maintenance and testing, or during emergency situations, such as power failures. To calculate emissions, the user must enter the engine rating (in horsepower), the anticipated maximum daily usage, and the anticipated maximum annual usage into the Emergency Generators and Fire Pumps sub-screen. The user may change the default load factor. The default emission factors for the equipment are shown on the separate Generators/Fire Pumps EF (emission factor) sub-screen. The user can replace the default emission factors, but needs to provide custom emission factors in the predefined units. See Appendix A for the sources of default emission factors and emission calculation methodology.





4.10.2 Process Boilers and Default Emission Factors

Two sub-screens allow the user to enter process boilers and to estimate emissions. Do not use this option for boilers providing space heating or building hot water, as these uses are included building energy use (See Subchapter 4.6). To calculate process boiler emissions, the user must enter the boiler rating (in million BTU/hr) and maximum anticipated daily and annual heat input in the Process Boilers sub-screen. The default emission factors for boilers are shown on the separate Boiler EF (emission factor) sub-screen. The user can replace the default emission factors, but needs to provide custom emission factors in the predefined units. See Appendix A for the sources of default emission factors and emission calculation methodology.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Fuel Type
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Daily Heat Input (MMBtu/day)
Annual Heat Input (MMBtu/yr)
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5
50
600
Boiler - Diesel (0 - 9999 MMBTU)
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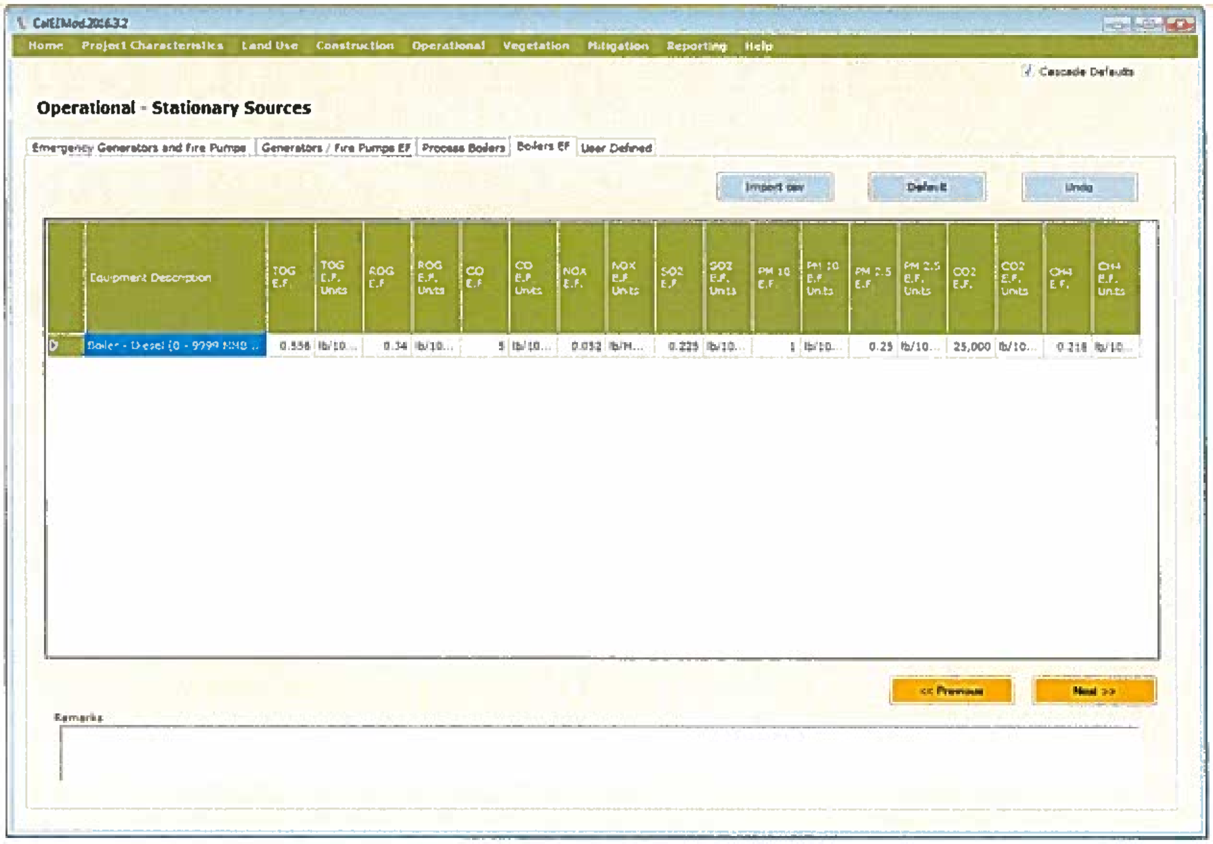
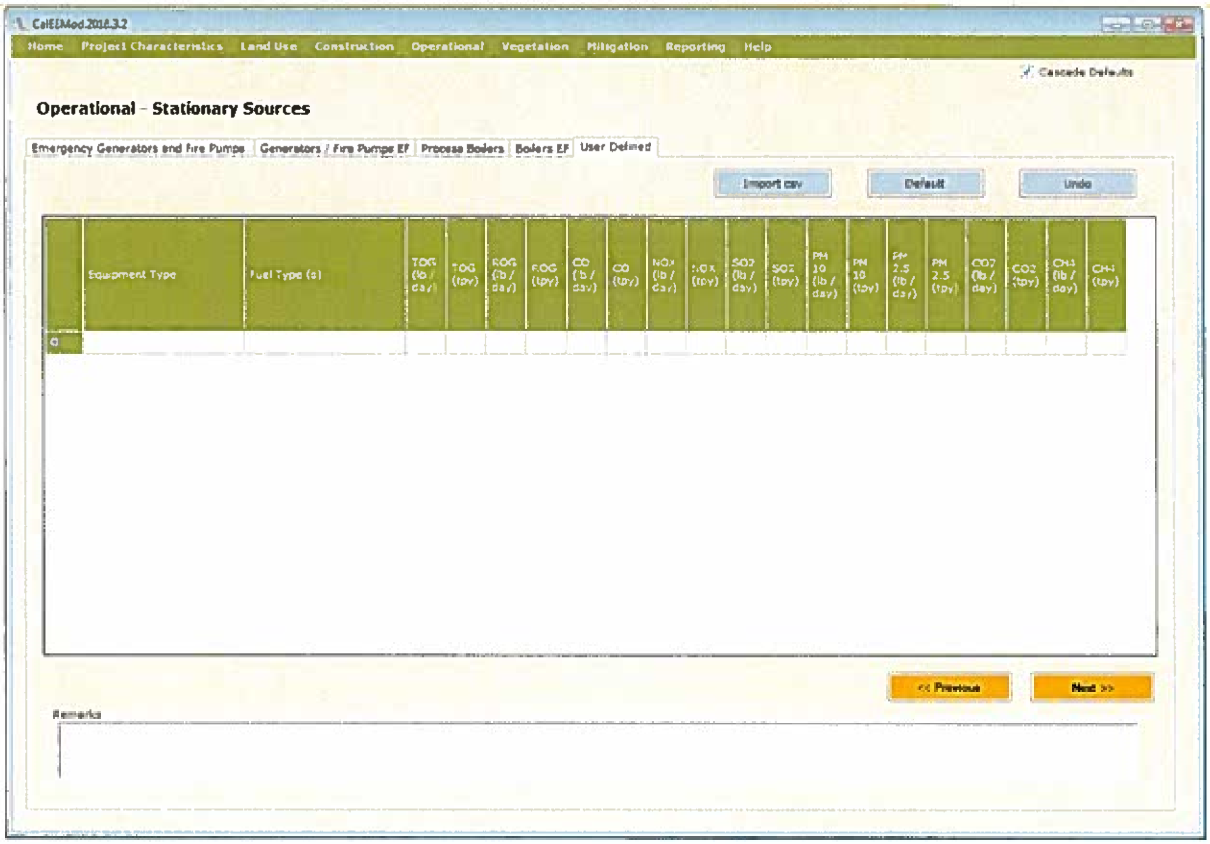


4.10.3 User Defined

An option for the user to define stationary sources other than emergency generators, fire pumps and process boiler has been included in the User Defined sub-screen. Emissions for this source would include any other miscellaneous sources that typically require permits to operate issued by an air district. Emissions may be manually entered here, either by transferring values from the permits to operate, or by calculating emissions outside of CalEEMod. Any emissions entered here will be transferred to the appropriate reports.



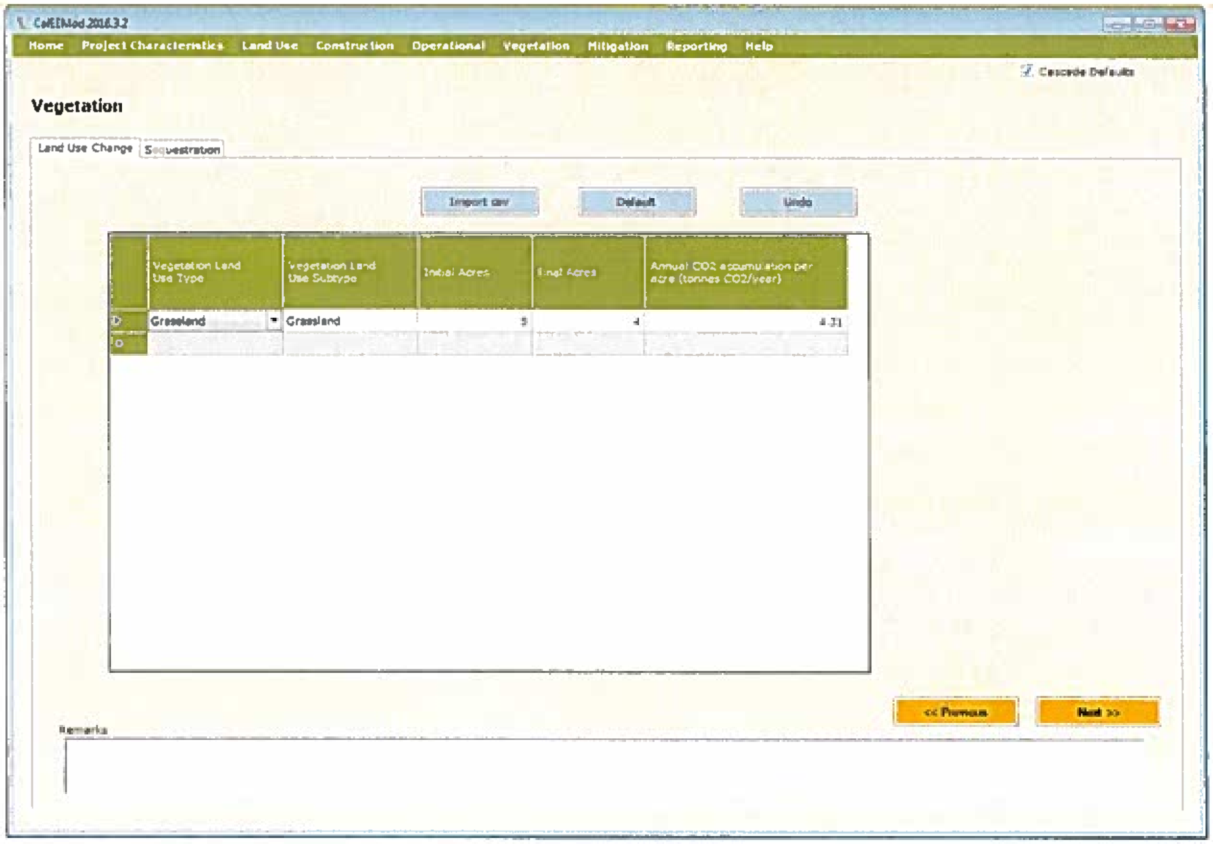
4.11 Vegetation

The vegetation screen is used to estimate the one-time change in carbon sequestration capacity due to a project. There are two sub-screens, Land Use Change and Sequestration. The methods used are based on IPCC[16].

4.11.1 Land Use Change

The Land Use Change sub-screen estimates GHG emissions due to a change in vegetation resulting from a change in land use type. The user enters the vegetation land use type, the initial and final acreage of the vegetation land use type, and the annual carbon dioxide equivalent accumulation per acre if the user chooses to override the default value. Settlement land use acreage is not considered since it is a net zero at steady state unless trees are added.

[16] IPCC. 2006. 2006 IPCC Guidelines for National Greenhouse Gas Inventories Volume 4. Available at http://www.ipcc-nggip.iges.or.jp/public/2006gl/index.html



4.11.2 Sequestration

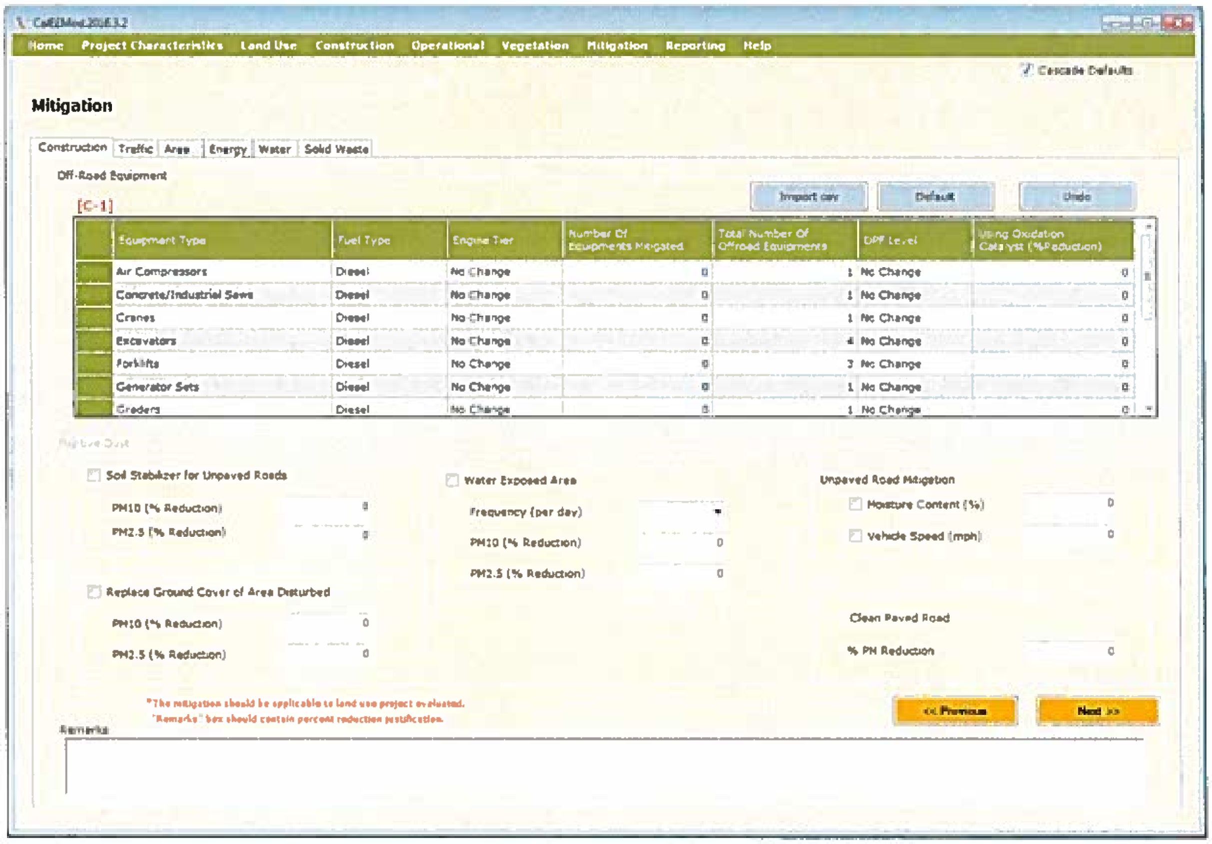
This sub-screen of Vegetation is used to estimate the GHG emissions associated with the sequestration of net new trees added to the project site. Consistent with IPCC recommendations a 20 year active growth period is assumed. The user enters the tree type or miscellaneous if it is not known, and the total number of trees. The user can override the default carbon sequestration rate.

4.12 Mitigation

The mitigation screen consists of six sub-screens that the user can indicate and supply the necessary information to estimate the emissions after mitigation measures have been implemented. The mitigation measures included in CalEEMod are largely based on the CAPCOA Quantifying Greenhouse Gas Mitigation Measures (http://www.capcoa.org/wp-content/uploads/downloads/2010/09/CAPCOA-Quantification-Report-9-14-Final.pdf) document. The CAPCOA measure numbers are provided next to the mitigation measures in CalEEMod to assist the user in understanding each measure by referencing back to the CAPCOA document. This User's Guide focuses on key aspects of the Mitigation sub-screens that users should pay particular attention.

4.12.1 Construction Mitigation

This sub-screen consists of a datagrid of off-road construction equipment to apply various mitigation measures and check boxes with supplemental information for fugitive dust emissions mitigation.



To apply mitigation to construction equipment, the user selects the equipment type, notes the number of equipment mitigated (of the total number of off-road equipment listed), and type of mitigation that applies. If substantial evidence supporting reductions was available at the time of development, options include fuel type (diesel, CNG, electric, hybrid, biodiesel), engine tier (typically select Tier 4), diesel particulate filter tiers (Tier 3 being the most effective), and use of oxidative catalysts. The program estimates how much if any increase or decrease in emissions to apply for each pollutant. Some mitigation measures have trade-offs in pollutant reductions and therefore may result in increases of some pollutants. The mitigation option to use alternative fuel for construction equipment is consistent with mitigation measure C-1 in the CAPCOA Quantifying GHG Mitigation document.

To apply mitigation to construction fugitive dust, the user selects the check box in front of the mitigation measure name, and enters in the appropriate information in the drop down or text boxes. Some fugitive dust mitigation required by some air districts do not appear here since the fugitive dust source they mitigate is not quantified by CalEEMod, in particular this includes fugitive dust generated by wind over land and storage piles. Since the fugitive dust source is not quantified it is not appropriate to apply the reduction.

For Unpaved Road Mitigation for construction fugitive dust, the maximum vehicle speed and the minimum moisture content for unpaved roads are entered. Defaults for these values are those entered on the On-Road Fugitive Dust screen. Mitigated emissions are calculated using the VMT from on-road vehicles traveling along unpaved roads, previously calculated from the percentages entered on the On-road Fugitive Dust Screen (e.g., % Pave Worker, % Pave Vendor or % Pave Hauling).

Users may check the boxes and provide a lower vehicle speed and a higher moisture content to conduct the mitigation calculation. If during a particular construction phase the user defined mitigated vehicle speed is higher than the unmitigated vehicle speed and/or the user defined mitigated moisture content is lower than the unmitigated moisture content, a warming message will be displayed. In this case, the unmitigated values will be used, resulting in no mitigation being calculated.

4.12.2 Traffic Mitigation

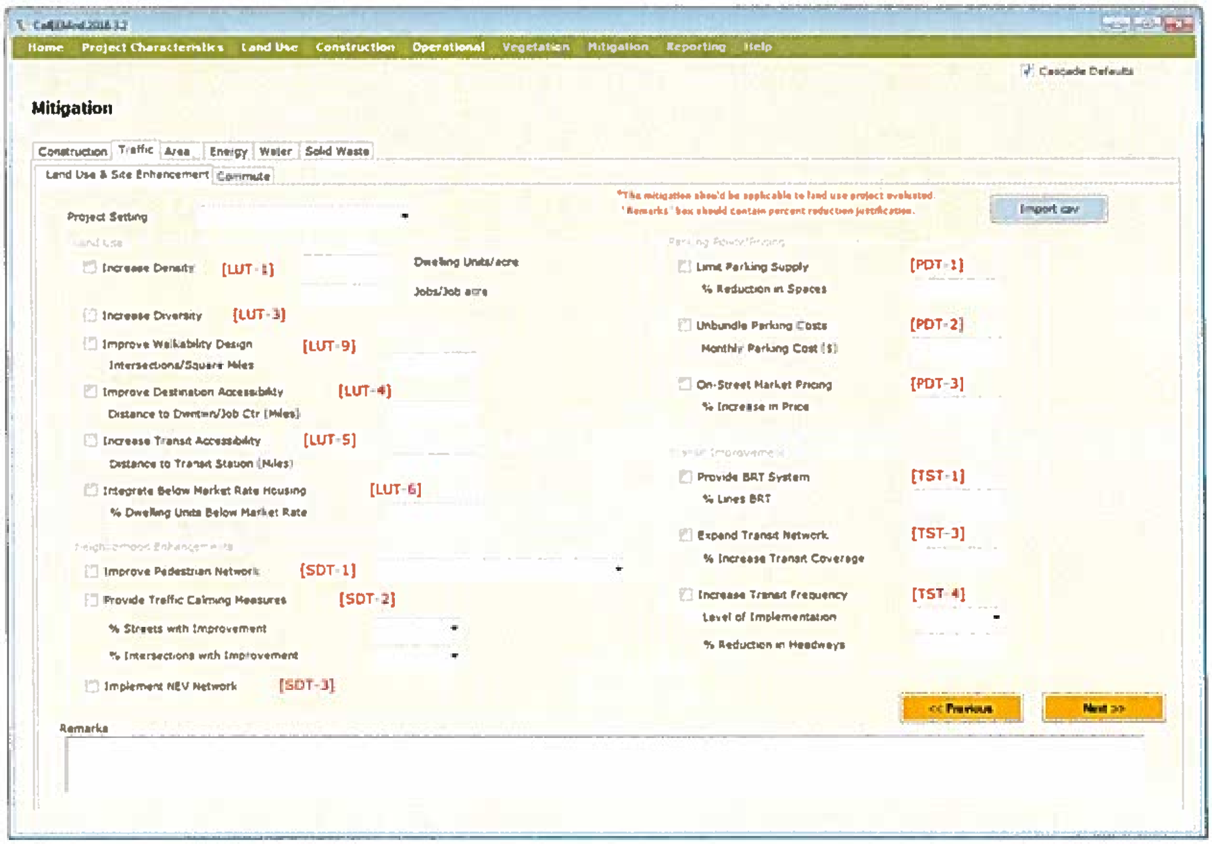
There are two traffic mitigation sub-screens that the user can select from, Land Use & Site Enhancement and Commute. First, the user must select the Project Setting as defined in the CAPCOA document (pp. 59-60).

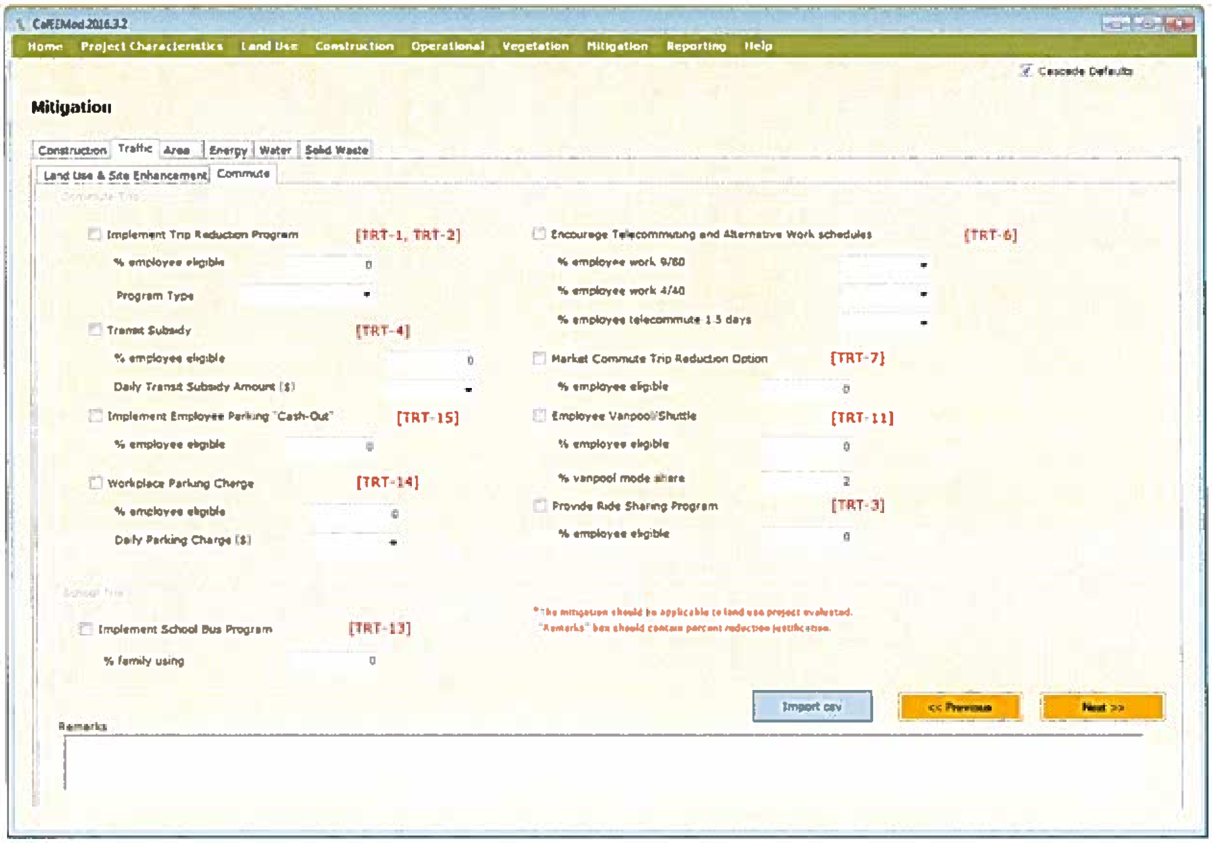
- Low Density Suburban: An area characterized by dispersed, low-density, single-use, automobile dependent land use patterns, usually outside of the central city (a suburb).
- Suburban Center: An area that serves the population of the suburb with office, retail and housing which is denser than the surrounding suburb.
- Urban: An area which is located within the central city with higher density of land uses than you would find in the suburbs. It may be characterized by multi-family housing and located near office and retail.
- Urban Center *(referred to as Compact Infill in the CAPCOA document)*: An area which is located within or contiguous with the central city. Examples may include redevelopment areas, abandoned sites, or underutilized older buildings/sites.

If the CAPCOA measure did not distinguish between Suburban Center and Low Density Suburban, values for Low Density Suburban were used. Similarly, if Urban Center and Urban values were not distinguished, Urban values were used.

The user checks the box next to each mitigation measure and fills in the appropriate information as required. The maximum reduction caps defined in the CAPCOA Quantifying GHG Mitigation document are integrated into these calculations. The CAPCOA traffic mitigation measure numbers included in CalEEMod are the following: LUT-1, LUT-3, LUT-9, LUT-4, LUT-5, LUT-6, SDT-1, SDT-2, SDT-3, PDT-1, PDT-2, PDT-3, TST-1, TST-3, TST-4, TRT-1, TRT-2, TRT-4,

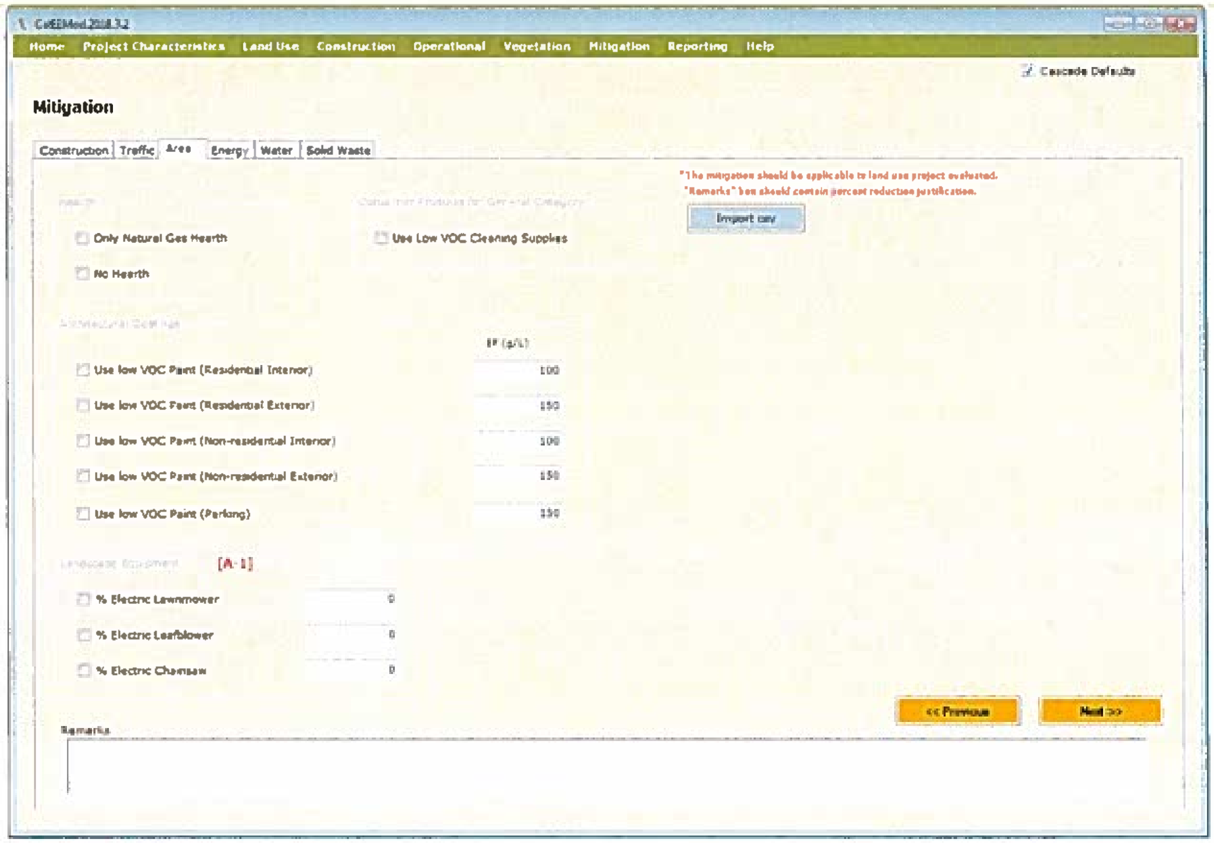
TRT-15, TRT-14, TRT-6, TRT-7, TRT-11,TRT-3, and TRT-13. The NEV network mitigation measure (SDT-3) assumes the low end of the CAPCOA recommendations.





4.12.3 Area Mitigation

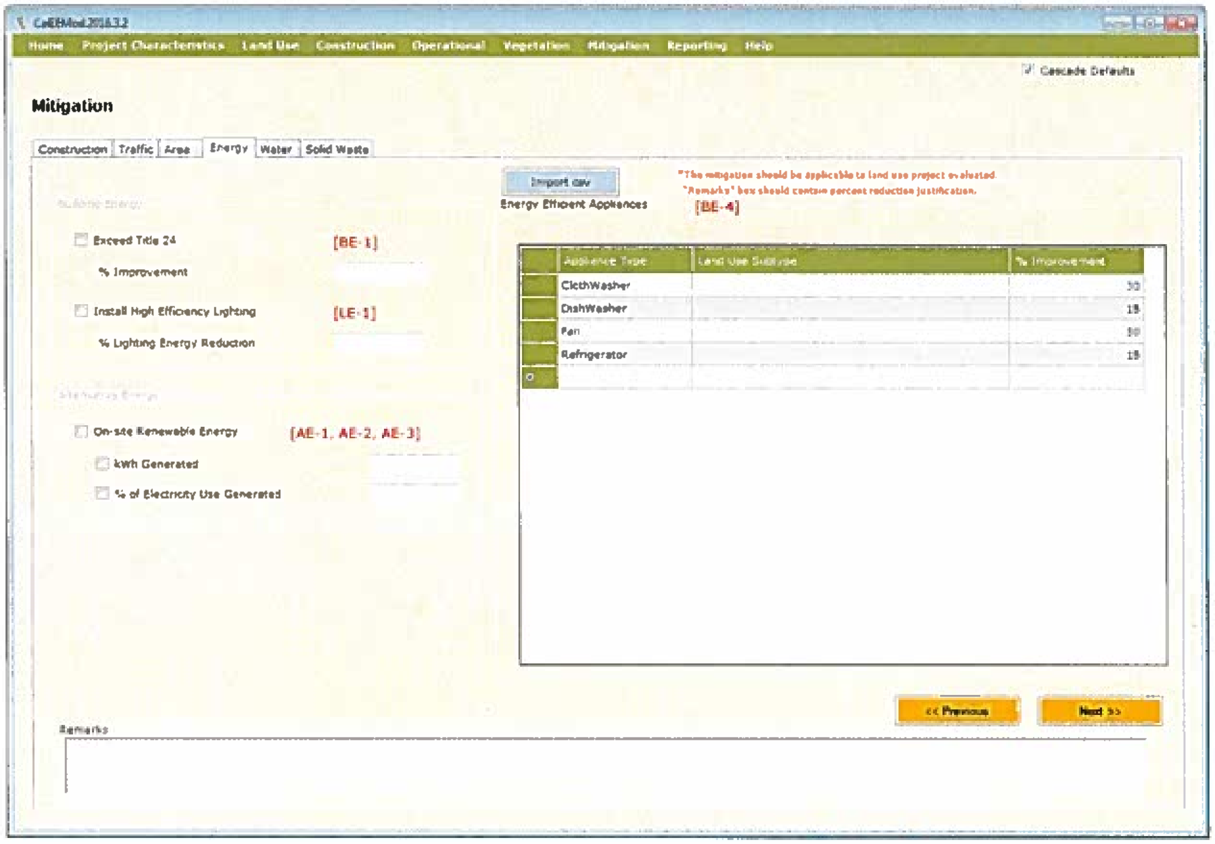
The user can select from a few area source mitigation measures on the Area sub-screen by checking the appropriate box and supplying any additional information in the text boxes. These measures include all natural gas hearths, no hearths, electric landscaping equipment use, reduced ROG coatings, and reduced general category consumer product ROG content. The area landscaping mitigation to prohibit gas powered landscape equipment is consistent with mitigation A-1 in the CAPCOA Quantifying GHG Mitigation document.



4.12.4 Energy Mitigation

The user selects energy mitigation measures on the Energy sub-screen by using the check boxes or the datagrid. These correspond to CAPCOA Mitigation Measures LE-1, BE-1, AE-1, AE-2, AE-3 and BE-4 as listed in the CAPCOA Quantifying GHG Mitigation document. The lighting is a percentage reduction in lighting as supplied by the user. The datagrid is used to enter the land use subtypes that will use energy efficient appliances. The percent improvement is the typical percent improvement above standard appliances according to the 2008 Energy Star Annual Report[17]. Alternative Energy has two methods to enter the amount of alternative energy. The first is the amount of kW-hr generated. The second is the percentage of the total electricity use by buildings that is generated. At this time alternative energy methods that are not carbon neutral are not quantified. To apply the amount of alternative energy only one of the two methods (kW-hr or percentage) needs to be entered for CalEEMod to calculate emission reductions.

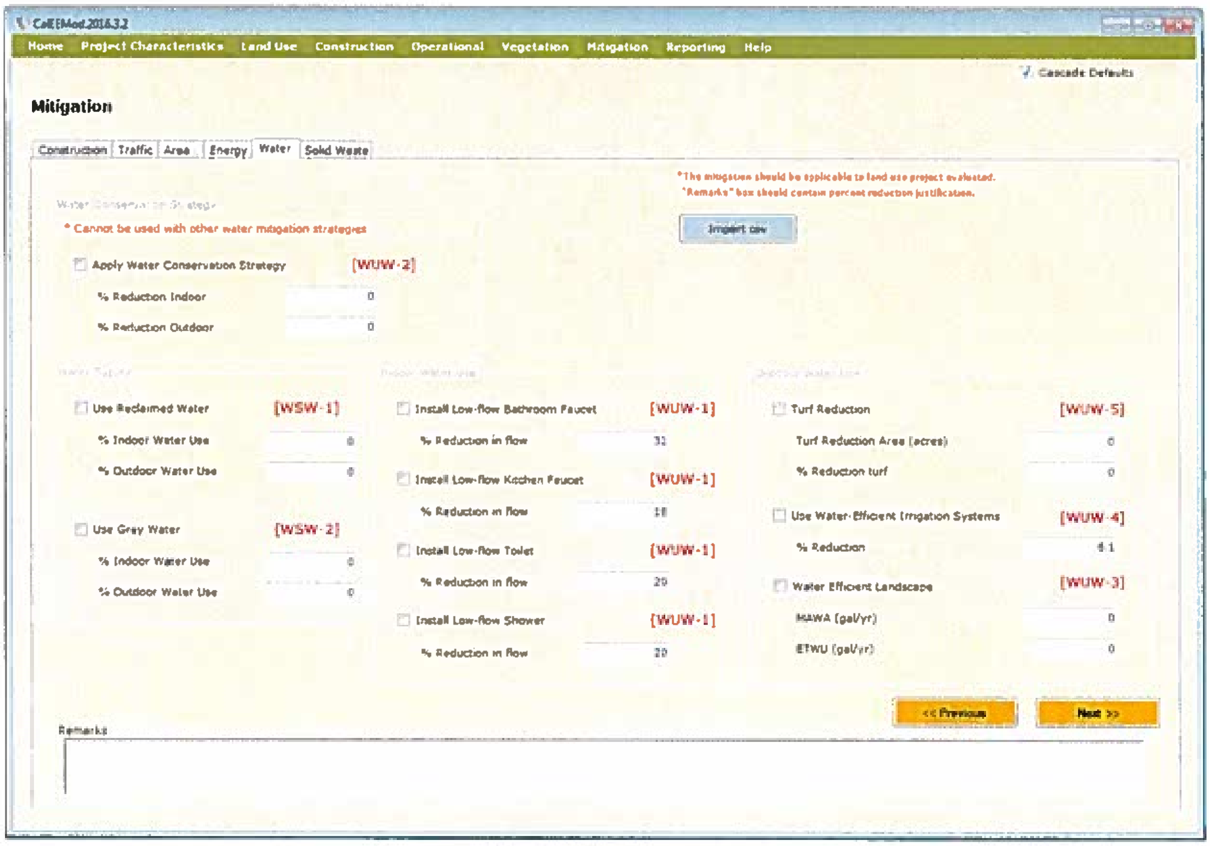
[17] Available at: https://www.energystar.gov/ia/partners/annualreports/annual_report_2008.pdf



4.12.5 Water Mitigation

On the Water sub-screen, water mitigation can either be estimated as the percent reduction based on a water conservation strategy or the other individual mitigation measures. The CAPCOA Quantifying GHG Mitigation document includes water supply and use measures WSW-1 & 2, and WUW-1 through 5.

For CAPCOA Mitigation Measure WSW- 3 (Use Locally Sourced Water Supply), using locally-sourced water or water from less energy-intensive sources reduces the electricity and indirect CO_2 emissions associated with water supply and transport because water from local or nearby groundwater basins, nearby surface water and gravity-dominated systems have smaller energy-intensity factors. This mitigation measure is not included in the Water mitigation sub-screen, therefore, to implement WSW-3, the user should alter the energy intensity values in water and run a separate CalEEMod run to accommodate these values.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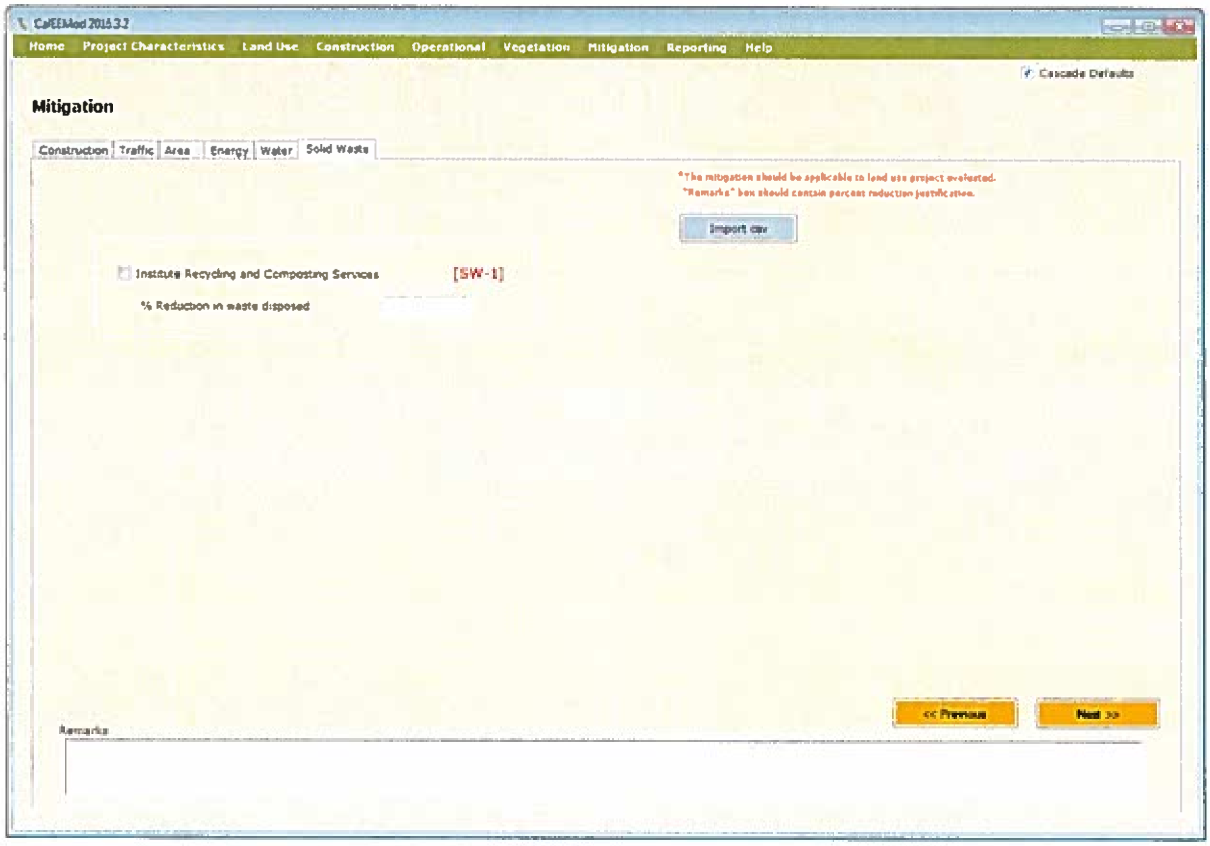
Import csv
Apply Water Conservation Strategy
[WUW-2]
% Reduction Indoor
% Reduction Outdoor
Use Reclaimed Water
[WSW-1]
% Indoor Water Use
% Outdoor Water Use
Use Gray Water
[WSW-2]
Install Low-flow Bathroom Faucet
[WUW-1]
% Reduction in flow
Install Low-flow Kitchen Faucet
Install Low-flow Toilet
Install Low-flow Shower
Turf Reduction
[WUW-5]
Turf Reduction Area (acres)
% Reduction turf
Use Water-Efficient Irrigation Systems
[WUW-4]
% Reduction
Water Efficient Landscape
[WUW-3]
MAWA (gal/yr)
ETWU (gal/yr)
Remarks

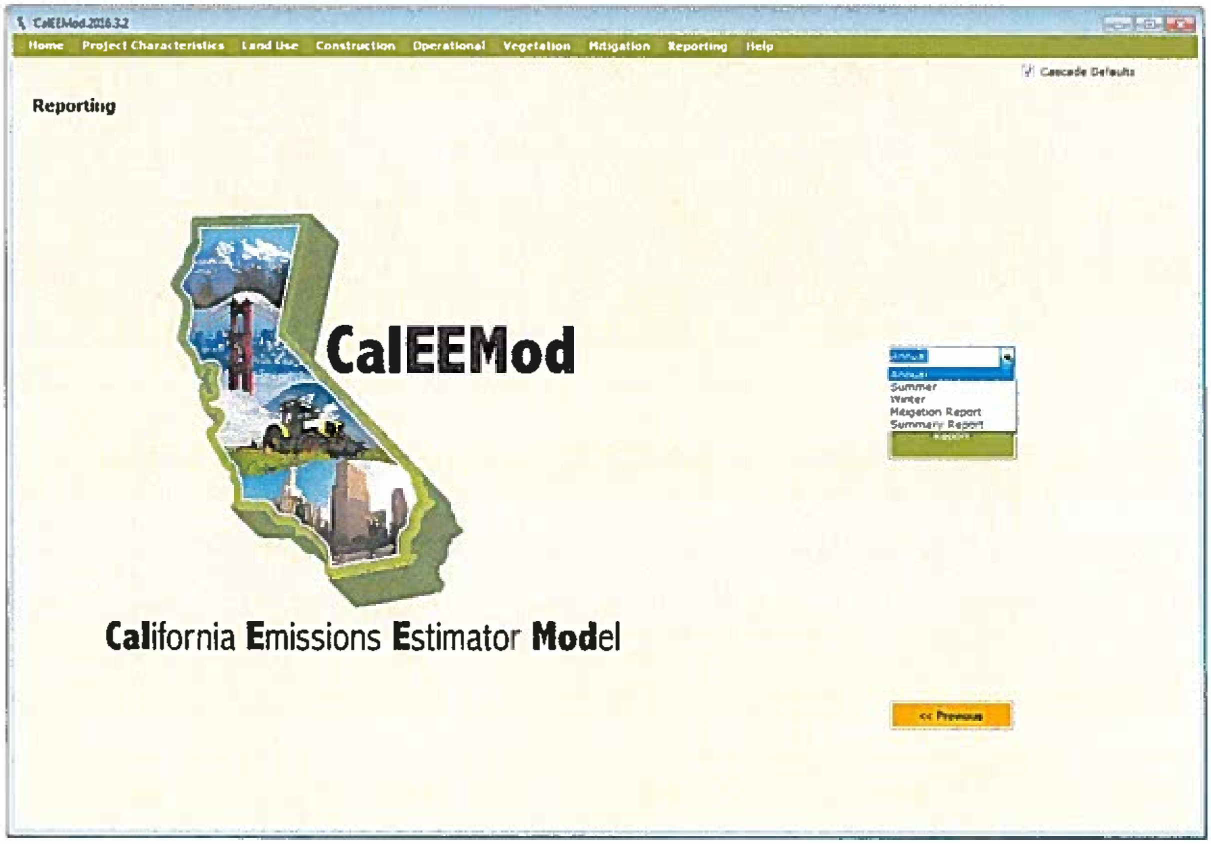
4.12.6 Solid Waste Mitigation

The user can calculate an emissions reduction for recycling waste. This mitigation measure corresponds to CAPCOA Mitigation Measure: SW-1.



4.13 Reporting

The user initiates final emission calculations by selecting the report and clicking on the Recalculate All Emissions and Run Report button. The available reports include: Annual, Summer (peak) Daily, Winter (peak) Daily, Mitigation and Summary of peak daily emissions and annual GHG emissions. A separate report viewer will appear on the screen. From this report viewer, the user can view the each selected report on-screen, print each report, save each report as either a Microsoft Excel .xls file, an Adobe Acrobat .pdf file, or in the case of the Mitigation report, a Microsoft Word .doc file. It is important to note that the data presented in the Excel file has already been calculated and the calculated results are placed in the grids as text. For this reason, the user cannot change an emission value presented in an Excel file and expect the report to calculate a revised value. These values, however, can be copied to new Excel spreadsheet for any further desired calculation with the data. If the user elects to generate a Summary report, the project needs to use only the CalEEMod defaults and there can be no remarks on any page.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